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WEIR EXPERIMENTS, COEFFICIENTS, AND FORMULAS

BY

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WEIR EXPERIMENTS, COEFFICIENTS, AND FORMULAS.

By Robert E. Horton.

INTRODUCTION.

DEFINITIONS OF TERMS.

The word "weir" will be used to describe any structure used to determine the volume of flow of water from measurements of its depth on a crest or sill of known length and form. In this general sense timber and masonry dams having various shapes of section, reservoir overflows, and the like may be weirs. Terms, more or less synonymous, used to describe such weirs are "comb," "wasteway," "spillway," "overwash," "rollway," and "overfall."

The French term "nappe," suggesting the curved surface of a cloth hanging over the edge of a table, has been fittingly used to designate the overfalling sheet of water.

The expression "wetted underneath" has been used to describe the condition of the nappe designated by Bazin as "noyées en dessous," signifying that the water level between the nappe and the toe of the weir is raised by vacuum above the general water level below the weir.

"Thin-edged weir" and "sharp-crested weir" are used to designate a weir in which the nappe, or overfalling sheet, touches only the smooth, sharp upstream corner or edge of the crest, the thickness of which is probably immaterial so long as this condition is fulfilled.

A "suppressed weir" has a channel of approach whose width is the length of the weir crest.

A "contracted weir" has a crest length that is less than the width of the channel of approach.

The term "channel of approach," or "leading channel," defines the body of water immediately upstream from the weir, in which is located the gage by which the depth of overflow is measured.

"Section of approach" may refer to the cross section of the leading channel, if the depth and width of the leading channel are uniform; otherwise it will, in general, apply to the cross section of the channel of approach in which the gage is located.

"Weir section" refers to the cross section of the overflowing stream in the plane of the weir crest.

"Crest contraction" refers to the diminished cross section of the overflowing stream resulting from the upward curvature of the lower water filaments in passing the crest edge. It does not include the downward curvature of the water surface near the weir crest.

The "vertical contraction of the nappe" includes both the crest contraction and the surface contraction.

"Incomplete contraction" may take place either at the crest or at the ends of a weir, and will occur when the bottom or side walls of the channel of approach are so near the weir as to prevent the complete curvature of the water filaments as they pass the contracting edge.

Dimensions are uniformly expressed in feet and decimals, velocities in feet per second, and quantities of flow in cubic feet per second, unless otherwise stated in the text.

In the preparation of this paper much computation has been involved and it is expected that errors will appear, which, if attention is called to them, may be corrected in the future. Information concerning such errors will be gratefully received.

NOTATION.

The symbols given below are used in the values indicated. The meaning of additional symbols as used and special uses of those that follow are given in the text:

D=Measured or actual depth on the crest of weir, usually determined as the difference of elevation of the weir crest and the water level, taken at a point sufficiently far upstream from the weir to avoid the surface curve.

H=The head corrected for the effect of velocity of approach, or the observed head where there is no velocity of approach. As will be explained, D is applied in formulas like Bazin's, in which the correction for velocity of approach is included in the coefficient. H is applied in formulas where it is eliminated.

v=Mean velocity of approach in the leading channel, usually taken in a cross section opposite which D is determined.

h=Velocity head$=\alpha\frac{v^2}{2g}$.

g=Acceleration by gravity. Value here used 32.16.

P=Height of weir crest above bottom of channel of approach, where channel is rectangular.

W=Width of channel of approach where D is measured.

A=Area of cross section of channel of approach.

G=Area of channel section where D is measured, per unit length of crest.

a=Area of weir section of discharge$=D\,L$.

L=Actual length of weir crest for a suppressed weir, or length corrected for end contractions, if any.

L'=Actual length of crest of a weir with end contractions.

N=Number of complete end contractions.

B=Breadth of crest of a broad-crested weir.

S=Batter or slope of crest, feet horizontal to one vertical.

d=Depth of crest submergence in a drowned or submerged weir.
Q=Volume of discharge per unit of time.
C, M, m, μ, α, f, etc., empirical coefficients.

BASE FORMULAS.

The following formulas have been adopted by the engineers named:

$Q=\frac{2}{3}MLH\sqrt{2gH}$.	Hamilton Smith (theoretical).
$=\mu LH\sqrt{2gH}$.	Bazin, with no velocity of approach.
$=mLD\sqrt{2gD}$.	Bazin, with velocity of approach.
$=CLH^{\frac{3}{2}}$.	Francis[a] (used here).
$=CLH^{\frac{3}{2}}+fL$.	Fteley and Stearns.

EQUIVALENT COEFFICIENTS.

The relations between the several coefficients, so far as they can be given here, are as follows:

$$\mu=\frac{2}{3}M.$$

M is a direct measure of the relation of the actual to the theoretical weir discharge.

$$C=\mu\sqrt{2g}=\frac{2}{3}M\sqrt{2g}=8.02\ \mu=5.35\ M.$$

$$M=\frac{3}{2}\mu=\frac{3}{2}\frac{C}{\sqrt{2g}}=0.1870\ C.$$

$$\mu=\frac{C}{8.02}=0.1247\ C.$$

APPROXIMATE RELATIVE DISCHARGE OVER WEIRS.

For a thin-edged weir, the coefficient C in the Francis formula is $3.33=\frac{10}{3}$. Let C'' be the coefficient for any other weir, and x the relative discharge as compared with the thin-edged weir, then

$$\frac{10}{3}:C''::1:x$$

$$x=\frac{3C''}{10} \qquad (1)$$

or, as a percentage,

$$x_1=100\ x=30\ C''.$$

[a] The coefficient C of Francis includes all the constant or empirical factors appearing in the formula, which is thus thrown into the simplest form for computation.

This expression will be found convenient in comparing the effect on discharge of various modifications of the weir cross section. For a broad-crested weir with stable nappe, $C_1=2.64$, see p. 121. The discharge over such a weir is thus seen to be 79.2 per cent of that for a thin-edged weir by the Francis formula.

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THEORY OF WEIR MEASUREMENTS.

DEVELOPMENT OF THE WEIR.

The weir as applied to stream gaging is a special adaptation of mill dam, to which the term weir, meaning a hindrance or obstruction, has been applied from early times. The knowledge of a definite relation between the length and depth of overflow and the quantity also probably antedates considerably the scientific determination of the relation between these elements.

In theory a weir or notch[a] is closely related to the orifice; in fact, an orifice becomes a notch when the water level falls below its upper boundary.

THEOREM OF TORRICELLI.

The theorem of Torricelli, enunciated in his De Motu Gravium Naturaliter Accelerato, 1643, states that *the velocity of a fluid passing through an orifice in the side of a reservoir is the same as that which would be acquired by a heavy body falling freely through the vertical*

[a] Commonly applied to a deep, narrow weir.

height measured from the surface of the fluid in the reservoir to the center of the orifice.

This theorem forms the basis of hydrokinetics and renders the weir and orifice applicable to stream measurement. The truth of this proposition was confirmed by the experiments of Mariotte, published in 1685. It can also be demonstrated from the laws of dynamics and the principles of energy.[a]

ELEMENTARY DEDUCTION OF THE WEIR FORMULA.

In deducing a theoretical expression for flow over a weir it is assumed that each filament or horizontal lamina of the nappe is actuated by gravity acting through the head above it as if it were flowing through an independent orifice. In fig. 1 the head on the successive orifices being H_1, H_2, H_3, etc., and their respective areas A_1, A_2, A_3, etc., the total discharge would be

$$Q=C\sqrt{2g}\left[A_1 H_1^{\frac{1}{2}}+A_2 H_2^{\frac{1}{2}} \text{ to } +A_n H_n^{\frac{1}{2}}\right]. \quad . \quad . \quad . \quad (2)$$

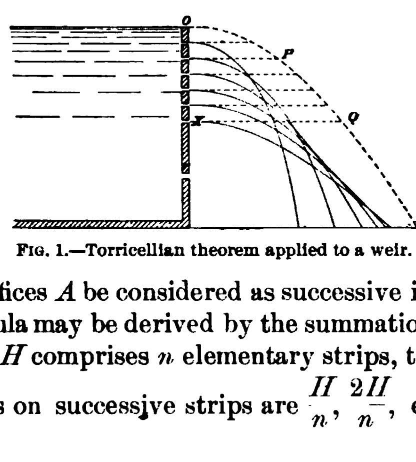

FIG. 1.—Torricellian theorem applied to a weir.

If the small orifices A be considered as successive increments of head H, the weir formula may be derived by the summation of the quantities in parentheses. H comprises n elementary strips, the breadth of each is $\frac{H}{n}$. The heads on successive strips are $\frac{H}{n}$, $\frac{2H}{n}$, etc., and the total becomes

$$Q=\frac{LH}{n}\sqrt{2g}\left(\sqrt{\frac{H}{n}}+\sqrt{\frac{2H}{n}}+\sqrt{\frac{3H}{n}}+\right) \quad . \quad . \quad . \quad . \quad (3)$$

where $\frac{LH}{n}=A+A_1$, etc., for a rectangular weir. The sum of the series $\sqrt{1}+\sqrt{2}+\sqrt{3}+$ to $\sqrt{n}=\frac{2}{3}n^{\frac{3}{2}}$.

Hence the discharge is

$$Q=\frac{LH}{n}\sqrt{2g}\sqrt{\frac{H}{n}}\cdot\frac{2}{3}n^{\frac{3}{2}}.$$

$$=\frac{2}{3}LH\sqrt{2gH}.$$

The above summation is more readily accomplished by calculus.

[a] See Wood, Elementary Mechanics, p. 167, also p. 291.

APPLICATION OF THE PARABOLIC LAW OF VELOCITY TO WEIRS.

The following elementary demonstration clearly illustrates the character of the weir:

According to Torricelli's theorem (see fig. 1), the velocity (v) of a filament at any depth (x) below surface will be $v=\sqrt{2gx}$. This is the equation of a parabola having its axis OX vertical and its origin O at water surface. Replacing the series of jets by a weir with crest at X, the mean velocity of all the filaments will be the average ordinate of the parabola OPQ. The average ordinate is the area divided by the height, but the area of a parabola is two-thirds that of the circumscribed rectangle; hence the mean velocity of flow through the weir is two-thirds the velocity at the crest, i. e., two-thirds the velocity due to the total head H on the crest. The discharge for unit length of crest is the head H, or area of opening per unit length, multiplied by the mean velocity. This quantity also represents the area of the parabolic velocity curve $OPQX$. The mean velocity of flow in the nappe occurs, theoretically, at two-thirds the depth on the crest.

The modification of the theoretical discharge by velocity of approach, the surface curve, the vertical contraction at the crest, and the various forms that the nappe may assume under different conditions of aeration, form of weir section, and head control the practical utility of the weir as a device for gaging streams.

GENERAL FORMULA FOR WEIRS AND ORIFICES.[a]

Consider first a rectangular opening in the side of a retaining vessel. The velocity of flow through an elementary layer whose area is Ldy will be from Torricelli's theorem:

$$v=\sqrt{2gy}.$$

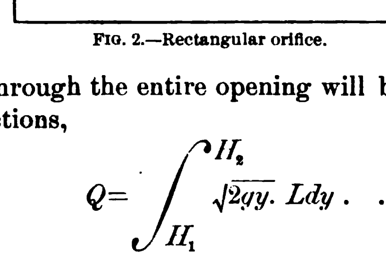

FIG. 2.—Rectangular orifice.

The discharge through the entire opening will be, per unit of time, neglecting contractions,

$$Q=\int_{H_1}^{H_2} \sqrt{2gy}.\ Ldy \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

[a] The correlation of the weir and orifice has been given by Merriman. See Hydraulics, pp. 42–43.

This is a general equation for the flow through any weir or orifice, rectangular or otherwise, Q being expressed as a function of y. In the present instance L is constant. Integrating,

$$Q=\frac{2}{3}\,L\,\sqrt{2g}\left(H_2^{\frac{3}{2}}-H_1^{\frac{3}{2}}\right) \quad . \quad . \quad . \quad . \quad . \quad . \qquad (5)$$

For a weir or notch, the upper edge will be at surface, $H_1=O$, and calling $H_2=H$ in equation (5),

$$Q=\frac{2}{3}L\,\sqrt{2g}\;H^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (6)$$

In the common formula for orifices, only the head on the center of gravity of the opening is considered.

Expressing H_2 and H_1 in terms of the depth H on the center of gravity of the opening and the height of opening d, Merriman obtains, after substituting these values in and expanding equation (5) by the binomial theorem, the equivalent formula,

$$Q=dL\sqrt{2gh}\left[1-\frac{1}{96}\,\frac{d^2}{H^2}-\frac{1}{2048}\,\frac{d^4}{H^4}-\frac{1}{21845}\,\frac{d^6}{H^6},\ \text{etc.}\right] \quad . \quad . \qquad (7)$$

The sum of the infinite series in brackets expresses the error of the ordinary formula for orifices as given by the remainder of the equation. This error varies from 1.1 per cent when $h=d$ to 0.1 per cent when $h=3d$.

VERTICAL CONTRACTION.

Practical weir formulas differ from the theoretical formula (6) in that velocity of approach must be considered and the discharge must be modified by a contraction coefficient to allow for diminished section of the nappe as it passes over the crest lip. Velocity of approach is considered on pages 14 to 20. Experiments to determine the weir coefficient occupy most of the remainder of the paper. The nature of the contraction coefficient is here described.

Vertical contraction expresses the relation of the thickness of nappe, s, in the plane of the weir crest, to the depth on the crest, H. If the ratio s/H were unity, the discharge would conform closely with the expression

$$Q=2/3\;LH\,\sqrt{2gH}.$$

The usual coefficient in the weir formula expresses nearly the ratio s/H.

The vertical contraction comprises two factors, the surface curve or depression of the surface of the nappe and the contraction of the under surface of the nappe at the crest edge. The latter factor in particular will vary with form of the weir cross section, and in general variation in the vertical contraction is the principal source of variation in the discharge coefficient for various forms of weirs.

The usual base weir formula, $Q=2/3\ LH\sqrt{2gH}$, is elsewhere given for an orifice in which the upper edge is a free surface. If instead the depth on the upper edge of the orifice is d, the surface contraction, there results the formula

$$Q=\frac{2}{3}ML\sqrt{2g}\left(H^{\frac{3}{2}}-d^{\frac{3}{2}}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

This is considered as the true weir formula by Merriman.[a] In this formula only the crest-lip contraction modifies the discharge, necessitating the introduction of the coefficient. The practical difficulties of measuring d prevent the use of this as a working formula.

Similarly a formula may be derived in which only the effective cross section s is considered, but even this will require some correction of the velocity. Such formulas are complicated by the variation of s and d with velocity of approach.[b] Hence, practical considerations included, it has commonly been preferred to adopt the convenient base formula for weirs, $Q=\frac{2}{3}\ MLH\sqrt{2gH}$, or an equivalent, and throw all the burden of corrections for contraction into the coefficient M.

VELOCITY OF APPROACH.

THEORETICAL FORMULAS.

Before considering the various practical weir formulas in use some general considerations regarding velocity of approach and its effect on the head and discharge may be presented.

In the general formula (4) for the efflux of water when the water approaches the orifice or notch with a velocity v, then with free discharge, writing $D+h$ in place of H, for a rectangular orifice, we have

$$Q=\int_{D_1+h}^{D_2+h}\sqrt{2gy}\ .\ Ldy \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

D_1 and D_2 being the measured depth on upper and lower edges of the orifice, and $h=\frac{v^2}{2g}$, the velocity head.

To assume that $D+h$ equals H is to assume that the water level is

[a] Hydraulics, p. 123.

[b] See Trautwine and Marichal's translation of Bazin's Experiments, pp. 231–307, where may also be found other data, including a résumé of M. Boussinesq's elaborate studies of the vertical contraction of the nappe, which appeared in Comptes Rendus de l'Académie des Sciences for October 24, 1887.

increased by the amount h, or, as is often stated, that H is "measured to the surface of still water." This is not strictly correct, however, because of friction and unequal velocities, which tend to make $H-D>h$, as explained below.

For a weir, D_1 equals zero; integrating,

$$Q=\frac{2}{3}L\sqrt{2g}\left[(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}\right]$$

Since $Q=\frac{2}{3}L\sqrt{2g}H^{\frac{3}{2}}$, we have

$$H=\left\{(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}\right\}^{\frac{2}{3}} \quad . \quad . \quad . \quad . \quad . \quad (9a)$$

This is the velocity correction formula used by James B. Francis.[a] Since h appears in both the superior and inferior limits of integration, it is evident that h increases the velocity only, and not the section of discharge. The criticism is sometimes made that Francis's equation has the form of an increase of the height of the section of discharge as well as the velocity.

The second general method of correcting for velocity of approach consists of adding directly to the measured head some function of the velocity head, making

$$H=D+\alpha h$$

in the formula

$$Q=CLH\sqrt{2gH}$$

or

$$Q=CL(D+\alpha h)\sqrt{2g(D+\alpha h)}. \quad . \quad . \quad . \quad . \quad . \quad 9b$$

This is the method employed by Boileau, Fteley and Stearns, and Bazin. No attempt is made to follow theory, but an empirical correction is applied, affecting both the velocity and area of section.

By either method v must be determined by successive approximations unless it has been directly measured.

Boileau and Bazin modify ($9b$) so as to include the area of section of channel of approach, and since the velocity of approach equals Q/A, a separate determination of v is unnecessary. Bazin also combines the factor for velocity of approach with the weir coefficient.

The various modifications of the velocity correction formulas are given in conjunction with the weir formulas of the several experimenters.

[a] Bovey gives similar proof of this formula for the additional cases of (1) an orifice with free discharge, (2) a submerged orifice, (3) a partially submerged orifice or drowned weir, thus establishing its generality.

DISTRIBUTION OF VELOCITY IN CHANNEL OF APPROACH.

The discharge over a weir takes place by virtue of the potential energy of the layer of water lying above the level of the weir crest, which is rendered kinetic by the act of falling over the weir. If the water approaches the weir with an initial velocity, it is evident that some part of the concurrent energy will facilitate the discharge.

The theoretical correction formulas may not truly represent the effect of velocity of approach for various reasons:

1. The fall in the leading channel adjacent to the measuring section is the source of the velocity of approach, and this fall will always be greater than that required to produce the existing velocities, because some fall will be utilized in overcoming friction.

2. The velocity is seldom uniform at all parts of the leading channel and the energy of the water varies accordingly. This effect is discussed later (p. 17).

3. It is not certain just what portion of the energy of the water in the section of the leading channel goes to increase the discharge.

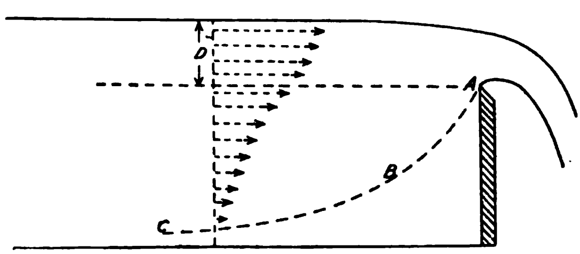

FIG. 3.—Distribution of velocities.

In general the threads of the water in the cross section of the channel of approach to a weir have varying velocities. It follows that, as will be shown, the ratio of the actual energy of the approaching water to the energy due to the mean velocity will be greater than unity, and for this reason the correction for velocity of approach will be greater than if the energy were that due to a fall through a head produced by the mean velocity v. The more nearly uniform is the velocity of the water in the leading channel the smaller will be the necessary coefficient α in the velocity head formula. The velocity may be rendered very nearly uniform by the use of stilling racks or baffles. Where this was done in the experiments on which a formula was based (that of Francis, for example) a larger velocity of approach correction than that obtained by the author may be necessary in applying the formula to cases where there is wide variation in the velocity in the leading channel. To avoid such a contingency it is desirable, when practicable, to measure head to surface of still water, because more accurate results can be obtained and wash against instruments prevented.

The vertical and horizontal velocity curves in an open channel usually closely resemble parabolas. A weir interposes an obstruction in the lower part of the channel, checking the bottom velocities. The velocity is not, however, confined to the filaments in line with the section of the discharge opening of the weir. As a result of viscosity of the liquid, the upper rapidly moving layers drag the filaments underneath, and the velocity may extend nearly or quite to the channel bottom. There will usually, however, be a line (A B C, fig. 3), rising as the weir is approached, below which there is no forward velocity.

The line A B C is the envelope of the curves of vertical velocity in the channel of approach.

There will be a similar area of low velocity at each side of the channel for a contracted weir. The inequality of velocities for such weirs being usually greater than for suppressed weirs, it follows that a larger coefficient in the formula for velocity of approach may be required. This is confirmed by experiment.

Various assumptions have been made as to what portion of the energy of the approaching stream goes to increase the discharge, (*a*) that resulting from the mean velocity deduced from the discharge divided by the area of the entire section of the channel of approach; (*b*) that of the mean velocity obtained by using the sectional area of the moving water, above the line A B C, fig. 3; (*c*) that of the filaments lying in line with or nearest to the section of the weir opening, determined approximately by the surface velocity.[a]

DISTRIBUTION OF ENERGY IN CHANNEL OF APPROACH.

Consider unit width of the channel of approach:

Let v_s = Surface velocity.
v_m = Mean velocity.
v_b = Bottom velocity.
v = Velocity at a height x above bottom.
X = Depth of water in channel of approach.
w = Weight of unit volume.

The general formula for kinetic energy is

$$\text{K. E.}=\frac{Wv^2}{2g} \qquad (10)$$

where W= weight of the moving mass.

If the velocity increases uniformly from bottom to surface, the velocity at height x will be

$$v=v_b+\frac{x}{X}(v_s-v_b).$$

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[a] Smith, Hamilton, Hydraulics, p. 68.

Let dx be the thickness of a lamina one unit wide at height x. The total kinetic energy for the depth X will be

$$\text{K. E.} = \int_0^X \left(v_b + \frac{x}{X}(v_s - v_b)\right)^3 \frac{w}{2g}\, dx \quad . \quad . \quad . \quad . \quad (11)$$

If the velocity is uniform, the total kinetic energy per unit width is found by integration to be

$$\text{K. E.} = \frac{wXv_m^3}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Integrating for the simple case where $v_b = 0$ and the velocity increases uniformly from the bottom to the surface so that $v_m = \frac{v_s}{2}$, we have

$$\text{K. E.} = \frac{wXv_m^3}{g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

Comparing this with the expression for kinetic energy of a stream flowing with the uniform velocity v (formula 12), we find the mass energy of the stream with uniformly varying velocity to be twice as great as for the uniform velocity.

By a similar integration the ratio of the total kinetic energy to the kinetic energy corresponding to the mean velocity in the channel of approach can be obtained for any assumption as to the distribution of velocities in the leading channel. The resulting ratio will depend upon the relative areas of section with low and high velocities which go to make up the mean, and in practice it will generally exceed unity.

The lowering of the water surface from the level of a still pond will also be greater in the case of unequal velocities than in the case of a uniform velocity equal to their mean. The theoretical weir formula indicates the same discharge in case of a uniform velocity of approach v as in case of varying velocities whose mean is equal to v, although in the former case the actual drawing down of the head if it were measured would be found greater. If h were the velocity head corresponding to the mean velocity, and if v_1, v_2, v_3, etc., v_n were the actual velocities in the n unit areas of cross section, the actual velocity head h' will be such that

$$\frac{Qw}{2g}(v_1^2 + v_2^2 + \text{etc. } v_n^2) = wQh' = \text{Integral K. E.}$$

Now,

$$\frac{wQ}{2g} v^2 = wQh = \text{K. E. of average velocity.}$$

As shown above, the integral K. E. is the greater.

It follows that $h'>h$.

If $$\alpha=\frac{h'}{h}$$

Then $$h'=\alpha h.$$

Introducing velocity of approach in the discharge formula we substitute $D+h$ for H, and integrate between the limits zero and D. Hence, for the same discharge, the area of weir section is greater without velocity of approach by nearly the amount hL.

For a given measured head D, the effect of velocity of approach, whatever it may be, appears as an increase in the mean velocity of discharge in the plane of the weir. The relation of the mean velocity of discharge for a weir with velocity of approach to that for a weir without such velocity is shown by the following expression, the mean head being the same in both cases:

$$\text{Mean velocity in the plane of the weir}=\frac{Q}{D},$$

then $$\frac{Q}{D}:\frac{Q_1}{D}::D^{\frac{3}{2}}:(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}.$$

It will be seen that the discharge over a weir with velocity of approach is less than that for the same total head and greater than that for the same measured head without velocity of approach, and that with a given measured head the greater the velocity of approach the greater will be the discharge.

In a weir section opening out of still water there is always a considerable surface velocity, the parabolic law (see fig. 3) being modified by fluid friction, which tends to equalize the velocities. Velocity of approach, being usually greater at the surface, furthers this equalization. Some of the kinetic energy of the swifter-moving filaments is transferred to their slower-moving neighbors, the result being that while the kinetic energy of the whole mass Q passing the weir per second remains constant, yet the *average* velocity is accelerated and the discharge rate is increased as compared with the theoretical quantities. This will be clearer if we consider two contiguous filaments, each having unit section a, one with a velocity of 1, the other of 2 feet per second. The two will discharge 2+1 units flow per second, having the total kinetic energy indicated below:

$$\text{K. E.}=\frac{1\times1^2}{2g}\,aw+\frac{2\times2^2}{2g}\,aw=9\frac{aw}{2g}.$$

If, now, the velocities are equalized, 9 units of kinetic energy will be equally divided between the two filaments, so that, the new velocity being v,

$$\frac{2awv \times v^2}{2g} = \frac{9aw}{2g}.$$

and

$$v = \sqrt[3]{\frac{9}{2}} = 1.651.$$

The average velocity before equalization was 1.5.

The discharge from two filaments having equal velocities will be 3.302 units, as compared with 3.00 for two filaments having unequal velocities.

THE THIN-EDGED WEIR.

EARLIER EXPERIMENTS AND FORMULAS.

Prior to 1850 the practice of weir measurement was in a somewhat chaotic condition, especially in England, Germany, and the United States. There were many experimental results, but the experiments were made on so small a scale that the various influences affecting the measurements and the lack of proper standards made the results erratic and untrustworthy in detail. Greater advancement had been made in France by such savants as Dubuat, Eytelwein, D'Aubuisson, Castel, Poncelet, Lesbros, and Boileau. Some of the work of the early French experimenters has proved, in the light of wider experience, to be of considerable value.

EXPERIMENTS OF CASTEL.

The first experiments deserving consideration are of those of M. Castel, conducted at the waterworks of Toulouse in 1835 and 1836.[a] Castel erected his apparatus on a terrace in conjunction with the water tower, which received a continuous supply of 1.32 cubic feet per second, capable of being increased to 1.77 cubic feet per second. The weir consisted of a wooden dam, surmounted by a crest of copper 0.001 foot in thickness, situated in the lower end of a leading channel, 19.5 feet long, 2.428 feet wide, and 1.772 feet deep. Screens were placed across the upper end of the channel to reduce oscillations. The head was measured at a point 1.60 feet upstream from the weir by means of a point gage. The overflow was measured in a zinc-lined tank having a capacity of 113.024 cubic feet. The length of the crest for weirs with suppressed contractions varied from 2.393 to 2.438 feet. Heights of weirs varying from 0.105 to 0.7382 were used, and

[a] Originally published in Mémoires Acad. Sci. Toulouse, 1837. See D'Aubuisson's Hydraulics, Bennett's translation, pp. 74–77. Data recomputed by Hamilton Smith in his Hydraulics, pp. 80–82 and 138–145. The recomputed coefficients will be found valuable in calculating discharge for very small and very low weirs.

a similar series of experiments was performed on suppressed weirs 1.1844 feet long. The head varied for the longer weirs from about 0.1 to 0.25 foot. Additional experiments were made on contracted weirs having various lengths, from 0.0328 to 1.6483 feet, in a channel 2.428 feet wide, and for lengths from 0.0328 to 0.6542 foot in a channel 1.148 feet wide. The experiments on these narrow slit weirs included depths varying from 0.1 or 0.2 foot to a maximum of about 0.8 foot.

D'Aubuisson gives the following formula, derived from the experiments of Castel for a suppressed weir:

$$Q=3.4872LD\sqrt{D+0.035W^2} \quad . \quad . \quad . \quad . \quad . \quad (14)$$

where W is the measured central surface velocity of approach, ordinarily about $1.2v$.

EXPERIMENTS OF PONCELET AND LESBROS.

The experiments made by Poncelet and Lesbros, at Metz, in 1827 and 1828, under the auspices of the French Government, were continued by Lesbros in 1836. The final results were not published, however, until some years later.[a]

The experiments of Poncelet and Lesbros and of Lesbros were performed chiefly on a weir in a fixed copper plate, length 5.562 feet. The head was measured in all cases in a reservoir 11.48 feet upstream, beyond the influence of velocity of approach. The crest depth varied from about 0.05 to 0.60 or 0.80 foot. The experiments of Lesbros are notable from the fact that a large number of forms of channel of approach were employed, including those with contracted and convergent sides, elevated bottoms, etc. The experiments of Lesbros on these special forms of weirs have been carefully recomputed by Hamilton Smith, and may be useful in determining the discharge through weirs having similar modifications.[b]

EXPERIMENTS OF BOILEAU.

The experiments of Boileau[c] at Metz, in 1846, included 3 suppressed weirs, having lengths and heights as follows:

(1) Length 5.30 feet, height 1.54 feet.
(2) Length 2.94 feet, height 1.12 feet.
(3) Length 2.94 feet, height 1.60 feet.

The depth of overflow varied from 0.19 to 0.72 foot. Boileau obtained the following formula for a suppressed weir:

$$Q=3.3455\frac{P+D}{\sqrt{(P+D)^2-D^2}}LD^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad (15)$$

[a] Expériences hydrauliques sur les lois de l'écoulement de l'eau, Paris, 1852.
[b] Smith, Hamilton, Hydraulics, pp. 96 and 97 and 104–107. Also plates 1–2 and 8.
[c] Gaugeage de cours d'eau, etc., Paris, 1850.

This formula includes the correction for velocity of approach. The coefficient C, it will be noticed, is given as a constant. Boileau afterwards gave a table of corrections varying with the depth, indicating a discharge from 96 to 107 per cent of that obtained with the constant coefficient. Additional experiments by Boileau on suppressed weirs having a crest length of about 0.95 foot have been recomputed by Hamilton Smith.[a] The heights of weirs were, respectively, 2.028, 2.690, 2.018, and 2.638 feet. In these experiments the discharge was determined by measurement through orifices.

EAST INDIAN ENGINEERS' FORMULA.[b]

The East Indian engineers' formula for thin-edged weirs is

$$Q=\frac{2}{3}ML\sqrt{2gH^3}=CLH^{\frac{3}{2}}$$

where

$$\left.\begin{aligned} C&=\frac{2}{3}\sqrt{2g}\,M=5.35\,M \\ M&=1-\left(\frac{0.04\,[34.6+H]}{4}\right). \end{aligned}\right\} \quad \ldots\ldots \quad (16)$$

Reducing,

$$\left.\begin{aligned} M&=0.654\;-0.01\,H \\ C&=3.4989-0.0535\,H \end{aligned}\right\} \quad \ldots\ldots \quad (17)$$

This formula applies to a suppressed weir. Method of correction for velocity of approach is not stated. Coefficient M has a maximum value 0.654, and decreases slowly as the head increases. Limits of applicability of formula are not stated. Values of C are given below:

Coefficient C for thin-edged weirs, East Indian engineers' formula.[c]

H in feet.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	3.499	3.494	3.488	3.483	3.478	3.472	3.467	3.462	3.456	3.451
1	3.445	3.440	3.435	3.429	3.424	3.419	3.413	3.408	3.403	3.397
2	3.392	3.386	3.381	3.376	3.370	3.365	3.360	3.354	3.349	3.344
3	3.338	3.333	3.328	3.322	3.317	3.312	3.306	3.301	3.296	3.290
4	3.285	3.280	3.274	3.269	3.264	3.258	3.253	3.248	3.242	3.237
5	3.221	3.226	3.221	3.215	3.210	3.205	3.199	3.194	3.189	3.183
6	3.178	3.172	3.167	3.162	3.156	3.151	3.146	3.140	3.135	3.130
7	3.124	3.119	3.114	3.108	3.103	3.098	3.092	3.087	3.082	3.076
8	3.071	3.066	3.060	3.055	3.050	3.044	3.039	3.034	3.028	3.023
9	3.017	3.012	3.007	3.001	2.996	2.991	2.985	2.980	2.975	2.969

[a] Hydraulics, pp. 133-135.

[b] Given in J. Mullins's Irrigation Manual, introduced in United States by G. W. Rafter and used in region of upper Hudson River. Not given in Bellasis's recent East Indian work on hydraulics.

[c] For East Indian engineers' broad-crested weir formula, using coefficients derived from the above, see p. 114.

EXPERIMENTS AND FORMULA OF JAMES B. FRANCIS.

The experiments on discharge over thin-edged weirs,[a] upon which the Francis formula is based, were made in October and November, 1852, at the lower locks of the Pawtucket canal, leading from Concord River past the Lowell dam to slack water of Merrimac River. Additional experiments were made by Francis in 1848[b] at the center vent water wheel at the Boott Cotton Mills in Lowell, with gates blocked open and with constant head. A uniform but unknown volume of water was thus passed through the turbine and over a weir having various numbers of end contractions, the effect of which was thus determined. Similar experiments were made in 1851 at the Tremont turbine,[c] where a constant volume of water was passed over weirs of lengths ranging from 3.5 to 16.98 feet and with from two to eight end contractions. These experiments were made to determine the exponent n in the weir formula

$$Q = CLH^n.$$

Francis here found $n=1.47$, but adopted the value $n=1.5=3/2$, in the experiments of 1852.

The Pawtucket canal lock was not in use at the time of the Lowell experiments in 1852 and the miter gates at the upper lock chamber were removed and the weir was erected in the lower hollow quoin of the gate chamber. The middle gates at the foot of the upper chamber were replaced by a bulkhead having a sluice for drawing off the water. A timber flume in the lower chamber of the lock was used as a measuring basin to determine the flow over the weir. Its length was 102 feet and its width about 11.6 feet. A swinging apron gate was so arranged over the crest of the weir that, when opened, the water flowed freely into the measuring basin below, and when closed, with its upper edge against the weir, the overflow passed into a wooden diverting channel, placed across the top of the lock chamber, and flowed into Concord River. An electric sounder was attached to the gate framework, by which a signal was given when the edge of the swinging gate was at the center of the nappe, when either opening or closing. By this means the time of starting and stopping of each experimental period was observed on a marine chronometer. The depth on the weir was observed by hook gages. The readings were taken in wooden stilling boxes, 11 by 18 inches square, open at the top, and having a 1-inch round hole through the bottom, which was about 4 inches below the weir crest. The weir was in the lower quoin of the gate recess, and the hook gage boxes were in the upper quoin, projecting slightly beyond the main lock walls. In weirs with end

[a] Francis, J. B., Lowell Hydraulic Experiments, pp. 103-135. [b] Idem, pp. 96-102. [c] Idem, pp. 76-95.

contractions the full width of the channel was used. For suppressed weirs, a leading channel having a width equal to the length of the weir crest was formed by constructing vertical timber walls within the main canal, extending 20 feet upstream from the weir and having their upper ends flaring about 1 foot toward the canal walls. Water was freely admitted on both sides of these timber walls. The hook gage boxes were outside of this channel. The holes in the bottom were plugged, and flush piezometer pipes were used to connect the hook-gage boxes with the inner face of the side walls of the channel of approach. Observations of the head by hook gage were taken at intervals of about 15 seconds. Each experimental period covered from 190 to 900 seconds. The hook-gage readings were reduced to weir crest level as a datum and arranged in groups of two or three, which agreed closely. The mean head was determined by the correction formula (48). In one period, 18 observations of heads ranged from 0.6310 to 0.6605 foot; their arithmetical mean was 0.6428; the computed correction was minus 0.0008.

The measured head was corrected for velocity of approach by using the theoretical formula given below. The range and character of the experiments, together with the general results, are shown in the following table:

Thin-edged weir experiments of J. B. Francis at the lower locks, Lowell, Mass., 1852.

Serial numbers of experiments.		Total number.	Width of channel at upstream side of weir.	Depth of channel below top of weir at hook gage boxes.	Range of observed head, in feet.		Range of velocity of approach, in feet per second.		Length of weir crest in feet.	Number end contractions.	Approximate mean corrected head H, in feet.	Discharge coefficient C.		
From—	To—				From—	To—	From—	To—				Maximum.	Minimum.	Mean.
1	4	4	13.96	5.048	1.52430	1.56910	0.7682	0.7889	9.997	2	1.56	3.3318	3.3002	3.3181
5	10	6	13.96	5.048	1.23690	1.25490	.5904	.6000	9.997	2	1.25	3.3412	3.3159	3.3338
11	33	23	13.96	5.048	.91570	1.06920	.3951	.4863	9.997	2	1.00	3.3333	3.3110	3.3223
34	35	2	13.96	5.048	1.01025	1.02625	.3527	.3596	7.997	4	1.02	3.3617	3.3586	3.3601
36	43	8	13.96	2.014	1.02805	1.07945	.9496	1.0049	9.997	2	1.06	3.3567	3.3498	3.3527
44	50	7	9.992	5.048	.97450	.98675	.5376	.5455	9.995	0	0.98	3.3437	3.3366	3.3409
51	55	5	9.992	5.048	.99240	1.00600	.5477	.5589	9.995	0	1.00	3.3349	3.3243	3.3270
56	61	6	13.96	5.048	.77690	.81860	.3170	.3405	9.997	2	0.80	3.3287	3.3188	3.3246
62	66	5	13.96	2.014	.77115	.88865	.6694	.7963	9.997	2	0.83	3.3435	3.3376	3.3403
67	71	5	9.992	5.048	.7362	.81495	.3659	.4213	9.995	0	0.80	3.3424	3.3341	3.3393
72	78	7	13.96	5.048	.59190	.65525	.2182	.2509	9.997	2	0.62	3.3306	3.3237	3.3275
79	84	6	13.96	2.014	.63135	.65385	.5193	.5496	9.997	2	0.65	3.3278	3.3244	3.3262
85	88	4	13.96	2.014	.66940	.68815	.4382	.4526	7.997	4	0.68	3.3382	3.3333	3.3368

From a discussion of these experiments Francis presents the final formula—

$$\left.\begin{array}{l} Q=3.33LH^{\frac{3}{2}}. \\ \text{If there are end contractions,} \\ L=L'-0.1NH. \\ \text{If there is velocity of approach,} \\ H^{\frac{3}{2}}=\left[(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}\right]. \end{array}\right\} \quad . \quad . \quad (18)$$

The mean velocity v was determined by successive approximations; h was determined by the usual formula—

$$h=\frac{v^2}{2g}$$

The Francis formula for velocity of approach correction is cumbersome, and several substitutes have been devised, some of which are described in the following paragraphs.

(1) Determine the approximate velocity of approach v_1 by a single trial computation of Q, using $D=H$. Then use

$$H=D+\frac{v_1}{2g}=D+h$$

to determine the final value of Q. For a given value of v this gives too large a value of H, but the approximate value of v_1 is somewhat too small, partially counterbalancing the error and usually giving a final value of Q sufficiently precise.

(2) By developing into series and omitting the powers h/D above the first, h being always relatively small, the following closely approximate equivalent of the Francis correction formula, given by Emerson,[a] is obtained:

$$H=D+h-\frac{2}{3}\sqrt{\frac{h^3}{D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

(3) Hunking and Hart[b] derive from the Francis correction formula the following equivalent expression:

$$KD^{\frac{3}{2}}=H^{\frac{3}{2}}=(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

$$K=\left[1+\frac{C^2}{2g}\left(\frac{D}{G}\right)^2K^2\right]^{\frac{3}{2}}-\left[\frac{C^2}{2g}\left(\frac{D}{G}\right)^2K^2\right]^{\frac{3}{2}} \quad . \quad . \quad (21)$$

where G is the area of channel section in which D is measured, per unit length of crest.

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[a] Hydrodynamics, p. 286.

[b] Jour. Franklin Inst., Phila., August, 1884, pp. 121-126.

For a suppressed weir,

$$G=P+D.$$

For a contracted weir,

$$G=\frac{A}{L'-0.1ND}. \qquad \ldots\ldots (22)$$

Hunking and Hart have computed values of K by the solution of the above formula for each 0.005 increment in D/G to 0.36. The results extended by formula (23) are given below.

Velocity of approach correction, factor K, Hunking and Hart formula, $H^{\frac{3}{2}}=KD^{\frac{3}{2}}$.

D/G	0.0	0.1	0.2	0.3	0.4	0.5	0.6
.000	1.00000	1.002528	1.009980	1.022359	1.039840	1.062250	1.08964
.005	1.000006	1.002785	1.010480	1.023110	1.040836	1.063495	1.091134
.010	1.000026	1.003053	1.010994	1.023875	1.041832	1.064740	1.092628
.015	1.000058	1.003335	1.011519	1.024653	1.042828	1.065985	1.094122
.020	1.000103	1.003628	1.012057	1.025444	1.043824	1.067230	1.095616
.025	1.000161	1.003933	1.012607	1.026248	1.045069	1.068724	1.097359
.030	1.000231	1.004251	1.013169	1.027065	1.046065	1.069969	1.098853
.035	1.000314	1.004581	1.013744	1.027895	1.047061	1.071214	1.100347
.040	1.000409	1.004923	1.014331	1.028739	1.048306	1.072708	1.102090
.045	1.000518	1.005278	1.014931	1.029596	1.049302	1.073953	1.103584
.050	1.000638	1.005644	1.015543	1.030467	1.050298	1.075198	1.105078
.055	1.000772	1.006023	1.016167	1.031350	1.051543	1.076692	1.106821
.060	1.000917	1.006414	1.016805	1.032248	1.052788	1.078186	1.108564
.065	1.001075	1.006817	1.017455	1.033117	1.053784	1.079431	1.110058
.070	1.001246	1.007232	1.018107	1.034113	1.055029	1.080925	1.111801
.075	1.001429	1.007659	1.018792	1.035109	1.056274	1.082419	1.113544
.080	1.001624	1.008099	1.019480	1.035856	1.057270	1.083664	1.115038
.085	1.001832	1.008551	1.020180	1.036852	1.058515	1.085158	1.116781
.090	1.002051	1.009015	1.020893	1.037848	1.059760	1.086652	1.118524
.095	1.002284	1.009491	1.021620	1.038844	1.061005	1.088146	1.120267

The general formula for K is too complex for common use. The expressions

$$K=1+0.2489\left(\frac{D}{G}\right)^2 \qquad \ldots\ldots (23)$$

and

$$K=1+\left(\frac{D}{2G}\right)^2 \qquad \ldots\ldots (24)$$

are stated to give results correct within one-hundredth and one-fiftieth of 1 per cent, respectively, for values of K less than 0.36.

EXPERIMENTS AND FORMULAS OF FTELEY AND STEARNS.

The first series of experiments by Fteley and Stearns on thin-edged weir discharge[a] were made in March and April, 1877, on a suppressed weir, with crest 5 feet in length, erected in Sudbury conduit below Farm Pond, Metropolitan waterworks of Boston.

Water from Farm Pond was let into the leading channel through

[a] Fteley, A., and Stearns, F. P., Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, Jan., Feb., Mar., 1883, pp. 1-118.

head-gates until the desired level for the experiment, as found by previous trial, was reached. A swinging gate was then raised from the crest of the weir and the water was allowed to flow over. The maintenance of a uniform regimen was facilitated by the large area and the consequent small variation of level in Farm Pond, so that the outflow from the gates was sensibly proportional to the height they were raised. The water flowed from the weir into the conduit channel below, and was measured volumetrically. For the smaller heads the length of the measuring basin was 22 feet, and for the larger heads 367 feet.

The crest depth was observed by hook gage in a pail below the weir, connected to the channel of approach by a rubber tube entering the top of the side wall, 6 feet upstream from the weir crest. Hook-gage readings of head were taken every half minute until uniform regimen was established, and every minute thereafter. The depths in the measuring basin were also taken by hook gage. The bottom of the conduit was concave, and was graded to a slope of 1 foot per mile. It was covered with water previous to each experiment, leaving a nearly rectangular section.

The experiments in 1877 included 31 depths on a suppressed weir of 5 feet crest length, 3.17 feet high. The observed heads varied from 0.0735 to 0.8198 foot.

In 1879 a suppressed weir, with a crest length of 19 feet, was erected in Farm Pond Gate House. Head-gates and screens were close to weir; otherwise the apparatus for measuring head and starting and stopping flow was similar to that used in previous experiments. The crest of the weir was an iron bar 3½ inches wide and one-fourth inch thick, planed and filed and attached to the upper weir timber with screws. No variation in level of the weir crest occurred. As in the preceding experiments, no by-pass was provided, and the entire overflow entered Sudbury conduit below the weir. The conduit was partly filled with water at the start, leaving a nearly rectangular section, 11,300 feet in length and about 9 feet wide. A difference of 3 feet in water level was utilized in measuring discharge, the total capacity being 300,272 cubic feet. Semipartitions were provided to reduce oscillation of the water. Many observations, covering a considerable period of time, were required to determine the true water level. This series of experiments included 10 depths on a suppressed weir 19 feet long and 6.55 feet high, with measured heads varying from 0.4685 to 1.6038 feet and velocities of approach ranging from 0.151 to 0.840 foot per second.

From measurements on weirs 5 and 19 feet in length, respectively, and from a recalculation of the experiments of James B. Francis, Fteley and Stearns obtained the final formula

$$Q=3.31LH^{\frac{3}{2}}+0.007L \quad . \quad . \quad . \quad . \quad . \quad . \quad (25)$$

In the above, if there is velocity of approach,

$H=D+\alpha h$.

$\alpha=1.5$ for suppressed weirs.

$\alpha=2.05$ for weirs with end contractions.

The value of the velocity head coefficient α was determined from 94 additional experiments on the 5-foot weir in 1878. These involved measured heads ranging from 0.1884 to 0.9443 foot, heights of weir ranging from 0.50 to 3.47 feet, and velocities of approach reaching a maximum of 2.35 feet per second. Also 17 experiments were made on weirs 3, 3.3, and 4 feet long respectively; the first with two and the last two with one end contraction. These experiments included measured heads varying from 0.5574 to 0.8702 foot, and velocities of approach from 0.23 to 1.239 feet per second.

In all experiments on velocity of approach, the head was measured 6 feet upstream from crest. The width of channel was 5 feet.[a]

Fteley and Stearns found the following values of α for suppressed weirs:

Fteley and Stearns's value of α for suppressed weirs.

Measured depth on weir, in feet.	Depth of channel of approach below weir crest, in feet.			
	0.50	1.00	1.70	2.60 [b]
0.2	1.70	1.87	1.66	1.51
.3	1.53	1.83	1.65	1.50
.4	1.53	1.79	1.63	1.49
.5	1.53	1.75	1.62	1.48
.6	1.52	1.71	1.60	1.47
.7	1.51	1.68	1.59	1.46
.8	[c] 1.50	[c] 1.65	1.57	1.45
.9	1.49	1.63	1.56	[c] 1.44
1.0	1.48	1.61	1.54	1.43
1.1	....	1.59	1.53	1.42
1.2	....	1.57	1.51	1.41
1.3	....	1.55	1.49	1.40
1.4	....	1.54	1.48	1.39
1.5	....	1.52	1.46	1.38
1.6	....	1.51	1.44	1.37
1.7	....	1.49	1.43	1.36
1.8	....	....	1.41	1.35
1.9	....	....	1.40	1.34
2.0	....	....	1.38	1.33

[a] Fteley and Stearns, idem, pp. 5-23.
[b] Applicable to greater heights of weir.
[c] Limit of experiments.

Current-meter measurements showed a nearly uniform distribution of velocities in the channel of approach above the 19-foot weir, a fact to be taken account of when the formulas are applied to cases where the velocity of approach varies in different portions of the leading channel.

If there are end contractions, the net length of weir should be determined by the Francis formula,

$$L = L' - 0.1NH.$$

The head should be measured at the surface of the channel of approach, 6 feet upstream from the weir crest.

BAZIN'S EXPERIMENTS.

Bazin's experiments on thin-edged weirs were performed in the side channel of the Canal de Bourgogne, near Dijon, France, and were begun in 1886. Their results were published in Annales des Ponts et Chaussées and have been translated by Marichal and Trautwine.[a]

The standard weir consisted of horizontal timbers 4 inches square, with an iron crest plate 0.276 inch in thickness. Air chambers were placed at the ends of the weir on the downstream side, to insure full aeration of the nappe. End contractions were suppressed. The height of the first weir was 3.27 feet above channel bottom, and the head was measured in "Bazin pits," one at each side of the channel 16.40 feet upstream from the weir crest. The pit consisted of a lateral chamber in the cement masonry forming the walls of the canal. The chamber was square, 1.64 feet on each side, and communicated with the channel of approach by a circular opening 4 inches in diameter, placed at the bottom of the side wall and having its mouth exactly flush with the face of the wall. The oscillations of the water surface in the lateral chamber were thus rendered much less prominent than in the channel of approach. The water level in the Bazin pit was observed by dial indicators attached to floats, the index magnifying the variations in water level four times, the datum for the indicators having been previously determined by means of hook gages placed above the crest of the weir and by needle-pointed slide gages in the leading channel.

A drop gate was constructed on the crest of the weir to shut off the discharge at will. In each experiment the head-gates through which the water entered the leading channel were first raised and the water was allowed to assume the desired level. The weir gate was then raised, and the head-gates were manipulated to maintain a nearly con-

[a] Bazin, H., Recent experiments on flow of water over weirs, translated from the French by Marichal and Trautwine: Proc. Engineers' Club Phila., vol. 7, Jan., 1890, pp. 259–310; vol. 9, pp. 231–244, 287–319; vol. 10, pp. 121–164.

stant inflow. The arithmetical mean of the observations during each period of uniform regimen was used as the measured head for that experiment.

The overflow passed into a measuring channel, 656.17 feet in length, whose walls were made of smooth Portland cement concrete. The channel was 6.56 feet wide, its side walls were 3.937 feet high, and its lower end was closed by water-tight masonry. Its bottom was graded to a slope of about 1:1,000. The volume of inflow was determined by first covering the channel bottom with water, then noting the change of level during each experimental period, the capacity of the channel at various heights having previously been carefully determined. A slight filtration occurred, necessitating a correction of about one-eighth of 1 per cent of the total volume. The observations for each regimen were continued through a period of 12 to 30 minutes.

Sixty-seven experiments were made on a weir 3.72 feet high, including heads from the least up to 1.017 feet. Above this point the volumetric measuring channel filled so quickly as to require the use of a shorter weir. Thirty-eight experiments were made with a standard weir, 3.28 feet long and 3.72 feet high, with heads varying from the least up to 1.34 feet. For heads exceeding 1.34 feet it was necessary to reduce the height of the weir in order that the depth above the weir should not exceed that of the channel of approach. Forty-eight experiments were made on a weir 1.64 feet long and 3.297 feet high, with heads ranging from the least up to 1.780 feet. These experiments sufficed to calibrate the standard weir with a degree of accuracy stated by Bazin as less than 1 per cent of error.

In order to determine the effect of varying velocities of approach the following additional series of experiments were made on suppressed weirs 2 meters (6.56 feet) in length.

Experiments on suppressed weirs 2 meters in length.

Number of experiments.	Range of head in feet.		Height of experimental weir, in feet.
	From—	To—	
28+30	0.489	1.443	2.46
29+29	.314	1.407	1.64
27+41	.298	1.338	1.15
44	.296	1.338	0.79

The standard weir was 3.72 feet high, and the experimental weirs were placed 46 to 199 meters downstream. The discharge was not measured volumetrically. A uniform regimen of flow was established and the depths on the two weirs were simultaneously observed during each period of flow.

These experiments afforded data for the determination of the relative effect of different velocities of approach, corresponding to the different depths of the leading channel.

From these experiments Bazin deduces coefficients for a thin-edged weir 3.72 feet high, for heads up to 1.97 feet, stated to give the true discharge within 1 per cent.[a]

BAZIN'S FORMULAS FOR THIN-EDGED WEIRS.

Starting with the theoretical formula for a weir without velocity of approach, in the form

$$Q=\mu LH\sqrt{2gH}$$

and substituting

$$D+\alpha\frac{v^2}{2g}$$

for H, in the case of a weir having velocity of approach, there results,

$$Q=\mu L\left(D+\alpha\frac{v^2}{2g}\right)\sqrt{2g\left(D+\alpha\frac{v^2}{2g}\right)}.$$

Bazin obtained, by mathematical transformation, the equivalent[b]

$$Q=\mu LD\sqrt{2gD}\left(1+\alpha\frac{v^2}{2gD}\right)^{\frac{3}{2}},$$

or

$$Q=\mu\left(1+\alpha\frac{v^2}{2gD}\right)^{\frac{3}{2}}LD\sqrt{2gD}.$$

Bazin writes

$$m=\mu\left(1+\alpha\frac{v^2}{2gD}\right)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

for which equation he obtains, by mathematical transformation, the approximate equivalent[c]

$$m=\mu\left(1+\frac{3}{2}\alpha\frac{v^2}{2gD}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

The calculation of the factor v appearing in this formula requires the discharge Q to be known.

Assuming that the channel of approach has a constant depth P below the crest of the weir, and that its width is equal to the length of the

[a] Bazin, H., Expériences nouvelles sur l'écoulement en déversoir: Ann. Ponts et Chaussées, Mém. et Doc., 1898, 2me trimestre. See translation by Marichal and Trautwine in Proc. Eng. Club Phila., vol. 7, pp. 259–310; vol. 9, pp. 231–244.

[b] The steps in the derivation of this formula are given by Trautwine and Marichal in their translation of Bazin's report of his experiments, in Proc. Eng. Club Phila., vol. 7, p. 280.

[c] The steps in detail are given by Trautwine and Marichal in their translation of Bazin, in Proc. Eng. Club Phila., vol. 7, No. 5, p. 281.

weir, v may be expressed in terms of these factors, and of the discharge ($Q=mLD\sqrt{2gD}$).

$$v=\frac{Q}{L(P+D)}.$$

Using this value of v, Bazin obtains the expression

$$m=\mu\left[1+\omega\left(\frac{D}{D+P}\right)^2\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

where $\omega=\frac{3}{2}\alpha m^2$. ω is a nearly constant factor, varying only with m^2. The value of ω as well as that of α can be determined by comparative experiments on thin-edged weirs of different heights.[a]

From a discussion of his own experiments and those of Fteley and Stearns, Bazin finally obtained the formulas

$$\left.\begin{array}{l} Q=\mu LH\sqrt{2gH}, \text{ no velocity of approach;} \\ Q=mLD\sqrt{2gD}, \text{ with velocity of approach.} \end{array}\right\} \quad . \quad . \quad (29)$$

$$\mu=0.405+\frac{0.003\times3.281}{D}=0.405+\frac{0.00984}{D}\,^{b} \quad . \quad . \quad . \quad (30)$$

For a weir with velocity of approach $\alpha=\frac{5}{3}$ and $\omega=0.55$. Substituting in equations (27) and (28),

$$m=\mu\left(1+\frac{3}{2}\cdot\frac{5}{3}\cdot\frac{v^2}{2gD}\right)=\mu\left(1+2.5\frac{v^2}{2gD}\right) \quad . \quad . \quad . \quad (31)$$

$$m=\mu\left[1+0.55\left(\frac{D}{P+D}\right)^2\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

These formulas give values of m agreeing with the results of the experiments within 1 per cent for weirs exceeding about 1 foot in height within the experimental range of head.

Approximately, for heads from 4 inches to 1 foot,

$$m=0.425+0.21\left(\frac{D}{P+D}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

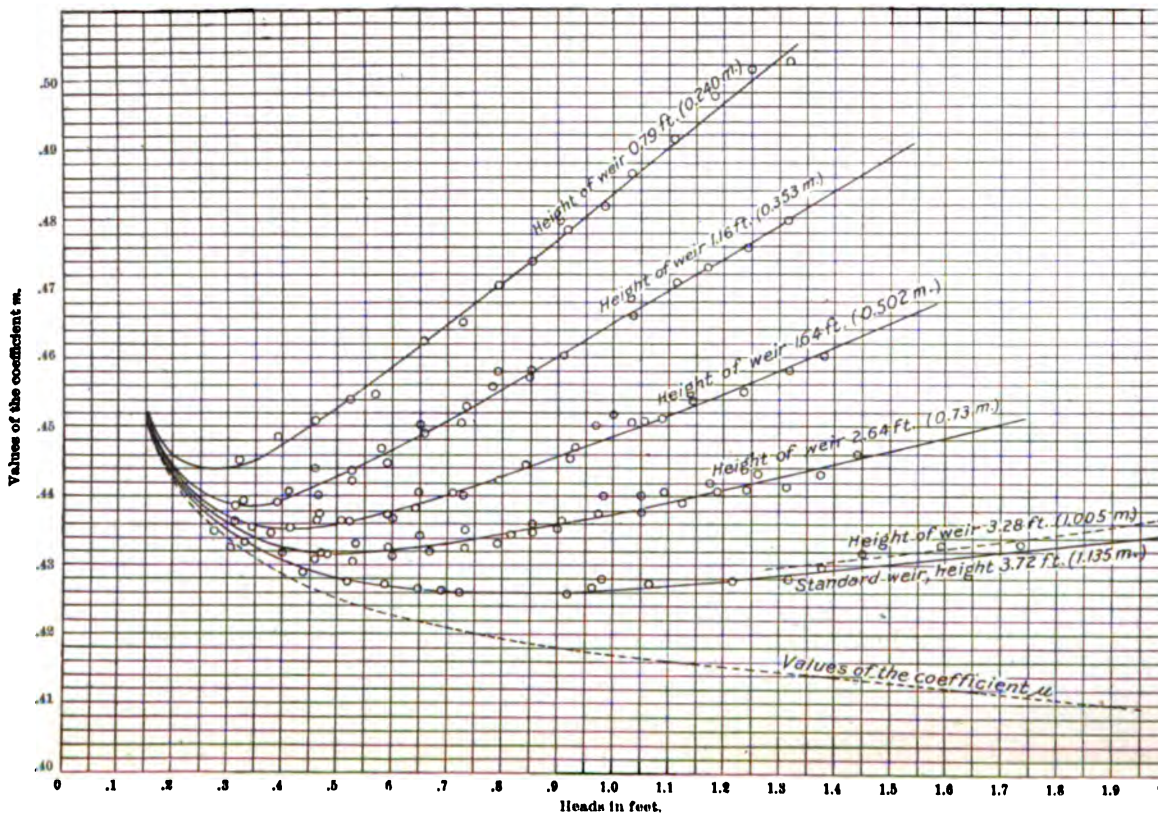
correct within 2 to 3 per cent.

The following table gives Bazin's experimental coefficients, the head and height of weir (originally meters) having been reduced to feet:

---

[a] For detailed analysis see Trautwine and Marichal, Proc. Eng. Club Phila., vol. 7, pp. 282-283.

[b] Experimental tabular values of μ differing very slightly from the formula within the range of Bazin's experiments are also given.



BAZIN'S EXPERIMENTAL COEFFICIENT m FOR THIN-EDGED WEIRS OF VARYING HEIGHT, FOR USE IN THE FORMULA $Q = mLH\sqrt{2gH}$

Values of the Bazin coefficient C in the formula $Q=CLH^{\frac{3}{2}}$ for a thin-edged weir, without end contraction.

Measured head D.	Height of crest of weir above bed of channel of approach, in feet.										Measured head D.
	0.66	0.98	1.31	1.64	1.97	2.62	3.28	4.92	6.56	∞	
Feet.	C	C	C	C	C	C	C	C	C	C	Meters.
0.164	3.673	3.633	3.617	3.609	3.601	3.601	3.601	3.593	3.593	3.594	0.05
.197	3.657	3.609	3.585	3.569	3.569	3.561	3.553	3.553	3.553	3.550	.06
.230	3.649	3.593	3.569	3.553	3.545	3.537	3.529	3.529	3.521	3.522	.07
.262	3.657	3.585	3.553	3.537	3.529	3.513	3.513	3.505	3.505	3.499	.08
.295	3.665	3.585	3.545	3.529	3.513	3.497	3.497	3.489	3.481	3.481	.09
.328	3.681	3.585	3.545	3.521	3.505	3.489	3.481	3.473	3.473	3.466	.10
.394	3.705	3.593	3.545	3.513	3.497	3.478	3.465	3.449	3.449	3.441	.12
.459	3.737	3.609	3.553	3.513	3.489	3.465	3.449	3.432	3.432	3.422	.14
.525	3.777	3.633	3.561	3.513	3.489	3.457	3.440	3.424	3.416	3.405	.16
.591	3.810	3.657	3.569	3.521	3.489	3.457	3.432	3.416	3.408	3.392	.18
.656	3.850	3.681	3.585	3.529	3.497	3.457	3.432	3.408	3.392	3.380	.20
.722	3.882	3.705	3.601	3.545	3.505	3.457	3.432	3.400	3.392	3.371	.22
.787	3.914	3.729	3.625	3.561	3.513	3.465	3.432	3.400	3.384	3.364	.24
.853	3.946	3.753	3.649	3.577	3.529	3.465	3.440	3.400	3.384	3.358	.26
.919	3.978	3.785	3.665	3.593	3.537	3.473	3.440	3.400	3.384	3.353	.28
.984	4.010	3.810	3.689	3.609	3.553	3.481	3.449	3.400	3.376	3.348	.30
1.050	....	3.834	3.705	3.625	3.561	3.497	3.449	3.400	3.376	3.343	.32
1.116	....	3.858	3.721	3.641	3.577	3.505	3.457	3.400	3.376	3.338	.34
1.181	....	3.874	3.745	3.657	3.593	3.513	3.465	3.400	3.376	3.333	.36
1.247	....	3.898	3.761	3.678	3.601	3.521	3.465	3.400	3.376	3.328	.38
1.312	....	3.922	3.785	3.681	3.617	3.529	3.473	3.400	3.376	3.323	.40
1.378	....	3.938	3.801	3.697	3.625	3.537	3.481	3.408	3.376	3.319	.42
1.444	....	3.962	3.818	3.713	3.641	3.545	3.489	3.408	3.376	3.316	.44
1.509	....	3.978	3.834	3.729	3.657	3.553	3.489	3.408	3.376	3.311	.46
1.575	....	....	3.850	3.745	3.665	3.561	3.497	3.408	3.376	3.306	.48
1.640	....	....	3.866	3.753	3.681	3.569	3.505	3.416	3.376	3.303	.50
1.706	....	....	3.874	3.769	3.689	3.577	3.513	3.416	3.376	3.298	.52
1.772	....	....	3.890	3.785	3.697	3.585	3.513	3.416	3.376	3.294	.54
1.837	....	....	3.906	3.793	3.713	3.593	3.521	3.424	3.376	3.289	.56
1.903	....	....	3.922	3.810	3.721	3.601	3.529	3.424	3.376	3.285	.58
1.969	....	....	3.930	3.818	3.737	3.617	3.537	3.424	3.376	3.282	.60
Meters.	0.20	0.30	0.40	0.50	0.60	0.80	1.00	1.50	2.00	∞	....

This table, unfortunately, is inconvenient for interpolation in English units. The values also differ slightly from those computed from the formulas. The table illustrates the difficulty of practical application of a weir formula in which the coefficient varies rapidly both with head and height of weir.

A table has been added giving values of μ computed by formula (30) for a thin-edged weir without velocity of approach.

Values of μ in the Bazin formula for weirs of infinite height, with no velocity of approach.

H. Feet.	0.	0.01.	0.02.	0.03.	0.04.	0.05.	0.06.	0.07.	0.08.	0.09.	0.1.
0.0	....	1.389	0.8970	0.7331	0.6510	0.6018	0.5693	0.5457	0.5280	0.5142	0.5034
.1	0.5034	.4944	.4870	.4807	.4753	.4706	.4665	.4628	.4596	.4568	.4542
.2	.4542	.4518	.4497	.4478	.4460	.4444	.4429	.4414	.4401	.4389	.4378
.3	.4378	.4367	.4357	.4348	.4339	.4331	.4324	.4316	.4309	.4302	.4296
.4	.4296	.4290	.4284	.4278	.4273	.4268	.4264	.4260	.4255	.4251	.4247
.5	.4247	.4243	.4239	.4236	.4232	.4229	.4225	.4222	.4219	.4216	.4214
.6	.4214	.4211	.4208	.4206	.4204	.4202	.4200	.4197	.4195	.4193	.4191
.7	.4191	.4189	.4187	.4185	.4183	.4181	.4180	.4178	.4176	.4174	.4173
.8	.4173	.4171	.4170	.4168	.4167	.4166	.4164	.4163	.4162	.4160	.4159
.9	.4159	.4158	.4157	.4156	.4154	.4153	.4152	.4151	.4150	.4149	.4148
1.0	.4148	.4147	.4146	.4146	.4145	.4144	.4143	.4142	.4141	.4140	.4139
1.1	.4139	.4139	.4138	.4137	.4136	.4136	.4135	.4134	.4133	.4133	.4132
1.2	.4132	.4131	.4131	.4130	.4129	.4129	.4128	.4127	.4127	.4126	.4126
1.3	.4126	.4125	.4124	.4124	.4123	.4123	.4122	.4122	.4121	.4121	.4120
1.4	.4120	.4120	.4119	.4119	.4118	.4118	.4117	.4117	.4116	.4116	.4116
1.5	.4116	.4115	.4115	.4114	.4114	.4113	.4113	.4113	.4112	.4112	.4112
1.6	.4112	.4111	.4111	.4110	.4110	.4110	.4109	.4109	.4108	.4108	.4108
1.7	.4108	.4108	.4107	.4107	.4107	.4106	.4106	.4106	.4105	.4105	.4105
1.8	.4105	.4104	.4104	.4104	.4103	.4103	.4103	.4103	.4102	.4102	.4102
1.9	.4102	.4102	.4101	.4101	.4101	.4100	.4100	.4100	.4100	.4099	.4099
2.0	.4099	....	....	....	....	....	....	....	....	....	....

DERIVED FORMULAS FOR THIN-EDGED RECTANGULAR WEIRS.

A number of weir formulas have been derived from subsequent analysis or recomputation of the experiments of Francis, Fteley and Stearns, and Bazin, differing more or less from those given by the experimenters.

FTELEY AND STEARNS-FRANCIS FORMULA.[a]

$$Q=3.33LH^{\frac{3}{2}}+0.007L \quad . \quad . \quad . \quad . \quad . \quad (34)$$

Correction for end contractions is to be made by the Francis formula; velocity of approach correction by the Fteley and Stearns formulas

$H=D+1.5h,$ for suppressed weir.
$H=D+2.05h,$ for contracted weir.

HAMILTON SMITH'S FORMULA.[b]

The base formula adopted is

$$Q=\frac{2}{3}MLH\sqrt{2gH} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

[a] Fteley and Stearns, Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, p. 82.
[b] Smith, Hamilton, Hydraulics, pp. 123–132.

The velocity of approach correction is made by the use of the formulas

$H=D+1.4h,$ for contracted weirs.[a]
$H=D+1\frac{1}{3}h,$ for suppressed weirs.

A diagram and tables of values of the coefficient M are given by the author. The correction for partial or complete contraction is included in the coefficient, separate values of M being given for suppressed and contracted weirs.

Making $C=\frac{2}{3}M\sqrt{2g},$ the Smith formula (35) may be written

$$Q=CLH^{\frac{3}{2}},$$

which is directly comparable with the Francis formula.

Smith's coefficients in the above form are given in the following tables.

Hamilton Smith's coefficients for weirs with contraction suppressed at both ends, for use in the formula $Q=CLH^{\frac{3}{2}}$.

H= Head, in feet.	L=length of weir, in feet.								
	19	15	10	7	5	4	3[a]	2[a]	0.66[b]
0.1	3.515	3.515	3.520	3.520	3.526	....	....	....	3.611
.15	3.440	3.445	3.445	3.451	3.451	3.461	3.472	3.488	3.542
.2	3.397	3.403	3.408	3.408	3.413	3.429	3.435	3.450	3.510
.25	3.371	3.376	3.381	3.386	3.392	3.403	3.413	3.429	3.494
.3	3.349	3.354	3.360	3.365	3.376	3.386	3.403	3.418	3.483
.4	3.322	3.328	3.333	3.344	3.360	3.371	3.386	3.403	3.478
.5	3.312	3.317	3.322	3.338	3.354	3.371	3.386	3.408	3.478
.6	3.306	3.312	3.317	3.333	3.354	3.371	3.392	3.413	3.483
.7	3.306	3.312	3.317	3.338	3.360	3.376	3.397	3.424	3.494
.8	3.306	3.317	3.322	3.344	3.365	3.386	3.408	3.441	3.510
.9	3.312	3.317	3.328	3.354	3.375	3.397	3.418	3.451	....
1.0	3.312	3.322	3.338	3.360	3.386	3.408	3.429	3.467	....
1.1	3.317	3.328	3.344	3.371	3.397	3.419	3.445	....	....
1.2	3.317	3.333	3.349	3.381	3.403	3.429	3.456	....	....
1.3	3.322	3.338	3.360	3.386	3.413	3.440	3.467	....	....
1.4	3.328	3.344	3.365	3.392	3.424	3.445	....	....	....
1.5	3.328	3.344	3.371	3.403	3.429	3.456	....	....	....
1.6	3.333	3.349	3.376	3.408	3.435	3.461	....	....	....
1.7	3.338	3.349	3.381	3.413	....	....	....	....	....
2.0	....	....	....	....	....	....	....	....	....

[a] The use of the head corresponding to central surface velocity without correction, to determine D, is also recommended.

[b] Approximate.

Hamilton Smith's coefficients for weirs with two complete end contractions, for use in the formula $Q=CLH^{\frac{3}{2}}$.

H= Head.	L′=length of weir, in feet.										
	0.66	1 [a]	2	2.6	3	4	5	7	10	15	19
0.1	3.381	3.419	3.456	3.478	3.488	3.494	3.494	3.499	3.504	3.504	3.510
.15	3.312	3.344	3.392	3.408	3.413	3.419	3.424	3.424	3.429	3.435	3.435
.2	3.269	3.306	3.349	3.365	3.371	3.376	3.376	3.381	3.386	3.392	3.392
.25	3.237	3.274	3.322	3.333	3.338	3.344	3.349	3.354	3.360	3.360	3.365
.3	3.215	3.253	3.296	3.306	3.312	3.322	3.322	3.333	3.338	3.338	3.344
.4	3.183	3.215	3.258	3.274	3.280	3.285	3.290	3.301	3.306	3.312	3.317
.5	3.156	3.189	3.237	3.247	3.253	3.264	3.269	3.280	3.290	3.295	3.301
.6	3.140	3.172	3.215	3.231	3.237	3.247	3.253	3.269	3.280	3.285	3.290
.7	3.130	3.156	3.199	3.210	3.226	3.231	3.242	3.258	3.274	3.280	3.285
.8	....	....	3.183	3.199	3.215	3.221	3.231	3.247	3.269	3.274	3.280
.9	....	....	3.167	3.189	3.199	3.210	3.226	3.242	3.258	3.269	3.274
1.0	....	....	3.156	3.172	3.183	3.199	3.215	3.231	3.253	3.264	3.269
1.1	....	....	3.140	3.162	3.172	3.189	3.205	3.226	3.242	3.258	3.264
1.2	....	....	3.130	3.151	3.162	3.178	3.194	3.215	3.237	3.253	3.264
1.3	....	....	3.114	3.135	3.151	3.167	3.199	3.205	3.231	3.247	3.258
1.4	....	....	3.103	3.124	3.140	3.156	3.178	3.199	3.221	3.242	3.258
1.5	....	....	....	3.114	3.130	3.151	3.167	3.189	3.215	3.237	3.253
1.6	....	....	....	3.103	3.114	3.140	3.162	3.183	3.210	3.231	3.247
1.7	....	....	....	....	....	....	....	3.178	3.205	3.226	3.247
2.0	....	....	....	....	....	....	....	....	....	....	....

[a] Approximate.

Hamilton Smith's coefficient C for long weirs.

H	0.1	0.2	0.3	0.4	0.5	0.6	0.7
0.00	3.5096	3.3972	3.3438	3.3170	3.3010	3.2956	3.2849
.01	3.4957	3.3908	3.3411	3.3154	3.3005	3.2945	3.2838
.02	3.4818	3.3844	3.3384	3.3138	3.2999	3.2935	3.2828
.03	3.4678	3.3780	3.3358	3.3122	3.2994	3.2924	3.2817
.04	3.4539	3.3716	3.3331	3.3106	3.2988	3.2913	3.2806
.05	3.4400	3.3652	3.3304	3.3090	3.2983	3.2902	3.2796
.06	3.4314	3.3537	3.3277	3.3074	3.2978	3.2892	3.2785
.07	3.4229	3.3512	3.3250	3.3058	3.2972	3.2881	3.2773
.08	3.4143	3.3488	3.3224	3.3042	3.2967	3.2870	3.2762
.09	3.4058	3.3463	3.3197	3.3026	3.2961	3.2860	3.2752

Hamilton Smith's formula is based on a critical discussion of the experiments of Lesbros, Poncelet and Lesbros, James B. Francis, Fteley and Stearns, and Hamilton Smith; including series with and without contractions and having crest lengths from 0.66 to 19 feet.

SMITH-FRANCIS FORMULA.

The Smith-Francis formula,[a] based on Francis's experiments, reduced to the basis of correction for contractions and velocity of approach used with Hamilton Smith's formula, is,

for a suppressed weir,

$$Q=3.29\left(L+\frac{H}{7}\right)H^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

for weir of great length or with one contraction,

$$Q=3.29LH^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (37)$$

for weir with full contraction,

$$Q=3.29\left(L-\frac{H}{10}\right)H^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (38)$$

If there is velocity of approach,

$H=D+1.4\ h$, for a contracted weir.
$H=D+1\frac{1}{3}\ h$, for a suppressed weir.

PARMLEY'S FORMULA.[b]

Parmley's formula is

$$Q=CKLD^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

If there are end contractions, the correction is to be made by the Francis formula,

$$L=L'-0.1NH$$

The factor K represents the correction for velocity of approach.

The factor has been derived by comparing the velocity correction factor in the Bazin formula (formula 32), written in the form

$$K=\left[1+0.55\left(\frac{a}{A}\right)^2\right],$$

with the approximate Francis correction as deduced by Hunking and Hart (formula 23), written in the form

$$K=\left[1+0.2489\left(\frac{a}{A}\right)^2\right],$$

where a is the area of the section of discharge, for either a suppressed or contracted weir, and A is the section of the leading channel. It is observed that there is an approximately constant relation between the two corrections, that of Bazin being 2.2 times that of Francis.

[a] Smith, Hamilton, Hydraulics, pp. 99 and 137.

[b] Rafter, G. W., On the flow of water over dams: Trans. Am. Soc. C. E., vol. 44, pp. 350-359, discussion by Walter C. Parmley.

Parmley adopts the Bazin correction and gives the following table, which may also conveniently be applied in computing discharge by Bazin's formula.

The discharge coefficient C used by Parmley is that for a weir with no velocity of approach, as in the Francis formula. It is not, however, constant. Its values have been deduced from a mean curve representing the experiments of Francis, Fteley and Stearns, and Bazin.

Velocity of approach correction, K, Parmley and Bazin formulas.

$\frac{a}{A}$	0	0.1	0.2	0.3	0.4	0.5
0.00	....	1.0055	1.0220	1.0495	1.0880	1.1375
.01	1.0001	1.0066	1.0243	1.0529	1.0925	1.1431
.02	1.0002	1.0079	1.0266	1.0563	1.0970	1.1487
.03	1.0005	1.0093	1.0291	1.0599	1.1017	1.1545
.04	1.0009	1.0108	1.0317	1.0636	1.1065	1.1604
.05	1.0014	1.0124	1.0344	1.0674	1.1114	1.1664
.06	1.0020	1.0141	1.0372	1.0713	1.1164	1.1725
.07	1.0027	1.0159	1.0401	1.0753	1.1215	1.1787
.08	1.0035	1.0178	1.0431	1.0794	1.1267	1.1850
.09	1.0044	1.0198	1.0463	1.0837	1.1321	1.1915

Parmley's weir formula, coefficient C.

D, feet.	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	....	....	....	....	....	....	....	....	....	....
.1	3.580	3.568	3.556	3.544	3.532	3.520	3.512	3.503	3.495	3.486
.2	3.478	3.471	3.464	3.458	3.451	3.444	3.439	3.434	3.430	3.425
.3	3.420	3.416	3.412	3.408	3.404	3.400	3.397	3.394	3.391	3.388
.4	3.385	3.383	3.381	3.380	3.378	3.376	3.374	3.373	3.371	3.370
.5	3.368	3.367	3.366	3.364	3.363	3.362	3.361	3.360	3.360	3.359
.6	3.358	3.357	3.356	3.356	3.355	3.354	3.353	3.353	3.352	3.352
.7	3.351	3.351	3.350	3.350	3.349	3.349	3.348	3.348	3.347	3.347
.8	3.346	3.345	3.345	3.344	3.344	3.343	3.342	3.342	3.341	3.341
.9	3.340	3.339	3.339	3.338	3.338	3.337	3.336	3.336	3.335	3.335
1.0	3.334	3.334	3.333	3.332	3.332	3.332	3.331	3.330	3.330	3.329
1.1	3.329	3.328	3.328	3.328	3.327	3.326	3.326	3.326	3.325	3.324
1.2	3.324	3.324	3.323	3.322	3.322	3.322	3.321	3.320	3.320	3.320
1.3	3.319	3.318	3.318	3.317	3.317	3.316	3.315	3.315	3.314	3.314
1.4	3.313	3.312	3.312	3.311	3.311	3.310	3.309	3.309	3.308	3.308
1.5	3.307	3.306	3.306	3.305	3.305	3.304	3.303	3.303	3.302	3.302
1.6	3.301	3.301	3.300	3.300	3.299	3.298	3.298	3.298	3.297	3.296
1.7	3.296	3.295	3.295	3.294	3.294	3.293	3.292	3.292	3.291	3.291
1.8	3.290	3.290	3.289	3.288	3.288	3.288	3.287	3.286	3.286	3.285
1.9	3.285	3.285	3.284	3.284	3.283	3.282	3.282	3.282	3.281	3.280
2.0	3.280	....	....	....	....	....	....	....	....	....

EXTENSION OF THE WEIR FORMULA TO HIGHER HEADS.

It will be noticed that all the accepted formulas for discharge over thin-edged rectangular weirs are based on experiments in which the head did not exceed 2 feet above crest. It is often desirable to utilize the weir for stream gagings where the head is greater, especially for the determination of maximum discharge of streams, the head frequently being as large as 6, 8, or even 10 or 12 feet.

In the experiments at Cornell University on weirs of irregular section it was often necessary to utilize depths on the standard weir exceeding the known limit of the formula. A series of experiments was accordingly carried out in which a depth on a standard thin-edged weir (16 feet long) not exceeding the limit of the formula was utilized to determine the discharge over a similar but shorter standard thin-edged weir (6.56 feet long) for depths up to approximately 5 feet.[a] The results of these experiments, as recomputed, eliminating slight errors in the original, are given below.

It will be noted that the weir was short and the velocity of approach relatively large, yet, according to the results when corrected by the Francis method, the average value of C for heads from 0.75 to 4.85 feet is 3.296, or 98.88 per cent of the Francis coefficient for a thin-edged weir. The average value of C for heads from 0.746 foot to 2 feet is 3.266, and for heads from 2 to 4.85 feet, 3.278.

United States Deep Waterways experiments at Cornell hydraulic laboratory for extension of thin-edged weir formula.

Standard weir, 16 feet long, 13.13 feet high.		Lower thin-edged weir: P=5.2, L=6.56.					Q, cubic feet per second, per foot (corrected).	$C=\frac{Q}{H^{\frac{3}{2}}}$
Cor. D, longitudinal, piezometer, centimeters.	Q, Bazin formula, in cubic feet per second.	Observed P, flush, piezometer, centimeters.	D, in feet.	$\frac{D}{P+D}$	K Hunking and Hart.	$H^{\frac{3}{2}}$		
1	2	3	4	5	6	7	8	9
12.28	14.12	22.744	0.7462	0.1255	1.0041	0.6469	2.1066	3.256
15.30	19.42	27.855	.9139	.1495	1.0056	.8787	2.9143	3.317
18.39	25.35	33.175	1.0885	.1731	1.0075	1.1434	3.8183	3.331
21.65	32.24	39.419	1.2933	.1992	1.0099	1.4849	4.8685	3.279
24.16	37.86	44.000	1.4436	.2173	1.0122	1.7564	5.7252	3.260
27.21	45.13	49.699	1.6306	.2387	1.0141	2.1116	6.8333	3.236
30.16	52.62	55.213	1.8115	.2583	1.0166	2.4787	7.9750	3.218
30.22	52.77	55.128	1.8088	.2581	1.0166	2.4730	7.9977	3.234
37.90	73.46	68.238	2.2389	.3010	1.0225	3.4254	11.1516	3.226
44.22	92.79	80.566	2.6434	.3370	1.0283	4.4193	14.0960	3.190
59.00	143.90	105.639	3.4660	.4000	1.0398	6.6095	21.8902	3.312
74.22	202.37	130.286	4.2747	.4512	1.0504	9.2867	30.8008	3.317
81.69	233.81	142.557	4.6773	.4735	1.0557	10.6789	35.5933	3.333

[a] Rafter, G. W., On the flow of water over dams: Trans. Am. Soc. C. E., vol. 44, p. 397.

If it is borne in mind that the influences which go to make up variation in the weir coefficient are more potent for low than for larger heads, it may be confidently asserted that the Francis formula is applicable within 2 per cent for heads as great as 5 feet, and by inference it is probably applicable for much greater heads as well.

COMPARISON OF WEIR FORMULAS.

The later weir formulas all give results agreeing, for the range of heads covered, within the limit of accuracy of ordinary stream measurements. Which of the several formulas to use will be determined by convenience and by the conditions attending the measurements.

The Francis formula is applicable for weirs with perfect bottom contraction and for any head above 0.50 foot.

The Hamilton Smith, Fteley and Stearns, and Bazin formulas are more accurate for very slight heads, or where bottom contraction is imperfect, this element, which tends to increase discharge, being included in the larger velocity of approach correction. These formulas are, however, based on experiments none of which exceeded 2 feet head, and they have not been extended.

For suppressed weirs in rectangular channels having conditions closely duplicating Bazin's experiments, his formula is probably most applicable. The head should preferably be measured in a Bazin pit, opening at the bottom of the channel, 16.4 feet upstream from the weir. In a suppressed weir, if the nappe is allowed to expand laterally after leaving the weir, the computed discharge by any of the formulas should be increased from one-fourth to one-half of 1 per cent.

Comparative discharge by various formulas over weirs of great height and length; no end contractions nor velocity of approach.[a]

Formula.	Coefficient C, for heads ranging from 0.20 to 4 feet.				Per cent of discharge by Francis formula for heads ranging from 0.20 to 4 feet.			
	0.20	0.50	1.00	4.00	0.20	0.50	1.00	4.00
Castel	3.4872	3.4872	3.4872	3.4872	104.616	104.616	104.616	104.616
Boileau	3.3455	3.3455	3.3455	3.3455	100.365	100.365	100.365	100.365
Weisbach	3.4025	....	3.3136	....	102.075	....	99.408	....
Francis	3.33	3.33	3.33	3.33	100.00	100.0	100.0	100.0
Fteley and Stearns	3.5004	3.3269	3.317	3.3109	105.012	99.807	99.51	99.327
Bazin	3.642684	3.406094	3.326696	3.26783	109.281	102.183	99.801	98.035
Fteley-Stearns-Francis	3.3800	3.3300	3.319	3.31375	101.400	99.90	99.570	99.412
Hamilton Smith	3.3972	3.3010	3.284	3.284	101.916	99.030	98.520	....
Smith-Francis	3.29	3.29	3.29	3.29	98.70	98.70	98.70	....
Parmley	3.478	3.368	3.334	....	104.340	101.040	100.020	....
East Indian engineers	3.488	3.472	3.445	3.285	104.640	104.16	103.35	98.560

[a] Computed by H. R. Beebe, C. E.

Table showing comparative discharge per foot of crest for suppressed weirs of various lengths, heads, and velocities of approach.[a]

Length (L)	2	2	10	10	10
Height (P)	1	2	2	4	4
Head (D)	1.0	1.0	1.0	1.0	4
Approximate velocity of approach (v)	1.90	1.18	1.16	.68	2.15
Castel	3.7822	3.6127	3.6217	3.5308	30.3037
Boileau	3,8630	3.5484	3.5484	3.4144	30.9046
Francis	3.5373	3.4218	3.4218	3.3632	28.2983
Fteley and Stearns	3.7268	3.4729	3.4730	3.3669	29.7470
Bazin	3.7845	3.3766	3.3766	3,4002	29.7555
Fteley-Stearns-Francis	3.7297	3.4752	3.4752	3.3690	29.7000
Hamilton Smith	3.9220	3.6392	3.4872	3.3878	....
Smith-Francis	4.0581	3.7109	3.4847	3.3876	31.573
Parmley	3.7924	3.5337	3.5337	3.3347	....
Average	3.800	3.532	3.490	3.395	30.040

[a] Computed by H. R. Beebe, C. E.

COMPARISON OF VARIOUS VELOCITY OF APPROACH CORRECTIONS.

The various modes of correction for velocity of approach used by different investigators can be rendered nearly identical in form, varying, however, in the value of the coefficient α adopted.

Comparative coefficients of correction for velocity of approach for thin-edged weirs with end contractions suppressed.

Experimenter.	Value of α in the formula $H=D+\alpha\frac{v^2}{2g}$	Values of ω in the formula $K=1+\omega\left(\frac{a}{A}\right)^2$.
Boileau	$\alpha=1.8$	
Lesbros	$\alpha=1.56$	
Fteley and Stearns	$\alpha=1.5$	
Francis	[a] $\alpha=1-\frac{2}{3}\sqrt{\frac{h}{D}}$	[b] $\omega=0.2489$
Bazin	$\alpha=1.69$ or $\frac{5}{3}$	$\omega=0.55$

[a] Emerson. [b] Hunking and Hart.

The above values were all derived from experiments on thin-edged weirs. Bazin's experiments covered the larger range of velocities and were most elaborate. It may be noted that the correction applied by Bazin is two and two-tenths times that of Francis for a given velocity of approach. Bazin's correction is, in effect, an increase in the measured head of 1.69 times the velocity head, while Francis increases the measured head by an amount $\frac{2}{3}\sqrt{\frac{h}{D}}$ *less than* the velocity head according to Emerson's formula.

Ratio of the various corrections for velocity of approach for suppressed weirs.

	Bazin.	Fteley and Stearns.	Hamilton Smith.	Francis.
Bazin	1.000	1.127	1.271	2.2
Fteley and Stearns	.887	1.000	1.128	1.957
Hamilton Smith	.789	.887	1.000	1.736
Francis	.454	.511	.576	1.000

The factors in the above table are not strictly accurate, for the reason that the expressions used to deduce the equivalents from the different formulas are in some cases approximations. They serve to illustrate the relative magnitude of the different corrections for thin-edged weirs without end contractions. For thin-edged weirs with end contraction, Hamilton Smith uses the coefficient $\alpha=1.4$ and Fteley and Stearns give the coefficient $\alpha=2.05$.

There are no experiments available relative to the value of the velocity correction for other than thin-edged weirs. It is necessary, therefore, to utilize the values above given for weirs of irregular section. It will be seen that it matters little in what manner the correction for velocity of approach is applied, either by directly increasing the observed head, as in the formulas of Hamilton Smith and Fteley and Stearns, or by including the correction in the weir coefficient, as is done by Bazin, or by utilizing a special formula to derive the corrected head, after the manner of James B. Francis. The three methods can be rendered equivalent in their effect.

The important point is that the corrected result must be the same as that given by the author of the formula which is used to calculate the discharge. As to the relative value of the different modes of applying the correction, it may be said of that of Francis, that in its original form it is cumbersome, but it renders the correction independent of dimensions of the leading channel, as do also the formulas for correction used by Hamilton Smith, and Fteley and Stearns. Inasmuch as the velocity head is a function of the discharge, successive approximations are necessary to obtain the final corrected head by any one of these three formulas.

By using the Hunking and Hart formula the correction for the Francis weir formula becomes fairly simple, as it does not require the determination of the mean velocity of approach by successive approximations, but to apply this formula it is necessary to know the dimensions of the leading channel and of the weir section. The approximation given by Emerson is also much simpler than the original Francis formula.

Bazin's method of including the velocity correction in the coefficient makes the weir coefficients obtained by the experiments comparable one with another only when both the head and velocity of approach are the same in both cases.[a] His correction also involves the dimensions of the leading channel as factors. Obviously, in the case of many broad-crested weirs utilized for measuring flow, the dimensions of the leading channel can not be ascertained accurately and there is great variation of velocity in different portions of the section of approach. It becomes necessary that the correction should be in such a form that it is a function of the velocity and not of the channel dimensions.

It is to be noticed that where an attempt has been made in the weir experiments to eliminate velocity of approach effect from the coefficient the velocity has been nearly equalized by screens and has been determined by successive approximations. It is suggested that where the velocities vary widely they be determined by current meter in several subdivisions of the section, the approximate integral kinetic energy estimated, and a value of α selected depending on the ratio of $\frac{h'}{h}$ so obtained, where h is the velocity head corresponding to the mean velocity and h' is the velocity head which would result if the actual velocities were equalized. Inasmuch as the surface velocity usually exceeds the mean velocity in the channel of approach in about the same ratio that h' exceeds h, the suggestion is made by Hamilton Smith[b] that where the velocity of approach is unavoidably variable, or the boundaries of the current are uncertain, the surface velocity v_s be measured by floats and applied directly in the determination of the quantity h.

The variations in discharge over a thin-edged weir, by the different formulas, are often less than the difference in the correction for velocity of approach would indicate. In the formula of Fteley and Stearns, as compared with Francis, for example, the larger velocity correction is in part compensated by a smaller weir coefficient, and the same is true of the formulas of Hamilton Smith and Bazin for cases where the head is large.

[a] See special discussion of the point, p. 63. [b] Smith, Hamilton, Hydraulics, p. 84.

END CONTRACTIONS—INCOMPLETE CONTRACTION.

The formula for end contractions deduced by James B. Francis is very generally used. The correction is made to the length of weir, the result obtained being the length of a suppressed weir that will give the same discharge.

$$L = L' - b\,NH \quad . . . . (40)$$

b = A coefficient, the value of which, deduced by Francis, is $b=0.1$.

L' = Actual length of weir crest.

L = Length of equivalent suppressed weir crest.

N = Number of end contractions.

H = Effective head, feet.

The experiments of Fteley and Stearns,[a] while somewhat discordant, indicate an average value of b for heads from 0.3 to 1 foot, of about 0.1. The value of b apparently decreases as the head increases. It also decreases if the end contraction piece is so near the side of the channel as to render the contraction incomplete.

Hamilton Smith shows that side contractions and bottom or crest contraction are mutually related, and that the side width of the channel of approach should be fully three times the least dimension of the weir. Usually L is much greater than H, and the side width may be made at least as great as $3H$. The specification of Francis is, side width $\geqq H$.

Smith's rule indicates that to provide complete contraction the area of leading section A must bear a relation to the area of weir section a depending upon the relative head and length of crest.

For three weirs of equal section a, the following values of A, the necessary channel-section area, are given:

$L'=12$	$H=1$	$a=12$	$A=72=6a$
$L'=4$	$H=3$	$a=12$	$A=264=2.2a$
$L'=1$	$H=12$	$a=12$	$A=105=8.7a$

Hamilton Smith prefers to use separate coefficients for suppressed weirs from those for contracted weirs, the relation between the coefficients being expressed by the formula

$$C_p = C_c\left(1 + z\,\frac{S}{R}\right) \quad . . . . (41)$$

C_p = Coefficient for partially suppressed weir, as with complete suppression on sides and full contraction at bottom.

[a] Fteley and Stearns, Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, pp. 108–113.

C_c = Coefficient for completely contracted weir.
X = Least dimension of weir, whether L or H.
R = Wetted perimeter of weir $= L+2H$.
Y = Distance from any side of weir to the respective side of channel, where there is partial suppression.
S = Length of sides on which there is partial suppression.

Smith's values of contraction coefficient z in formula 31 are

Y/X	z
3	0.000
2	.005
1	.025
½	.06
0	.16

The ratio Y/X approximately measures the amount of contraction.[a]

Bazin does not give a formula for weirs with end contractions. The Bazin formula may be applied to weirs in which the height of weir is so small that the bottom contraction is partially suppressed. The Bazin coefficient then includes:

1. Effect of contraction from surface curve.
2. Effect of crest contraction and its modification by both velocity of approach and by partial suppression, if any.
3. Effect of velocity of approach proper.
4. Effect of distribution of velocities in channel of approach.
5. Loss of head from friction and eddies.

As the Bazin weirs were very low, and these factors go to increase the correction necessary, it will be seen that the relatively large velocity of approach correction required by Bazin's formula may be readily accounted for.

The experiments of Flinn and Dyer on the Cippoletti weir (see p. 48) indicate that the effect of end contraction may be somewhat greater than that indicated by the Francis formula. Any experiments in which similar volumes of water have been successively passed over weirs with and without end contractions may be utilized to determine the effect of such contractions.

It may be added that a more elaborate study of end contractions is desirable. It is to be borne in mind, however, that to secure greater accuracy in this regard a more complicated or variable correction than that of Francis must probably be used, and the result will be to greatly increase the labor of weir computations in the interest of what is usually a comparatively small matter, the better remedy being probably the use of weirs with end contractions suppressed, wherever practicable.

[a] Smith, Hamilton, Hydraulics, pp. 118-123. Smith's critical discussion of this subject will be found of value in calculating discharge for weirs with partially suppressed contraction either at sides or bottom.

COMPOUND WEIR.

A weir with a low-water notch depressed below the general crest level may sometimes be used to advantage in gaging small, variable streams. The discharge over such a weir, constructed with end contractions on both sections, can be calculated as for two separate weirs, the lower short section having end contractions for all heads. The flow over the two upper sections is computed as for a suppressed weir.

Such a weir has been used for the determination of the low-water flow of very small streams, for which purpose it is well adapted, the entire stream when at low stages flowing in the central notch, in a stream relatively deep and narrow.

The measurement of very thin sheets of water on a broad weir is subject to peculiar difficulties, including uncertainty of coefficient, adhesion of nappe to weir face, dispersion by winds, and a large percentage error in the results if there is a small error in measuring the head.

TRIANGULAR WEIR.

GENERAL FORMULA.

Referring to fig. 4, we may write

$$l : H-y :: L : H.$$

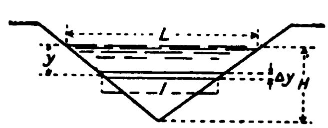


FIG. 4.—Triangular weir.

Substituting, in equation (4),

$$Q=\int_0^H \sqrt{2gy}\,\frac{L(H-y)}{H}\,dy$$

$$=\frac{4}{15}\,L\,\sqrt{2g}\,H^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (42)$$

THOMSON'S EXPERIMENTS.

The mean coefficient of contraction for a thin-edged triangular weir deduced experimentally by Prof. James Thomson, of Belfast, is $M=0.617$,[a] the formula being

$$Q=\frac{4}{15}ML\,\sqrt{2g}\,H^{\frac{3}{2}}=1.32LH^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad (43)$$

[a] British Association Report, 1858 (original not consulted). Merriman gives the mean value of M for heads between 0.2 and 0.8 foot as 0.592.

For a right-angled notch,

$$L=2H \text{ and } Q=2.64H^{\frac{5}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (44)$$

The length of the contracting edges in a triangular notch being proportional to the depth, it is believed that the coefficient of discharge is somewhat more constant than for a rectangular weir.[a]

TRAPEZOIDAL WEIR.

The discharge in this case may be determined directly from the integral formula (4) as for a triangular weir, by integrating between the limits AD and CE, fig. 5. It may also be derived as follows:

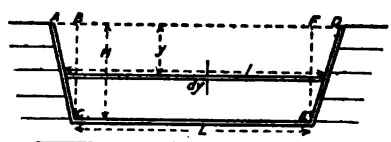


FIG. 5.—Trapezoidal weir.

z=slope of one side to the vertical.

$$Q=\int_{y=o}^{y=H} \sqrt{2gy}\,[L+2z(H-y)]\,dy$$

By integration,

$$Q=\frac{2}{3}\sqrt{2g}\,LH^{\frac{3}{2}}+\frac{8}{15}z\sqrt{2g}\,H^{\frac{5}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (45)$$

in which coefficients of contraction for the horizontal crest and for the end slopes must be introduced.

THE CIPPOLETTI TRAPEZOIDAL WEIR.

The discharge over a trapezoidal without contraction would be the sum of that for a rectangular weir added to that for two triangular weirs forming the ends. From the experiments of James B. Francis[b] it appears that each end contraction reduces the effective length of the weir $0.1H$. The contraction decreases the discharge by the amount

$$Q=\frac{2}{3}\,M\times 0.1H\sqrt{2g}\,H^{\frac{3}{2}}=\frac{2}{3}\frac{M}{10}\sqrt{2g}H^{\frac{5}{2}}$$

If the ends of the weir, instead of being vertical, are inclined outward in such manner that the discharge through the added area counterbalances the decrease from the end contraction, then the effective

[a] The coefficient 2.64 is the same as that deduced for broad crest weirs with stable nappe. A table of values of $2.64H^{\frac{5}{2}}$ is given on page 177, which may be applied in calculating flow over triangular weirs.

[b] Lowell Hydraulic Experiments.

length of the weir will remain constant as the head increases, the same as in a suppressed weir. The discharge through the end triangle ABC will be, from equation (42),

$$Q=\frac{4}{15}Mz\sqrt{2g}H^{\frac{3}{2}}$$

Where z is the width or base of the end triangle. Equating the two expressions for Q, and solving for z, we find, assuming M to have the same value in both cases,

$$z=\frac{1}{4}H \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (46)$$

This condition defines the Cippoletti weir.[a]

CIPPOLETTI'S FORMULA.

Cippoletti derived his formula from a discussion of the experiments of James B. Francis, selecting a coefficient 1 per cent greater, making

$$Q=\frac{2}{3}\times 0.629\ LH\sqrt{2gH}=3.367LH^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad (47)$$

L is the length of the crest or base of the trapezoid.

Flinn and Dyer[b] experimented at the testing flume of the Holyoke Water Power Company by passing the same volume of water successively over a trapezoidal experimental weir and over the gaging weir of the turbine testing flume 19.7 feet downstream. The latter, it is stated, complied in form with Francis's specifications.

The depths were observed by hook gage; eleven readings, as a rule, being taken and their arithmetical mean used for the determination of a head. The thirty-two series of valid experiments range from 0.3 foot depth on a weir with sill length of 3 feet to a head of 1.25 feet on a sill 9 feet long.

The discharge over the standard weir was calculated by the formulas of J. B. Francis and of Hamilton Smith. The correction for velocity of approach at the experimental weir was made by the formula of Hamilton Smith, for use with contracted rectangular weirs,

$$H=D+1.4h.$$

Flinn and Dyer's coefficients are as follows:

Mean of 32 experiments,	$C=3.283$
Mean after rejecting 5 diminished weights,	$C=3.301$

In general, the coefficient diminished as the head increased, suggesting that the end inclination should slightly exceed $\frac{1}{4}H$ in the Cippoletti weir, to provide complete compensation, and that the end contraction

[a] First described by Cesare Cippoletti in Canal Villoresi, Modulo per la Dispensa delle Acque, 1887.

[b] Flinn, A. D., and Dyer, C. W. D., The Cippoletti trapezoidal weir: Trans. Am. Soc. C. E., vol. 32, 1894, pp. 9-33.

coefficient in the trapezoidal weir may be greater than $0.1H$, as used by Francis.

The question is complicated by velocity of approach. For example, had the Francis velocity-correction formula been used by Flinn and Dyer, their values of C would have been larger. As a tentative conclusion it is probable that the application of either the Francis formula with his velocity-head correction or the Flinn and Dyer coefficient with the Smith velocity correction will, when applied to a Cippoletti weir, give results as accurate as the precision of the coefficients will justify.

REQUIREMENTS AND ACCURACY OF WEIR GAGINGS.

PRECAUTIONS FOR STANDARD WEIR GAGING.

Certain specifications were laid down by James B. Francis as guides in cases where the utmost precision is desired in weir measurements.[a] The limits of applicability of the weir have been greatly extended since 1852, and some of the uncertainties as to the effect of various modifications of weir construction have been removed.

In general, for standard thin-edged weirs—

1. The upstream crest edge should be sharp and smooth.
2. The overflowing sheet should touch only the upstream crest corner.
3. The nappe should be perfectly aerated.
4. The upstream face of the weir should be vertical.
5. The crest should be level from end to end.
6. The measurements of head should show the true actual elevation of water surface above the level of the weir crest.
7. The depth of leading channel should be sufficient to provide complete crest contractions, and, if they are not suppressed, the width of channel should be sufficient to provide complete end contractions.
8. A weir discharging from a quiet pond is to be preferred. If this is not available, the velocity of approach in the leading channel should be rendered as uniform as possible and correction made therefor by the method employed by the experimenter in deriving the formula.

In order to fulfill these requirements, certain secondary conditions are necessary. The depth on the weir should be measured at a point far enough upstream from the crest to be unaffected by the surface curvature, caused by the discharge.

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[a] Francis, J. B., Lowell Hydraulic Experiments, pp. 133-135.

The distance upstream to the point of measuring the head has been as follows:

Distance upstream from weir to gage used by various experimenters.

Experimenter.	Date.	Distance upstream, in feet.
Poncelet and Lesbros	1828	11.48
Lesbros	1834	11.48
Francis	1852	6.00
Hamilton Smith, jr.	1874–1876	7.60
Fteley and Stearns	1878	6.00
Bazin	1886	16.40

Six feet upstream from crest is a distance frequently used, but this may be insufficient for suppressed weirs, and also for those having irregular cross sections or upstream slopes. Boileau considered the origin of the surface curvature to be at a distance from the weir equal to about 2.5 times the height of crest above the bottom of the channel of approach, indicating that for a suppressed weir the head should be measured at least this distance from the crest.[a] For a weir discharging from a still pond the head can be measured at any considerable distance from the weir. Hamilton Smith[b] states that, for weirs with full contraction, H can be measured at any convenient point from 4 feet to 10 feet from the crest.

The head may be measured directly by a graduated scale or hook gage, or by means of a piezometer tube having its orifice flush with the side wall of the leading channel, and at right angles to the direction of flow of the water.

The depth of the leading channel in Francis's experiments was 4.6 feet below crest, and Francis lays down the rule that the depth of the leading canal should be at least three times the head on the weir. Hamilton Smith fixes the minimum depth of the leading channel below the crest at $2H$.

Fteley and Stearns[c] state that the depth of the leading channel below weir crest should be at least 0.5 foot, in order that correction for velocity of approach may be reliably made for depths occurring in their measurements, and that a greater depth of leading channel is to be preferred.

To provide complete end contractions, Francis states that the distance from the side of the channel of approach to the end of the weir overflow should be at least equal to the depth on the weir. Hamilton

[a] Fteley and Stearns, Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, p. 47.
[b] Smith, Hamilton, Hydraulics, pp. 129–131.
[c] Ibid., pp 112–114.

Smith considers that the distance from the end of the weir to the side of the channel should be at least $2H$, and that the depth of channel below crest, also the side distance, should in no case be less than 1 foot. Francis further specifies that the length of weir crest should be at least three times the depth of overflow. The nappe should not be allowed to expand laterally immediately below a suppressed weir.

In order that the nappe may be perfectly aerated, Francis considers that the fall below crest level on the downstream side should be not less than $\frac{1}{2}H$, increasing for very long weirs or in cases where the downstream channel is shallow. He found, however, no perceptible difference in the discharge for a head of 0.85 foot, whether the water on the downstream side was 1.05 feet or 0.0255 foot below crest level. Fteley and Stearns and Hamilton Smith agree that, if the water is

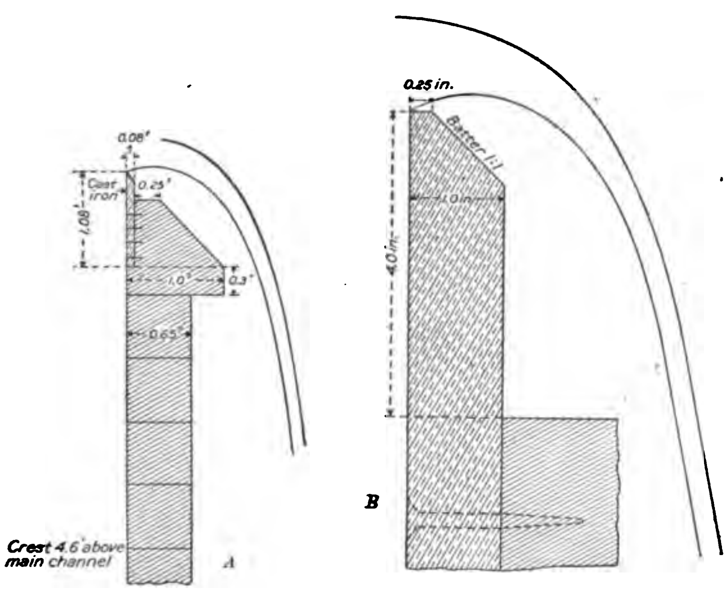


FIG. 6.—Sections of the Francis weir. *A*, General section of weir; *B*, detail of crest.

deep below, it may rise to crest level on downstream side of weir without sensible error, and Fteley and Stearns add that a weir may be submerged to a depth of 15 per cent of the head without an error exceeding 1 per cent.

The thickness of crest lip is immaterial so long as the edge is sharp and square and the nappe cuts free and is freely aerated. The latter conditions require, however, that the crest shall be thin, especially where the head is slight.

Fig. 6 shows cross sections of the crest of the weir used by James B. Francis at the Lower Merrimac locks at Lowell, in 1852, in deriving his formula. The crest consisted of a cast-iron plate 13 inches wide and 1 inch thick, planed true and smooth on all surfaces. Its upper edge was chamfered on the downstream side at an angle of 45° to a thickness of 0.25 inch at the edge. As shown, the nappe cut clear from the top of the crest in an unbroken sheet. The lowest head used by Francis was over 0.5 foot. For very low heads the crest lip should be thinner. A wooden crest tends, by capillary attraction, to cause the nappe to adhere to the flat top surface under low heads. A wooden crest is cheap, easily adjusted, and convenient for temporary use, but it will, in time, tend to become somewhat rounded, reducing the vertical contraction of the nappe.

A cast-iron crest will usually have to be made to order. A large steel angle bar may often be obtained from stock sizes of the rolling mills more cheaply. Such a bar, with legs, say 3 and 6 inches, respectively, with the 6-inch flat face planed and its edge trued, will form a rigid and permanent crest. The 3-inch leg may be bolted to the top of the timbers forming the body of the weir.

It may be added that approximate corrections for rounding of upstream corner of the crest, inclination of the weir upstream or downstream, or incomplete contractions can be made from data now available. In constructing gaging weirs preference, however, should be given to those forms which render the determination of the discharge the most simple, and the extent to which the preceding specifications may be departed from judiciously will depend upon the exigencies of the case and the purposes for which the results are desired.

PLANK AND BEAM WEIRS OF SENSIBLE CREST WIDTH.

Experiments on weirs with crest boards 1, 2, or 4 inches in thickness were made by Blackwell, Fteley and Stearns, and Bazin. The results show that for depths exceeding 1.5 to 2 times the crest width the nappe will break free, and if properly aerated the coefficient will then be identical with that for a thin-edged weir.

When the nappe adheres to the crest the coefficients are very uncertain for such weirs, adhesion of nappe to downstream face of crest and modified aeration entering to give divergent values.

The precise stage at which the change from an adhering to a free nappe or the reverse occurs is not constant, but varies with velocity of approach and with rate of change of the head as the changing point is approached, being different for a sudden and for a gradual change, and also when the point of change is approached by an increasing as compared with a decreasing head.

REDUCTION OF THE MEAN OF SEVERAL OBSERVATIONS OF HEAD.

In measuring a constant volume of water, several observations of the head on the weir are desirable, the accuracy of the result, according to the theory of least squares, being proportional to the square root of the number of observations.

In weir experiments it is often impossible to maintain a perfectly uniform head or regimen. If the variations are minute the arithmetical mean may be used directly. If the variations are of wider range, or if the utmost precision is required, the following correction formula of Francis may be applied:[a]

Let D_1, D_2, D_3, etc., D_n represent the several successive observed heads.

Let t_1, t_2, t_3, etc., t_n represent the corresponding intervals of time between the several observations.

Let T represent their sum, or the total time interval.

Q=the total volume of water flowing over the weir in the time T.

D=the mean depth on the weir that would discharge the quantity Q in the time T.

L=the length of weir crest.

C=the weir coefficient.

We have, very nearly,

$$Q=\frac{t_1}{2}\ CLD_1^{\frac{3}{2}}+\frac{t_1+t_2}{2}\ CLD_2^{\frac{3}{2}}+\frac{t_2+t_3}{2}\ CLD_3^{\frac{3}{2}}+\text{etc.}+\frac{t_n}{2}\ CLD_n^{\frac{3}{2}}$$

Also,

$$Q=TCLD^{\frac{3}{2}}.$$

Equating, eliminating the common factor CL, and solving for D, we have

$$D=\left\{\frac{1}{T}\left(\frac{t_1}{2}\ D_1^{\frac{3}{2}}+\frac{t_1+t_2}{2}D_2^{\frac{3}{2}}+\frac{t_n+t_3}{2}D_3^{\frac{3}{2}}+\text{etc.}+\frac{t_n}{2}D_n^{\frac{3}{2}}\right)\right\}^{\frac{2}{3}} \qquad (48)$$

EFFECT OF ERROR IN DETERMINING THE HEAD ON WEIRS.[b]

Consider the formula

$$Q=CLH^{\frac{3}{2}}.$$

Differentiating, we have

$$dQ=\frac{3}{2}\ CL\sqrt{H}\,dH.$$

The error of any gaging when $H+dH$ is taken as the head instead of the true head H being used will be dQ, and the ratio of this quantity to the true discharge Q will be

$$\frac{dQ}{Q}=\frac{3CL\sqrt{H}}{2CLH^{\frac{3}{2}}}dH=\frac{3}{2}\frac{dH}{H} \quad . \quad . \quad . \quad . \quad . \qquad (49)$$

This formula will give nearly the correct value of the error if the increment dH approaches an infinitesimal.

[a] Francis, J. B., Lowell Hydraulic Experiments, p. 113.

[b] Rafter, G. W., On the flow of water over dams: Trans. Am. Soc. C. E., vol. 44, p. 686; data here given based on discussion by Walter C. Parmley.

In the following table is shown the effect of errors of one thousandth, five thousandths, one hundredth, and five hundredths foot, respectively, for various heads. This clearly illustrates both the necessity of proper care and the folly of ultra precision in measuring the relatively large values of H with which we are mainly concerned. The curves of error on Pl. II are equilateral hyperbolas, which have been reduced to straight lines by plotting on logarithmic scales.

Percentage error in discharge resulting from various errors in the measured head on weirs.

Head, in feet.	Error in measured head, in feet.			
	0.001	0.005	0.01	0.05
	Per cent.	*Per cent.*	*Per cent.*	*Per cent.*
0.1	1.5	7.5	15	....
.5	.3	1.5	3	15
1.0	.15	.75	1.5	7.5
5.0	.03	.15	.3	1.5
10.0	.015	.075	.15	.75

An error of a half-tenth foot under 5 feet head causes the same error in the result as an error of one-half hundredth foot with a head of one-half foot.

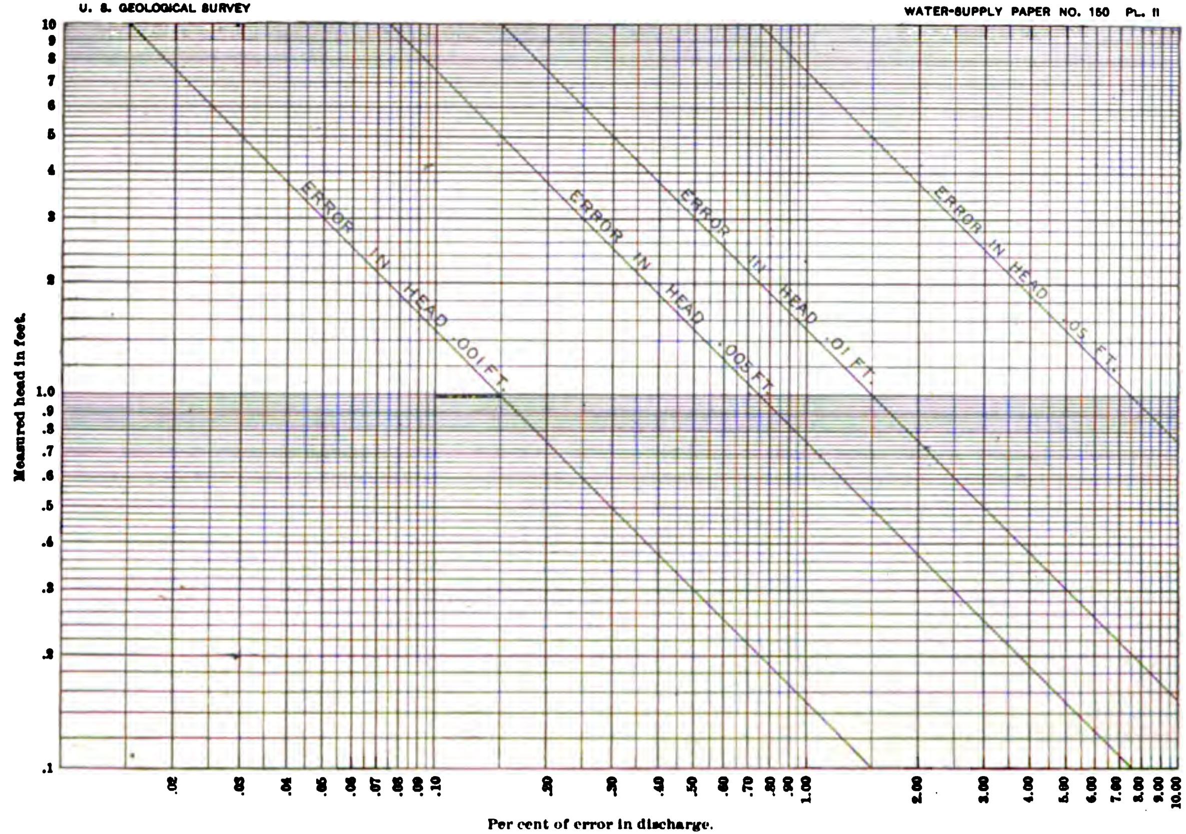
In weir experiments it is important to know the effect of an error in head H on the resultant coefficient of discharge C. The error in C is evidently equivalent to the error in Q found above, where H is constant.

ERROR OF THE MEAN WHERE THE HEAD VARIES.

In determining the volume of flow over dams where gaging records are kept, the method usually pursued has been to have readings taken twice daily, as at morning and evening, showing the depth flowing over the crest of the dam. The average of the two readings for each day has been found and the volume of flow corresponding to this average head has been taken as the mean rate of flow over the dam for the day.

It is evident, however, that as the discharge varies more rapidly than the head (usually considered to be proportional to the three-halves power of the head), the volume of discharge obtained as above described will be somewhat less than the amount which actually passes over the dam. The following analysis has been made to show the magnitude of the error introduced by using the above method.

Assuming that the initial depth on the crest of the dam is zero, but increases at a uniform rate to H_1 at the end of a time interval T, the



EFFECT OF ERRORS IN WEIR MEASUREMENTS.

mean head deduced from observations at the beginning and end of the period would be $\frac{1}{2}$ H_1, the head at any time t would be

$$H=ft$$

where f is a constant.

We may write the usual formula for weir discharge $Q=CLH^{\frac{3}{2}}$; then, if the head varies from zero to H_1, the total volume of flow in the time T will be

$$Q_t=\int_0^T Qdt=CLf^{\frac{3}{2}}\int_0^T t^{\frac{3}{2}}dt=\frac{2}{5}\ CLf^{\frac{3}{2}}T^{\frac{5}{2}}\ .\ .\ .\ (50)$$

The total discharge corresponding to the average head $\frac{1}{2}$ H_1 is

$$Q_{av}=CL\left(\frac{H_1}{2}\right)^{\frac{3}{2}}T=CL\left(\frac{f}{2}\right)^{\frac{3}{2}}T^{\frac{5}{2}}\ .\ .\ .\ .\ (51)$$

The ratio of the discharge is

$$\frac{\text{Volume by average head}}{\text{Actual volume}}=\frac{Q_{av}}{Q_t}=\frac{\left(\frac{1}{2}\right)^{\frac{3}{2}}}{\frac{2}{5}}=0.8840\ .\ .\ (52)$$

It appears that where the initial or terminal head is zero the volume of flow determined by using the average head will be 11.6 per cent too small. This percentage of error is the same whatever may be the maximum head H, and whether the stream is rising or falling. It is also independent of the rate of change in the head.

Conditions like those above discussed occur at milldams during the season of low water, when the pond is allowed to fill up at night and the water is drawn down to crest level or below during the day when mills are running.

The following example will illustrate. Suppose a sharp-crested weir without end contractions, with crest 1 foot long, on which the water rises to a depth of 1 foot in a period of 10 seconds—

Mean depth on a weir with varying head.

Time, in seconds	1	2	3	4	5	6	7	8	9	10
Head, in feet, at end of each second	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
Average head for period	.05	.1	.15	.2	.25	.3	.35	.4	.45	.5
Average head, second to second	.05	.15	.25	.35	.45	.55	.65	.75	.85	.95

Using the average head during each second the volume of flow may be approximately integrated by finite differences, as follows, the discharge being taken from Francis's tables:

Discharge over a weir with varying head.

Time, in seconds.	Average head, in feet.	Discharge, in second-feet.
0 to 1	0.05	0.037
1 to 2	.15	.194
2 to 3	.25	.416
3 to 4	.35	.690
4 to 5	.45	1.005
5 to 6	.55	1.358
6 to 7	.65	1.745
7 to 8	.75	2.163
8 to 9	.85	2.609
9 to 10	.95	3.083
Total	....	13.30

The average head for the entire period, 0.5 foot, gives a discharge for 10 seconds of 11.773 second-feet, or 88.5 per cent of that given above, the numerical result agreeing closely with that obtained by analysis. The volume of flow from average head equals seven-eighths of the true integral volume of flow, approximately.

If there is an initial head H_0, then when the head varies uniformly,

$$H = H_0 + ft$$
$$Q = CLH^{\frac{3}{2}} = CL\,(H_0 + ft)^{\frac{3}{2}}.$$

The total volume of flow in time T will be

$$Q_t = \int_0^T Q dt = CL \int_0^T (H_0 + ft)^{\frac{3}{2}}\, dt = \frac{2}{5f} CL \left(H_0 + fT\right)^{\frac{5}{2}} - \frac{2}{5f}\, CLH_0^{\frac{5}{2}}.$$

The average head during time T is

$$H_{av} = H_0 + \frac{1}{2} f T$$

The total volume of flow corresponding to this head is

$$Q_{av} = CL \left(H_0 + \frac{1}{2} fT\right)^{\frac{3}{2}} T$$

The ratio of the actual or integral discharge to the discharge by the average head is

$$\frac{\text{Volume by average head}}{\text{Integral volume}}=\frac{5}{2}f\frac{\left(H_0+\frac{1}{2}fT\right)^{\frac{3}{2}}T}{\left[\left(H_0+fT\right)^{\frac{5}{2}}-H_0^{\frac{5}{2}}\right]} \quad . \quad (53)$$

The value of this ratio is independent of the coefficient or length of weir, but varies with the rate of change of head.

WEIR NOT LEVEL.

If the crest of a gaging weir is not truly horizontal, but is a little inclined, the discharge may be closely approximated by the use of the average crest depth H in the ordinary formula, or more precisely by the formula below, applicable also to weirs of any inclination.

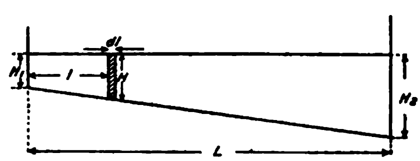


FIG. 7.—Inclined weir

The flow through the elementary width dl is

$$dQ=CH^{\frac{3}{2}}dl$$

$$H=H_1+\frac{H_2-H_1}{L}l$$

$$\text{Total discharge}=Q=\int_0^L CH^{\frac{3}{2}}\,dl=C\int_0^L\left(H_1+\frac{H_2-H_1}{L}\,l\right)^{\frac{3}{2}}dl$$

Integrating,

$$Q=\frac{2CL}{5\left(H_2-H_1\right)}\left(H_2^{\frac{5}{2}}-H_1^{\frac{5}{2}}\right) \quad . \quad . \quad . \quad . \quad . \quad (54)$$

In this formula either the mean coefficient deduced by Thomson (see p. 46) for a triangular weir, in which $\frac{2}{5}C=1.32$, or that of Francis, in which $\frac{2}{5}C=1.332$, may be used. If there are end contractions, the net length,

$$L=L'-0.2\left(\frac{H_2+H_1}{2}\right),$$

should be used.

The discharge using the average head,

$$H_a=\frac{H_2+H_1}{2},$$

is

$$Q=CL\left(\frac{H_2+H_1}{2}\right)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (55)$$

The extent of variation from the true discharge resulting from the use of formula (55) in place of the integral formula (54) is illustrated by the following:

Let $L=10$ feet, $H_a=1.0$ foot, $\frac{2}{5}\ C=1.332$.

Discharge by (55) for average head$=33.30$ cubic feet per second.
If $H_2-H_1=0.01$ foot—true discharge, $Q=33.30$ cubic feet per second.
If $H_2-H_1=0.10$ foot—true discharge, $Q=33.30$ cubic feet per second.
If $H_2-H_1=0.50$ foot—true discharge, $Q=33.54$ cubic feet per second.

In general, since the discharge varies more rapidly than the head, the effect of calculating the discharge from the average head will be to give too small discharge, the error increasing with the variation in crest level.

Hence the discharge obtained by using the average crest level for a weir having an inclined or uneven crest will be somewhat deficient. The magnitude of the variations in height of the crest will determine whether the average profile can be used or whether the crest should be subdivided into sections, each comprising portions having very nearly the same elevation (whether adjacent or not), and the discharge over each section computed as for a separate weir.

In general it may be stated that the error in the value of Q_1 increases directly in proportion as the ratio of the difference in the limiting heads to the average head is increased.

CONVEXITY OF WATER SURFACE IN LEADING CHANNEL.

If there are wide variations in velocity in the measuring section, the level of the water surface may be affected, since water in motion exerts less pressure than when at rest.

Conditions of equilibrium cause the swift-moving current to rise above the level of the slower-moving portions. If the head is measured near still water at the shore, the result may be slightly too small.

The difference in height[a] may be expressed in the form,

$$D_2-D_1=r\frac{v_2^2-v_1^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (56)$$

The coefficient r is often assumed equal to unity, but evidently varies with the distribution of velocities whose resultant effect it measures.

[a] Humphreys and Abbot, Physics and Hydraulics of the Mississippi River, 1876, p. 320.

RESULTS OF EXPERIMENTS ON VARIOUS FORMS OF WEIR CROSS SECTIONS.

THE USE OF WEIRS OF IRREGULAR SECTION.

Many cases arise where it is desired to estimate, approximately, at least, the flow over dams of peculiar cross section.

The construction of so-called standard or thin-edged weirs that shall be permanently useful to measure the flow of large and variable streams is so difficult and expensive as to be frequently impracticable. Existing milldams often afford a convenient substitute. In the following pages are presented the results of the leading experiments to determine proper coefficients for "irregular" weirs, followed by a grouping of experiments on similar models, whether all by one experimenter or not. The data are not always as complete or consistent as could be desired, but the need for fair working coefficients is very great, and, in the line of making use of all the available information, the several diagrams of comparison and the conclusions therefrom are presented, with the understanding that these are not final, although it is quite certain that the laws of coefficient variation are correctly outlined by the data at present available, and they form, therefore, a safe working hypothesis.

Weir models of irregular section are calibrated in order that existing dams of similar cross section may be used for stream gaging. It becomes necessary to calibrate the experimental models for a wider range of heads than has commonly been employed in experiments on standard thin-edged weirs, in order that the range of rise and fall of the stream from low water to high may be included.

While the recent experimental data include heads as great as from 4 to 6 feet, yet it is often necessary to determine the discharge for still greater heads, and experiments on certain forms with heads up to 10 or 12 feet are needed.

In this connection the greater relative facility of securing accurate results with weirs for high than for low heads may be noted.

The proportional error resulting from variations in crest level, as well as uncertainties as to the nappe form and consequent value of the coefficient, largely disappear as the head increases. The effect of form of crest and friction is also relatively diminished. It is probably true that the coefficients for many ordinary forms of weir section would tend toward a common constant value if the head were indefinitely increased. The above facts render milldams especially useful for the determination of the maximum discharge of streams. Dams can be used for this purpose when the presence of logs and drift carried down by the flood prechude the use of current meters or other gaging instruments.

MODIFICATIONS OF THE NAPPE FORM.

The elaborate investigations of Bazin relative to the physics of weir discharge set forth clearly the importance of taking into consideration the particular form assumed by the nappe. This is especially true in weirs of irregular section in which there is usually more opportunity for change of form than for a thin-edged weir. In general the nappe may—

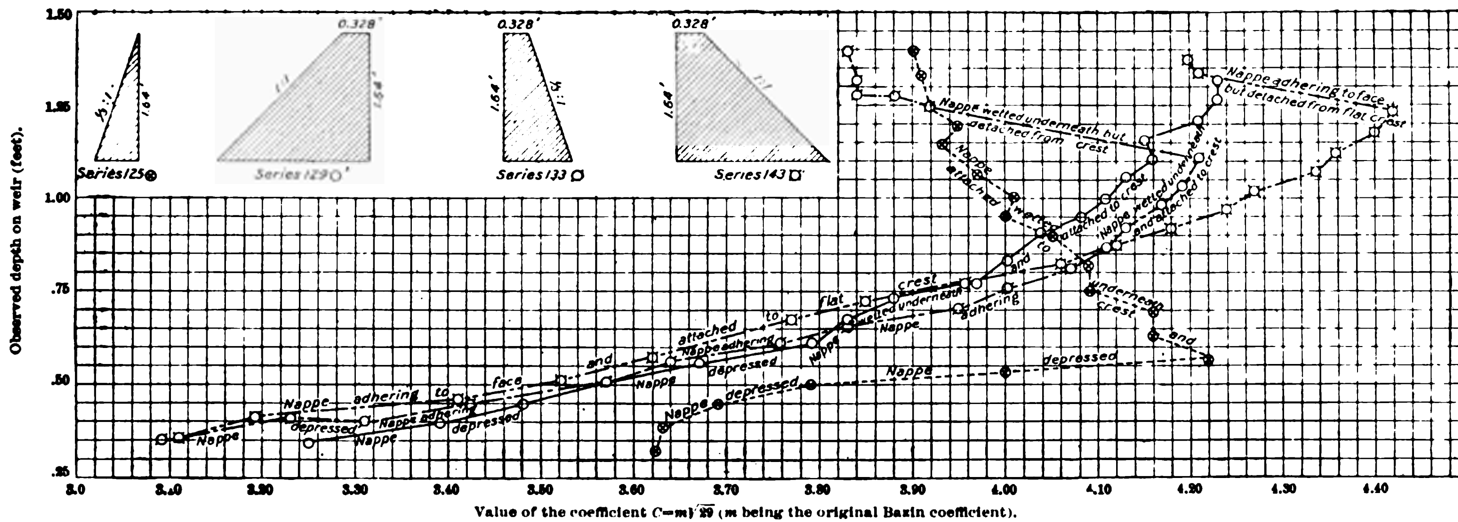
1. Discharge freely, touching only the upstream crest edge.
2. Adhere to top of crest.
3. Adhere to downstream face of crest.
4. Adhere to both top and downstream face.
5. Remain detached, but become wetted underneath.
6. Adhere to top, but remain detached from face and become wetted underneath.
7. In any of the cases where the nappe is "wetted underneath" this condition may be replaced by a depressed nappe, having air imprisoned underneath at less than atmospheric pressure.

The nappe may undergo several of these modifications in succession as the head is varied. The successive forms that appear with an increasing stage may differ from those pertaining to similar stages with a decreasing head. The head at which the changes of nappe form occur vary with the rate of change of head, whether increasing or decreasing, and with other conditions.

The law of coefficients may be greatly modified or even reversed when a change of form takes place in the nappe.

The effect of modifications of nappe form on various irregular weir sections is shown in Pl. III. The coefficients are those of Bazin and include velocity of approach. The coefficient curve for any form of weir having a stable nappe is a continuous, smooth line. When the nappe becomes depressed, detached, or wetted underneath during the progress of an experiment, the resulting coefficient curve may consist of a series of discontinuous or even disconnected arcs terminating abruptly in "*points d'arrêt*," where the form of nappe changes. The modifications of nappe form are usually confined to comparatively low heads, the nappe sometimes undergoing several successive changes as the head increases from zero until a stable condition is reached beyond which further increase of head produces no change. The condition of the nappe when depressed or wetted underneath can usually be restored to that of free discharge by providing adequate aeration.

The weir sections shown in Pl. III are unusually susceptible of changes of nappe form. Among weirs of irregular section there is a large class for which, from the nature of their section, the nappe can assume only one form unless drowned. Such weirs, it is suggested, may, if properly calibrated, equal or exceed the usefulness of the thin-edged



VARIATIONS IN WEIR COEFFICIENT WITH CHANGE OF NAPPE FORM.

weir for purposes of stream gaging, because of their greater stability of section and because the thin-edged weir is not free from modification of nappe form for low heads.

As an example, Bazin gives the following coefficients applying to a thin-edged weir 2.46 feet high, with a head of 0.656 foot, under various conditions:

Condition of nappe.	Bazin coefficient m.	$C=m\sqrt{2g}$	Per cent of the Francis coefficient.
Free discharge, full aeration	0.433	3.47	104.1
Nappe depressed, partial vacuum underneath	.460	3.69	110.7
Nappe wetted underneath, downstream water level, 0.42 foot below crest	.497	3.99	119.7
Nappe adhering to downstream face of weir, ressault at a distance	.554	4.45	133.5

These coefficients include velocity of approach effect, which tends to magnify their differences somewhat. There is, however, a range of 25 per cent variation in discharge between the extremes.[a]

The departure in the weir coefficient from that applying to a thin-edged weir, for most forms of weirs of irregular section, results from some permanent modification of the nappe form. Weirs with sloping upstream faces reduce the crest contraction, broad-crested weirs cause adherence of the nappe to the crest, aprons cause permanent adherence of the nappe to the downstream face.

EXPERIMENTAL DATA FOR WEIRS OF IRREGULAR CROSS SECTION.

The only experiments on irregular or broad-crested weirs in which the discharge has been determined volumetrically are those of Blackwell on weirs 3 feet broad, of Francis on the Merrimac dam, and of the United States Geological Survey for lower heads, on various forms of section. So far as the writer is aware, all other such experiments have been made by comparison with standard weirs.

In the following pages are included the results of the experiments of Bazin on 29 forms of cross section; also those of the United States Deep Waterways Board under the direction of George W. Rafter, and those of John R. Freeman at Cornell University hydraulic laboratory. The results of 20 series of experiments, chiefly on weirs with broad and ogee crest sections, made under the writer's direction at Cornell University hydraulic laboratory, are here for the first time published.

[a] Bazin's general discussion of the above and other modifications of the coefficient has been translated by the writer, and may be found in Rafter's paper, On the flow of water over dams: Trans. Am. Soc. C. E., vol. 44, pp. 254-261.

As it has been necessary to reduce the experimental data to a uniform basis for purposes of comparison the original data, together with the results obtained by recalculation, have been included for the Bazin, United States Deep Waterways, and Freeman experiments.

BASE FORMULA FOR DISCHARGE OVER WEIRS OF IRREGULAR CROSS SECTION.

Precedent to the opening of the hydraulic laboratory of Cornell University the most elaborate experiments on weirs of irregular cross section were those of Bazin. His experiments were all reduced in such manner as to include the velocity of approach correction in the discharge coefficient.

In America the formula most commonly used is that adopted by James B. Francis, in which velocity of approach is eliminated from the coefficient by correcting the head, thus reducing the conditions as nearly as possible to the basis of no velocity of approach before applying the formula.

In order to render Bazin's results comparable with the later experiments, it has been necessary to adopt a standard or base formula to which all the experiments should be reduced. The considerations leading to the adoption of the formula of Francis here used are given below.

In the process of gaging streams at dams the head is usually measured in comparatively still water in an open pond. This condition could not be duplicated in the Cornell experiments. As the formula of James B. Francis is most simple in form for the case of a weir with no velocity of approach, and as it is often convenient to compare the discharge over a dam with that for a thin-edged weir of standard form, a weir formula of the base form used by Francis has been adopted in reducing the experiments. In this formula,

$$Q=CLH^{\frac{3}{2}}.$$

L=Length of crest corrected for end contractions, if any.

H=Head on weir crest corrected for velocity of approach by the Francis correction formula or an equivalent method.

C=A coefficient determined from experiments on a model dam.

In this connection it may be remarked that the formula of Bazin includes the correction for velocity of approach in the weir coefficient; hence the coefficient for a given weir is comparable only with that for another weir under the same head when the velocity of approach is the same in both cases. Bazin's formula also expresses the velocity of approach implicitly by means of the depth and breadth of the leading channel. In actual gagings the leading channel is often of irregular form, hence it becomes necessary to eliminate the depth and breadth of the channel from the formula.

There is considerable variation in the magnitude of the correction for velocity of approach used by different experimenters. As a rule, the velocity of approach is negligible at gaging stations at dams. It became necessary, therefore, in reducing these experiments to determine from the measured discharge and observed head what the head would have been had the same discharge taken place over a weir in a still pond. To accomplish this the formula for correction for velocity of approach adopted by James B. Francis has been used. This being the case, it is to be noted that in applying the coefficients, which, as given, have been reduced as nearly as possible to the basis of no velocity of approach, the same method of velocity correction must be used, and if it is used no error will result where the actual velocity of approach is nearly the same as that which occurred in the experiments.

BAZIN'S EXPERIMENTS ON WEIRS OF IRREGULAR CROSS SECTION.

These include a wide variety of forms, many of which will seldom be found in America, and the use of which for purposes of gaging would be ill advised.

The small size of the models used, high velocity of approach, and narrow range of heads covered, limit the application of these results. No effort has been made to present all the results in this paper.[a] Certain series, useful for comparison, have been recomputed as described below, and by grouping similar sections we may determine the general effect of various slope and crest modifications.

BAZIN'S CORRECTION FOR VELOCITY OF APPROACH.

The base formula for weir discharge adopted by Bazin and the method of taking into account the velocity of approach are described in connection with his experiments on thin-edged weirs (p. 31).

The following discussion shows the complex character of the Bazin coefficients, and the fact that they do not express directly the relative discharging capacity of weirs of irregular section.

The effect of velocity of approach is to increase the discharge at a given observed head, *D*, over what it would be if the same head were measured in still water, as in a deep, broad pond.

Bazin's coefficients in the form published are not readily applicable in practice to weirs of other heights, or to weirs in ponds, or otherwise to any but weirs in restricted channels of the depth and width of the weir.

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[a] For complete original data, see Bazin, as translated by Marichal and Trautwine in Proc. Engineers Club Phila., vol. 7, pp. 259-310; vol. 9, pp. 231-244, 287-319; vol. 10, pp. 121-164; also numerous experiments reduced to English units by Rafter and others, Trans. Am. Soc. C. E., vol. 44, pp. 220-398.

The Bazin coefficients as published may be considered as comprising two principal factors. M being the Bazin coefficient, we may write

$M = FC$.

F = velocity of approach effect.

C = contraction effect.

Bazin uses a correction formula for velocity of approach, derived from the expression

$$H = D + 1.69 \frac{v^2}{2g} \text{ or } D + \alpha h.$$

Consider a standard weir and experimental weir both of the same height, but of different form, the measured depth being the same, and the Bazin coefficients being M and m, the velocity of approach and discharge V and v and Q and q, respectively, and C and C_1 the coefficients in a formula in which the velocity of approach correction is eliminated from the coefficient and applied to the head; then the discharge for the standard weir would be,

using the Bazin coefficients,

$$Q = MLD\sqrt{2gD} = M'D^{\frac{3}{2}},$$

where $M' = M\sqrt{2g}$ and $L = 1.0$;

using the coefficient C,

$$Q = CH^{\frac{3}{2}} = C(D + \alpha h)^{\frac{3}{2}},$$

taking roots

$$\left(\frac{C}{M'}\right)^{\frac{2}{3}} = \frac{D}{D + \alpha h}.$$

Bazin does not give the quantities of flow in the tables of results of his experiments, hence to determine h it is necessary to calculate Q. v, and h from the known values M and D and from P, the height of weir.

D being the same for both the standard and the experimental weirs, we have for the experimental weir

$$\left(\frac{C_1}{m}\right)^{\frac{2}{3}} = \frac{D}{D + \alpha h_1},$$

C_1 being the coefficient for the experimental weir, and h_1 the velocity head.

Hence, by multiplication,

$$\left(\frac{m}{M}\right)^{\frac{2}{3}}\left(\frac{C}{C_1}\right)^{\frac{2}{3}}=\frac{D}{D+\alpha h}\times\frac{D+\alpha h_1}{D},$$

and

$$\left(\frac{m}{M}\right)^{\frac{2}{3}}=\left(\frac{C_1}{C}\right)^{\frac{2}{3}}\times\frac{D+\alpha h_1}{D+\alpha h},$$

or

$$\frac{m}{M}=\frac{C_1}{C}\times\left(\frac{D+\alpha h_1}{D+\alpha h}\right)^{\frac{3}{2}}.$$

The velocity of approach for a given depth on a weir is proportional to C, hence, since h is proportional to v^2, we have

$$\frac{h_1}{h}=\left(\frac{C_1}{C}\right)^2.$$

Hence,

$$\frac{m}{M}=\frac{C_1}{C}\left(\frac{D+\left(\frac{C_1}{C}\right)^2\alpha h}{D+\alpha h}\right)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (57)$$

The ratio $\frac{m}{M}$ used by Bazin is not, therefore, precisely a measure of the relative discharging capacities of the two weirs under similar conditions of head and velocity of approach, for the reason that the velocity of approach will not be the same for both weirs if the Bazin coefficients are different. The ratio m/M is made up of two factors, one of which, C_1/C, expresses the absolute relative discharging capacities of the two weirs under similar conditions of head and velocity of approach, and the other expresses the effect of the change in discharging capacity on the velocity of approach for a given depth on a weir of given height.

Thus the coefficient M for any weir has, by Bazin's method of reduction, different values for every depth and for every height of weir that may occur.

For reasons elsewhere stated it is preferred to express by $\frac{C}{C_1}$ only the relative discharging capacities of the weirs where the velocity of approach is the same in both. It is then practically a measure of the vertical contraction of the nappe, and is constant for a given head for any height of weir, and may be sensibly constant for various depths on the weir.

RECOMPUTATION OF COEFFICIENTS IN BAZIN'S EXPERIMENTS.

In reporting the results of his experiments on weirs of irregular section, Bazin gives the observed heads on the standard weir of comparison, the absolute coefficient m applying for each depth on the experimental weir and the ratio m/M of the experimental and standard weir coefficients.

The results give coefficients which strictly apply only to weirs having both the same form of section and the same heights as those of Bazin. Although weirs of *sectional form* geometrically similar to Bazin's are common, yet few actual weirs have the same height as his. There appear to be two elements which may render inaccurate the application of Bazin's absolute coefficients to weirs of varying height: (1) The difference in velocity of approach; (2) the difference in contraction of the nappe for a higher or lower weir.

In order to render the results of Bazin's experiments comparable one with another and with later experiments, a number of series have been recomputed, the velocity of approach being treated in the same manner as in the computation of experiments at Cornell hydraulic laboratory.

The method is outlined below, the references being to the tables of Bazin's experiments given on pages 68 to 81.

Column 2 gives the observed head reduced to feet for the experimental weir.

Column 4 the absolute coefficient $C_1 = m\sqrt{2g}$.

(These have been reduced from Bazin's original tables.)

Column 5 gives the discharge per foot of crest over the experimental weir calculated by the formula

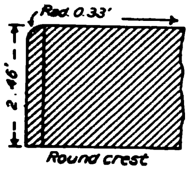
$$Q = mLD\sqrt{2gD} = C_1LD^{\frac{3}{2}},$$

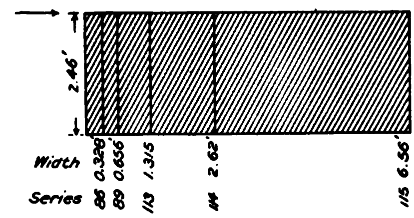
quantities in column 3 being taken directly from a table of three-halves powers.

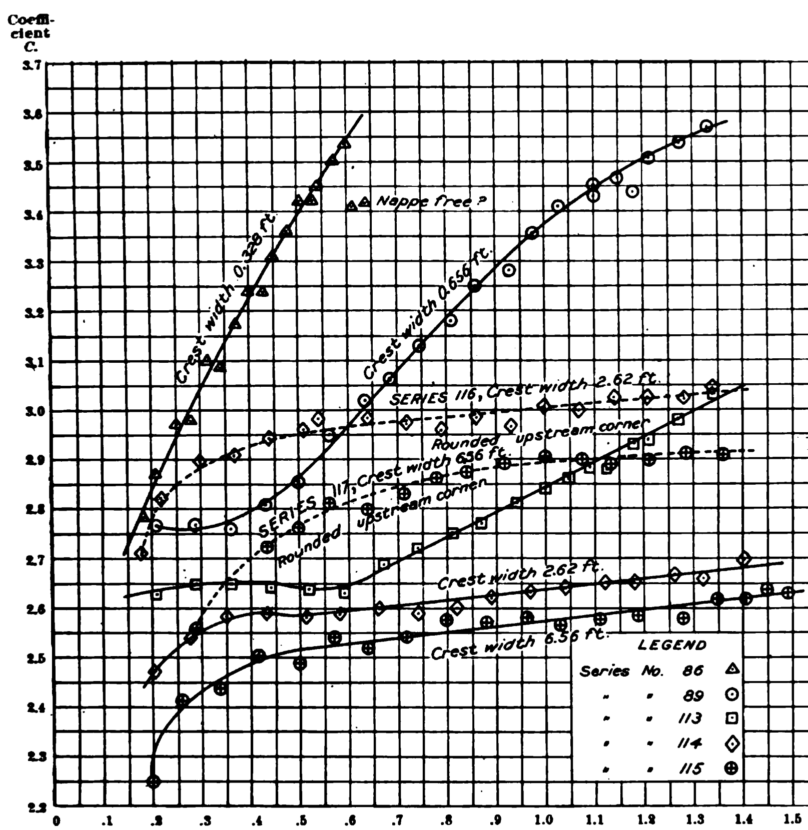
In column 6 the actual velocity of approach, $v = \frac{Q}{P+D}$, is given, and in column 7 the velocity head, $h = \frac{v^2}{2g}$.

The discharge over the standard weir was calculated by Bazin by using his own formula and velocity of approach correction. He does not give the discharge, however, and we have been obliged to work back and obtain it from the data given for the experimental weir.

Having determined the actual discharge and the observed head, we are now at liberty to assume such a law of velocity of approach correction in deducing our new coefficients as we choose. We will therefore deduce the coefficients in such form that when applied to a weir

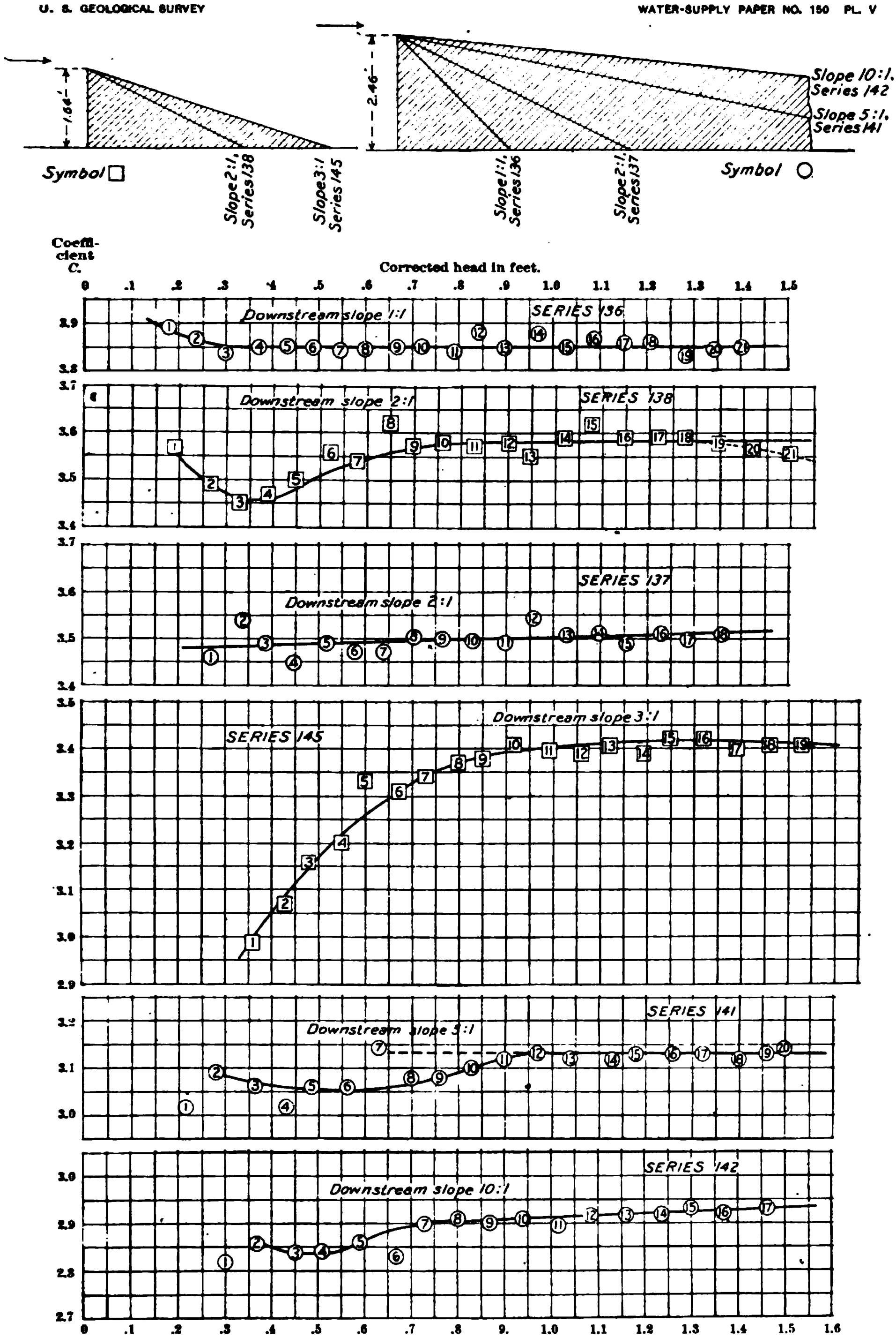






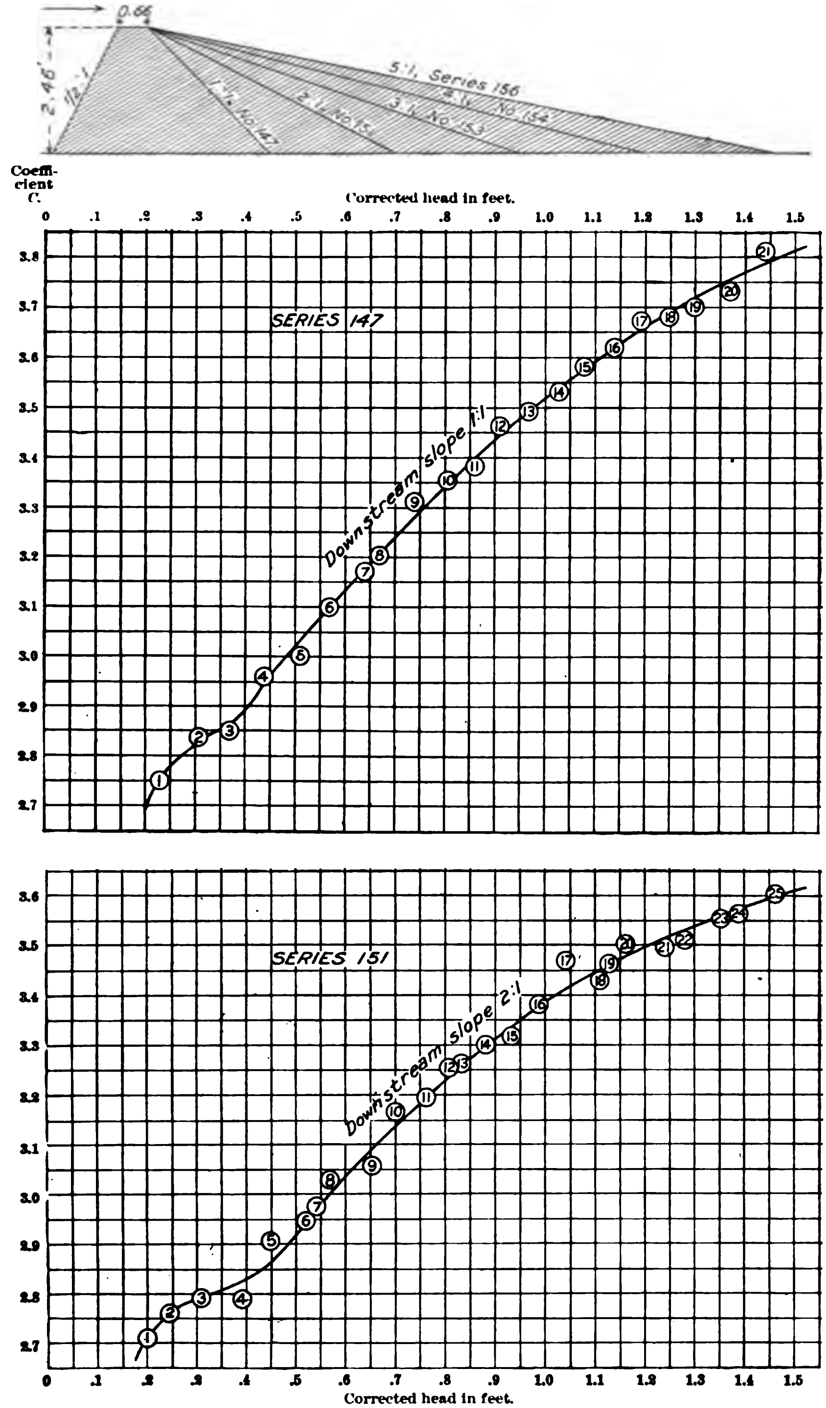
EXPERIMENTS OF BAZIN ON BROAD-CRESTED WEIRS.

Velocity-of-approach correction by the Francis method.



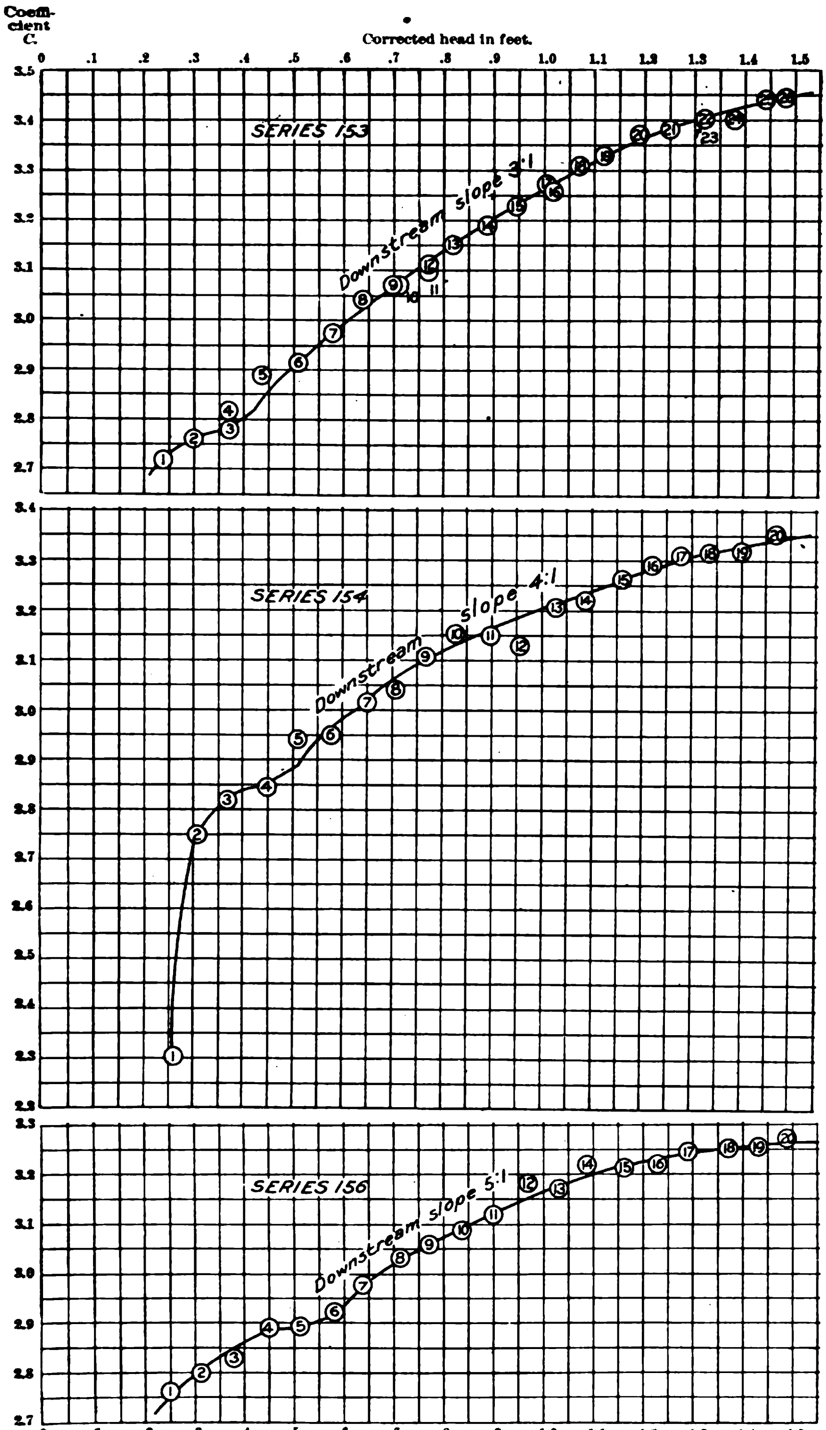
EXPERIMENTS OF BAZIN ON WEIRS OF TRIANGULAR SECTION WITH VARYING DOWNSTREAM SLOPE.

Velocity-of-approach correction by the Francis method.



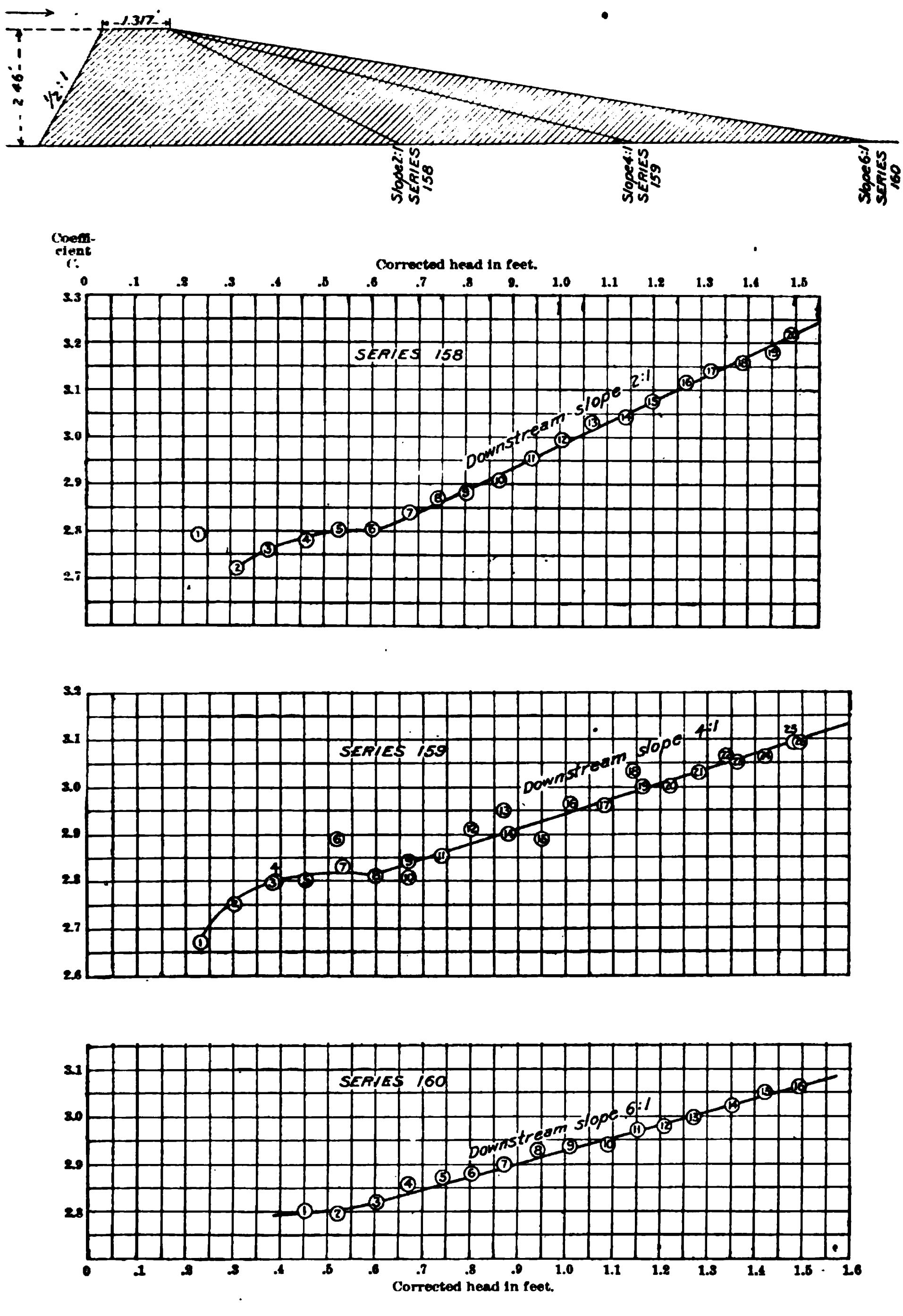
EXPERIMENTS OF BAZIN ON WEIRS OF TRAPEZOIDAL SECTION WITH VARYING DOWNSTREAM SLOPE.

Velocity-of-approach correction by the Francis method. (See also Pl. VII.)



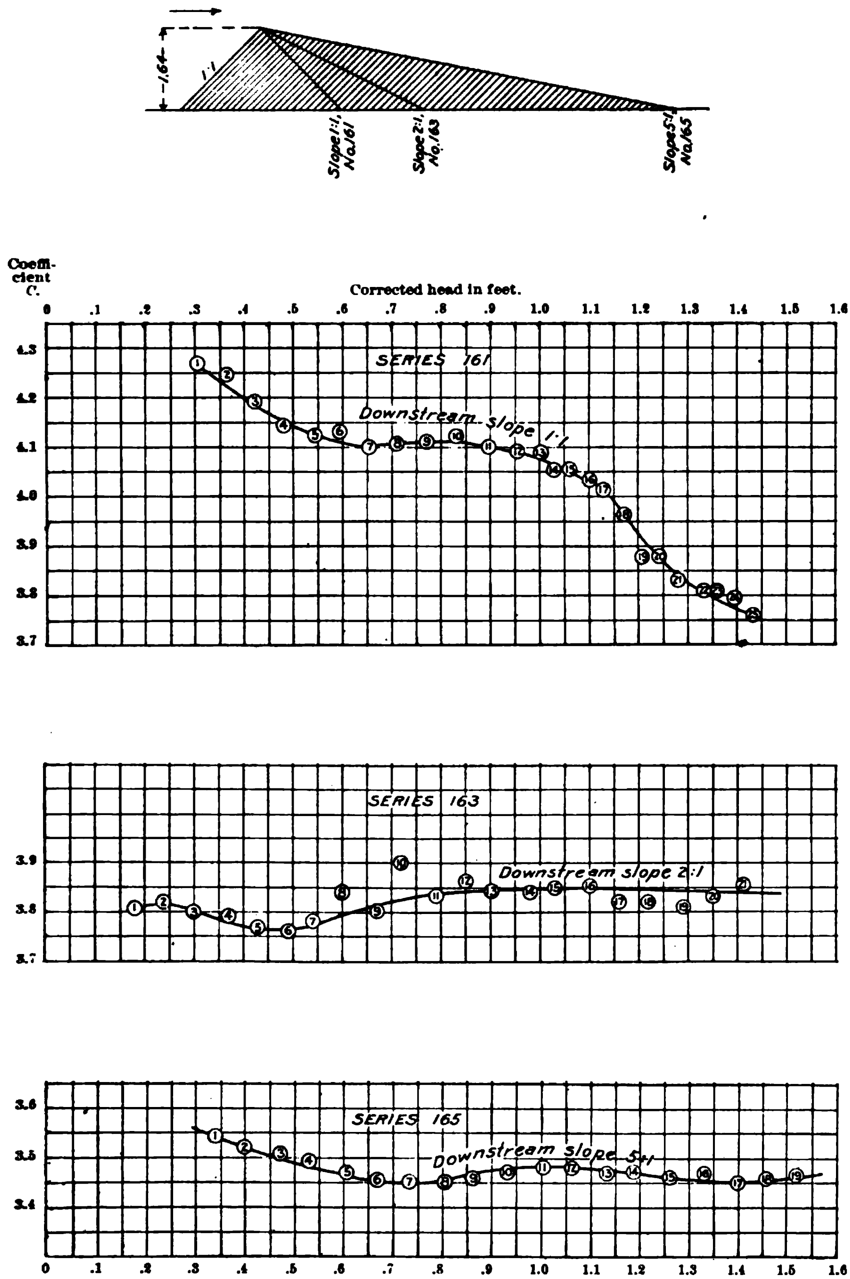
EXPERIMENTS OF BAZIN ON WEIRS OF TRAPEZOIDAL SECTION WITH VARYING DOWNSTREAM SLOPE.

Velocity-of-approach correction by the Francis method. (For cross section see Pl. VI.)



EXPERIMENTS OF BAZIN ON WEIRS OF TRAPEZOIDAL SECTION WITH VARYING DOWNSTREAM SLOPE.

Velocity-of-approach correction by the Francis method.



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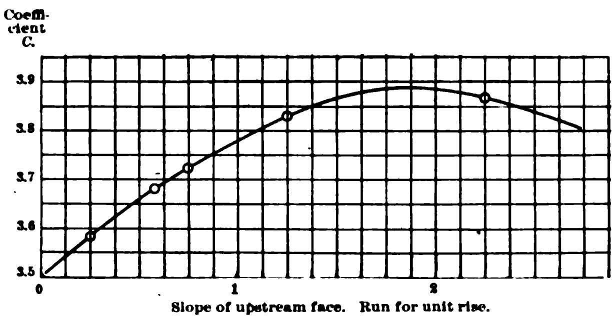
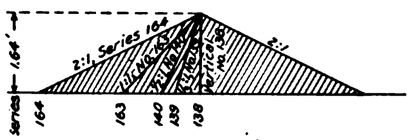
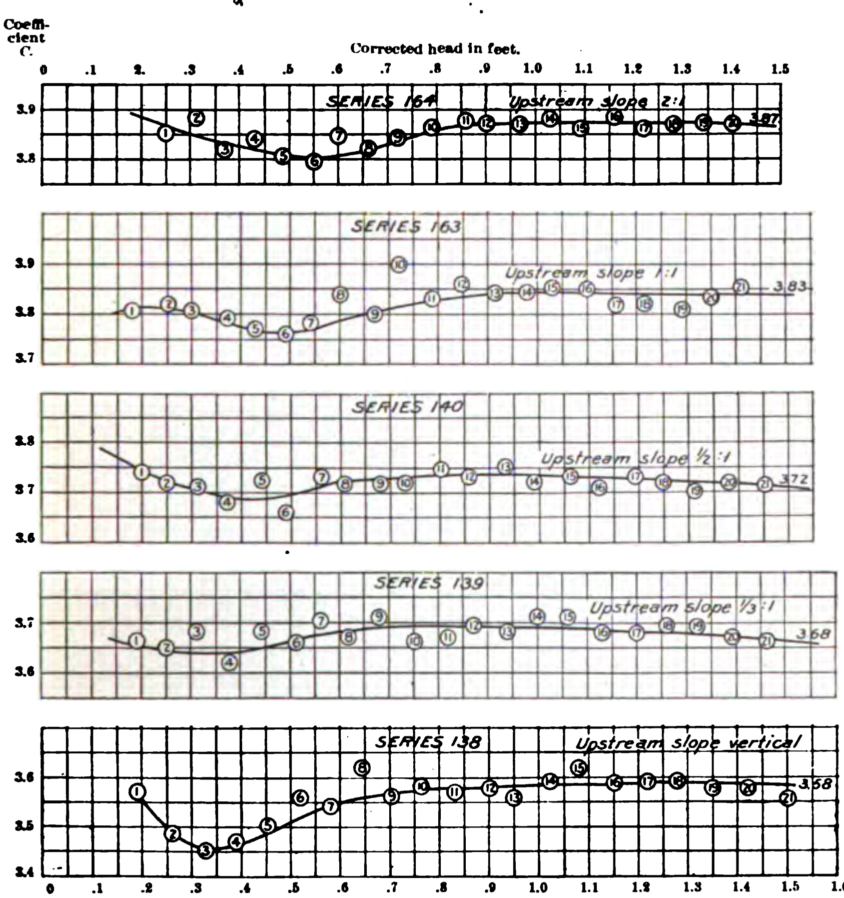
Velocity-of-approach correction by the Francis method.



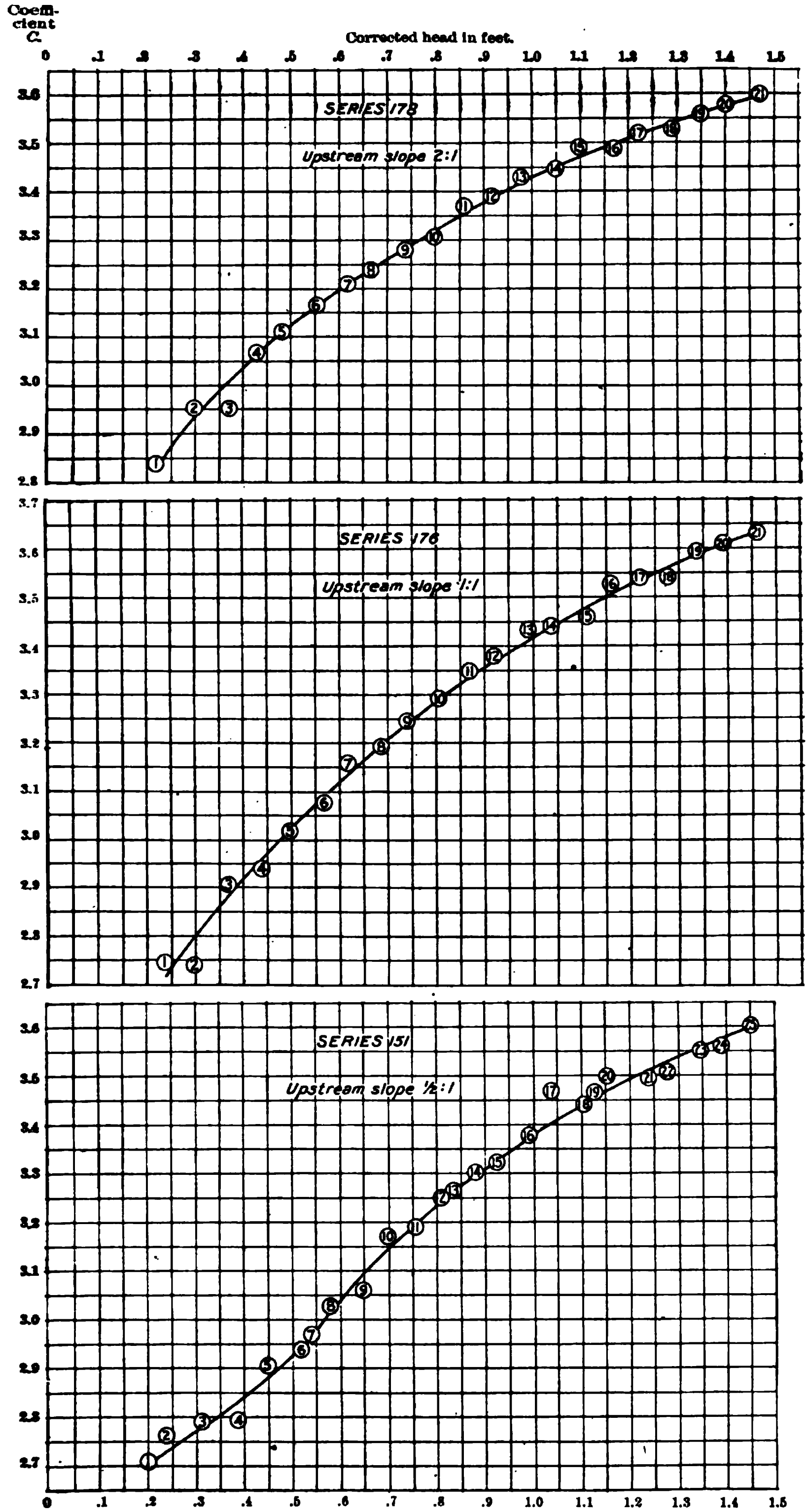
MEAN CONSTANT COEFFICIENTS FOR VARYING SLOPE OF UPSTREAM FACE.



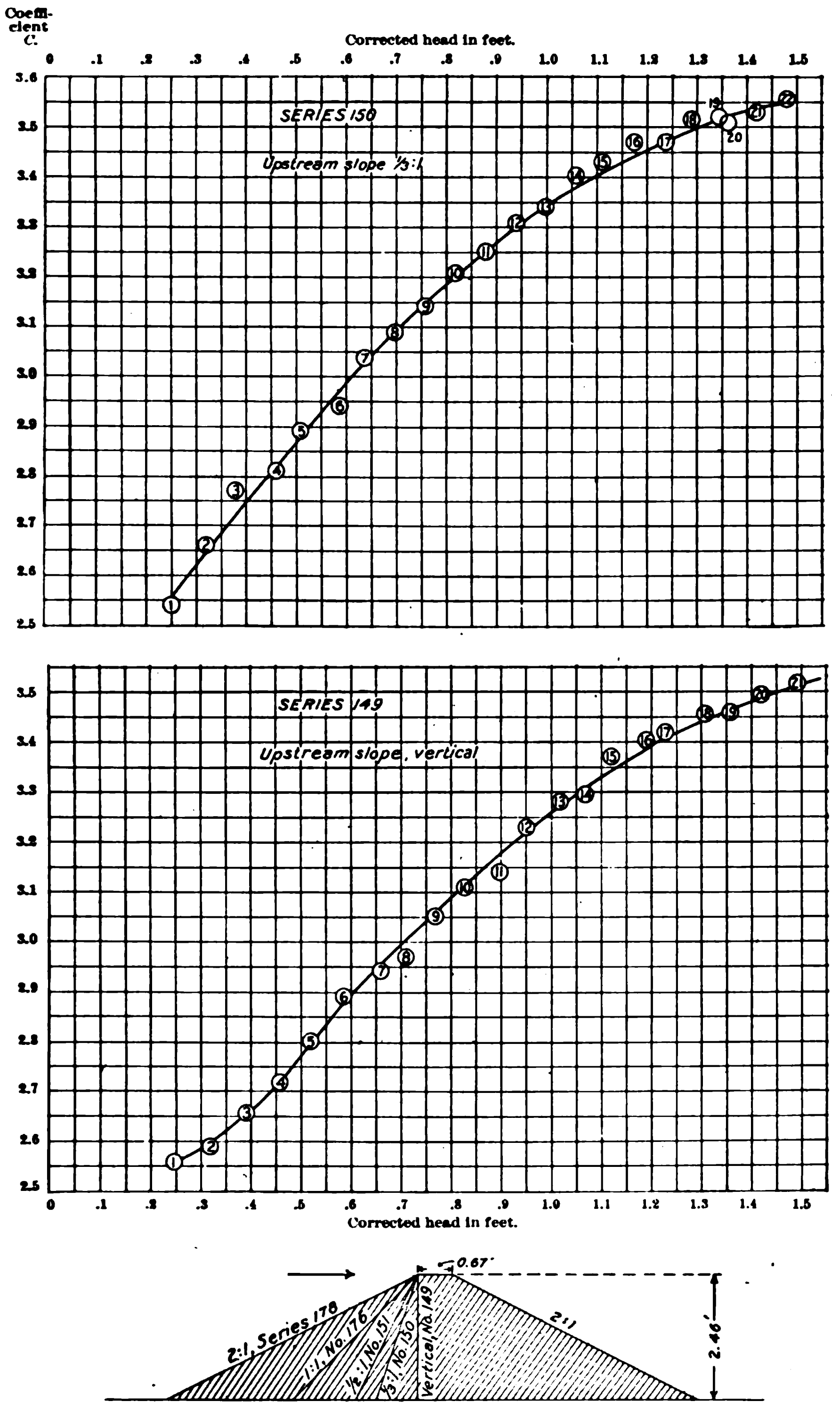


EXPERIMENTS OF BAZIN ON WEIRS OF TRIANGULAR SECTION WITH VARYING UPSTREAM SLOPE.

Velocity-of-approach correction by the Francis method.



EXPERIMENTS OF BAZIN ON WEIRS OF TRAPEZOIDAL SECTION WITH VARYING UPSTREAM SLOPE.



EXPERIMENTS OF BAZIN ON WEIRS OF TRAPEZOIDAL SECTION WITH VARYING UPSTREAM SLOPE.

Velocity-of-approach correction by the Francis method. (See also Pl. XI.)

in which there is velocity of approach we may apply the correction formula of Francis,

$$H=\left[(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}\right]^{\frac{2}{3}}.$$

A sufficient approximation to this formula for our present purposes may be obtained if we simply make

$$H=D+\frac{v^2}{2g},$$

where v is the velocity of approach corresponding to the trial discharge for the head D, no successive approximations being made, as would be necessary to determine the true head H by the Francis correction formula.

For example, in an extreme case, using a thin-edged weir

$$D=1.0, \quad P=1.0, \quad v \text{ (approx.)} \frac{Q}{P+D}=\frac{3.33}{2}=1.665$$

$$h=\frac{v^2}{2g}=0.0431 \quad \text{whence} \quad H=D+\frac{v^2}{2g}=1.0431,$$

and $Q=3.547$.

By the Francis correction formula we find, using three successive approximations,

$Q_1=3.5183$ giving $v=1.7591$
$Q_2=3.5387$ giving $v=1.7694$
$Q_3=3.541$ as the final discharge,

that the difference is 0.11 of 1 per cent. We are therefore justified in using this method to determine values of C to two places decimals, or to within one-fourth to one-half per cent.

We have also used $\sqrt{2g}=8.02$, as in the reduction of the Cornell experiments.

Column 8 gives the corrected head,

$$H=D+h.$$

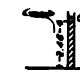
Column 10 gives the final coefficient C deduced by the formula

$$C=\frac{Q}{H^{\frac{3}{2}}}.$$

Pls. IV to XII show the resulting discharge coefficients.

Bazin's experiments on weirs of irregular section.

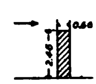
Bazin's Series, No. 86.
Crest length, 6.55 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.1820	0.0777	2.7829	0.2160	0.082	....	0.1820	0.0777	2.78
2	.2119	.0976	2.8712	.2801	.105	....	.2119	.0976	2.87
3	.2509	.1257	2.9674	.3733	.138	....	.2509	.1257	2.97
4	.2781	.1466	2.9754	.4369	.159	....	.2781	.1466	2.98
5	.3067	.1701	3.0957	.5273	.190	0.0006	.3073	.1701	3.10
6	.3392	.1974	3.0957	.6119	.218	.0008	.3400	.1983	3.09
7	.3678	.2232	3.1839	.7098	.251	.0010	.3688	.2241	3.17
8	.4016	.2549	3.2321	.8233	.288	.0013	.4029	.2539	3.24
9	.4251	.2771	3.2641	.9033	.303	.0014	.4265	.2786	3.24
10	.4527	.3049	3.3283	1.0153	.350	.0019	.4546	.3069	3.31
11	.4770	.3294	3.3844	1.1137	.379	.0022	.4792	.3315	3.36
12	.5075	.3616	3.4406	1.2449	.420	.0027	.5102	.3642	3.42
13	.5360	.3924	3.4807	1.3656	.455	.0033	.5393	.3957	3.45
14	.5639	.4235	3.5368	1.4992	.496	.0039	.5678	.4280	3.50
15	.5973	.4613	3.5930	1.6561	.542	.0045	.6018	.4671	3.54
16	a.5304	.3858	3.4566	1.3349	.446	.0031	.5335	.3897	3.42
17	a.6032	.4683	3.4486	1.6156	.528	.0044	.6076	.4740	3.41
18	a.6347	.5060	3.4646	1.7508	.566	.0051	.6398	.5120	3.42

Bazin's Series, No. 89.
Crest length, 6.55 feet.
Crest height, 2.46 feet.



Cross section.

1	0.2079	0.0948	2.7669	0.2626	0.098	....	0.2079	0.0948	2.77
2	.2873	.1538	2.7669	.4260	.155	....	.2873	.1538	2.77
3	.3641	.2196	2.7669	.6083	.216	0.0008	.3649	.2205	2.76
4	.4337	.2859	2.8230	.8062	.279	.0012	.4349	.2869	2.81
5	.4963	.3494	2.8792	1.0063	.340	.0018	.4981	.3515	2.86
6	.5619	.4213	2.9674	1.2513	.414	.0026	.5635	.4235	2.95
7	.6331	.5036	3.0476	1.5360	.498	.0039	.6370	.5084	3.02
8	.6890	.57195	3.0877	1.7673	.500	.0049	.6939	.5782	3.06
9	.7490	.6482	3.1679	2.0548	.640	.0064	.7554	.6561	3.13
10	.7985	.7135	3.2160	2.2975	.705	.0078	.8063	.7236	3.18
11	.8546	.7906	3.2962	2.6090	.785	.0097	.8643	.8031	3.25
12	.9228	.8867	3.3524	2.9704	.879	.0120	.9348	.9041	3.28
13	.9648	.9479	3.4326	3.2513	.949	.0140	.9788	.9687	3.36
14	1.0236	1.0362	3.4887	3.6163	1.038	.0168	1.0404	1.0606	3.41
15	1.0784	1.1193	3.5288	3.9511	1.118	.0195	1.0979	1.1505	3.43
16	1.1312	1.2028	3.5849	4.3060	1.201	.0224	1.1536	1.2396	3.47
17	1.1866	1.2932	3.6331	4.6943	1.292	.0259	1.2125	1.3359	3.51
18	1.2375	1.3767	3.6732	5.0525	1.364	.0288	1.2663	1.4245	3.54
19	1.2959	1.4754	3.7052	5.4737	1.456	.0331	1.3290	1.5321	3.57
20	a1.0807	1.1239	3.5368	3.9786	1.122	.0195	1.1002	1.1537	3.45
21	a1.1587	1.2477	3.5448	4.4169	1.219	.0231	1.1818	1.2850	3.44

a Nappe free from the crest.

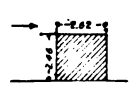
Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 113.
Crest height, 2.463 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.208	0.0948	2.64	0.2503	....	....	0.208	0.09484	2.63
2	.289	.1554	2.66	.4134	....	....	.289	.1562	2.65
3	.363	.2187	2.66	.5817	0.206	0.0007	.3637	.2196	2.65
4	.443	.2949	2.65	.7815	.269	.0011	.4441	.296	2.64
5	.518	.3728	2.66	.9916	.332	.0017	.5197	.375	2.64
6	.592	.4555	2.64	1.2025	.392	.0024	.5944	.4578	2.63
7	.667	.5447	2.71	1.4761	.472	.0034	.6704	.549	2.69
8	.736	.6314	2.75	1.7364	.542	.0045	.7405	.6377	2.72
9	.805	.7223	2.78	2.0080	.612	.0058	.8108	.7303	2.75
10	.863	.8017	2.80	2.2441	.672	.0070	.8700	.8115	2.77
11	.936	.9056	2.85	2.5810	.758	.0090	.9450	.91865	2.81
12	.989	.9835	2.88	2.8325	.821	.0105	.9995	.9925	2.84
13	1.055	1.0530	2.91	3.0641	.872	.0118	1.0468	1.068	2.86
14	1.076	1.1162	2.94	3.2816	.925	.0132	1.0892	1.1364	2.88
15	1.114	1.1758	2.95	3.4686	.968	.0143	1.1283	1.1980	2.88
16	1.159	1.2478	3.00	3.7434	1.032	.0165	1.1755	1.274	2.93
17	1.197	1.3096	3.01	3.9419	1.075	.0179	1.2149	1.339	2.94
18	1.252	1.4009	3.06	4.2868	1.154	.0206	1.2726	1.436	2.98
19	1.320	1.5166	3.11	4.7166	1.246	.0243	1.3443	1.558	3.03

Bazin's Series, No. 114.
Crest height, 2.46 feet.

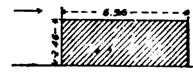


Cross section.

1	0.204	0.0921	2.47	0.2275	0.056	....	0.204	0.0921	2.47
2	.280	.1482	2.54	.3764	.137	....	.280	.1482	2.54
3	.352	.2089	2.59	.5411	.193	0.0006	.3526	.2097	2.58
4	.433	.28497	2.60	.7409	.256	.0011	.4341	.2860	2.59
5	.504	.3578	2.59	.9267	.318	.0015	.5055	.3594	2.58
6	.578	.4394	2.60	1.1424	.376	.0022	.5802	.4417	2.59
7	.657	.5325	2.62	1.3952	.446	.0031	.6601	.5362	2.60
8	.735	.6302	2.63	1.6511	.517	.0042	.7392	.6353	2.59
9	.810	.7290	2.63	1.9173	.587	.0054	.8154	.7358	2.60
10	.882	.8283	2.65	2.1950	.656	.0068	.8888	.8381	2.62
11	.958	.9377	2.66	2.4943	.728	.0083	.9663	.9494	2.63
12	1.034	1.0514	2.68	2.8178	.806	.0102	1.0442	1.0667	2.64
13	1.112	1.1727	2.69	3.1546	.883	.0120	1.1240	1.1917	2.65
14	1.171	1.2672	2.70	3.4214	.941	.0137	1.1847	1.2899	2.65
15	1.243	1.3858	2.73	3.7832	1.021	.0161	1.2591	1.4127	2.67
16	1.301	1.48[illegible]	2.73	4.0510	1.078	.0181	1.3191	1.5149	2.66
17	1.384	1.6282	2.76	4.4938	1.168	.0213	1.4053	1.6654	2.70

Bazin's experiments on weirs of irregular section—Continued.

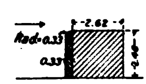
Bazin's Series, No. 115.
Crest height, 2.46 **feet.**



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.196	0.0868	2.25	0.1953	0.073	....	0.196	0.0868	2.25
2	.264	.1357	2.41	.3270	.120	....	.264	.1357	2.41
3	.342	.2001	2.45	.4902	.175	0.0005	.3425	.2005	2.44
4	.415	.2674	2.51	.6712	.233	.0008	.4158	.2683	2.50
5	.495	.3483	2.50	.8708	.290	.0013	.4963	.3494	2.49
6	.566	.4258	2.55	1.0858	.358	.0020	.5680	.4281	2.54
7	.638	.5096	2.54	1.2944	.418	.0027	.6407	.5132	2.52
8	.716	.6059	2.56	1.5511	.487	.0037	.7197	.6109	2.54
9	.792	.7049	2.60	1.8327	.563	.0049	.7969	.7115	2.58
10	.871	.8129	2.60	2.1135	.634	.0062	.8782	.8227	2.57
11	.948	.9230	2.60	2.4098	.706	.0078	.9558	.9347	2.58
12	1.023	1.0347	2.61	2.7006	.775	.0095	1.0325	1.0491	2.57
13	1.097	1.1490	2.63	3.0219	.849	.0112	1.1089	1.1679	2.58
14	1.178	1.2786	2.64	3.3755	.928	.0134	1.1914	1.2997	2.59
15	1.260	1.4144	2.65	3.7482	1.009	.0159	1.276	1.4414	2.58
16	1.330	1.5338	2.68	4.1106	1.085	.0181	1.348	1.5651	2.62
17	1.388	1.6353	2.69	4.3990	1.144	.0202	1.408	1.6707	2.62
18	1.424	1.6993	2.70	4.5881	1.18	.0216	1.446	1.7388	2.64
19	1.467	1.7768	2.70	4.797	1.26	.0247	1.492	1.8225	2.63

Bazin's Series, No. 116.
Crest height, 2.46 **feet.**

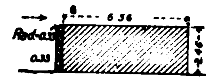


Cross section.

1	0.177	0.0745	2.71	0.2019	0.076	....	0.177	0.0745	2.71
2	.225	.1068	2.83	.3022	.112	....	.225	.1068	2.83
3	.296	.1611	2.90	.4672	.169	....	.296	.1611	2.90
4	.367	.2224	2.92	.6494	.229	0.0008	.3678	.2232	2.91
5	.435	.2870	2.95	.8467	.292	.0013	.4363	.2879	2.94
6	.504	.3578	2.98	1.0662	.360	.0020	.5060	.3599	2.96
7	.537	.3935	2.99	1.1776	.392	.0024	.5394	.3957	2.98
8	.639	.5108	3.01	1.5375	.497	.0039	.6429	.5156	2.98
9	.713	.6021	3.00	1.8063	.569	.0051	.7181	.6084	2.97
10	.781	.6902	3.00	2.0706	.640	.0064	.7874	.6982	2.96
11	.849	.7823	3.02	2.3625	.713	.0078	.8568	.7933	2.98
12	.917	.8781	3.02	2.6519	.793	.0097	.9267	.8925	2.97
13	.986	.9791	3.05	2.9863	.864	.0115	.9975	.9963	3.00
14	1.053	1.0805	3.06	3.3063	.942	.0137	1.0667	1.1021	3.00
15	1.120	1.1853	3.08	3.6507	1.019	.0162	1.1362	1.2108	3.02
16	1.185	1.28995	3.09	3.9859	1.092	.0185	1.2035	1.3203	3.02
17	1.251	1.3992	3.10	4.3375	1.169	.0213	1.2723	1.4346	3.02
18	1.317	1.5114	3.12	4.7156	1.250	.0243	1.3413	1.5529	3.04

Bazin's experiments on weirs of irregular section—Continued.

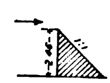
Bazin's Series, No. 117.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.158	0.06282	2.19	0.1376	0.043	....	0.158	0.0628	2.19
2	.204	.09212	2.64	.2432	.091	....	.204	.0921	2.64
3	.289	.1554	2.57	.3994	.145	....	.289	.1554	2.56
4	.361	.21691	2.65	.5126	.182	0.0005	.3615	.2178	2.35
5	.426	.27808	2.73	.7592	.263	.0011	.4271	.2791	2.72
6	.494	.34724	2.77	.9619	.321	.0016	.4956	.3493	2.76
7	.562	.42134	2.83	1.1924	.395	.0025	.5645	.4247	2.81
8	.635	.5060	2.82	1.4267	.461	.0033	.6383	.5096	2.80
9	.708	.59578	2.86	1.7049	.539	.0045	.7125	.6020	2.83
10	.771	.67702	2.89	1.9566	.606	.0058	.7768	.6849	2.86
11	.834	.76168	2.91	2.2155	.674	.0070	.8410	.7713	2.87
12	.912	.87096	2.93	2.5519	.758	.0090	.9210	.8839	2.89
13	.989	.98352	2.95	2.9014	.841	.0110	1.0000	1.0000	2.90
14	1.064	1.0975	2.95	3.2376	.919	.0132	1.0772	1.1177	2.90
15	1.129	1.1996	2.95	3.5388	.987	.0152	1.1442	1.2236	2.89
16	1.197	1.3096	2.97	3.8895	1.062	.0175	1.2145	1.3392	2.90
17	1.267	1.4262	2.98	4.2501	1.139	.0202	1.2872	1.4601	2.91
18	1.336	1.5442	2.99	4.6172	1.214	.0228	1.3588	1.5842	2.91

Bazin's Series, No. 136.
Crest length, 6.519 feet.
Crest height, 2.46 feet.

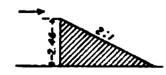


Cross section.

1	0.183	0.0783	3.90	0.305	0.12	0.0002	0.1832	0.0783	3.90
2	.244	.1206	3.86	.467	.17	.0004	.2444	.1206	3.87
3	.304	.1676	3.85	.647	.23	.0008	.3048	.1684	3.84
4	.364	.2196	3.86	.849	.30	.0014	.3654	.2206	3.85
5	.424	.2761	3.88	1.071	.37	.0021	.4261	.2781	3.85
6	.484	.3367	3.87	1.304	.44	.0030	.4870	.3399	3.84
7	.542	.3990	3.88	1.548	.52	.0042	.5462	.4035	3.84
8	.597	.4613	3.89	1.793	.59	.0054	.6024	.4671	3.84
9	.658	.5338	3.91	2.088	.67	.0070	.6650	.5423	3.85
10	.713	.6021	3.92	2.360	.74	.0085	.7215	.6135	3.85
11	.776	.6836	3.93	2.684	.83	.0107	.7867	.6982	3.84
12	.830	.7562	3.97	3.001	.91	.0129	.8427	.7740	3.88
13	.887	.8354	3.96	3.300	.99	.0152	.9022	.8567	3.85
14	.953	.9308	3.98	3.701	1.08	.0181	.9711	.9568	3.87
15	1.010	1.0150	3.97	4.029	1.16	.0209	1.0309	1.0468	3.85
16	1.068	1.1037	4.00	4.417	1.25	.0243	1.0923	1.1411	3.87
17	1.122	1.1885	3.99	4.748	1.33	.0275	1.1495	1.2316	3.86
18	1.179	1.2802	4.01	5.133	1.41	.0309	1.2099	1.3310	3.86
19	1.244	1.3875	4.01	5.564	1.50	.0350	1.2790	1.4446	3.85
20	1.299	1.4805	4.01	5.935	1.58	.0388	1.3378	1.5477	3.84
21	1.361	1.5878	4.03	6.408	1.68	.0439	1.4049	1.6654	3.85

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 137.
Crest length, 6.523 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.268	0.1388	3.47	0.482	0.18	0.0006	0.2686	0.1395	3.46
2	.332	.1974	3.45	.680	.24	.0009	.3329	.1922	3.54
3	.391	.2445	3.50	.856	.30	.0014	.3924	.2454	3.49
4	.451	.3029	3.47	1.051	.36	.0020	.4530	.3049	3.45
5	.513	.3674	3.53	1.295	.44	.0030	.5160	.3707	3.49
6	.578	.4394	3.51	1.540	.51	.0040	.5820	.4440	3.47
7	.637	.5084	3.51	1.783	.57	.0051	.6421	.5144	3.47
8	.700	.5857	3.55	2.080	.66	.0068	.7068	.5945	3.50
9	.765	.6692	3.56	2.382	.74	.0085	.7735	.6797	3.50
10	.822	.7452	3.56	2.652	.81	.0102	.8322	.7589	3.49
11	.887	.8354	3.56	2.973	.89	.0123	.8993	.8524	3.49
12	.946	.9201	3.62	3.334	.98	.0149	.9609	.9420	3.54
13	1.012	1.0180	3.59	3.662	1.05	.0171	1.0291	1.0438	3.51
14	1.078	1.1193	3.61	4.043	1.14	.0202	1.0982	1.1506	3.51
15	1.142	1.2204	3.60	4.392	1.22	.0231	1.1651	1.2575	3.49
16	1.201	1.3162	3.62	4.778	1.30	.0263	1.2273	1.3591	3.52
17	1.262	1.4178	3.62	5.140	1.38	.0296	1.2916	1.4686	3.50
18	1.322	1.5200	3.64	5.533	1.46	.0331	1.3551	1.5773	3.51

Bazin's Series, No. 138.
Crest length, 6.532 feet.
Crest height, 1.64 feet.

Cross section.

1	0.194	0.0854	3.57	0.305	0.17	0.0004	0.1944	0.0854	3.57
2	.263	.1349	3.50	.473	.25	.0010	.2640	.1357	3.48
3	.327	.1870	3.48	.651	.33	.0017	.3287	.1887	3.45
4	.391	.2445	3.50	.858	.42	.0027	.3937	.2473	3.47
5	.447	.2989	3.56	1.064	.50	.0039	.4519	.3039	3.50
6	.510	.3642	3.63	1.321	.61	.0058	.5158	.3706	3.56
7	.571	.4314	3.62	1.560	.70	.0076	.5786	.4405	3.54
8	.626	.4953	3.71	1.838	.81	.0102	.6362	.5072	3.62
9	.685	.5670	3.66	2.075	.89	.0123	.6973	.5820	3.56
10	.745	.6431	3.69	2.373	.99	.0152	.7602	.6626	3.58
11	.807	.7250	3.70	2.683	1.09	.0185	.8255	.7507	3.57
12	.873	.8157	3.72	3.036	1.21	.0228	.8958	.8481	3.58
13	.927	.8926	3.72	3.318	1.29	.0259	.9529	.9303	3.56
14	.992	.9880	3.76	3.715	1.41	.0309	1.0229	1.0347	3.59
15	1.045	1.0683	3.80	4.060	1.51	.0354	1.0804	1.1224	3.62
16	1.110	1.1695	3.78	4.422	1.61	.0403	1.1503	1.2332	3.58
17	1.176	1.2753	3.79	4.851	1.72	.0460	1.2220	1.3508	3.59
18	1.233	1.3691	3.81	5.220	1.82	.0515	1.2845	1.4550	3.59
19	1.289	1.4645	3.82	5.577	1.90	.0561	1.3451	1.5599	3.58
20	1.355	1.5773	3.82	6.036	2.01	.0628	1.4178	1.6885	3.57
21	1.429	1.7082	3.83	6.542	2.13	.0705	1.4995	1.8362	3.56

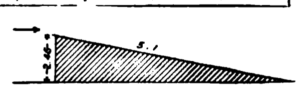
Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 145.
Crest length, 6.541 feet.
Crest height, 1.64 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.359	0.2151	3.02	0.649	0.32	0.0016	0.3606	0.2169	2.99
2	.424	.2761	3.10	.856	.42	.0027	.4267	.2790	3.07
3	.479	.3315	3.18	1.053	.50	.0039	.4809	.3335	3.16
4	.547	.4046	3.25	1.316	.60	.0056	.5526	.4112	3.20
5	.592	.4636	3.35	1.553	.69	.0074	.5994	.4636	3.33
6	.658	.5338	3.38	1.805	.78	.0095	.6675	.5447	3.31
7	.720	.6109	3.42	2.090	.89	.0123	.7323	.6263	3.34
8	.781	.6902	3.47	2.394	.99	.0152	.7962	.7102	3.37
9	.835	.7631	3.49	2.663	1.07	.0178	.8528	.7878	3.38
10	.902	.8567	3.53	3.025	1.19	.0220	.9240	.8882	3.41
11	.962	.9435	3.53	3.332	1.28	.0255	.9875	.9806	3.40
12	1.032	1.0484	3.53	3.707	1.39	.0300	1.0620	1.0944	3.39
13	1.087	1.1333	3.58	4.045	1.48	.0341	1.1211	1.1869	3.41
14	1.152	1.2364	3.58	4.408	1.58	.0388	1.1908	1.2997	3.39
15	1.210	1.3310	3.61	4.801	1.68	.0439	1.2539	1.4042	3.42
16	1.274	1.4380	3.61	5.198	1.78	.0493	1.3233	1.5218	3.42
17	1.334	1.5408	3.62	5.575	1.88	.0549	1.3889	1.6370	3.40
18	1.396	1.6494	3.64	6.006	1.98	.0609	1.4569	1.7586	3.42
19	1.467	1.7768	3.64	6.479	2.09	.0679	1.5349	1.9016	3.41

Bazin's Series, No. 141.
Crest length, 6.520 feet.
Crest height, 2.46 feet.

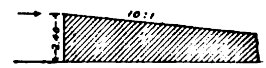


Cross section.

1	0.215	0.0997	3.02	0.301	0.11	0.0002	0.2152	0.0997	3.02
2	.281	.1490	3.09	.460	.17	.0004	.2814	.1490	3.09
3	.355	.2116	3.07	.650	.24	.0009	.3559	.2124	3.06
4	.425	.2771	3.04	.842	.29	.0013	.4263	.2781	3.02
5	.489	.3420	3.08	1.053	.37	.0021	.4911	.3441	3.06
6	.561	.4202	3.08	1.294	.43	.0029	.5639	.4235	3.06
7	.624	.4929	3.17	1.562	.51	.0040	.6280	.4976	3.14
8	.692	.5757	3.11	1.791	.57	.0051	.6971	.5820	3.08
9	.758	.6600	3.12	2.059	.64	.0064	.7644	.6678	3.08
10	.822	.7452	3.15	2.347	.72	.0081	.8301	.7562	3.10
11	.888	.8368	3.17	2.653	.79	.0097	.8977	.8509	3.12
12	.956	.9347	3.19	2.983	.87	.0118	.9678	.9523	3.13
13	1.029	1.0438	3.17	3.309	.95	.0140	1.0430	1.0652	3.12
14	1.113	1.1742	3.20	3.757	1.04	.0168	1.1298	1.2012	3.13
15	1.165	1.2575	3.21	4.045	1.12	.0195	1.1845	1.2884	3.14
16	1.237	1.3758	3.20	4.416	1.19	.0220	1.2590	1.4127	3.13
17	1.298	1.4788	3.22	4.766	1.27	.0251	1.3231	1.5218	3.13
18	1.369	1.6018	3.22	5.152	1.34	.0279	1.3969	1.6511	3.12
19	1.431	1.7118	3.24	5.540	1.42	.0313	1.4623	1.7677	3.13
20	1.463	1.7696	3.25	5.752	1.47	.0336	1.4966	1.8307	3.14

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 142.
Crest length, 6.523 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.300	0.1643	2.83	0.464	0.17	0.0004	0.3004	0.1643	2.82
2	.369	.2242	2.87	.643	.23	.0008	.3698	.2251	2.86
3	.447	.2989	2.87	.851	.29	.0013	.4483	.2999	2.84
4	.509	.3631	2.86	1.038	.35	.0019	.5109	.3652	2.84
5	.591	.4544	2.88	1.308	.43	.0029	.5939	.4578	2.86
6	.666	.5435	2.86	1.554	.50	.0039	.6699	.5484	2.83
7	.727	.6199	2.92	1.810	.57	.0051	.7327	.6263	2.89
8	.795	.7089	2.94	2.084	.64	.0064	.8014	.7155	2.91
9	.861	.7989	2.94	2.349	.71	.0078	.8688	.8101	2.90
10	.934	.9027	2.95	2.664	.78	.0095	.9435	.9158	2.91
11	1.007	1.0105	2.95	2.980	.86	.0115	1.0185	1.0286	2.89
12	1.079	1.1208	2.98	3.338	.94	.0137	1.0927	1.1427	2.92
13	1.149	1.2316	2.98	3.665	1.01	.0159	1.1649	1.2575	2.92
14	1.222	1.3508	2.99	4.037	1.10	.0188	1.2408	1.3824	2.92
15	1.285	1.4567	3.00	4.370	1.17	.0213	1.3063	1.4925	2.93
16	1.362	1.5895	3.00	4.770	1.25	.0243	1.3863	1.6317	2.92
17	1.430	1.7100	3.01	5.147	1.30	.0263	1.4563	1.7569	2.93

Bazin's Series, No. 139.
Crest length, 6.532 feet.
Crest height, 1.64 feet.

Cross section.

1	0.190	0.0828	3.66	0.303	0.17	0.0004	0.1904	0.0828	3.66
2	.253	.1273	3.68	.467	.25	.0010	.2540	.1280	3.65
3	.312	.1743	3.72	.647	.33	.0017	.3137	.1759	3.68
4	.375	.2297	3.66	.841	.42	.0027	.3777	.2323	3.62
5	.434	.2860	3.73	1.067	.52	.0042	.4382	.2899	3.68
6	.500	.3536	3.72	1.317	.62	.0060	.5060	.3600	3.66
7	.552	.4101	3.78	1.550	.71	.0078	.5598	.4191	3.70
8	.615	.4823	3.76	1.812	.80	.0099	.6249	.4941	3.67
9	.667	.5447	3.82	2.081	.90	.0126	.6796	.5607	3.71
10	.733	.6276	3.79	2.380	1.00	.0155	.7485	.6482	3.67
11	.798	.7128	3.80	2.709	1.11	.0192	.8172	.7385	3.67
12	.852	.7865	3.84	3.022	1.21	.0228	.8748	.8185	3.69
13	.915	.8753	3.86	3.378	1.32	.0271	.9421	.9143	3.68
14	.969	.9538	3.87	3.692	1.41	.0309	.9961	.9940	3.71
15	1.023	1.0347	3.92	4.038	1.52	.0359	1.0589	1.0897	3.71
16	1.092	1.1411	3.90	4.446	1.63	.0413	1.1333	1.2060	3.68
17	1.151	1.2348	3.90	4.816	1.72	.0460	1.1970	1.3096	3.68
18	1.210	1.3310	3.94	5.240	1.84	.0526	1.2626	1.4194	3.69
19	1.258	1.4110	3.95	5.570	1.92	.0573	1.3153	1.5080	3.69
20	1.326	1.5269	3.93	6.013	2.03	.0641	1.3901	1.6388	3.67
21	1.394	1.6459	3.93	6.484	2.13	.0705	1.4645	1.7714	3.66

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 140.
Crest length, 6.532 feet.
Crest height, 1.64 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.192	0.0835	3.77	0.315	0.17	0.0004	0.1924	0.0841	3.74
2	.252	.1265	3.74	.473	.25	.0010	.2530	.1273	3.72
3	.308	.1709	3.75	.641	.33	.0017	.3097	.1726	3.71
4	.371	.2260	3.71	.838	.42	.0027	.3727	.2278	3.68
5	.436	.2879	3.77	1.086	.52	.0042	.4402	.2919	3.72
6	.488	.3399	3.74	1.276	.60	.0056	.4936	.3472	3.66
7	.549	.4068	3.81	1.551	.71	.0078	.5568	.4157	3.73
8	.604	.4694	3.82	1.792	.80	.0099	.6139	.4811	3.72
9	.664	.5411	3.83	2.072	.90	.0126	.6766	.5570	3.72
10	.719	.6096	3.84	2.342	.99	.0152	.7342	.6289	3.72
11	.785	.6956	3.88	2.700	1.11	.0192	.8042	.7209	3.74
12	.837	.7658	3.88	2.968	1.20	.0224	.8594	.7961	3.73
13	.905	.8610	3.92	3.375	1.32	.0271	.9321	.8998	3.75
14	.961	.9421	3.90	3.674	1.41	.0309	.9919	.9880	3.72
15	1.023	1.0347	3.95	4.069	1.53	.0364	1.0594	1.0898	3.73
16	1.080	1.1224	3.93	4.402	1.62	.0408	1.1208	1.1869	3.71
17	1.143	1.2220	3.97	4.843	1.74	.0471	1.1901	1.2981	3.73
18	1.195	1.3063	3.96	5.187	1.83	.0521	1.2471	1.3925	3.72
19	1.254	1.4043	3.97	5.558	1.92	.0573	1.3113	1.5011	3.70
20	1.316	1.5097	3.99	6.024	2.03	.0641	1.3801	1.6211	3.72
21	1.375	1.6123	4.01	6.456	2.14	.0712	1.4462	1.7388	3.71

Bazin's Series, No. 147.
Crest length, 6.536 feet.
Crest height, 2.46 feet.

Cross section.

1	0.231	0.1110	2.75	0.305	0.11	0.0002	0.2312	0.1110	2.75
2	.308	.1709	2.85	.485	.18	.0005	.3085	.1709	2.84
3	.373	.2278	2.86	.652	.23	.0008	.3738	.2287	2.85
4	.438	.2899	2.97	.861	.30	.0014	.4394	.2909	2.96
5	.503	.3568	3.02	1.078	.37	.0021	.5051	.3589	3.00
6	.569	.4292	3.13	1.343	.44	.0030	.5720	.4326	3.10
7	.637	.5084	3.20	1.626	.54	.0045	.6415	.5132	3.17
8	.681	.5620	3.24	1.821	.58	.0052	.6862	.5682	3.20
9	.734	.6289	3.34	2.109	.66	.0068	.7408	.6378	3.31
10	.797	.7115	3.40	2.417	.74	.0083	.8053	.7223	3.35
11	.845	.7768	3.44	2.673	.81	.0102	.8552	.7906	3.38
12	.898	.8510	3.53	3.004	.89	.0123	.9103	.8681	3.46
13	.953	.9303	3.57	3.320	.97	.0146	.9676	.9523	3.49
14	1.015	1.0226	3.63	3.703	1.06	.0175	1.0825	1.0484	3.53
15	1.063	1.0960	3.67	4.037	1.15	.0206	1.0836	1.1286	3.58
16	1.115	1.1774	3.73	4.401	1.22	.0231	1.1381	1.2140	3.62
17	1.165	1.2575	3.79	4.775	1.31	.0267	1.1917	1.3013	3.67
18	1.217	1.3426	3.83	5.132	1.40	.0305	1.2475	1.3925	3.68
19	1.265	1.4228	3.85	5.467	1.47	.0336	1.2986	1.4805	3.70
20	1.332	1.5373	3.89	5.991	1.58	.0388	1.3708	1.6053	3.73
21	1.394	1.6459	3.98	6.567	1.68	.0439	1.4379	1.7244	3.81

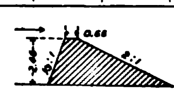
Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 149.
Crest length, 6.518 feet.
Crest height, 2.46 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.248	0.1235	2.55	0.316	0.12	0.0002	0.2482	0.1235	2.56
2	.317	.1785	2.58	.462	.17	.0004	.3174	.1785	2.59
3	.390	.2436	2.67	.651	.23	.0008	.3908	.2445	2.66
4	.455	.3070	2.73	.838	.29	.0013	.4563	.3080	2.72
5	.521	.3761	2.82	1.060	.36	.0020	.5230	.3782	2.80
6	.585	.4475	2.89	1.295	.43	.0030	.5850	.4475	2.89
7	.653	.5277	2.97	1.568	.51	.0040	.6570	.5325	2.94
8	.705	.5920	3.00	1.776	.56	.0049	.7099	.5983	2.97
9	.766	.6705	3.08	2.067	.64	.0064	.7724	.6783	3.05
10	.818	.7398	3.16	2.338	.71	.0078	.8258	.7507	3.11
11	.882	.8283	3.23	2.674	.80	.0099	.8979	.8509	3.14
12	.942	.9143	3.30	3.016	.89	.0123	.9543	.9318	3.23
13	.999	.9985	3.36	3.356	.97	.0146	1.0156	1.0241	3.28
14	1.051	1.0774	3.39	3.627	1.08	.0165	1.0665	1.1006	3.30
15	1.108	1.1584	3.45	4.002	1.12	.0195	1.1225	1.1885	3.37
16	1.165	1.2575	3.49	4.397	1.20	.0224	1.1874	1.2932	3.40
17	1.209	1.3294	3.52	4.682	1.27	.0251	1.2341	1.3708	3.42
18	1.281	1.4499	3.57	5.177	1.38	.0296	1.3106	1.5011	3.45
19	1.330	1.5338	3.60	5.508	1.45	.0327	1.3627	1.5912	3.46
20	1.385	1.6300	3.63	5.917	1.54	.0369	1.4219	1.6956	3.49
21	1.446	1.7388	3.67	6.386	1.64	.0418	1.4878	1.8151	3.52

Bazin's Series, No. 150.
Crest length, 6.518 feet.
Crest height, 2.46 feet.



Cross section.

1	0.248	0.1235	2.53	0.314	0.12	0.0002	0.2482	0.1235	2.54
2	.323	.1836	2.65	.488	.16	.0004	.3234	.1836	2.66
3	.379	.2333	2.78	.648	.23	.0008	.3798	.2342	2.77
4	.459	.3110	2.82	.877	.30	.0014	.4604	.3120	2.81
5	.512	.3664	2.91	1.065	.36	.0020	.5140	.3685	2.89
6	.586	.4486	2.96	1.329	.43	.0029	.5889	.4521	2.94
7	.637	.5084	3.07	1.560	.50	.0039	.6409	.5132	3.04
8	.698	.5832	3.12	1.819	.57	.0051	.7031	.5895	3.09
9	.751	.6508	3.18	2.070	.64	.0064	.7574	.6587	3.14
10	.814	.7344	3.26	2.393	.73	.0083	.8223	.7452	3.21
11	.869	.8101	3.31	2.681	.80	.0099	.8789	.8241	3.25
12	.928	.8940	3.37	3.013	.89	.0123	.9403	.9114	3.31
13	.982	.9732	3.42	3.328	.97	.0146	.9966	.9955	3.34
14	1.043	1.0652	3.47	3.713	1.06	.0175	1.0605	1.0913	3.40
15	1.095	1.1459	3.51	4.037	1.13	.0199	1.1149	1.1774	3.43
16	1.152	1.2364	3.56	4.414	1.22	.0231	1.1751	1.2737	3.46
17	1.215	1.3393	3.58	4.797	1.30	.0263	1.2413	1.3825	3.47
18	1.259	1.4127	3.63	5.118	1.38	.0296	1.2856	1.4584	3.51
19	1.315	1.5080	3.65	5.512	1.46	.0331	1.3481	1.5651	3.52
20	1.323	1.5218	3.65	5.548	1.47	.0336	1.3566	1.5807	3.51
21	1.380	1.6211	3.68	5.962	1.55	.0374	1.4174	1.6868	3.54
22	1.439	1.7262	3.73	6.416	1.64	.0418	1.4808	1.8023	3.56

Bazin's experiments on weirs of irregular section—Continued.

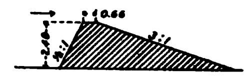
Bazin's Series, No. 151.
Crest length, 6.550 feet.
Crest height, 2.46 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.201	0.0901	2.71	0.244	0.09	0.0001	0.2011	0.0901	2.71
2	.240	.1176	2.81	.329	.12	.0002	.2422	.1191	2.76
3	.307	.1701	2.79	.474	.17	.0004	.3074	.1701	2.79
4	.391	.2445	2.79	.684	.24	.0009	.3919	.2454	2.79
5	.445	.2969	2.92	.867	.30	.0014	.4464	.2979	2.91
6	.514	.3685	2.95	1.089	.37	.0021	.5161	.3707	2.94
7	.537	.3935	2.98	1.174	.39	.0024	.5394	.3957	2.97
8	.573	.4337	3.05	1.324	.43	.0030	.5760	.4371	3.03
9	.643	.5156	3.09	1.594	.51	.0040	.6470	.5204	3.06
10	.695	.5795	3.20	1.856	.59	.0054	.7004	.5857	3.17
11	.756	.6574	3.24	2.129	.66	.0068	.7628	.6665	3.19
12	.800	.7155	3.30	2.362	.72	.0081	.8081	.7263	3.25
13	.826	.7507	3.31	2.486	.76	.0090	.8350	.7631	3.26
14	.867	.8073	3.36	2.712	.81	.0102	.8772	.8213	3.30
15	.921	.8839	3.39	2.997	.89	.0123	.9333	.9013	3.32
16	.975	.9628	3.46	3.332	.97	.0146	.9896	.9850	3.38
17	1.027	1.0408	3.51	3.653	1.04	.0168	1.0438	1.0667	3.47
18	1.090	1.1380	3.52	4.013	1.13	.0199	1.1099	1.1695	3.43
19	1.112	1.1727	3.57	4.177	1.17	.0213	1.1333	1.2060	3.46
20	1.140	1.2172	3.60	4.382	1.22	.0231	1.1631	1.2543	3.50
21	1.209	1.3294	3.61	4.801	1.31	.0267	1.2357	1.3741	3.49
22	1.248	1.3942	3.64	5.060	1.36	.0288	1.2768	1.4431	3.51
23	1.314	1.5063	3.68	5.557	1.47	.0336	1.3476	1.5651	3.55
24	1.352	1.5721	3.71	5.825	1.52	0359	1.3879	1.6352	3.56
25	1.416	1.6850	3.75	6.337	1.64	.0418	1.4578	1.7604	3.60

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 153.
Crest length, 6.515 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.237	0.1154	2.73	0.314	0.12	0.0002	0.2372	0.1154	2.72
2	.301	.1651	2.77	.457	.16	.0004	.3014	.1651	2.77
3	.372	.2269	2.79	.633	.22	.0008	.3728	.2278	2.78
4	.373	.2278	2.83	.645	.23	.0008	.3738	.2287	2.82
5	.440	.2919	2.90	.847	.29	.0013	.4413	.2929	2.89
6	.505	.3589	2.93	1.052	.35	.0019	.5069	.3610	2.91
7	.576	.4371	3.00	1.311	.43	.0030	.5790	.4406	2.98
8	.637	.5084	3.07	1.560	.50	.0039	.6409	.5132	3.04
9	.696	.5807	3.10	1.801	.57	.0051	.7011	.5870	3.07
10	.701	.5870	3.10	1.820	.58	.0052	.7062	.5933	3.07
11	.760	.6626	3.15	2.085	.65	.0066	.7666	.6717	3.10
12	.762	.6652	3.16	2.101	.65	.0066	.7686	.6743	3.12
13	.814	.7344	3.20	2.349	.72	.0081	.8221	.7452	3.15
14	.879	.8241	3.25	2.678	.80	.0099	.8889	.8381	3.20
15	.937	.9071	3.29	2.984	.88	.0120	.9490	.9245	3.23
16	.993	.9895	3.34	3.307	.96	.0143	1.0073	1.0105	3.27
17	1.001	1.0015	3.33	3.330	.96	.0143	1.0153	1.0226	3.26
18	1.055	1.0836	3.40	3.672	1.05	.0171	1.0721	1.1099	3.31
19	1.102	1.1569	3.41	3.956	1.11	.0192	1.1212	1.1869	3.33
20	1.170	1.2656	3.46	4.394	1.21	.0228	1.1928	1.3030	3.37
21	1.226	1.3575	3.48	4.733	1.28	.0255	1.2515	1.3992	3.38
22	1.290	1.4652	3.51	5.159	1.38	.0296	1.3196	1.5166	3.40
23	1.289	1.4635	3.52	5.139	1.37	.0292	1.3182	1.5132	3.40
24	1.347	1.5634	3.53	5.507	1.45	.0327	1.3791	1.6193	3.40
25	1.404	1.6636	3.58	5.943	1.54	.0369	1.4409	1.7298	3.44
26	1.436	1.7208	3.58	6.158	1.58	.0388	1.4748	1.7914	3.44

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 154.
Crest length, 6.516 feet.
Crest height, 2.46 feet.

Cross section.

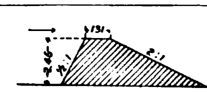
Period.	Observed head, experimental weir D, in feet.	$D^{3/2}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{3/2}$	C
1	2	3	4	5	6	7	8	9	10
1	0.236	0.1147	2.70	0.311	0.12	0.0002	0.2632	0.1349	2.30
2	.308	.1709	2.74	.469	.17	.0004	.3084	.1709	2.74
3	.373	.2278	2.83	.645	.23	.0008	.3738	.2287	2.82
4	.447	.2989	2.85	.852	.29	.0013	.4483	.2999	2.84
5	.508	.3621	2.95	1.068	.36	.0020	.5100	.3642	2.94
6	.577	.4382	2.97	1.301	.37	.0021	.5791	.4406	2.95
7	.643	.5156	3.04	1.569	.51	.0040	.6470	.5204	3.02
8	.706	.5933	3.07	1.821	.57	.0051	.7111	.5996	3.04
9	.760	.6626	3.17	2.102	.65	.0066	.7666	.6717	3.11
10	.823	.7466	3.20	2.389	.73	.0083	.8313	.7576	3.16
11	.888	.8368	3.20	2.678	.80	.0099	.8979	.8509	3.15
12	.946	.9201	3.24	2.981	.87	.0118	.9578	.9376	3.18
13	1.011	1.0165	3.28	3.334	.96	.0143	1.0253	1.0377	3.21
14	1.075	1.1146	3.31	3.674	1.03	.0165	1.0915	1.1396	3.22
15	1.138	1.2140	3.36	4.066	1.13	.0199	1.1579	1.2461	3.26
16	1.195	1.3063	3.37	4.415	1.20	.0224	1.2174	1.3426	3.29
17	1.250	1.3975	3.40	4.760	1.28	.0255	1.2755	1.4397	3.31
18	1.310	1.4994	3.43	5.145	1.36	.0288	1.3388	1.5494	3.32
19	1.370	1.6035	3.45	5.520	1.44	.0322	1.4022	1.6601	3.32
20	1.430	1.7100	3.48	5.951	1.53	.0364	1.4664	1.7750	3.35

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 156.
Crest height, 2.46 feet.
Crest width, 0.66 foot.
Upstream slope, ½ to 1.
Downstream slope, 5 to 1.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.246	0.1220	2.76	0.337	0.12	0.0002	0.2462	0.1220	2.76
2	.311	.1734	2.80	.486	.17	.0004	.3114	.1734	2.80
3	.382	.2361	2.84	.671	.24	.0009	.3829	.2370	2.83
4	.446	.2979	2.90	.864	.30	.0014	.4474	.2989	2.89
5	.508	.3621	2.91	1.054	.36	.0020	.5100	.3642	2.89
6	.576	.4371	2.95	1.289	.42	.0027	.5787	.4406	2.92
7	.638	.5096	3.01	1.534	.49	.0037	.6417	.5144	2.98
8	.703	.5895	3.06	1.804	.57	.0051	.7081	.5958	3.03
9	.764	.6678	3.10	2.070	.64	.0064	.7704	.6757	3.06
10	.834	.7617	3.13	2.384	.72	.0081	.8421	.7727	3.08
11	.888	.8368	3.17	2.653	.79	.0097	.8977	.8510	3.12
12	.956	.9347	3.24	3.028	.88	.0120	.9680	.9524	3.18
13	1.018	1.0272	3.22	3.309	.95	.0140	1.0320	1.0484	3.16
14	1.074	1.1130	3.30	3.673	1.04	.0168	1.0908	1.1396	3.22
15	1.139	1.2156	3.29	3.999	1.11	.0192	1.1582	1.2462	3.21
16	1.203	1.3194	3.31	4.367	1.19	.0220	1.2250	1.3558	3.22
17	1.267	1.4262	3.34	4.764	1.26	.0247	1.2917	1.4686	3.24
18	1.341	1.5529	3.36	5.218	1.37	.0292	1.3702	1.6035	3.25
19	1.394	1.6459	3.36	5.530	1.43	.0318	1.4258	1.7028	3.25
20	1.457	1.7587	3.39	5.962	1.52	.0359	1.4929	1.8241	3.27

Bazin's Series, No. 158.
Crest length, 6.520 feet.
Crest height, 2.46 feet.

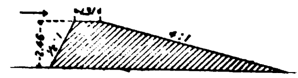


Cross section.

1	0.234	0.1132	2.79	0.316	0.12	0.0002	0.2342	0.1132	2.79
2	.312	.1743	2.72	.474	.17	.0004	.3124	.1743	2.72
3	.383	.2370	2.77	.656	.23	.0008	.3838	.2379	2.76
4	.457	.3090	2.79	.862	.29	.0013	.4583	.3100	2.78
5	.530	.3858	2.81	1.085	.36	.0020	.5320	.3880	2.80
6	.600	.4648	2.82	1.311	.43	.0030	.6030	.4683	2.80
7	.672	.5509	2.86	1.576	.50	.0039	.6759	.5557	2.84
8	.733	.6276	2.90	1.821	.57	.0051	.7381	.6340	2.87
9	.799	.7142	2.91	2.078	.64	.0064	.8054	.7223	2.88
10	.860	.7975	2.95	2.354	.71	.0078	.8678	.8087	2.91
11	.930	.8969	3.00	2.691	.79	.0097	.9397	.9114	2.95
12	.984	.9761	3.04	2.967	.86	.0115	.9955	.9925	2.99
13	1.055	1.0836	3.10	3.348	.95	.0140	1.0690	1.1053	3.03
14	1.125	1.1933	3.12	3.713	1.04	.0168	1.1418	1.2204	3.04
15	1.177	1.2769	3.15	4.022	1.10	.0188	1.1958	1.3029	3.08
16	1.243	1.3858	3.19	4.434	1.20	.0224	1.2654	1.4228	3.12
17	1.297	1.4771	3.22	4.766	1.27	.0251	1.3221	1.5200	3.14
18	1.361	1.5878	3.25	5.168	1.35	.0283	1.3893	1.6370	3.16
19	1.412	1.6779	3.30	5.544	1.43	.0348	1.4468	1.7406	3.18
20	1.457	1.7587	3.32	5.839	1.49	.0345	1.4915	1.8215	3.22

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 159.
Crest length, 6.511 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.234	0.1132	2.68	0.303	0.11	0.0002	0.2342	0.1132	2.68
2	.304	.1676	2.75	.462	.17	.0004	.3044	.1676	2.74
3	.379	.2333	2.82	.657	.23	.0008	.3798	.2342	2.80
4	.387	.2408	2.82	.680	.24	.0009	.3879	.2417	2.81
5	.457	.3090	2.81	.868	.30	.0014	.4584	.3100	2.80
6	.516	.3707	2.91	1.079	.36	.0020	.5180	.3728	2.89
7	.526	.3815	2.84	1.085	.36	.0020	.5280	.3836	2.83
8	.599	.4636	2.82	1.308	.43	.0030	.6020	.4671	2.81
9	.664	.5411	2.87	1.553	.50	.0039	.6679	.5460	2.84
10	.670	.5484	2.83	1.552	.49	.0037	.6737	.5533	2.80
11	.785	.6302	2.88	1.813	.56	.0049	.7399	.6366	2.85
12	.797	.7115	2.94	2.092	.64	.0064	.8034	.7196	2.91
13	.861	.7989	2.99	2.389	.72	.0081	.8693	.8101	2.95
14	.876	.8199	2.94	2.411	.72	.0081	.8843	.8311	2.90
15	.935	.9042	2.93	2.649	.78	.0095	.9445	.9172	2.89
16	.994	.9910	3.01	2.983	.86	.0115	1.0055	1.009	2.96
17	1.068	1.1037	3.03	3.333	.94	.0137	1.0817	1.1255	2.96
18	1.126	1.1948	3.10	3.704	1.03	.0165	1.1425	1.2204	3.04
19	1.145	1.2252	3.06	3.751	1.04	.0168	1.1618	1.2526	3.00
20	1.198	1.3112	3.08	4.035	1.10	.0188	1.2168	1.3425	3.00
21	1.261	1.4161	3.11	4.416	1.19	.0220	1.2830	1.4533	3.03
22	1.320	1.5166	3.15	4.777	1.27	.0251	1.3451	1.5599	3.06
23	1.332	1.5373	3.13	4.820	1.27	.0251	1.3571	1.5808	3.05
24	1.389	1.6370	3.14	5.150	1.33	.0275	1.4165	1.6850	3.06
25	1.445	1.7370	3.19	5.551	1.42	.0313	1.4763	1.7932	3.09
26	1.456	1.7569	3.19	5.614	1.43	.0348	1.4908	1.8188	3.09

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 160.
Crest height, 2.46 feet.
Crest width, 1.31 feet.
Upstream slope, ½ to 1.
Downstream slope, 6 to 1.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.451	0.3029	2.81	0.8540	0.29	0.0013	0.4523	0.3039	2.80
2	.522	.3772	2.82	1.0637	.36	.0020	.5240	.3793	2.80
3	.593	.4567	2.84	1.2970	.42	.0027	.5957	.4601	2.82
4	.663	.5399	2.88	1.5549	.50	.0039	.6669	.5447	2.86
5	.735	.6302	2.89	1.8213	.57	.0051	.7401	.6366	2.86
6	.798	.7128	2.91	2.0742	.64	.0064	.8044	.7209	2.88
7	.863	.8017	2.92	2.3410	.70	.0076	.8706	.8129	2.88
8	.930	.8969	2.97	2.6638	.78	.0095	.9395	.9100	2.93
9	.998	.9970	2.99	2.9810	.86	.0115	1.0095	1.0135	2.94
10	1.074	1.1130	3.02	3.3361	.95	.0140	1.0880	1.1349	2.94
11	1.129	1.1996	3.03	3.6348	1.01	.0159	1.1449	1.2252	2.97
12	1.193	1.3030	3.06	3.9872	1.09	.0185	1.2115	1.3334	2.98
13	1.254	1.4043	3.08	4.3252	1.16	.0209	1.2749	1.4397	3.00
14	1.326	1.5269	3.10	4.7334	1.25	.0243	1.3503	1.5686	3.02
15	1.389	1.6370	3.14	5.1402	1.34	.0279	1.4169	1.6867	3.05
16	1.457	1.7587	3.16	5.5575	1.42	.0313	1.4883	1.8151	3.06

Bazin's Series, No. 161.
Crest length, 6.543 feet.
Crest height, 1.64 feet.

Cross section.

1	0.298	0.1627	4.31	0.701	0.36	0.0020	0.3000	0.1643	4.27
2	.354	.2107	4.30	.906	.45	.0031	.3571	.2133	4.25
3	.413	.2654	4.26	1.131	.56	.0049	.4179	.2702	4.19
4	.472	.3243	4.23	1.371	.65	.0066	.4786	.3314	4.14
5	.529	.3847	4.22	1.625	.75	.0087	.5377	.3946	4.12
6	.581	.4429	4.25	1.883	.85	.0112	.5922	.4555	4.13
7	.639	.5108	4.24	2.167	.95	.0140	.6530	.5277	4.11
8	.693	.5770	4.26	2.458	1.05	.0171	.7101	.5983	4.11
9	.750	.6495	4.28	2.782	1.16	.0209	.7709	.6770	4.11
10	.804	.7209	4.31	3.107	1.27	.0251	.8291	.7548	4.12
11	.864	.8031	4.31	3.461	1.38	.0296	.8936	.8452	4.10
12	.919	.8810	4.32	3.806	1.49	.0345	.9535	.9303	4.09
13	.960	.9406	4.33	4.073	1.57	.0383	.9983	.9970	4.08
14	.992	.9880	4.30	4.248	1.61	.0403	1.0323	1.0484	4.05
15	1.019	1.0287	4.31	4.434	1.67	.0434	1.0624	1.0944	4.05
16	1.056	1.0851	4.28	4.665	1.72	.0460	1.1020	1.1569	4.03
17	1.083	1.1271	4.27	4.825	1.78	.0493	1.1323	1.2044	4.01
18	1.118	1.1821	4.24	5.003	1.81	.0509	1.1689	1.2640	3.96
19	1.157	1.2445	4.17	5.171	1.84	.0526	1.2096	1.3310	3.88
20	1.187	1.2932	4.16	5.380	1.90	.0561	1.2431	1.3858	3.88
21	1.225	1.3558	4.12	5.378	1.95	.0591	1.2841	1.4550	3.83
22	1.263	1.4194	4.09	5.808	2.00	.0622	1.3252	1.5252	3.81
23	1.289	1.4635	4.11	6.001	2.05	.0653	1.3543	1.5756	3.81
24	1.326	1.5269	4.08	6.242	2.10	.0686	1.3946	1.6476	3.79
25	1.359	1.5843	4.08	6.416	2.15	.0719	1.4309	1.7118	3.76

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 163.
Crest length, 6.535 feet.
Crest height, 1.64 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.184	0.0790	3.81	0.301	0.17	0.0004	0.1844	0.0790	3.81
2	.244	.1206	3.83	.463	.25	.0009	.2449	.1213	3.82
3	.303	.1668	3.84	.641	.33	.0017	.3047	.1684	3.81
4	.366	.2215	3.83	.850	.42	.0027	.3687	.2241	3.79
5	.423	.2751	3.83	1.053	.51	.0040	.4270	.2791	3.77
6	.486	.3388	3.82	1.295	.61	.0058	.4915	.3441	3.76
7	.536	.3924	3.86	1.513	.69	.0074	.5434	.4001	3.78
8	.593	.4567	3.94	1.799	.81	.0102	.6032	.4683	3.84
9	.653	.5277	3.91	2.063	.90	.0126	.6656	.5435	3.80
10	.702	.5882	4.04	2.376	1.01	.0159	.7179	.6084	3.90
11	.769	.6744	3.98	2.683	1.11	.0188	.7878	.6995	3.84
12	.827	.7521	4.02	3.023	1.22	.0231	.8501	.7837	3.86
13	.882	.8283	4.02	3.329	1.32	.0271	.9091	.8667	3.84
14	.949	.9245	4.04	3.735	1.44	.0322	.9812	.9717	3.84
15	.998	.9970	4.06	4.048	1.53	.0364	1.0344	1.0514	3.85
16	1.056	1.0851	4.06	4.425	1.64	.0418	1.0978	1.1505	3.85
17	1.114	1.1758	4.05	4.779	1.74	.0471	1.1611	1.2510	3.82
18	1.171	1.2672	4.07	5.169	1.84	.0526	1.2236	1.3541	3.82
19	1.231	1.3658	4.07	5.576	1.94	.0585	1.2895	1.4635	3.81
20	1.285	1.4567	4.12	6.014	2.06	.0660	1.3510	1.5703	3.83
21	1.339	1.5495	4.17	6.464	2.17	.0732	1.4122	1.6779	3.85

Bazin's Series, No. 164.
Crest length, 6.534 feet.
Crest height, 1.64 feet.

Cross section.

1	0.244	0.1206	3.86	0.467	0.25	0.0009	0.2449	0.1213	3.85
2	.305	.1685	3.91	.659	.34	.0018	.3068	.1701	3.88
3	.367	.2224	3.87	.859	.43	.0030	.3700	.2251	3.82
4	.425	.2771	3.90	1.080	.52	.0042	.4292	.2810	3.84
5	.482	.3346	3.87	1.296	.61	.0058	.4878	.3409	3.80
6	.540	.3968	3.87	1.536	.70	.0076	.5476	.4057	3.79
7	.592	.4555	3.94	1.797	.81	.0102	.6022	.4671	3.85
8	.651	.5252	3.94	2.069	.90	.0126	.6636	.5410	3.82
9	.702	.5882	3.97	2.334	.99	.0152	.7172	.6071	3.84
10	.766	.6705	4.00	2.684	1.11	.0188	.7848	.6955	3.86
11	.817	.7385	4.03	2.978	1.21	.0228	.8398	.7699	3.87
12	.877	.8213	4.05	3.325	1.32	.0271	.9041	.8595	3.87
13	.939	.9100	4.07	3.704	1.44	.0322	.9722	.9583	3.86
14	.993	.9895	4.10	4.055	1.54	.0369	1.0299	1.0453	3.88
15	1.052	1.0790	4.09	4.417	1.64	.0418	1.0938	1.1442	3.86
16	1.115	1.1774	4.12	4.862	1.76	.0482	1.1632	1.2543	3.88
17	1.162	1.2526	4.13	5.163	1.84	.0526	1.2146	1.3392	3.86
18	1.219	1.3459	4.15	5.602	1.96	.0597	1.2787	1.4465	3.87
19	1.277	1.4431	4.18	6.019	2.06	.0660	1.3430	1.5564	3.87
20	1.330	1.5338	4.19	6.411	2.16	.0725	1.4025	1.6601	3.86

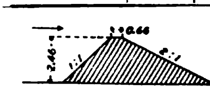
Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 165.
Crest length, 6.544 feet.
Crest height, 1.64 feet.

Cross section.

Period.	Observed head, experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.337	0.1957	3.56	0.698	0.35	0.0019	0.3389	0.1974	3.54
2	.401	.2540	3.56	.904	.44	.0030	.4040	.2568	3.52
3	.464	.3161	3.56	1.125	.54	.0045	.4685	.3202	3.51
4	.528	.3836	3.55	1.363	.63	.0062	.5342	.3902	3.49
5	.593	.4567	3.54	1.618	.73	.0083	.6013	.4660	3.47
6	.656	.5313	3.54	1.880	.82	.0105	.6665	.5435	3.46
7	.720	.6109	3.54	2.162	.92	.0132	.7332	.6276	3.44
8	.783	.6929	3.55	2.461	1.02	.0162	.7992	.7142	3.45
9	.843	.7740	3.58	2.771	1.11	.0192	.8622	.8003	3.46
10	.904	.8595	3.61	3.103	1.22	.0231	.9271	.8926	3.47
11	.969	.9538	3.63	3.462	1.33	.0275	.9965	.9940	3.48
12	1.029	1.0438	3.63	3.789	1.42	.0313	1.0603	1.0913	3.47
13	1.090	1.1380	3.64	4.150	1.52	.0359	1.1259	1.1948	3.47
14	1.153	1.2381	3.65	4.526	1.62	.0408	1.1938	1.3046	3.47
15	1.217	1.3426	3.66	4.904	1.71	.0455	1.2625	1.4178	3.46
16	1.279	1.4465	3.68	5.336	1.83	.0521	1.3311	1.5355	3.48
17	1.341	1.5529	3.68	5.704	1.92	.0573	1.3983	1.6530	3.45
18	1.401	1.6583	3.69	6.125	2.02	.0634	1.4644	1.7714	3.46
19	1.448	1.7424	3.73	6.490	2.10	.0686	1.5166	1.8680	3.47

Bazin's Series, No. 176.
Crest length, 6.519 feet.
Crest height, 2.46 feet.

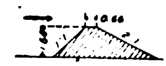


Cross section.

1	0.237	0.1154	2.76	0.317	0.12	0.0002	0.2372	0.1154	2.75
2	.296	.1611	2.74	.441	.16	.0004	.2964	.1611	2.74
3	.365	.2206	2.92	.645	.23	.0008	.3658	.2214	2.91
4	.439	.2909	2.95	.858	.30	.0014	.4404	.2919	2.94
5	.494	.3472	3.04	1.055	.36	.0020	.4960	.3494	3.02
6	.565	.4247	3.10	1.318	.43	.0030	.5680	.4281	3.08
7	.618	.4858	3.19	1.550	.50	.0039	.6219	.4905	3.16
8	.682	.5632	3.22	1.813	.58	.0052	.6872	.5695	3.18
9	.733	.6276	3.29	2.066	.65	.0066	.7396	.6366	3.24
10	.797	.7115	3.35	2.385	.73	.0083	.8073	.7250	3.29
11	.861	.7989	3.41	2.724	.82	.0105	.8715	.8129	3.35
12	.910	.8681	3.45	2.995	.89	.0123	.9223	.8853	3.38
13	.974	.9613	3.51	3.373	.98	.0149	.9889	.9835	3.43
14	1.027	1.0408	3.53	3.671	1.05	.0171	1.0441	1.0667	3.44
15	1.088	1.1349	3.57	4.034	1.14	.0202	1.1082	1.1663	3.46
16	1.139	1.2156	3.62	4.416	1.23	.0235	1.1625	1.2526	3.52
17	1.196	1.3079	3.65	4.782	1.30	.0263	1.223	1.3508	3.54
18	1.248	1.3942	3.68	5.115	1.38	.0296	1.2776	1.4448	3.54
19	1.303	1.4874	3.73	5.558	1.47	.0336	1.3366	1.5460	3.60
20	1.355	1.5773	3.75	5.925	1.55	.0374	1.3924	1.6423	3.61
21	1.420	1.6921	3.80	6.422	1.66	.0428	1.4628	1.7695	3.63

Bazin's experiments on weirs of irregular section—Continued.

Bazin's Series, No. 178.
Crest length, 6.518 feet.
Crest height, 2.46 feet.



Cross section.

Period.	Observed head experimental weir D, in feet.	$D^{\frac{3}{2}}$	C_1	Q, flow per foot, experimental weir, in cubic feet per second.	v	$\frac{v^2}{2g}$	H	$H^{\frac{3}{2}}$	C
1	2	3	4	5	6	7	8	9	10
1	0.222	0.1046	2.83	0.297	0.11	0.0002	0.2222	0.1046	2.84
2	.299	.1635	2.95	.482	.17	.0004	.2994	.1635	2.95
3	.367	.2224	2.96	.658	.23	.0008	.3678	.2233	2.95
4	.431	.2830	3.08	.872	.30	.0014	.4324	.2840	3.07
5	.491	.3441	3.13	1.077	.37	.0021	.4931	.3462	3.11
6	.556	.4146	3.19	1.323	.47	.0034	.5594	.4180	3.16
7	.614	.4811	3.24	1.558	.51	.0040	.6180	.4858	3.21
8	.669	.5472	3.28	1.794	.57	.0051	.6741	.5533	3.24
9	.732	.6263	3.33	2.085	.65	.0066	.7386	.6358	3.28
10	.789	.7009	3.36	2.355	.73	.0083	.7973	.7115	3.31
11	.847	.7796	3.43	2.675	.81	.0102	.8572	.7934	3.37
12	.906	.8624	3.46	2.983	.89	.0123	.9183	.8795	3.39
13	.966	.9494	3.51	3.331	.97	.0146	.9806	.9716	3.43
14	1.028	1.0423	3.53	3.671	1.05	.0171	1.0451	1.0683	3.44
15	1.083	1.1271	3.58	4.045	1.14	.0202	1.1032	1.1584	3.49
16	1.142	1.2204	3.60	4.392	1.22	.0231	1.1651	1.2575	3.49
17	1.195	1.3063	3.64	4.755	1.30	.0263	1.2213	1.3492	3.52
18	1.259	1.4127	3.66	5.170	1.39	.0300	1.2890	1.4635	3.53
19	1.314	1.5063	3.69	5.572	1.48	.0341	1.3481	1.5651	3.56
20	1.366	1.5965	3.72	5.952	1.55	.0374	1.4034	1.6618	3.58
21	1.424	1.6992	3.75	6.375	1.65	.0423	1.4663	1.7750	3.59

CORNELL UNIVERSITY HYDRAULIC LABORATORY.[a]

This laboratory, erected in 1898, includes a reservoir formed by a masonry dam on Fall Creek, at Ithaca, N. Y. An experimental channel is supplied with water from the pond and has, as its general dimensions, length, 400 feet; breadth, 16 feet; depth, 10 feet; bottom grade, 1:500. Fall Creek drains an area of 117 square miles, and affords a minimum water supply estimated at 12 second-feet. The hydraulic laboratory is located at Triphammer Falls, where a descent of 189 feet occurs. The weirs used in the experiments here described were erected in the concrete-lined experimental channel. The water supply was regulated by wooden head-gates, operated by lever, rack, and pinion, the outflow from the canal passing over the declivity below.

[a] In reducing the experiments at Cornell hydraulic laboratory the value of g for Ithaca, latitude 42° 27′, altitude 500 feet, has been taken as 32.16, making $\sqrt{2g}=8.02$, $\frac{1}{2g}=0.015547$, $\frac{2}{3}\sqrt{2g}=5.35$.

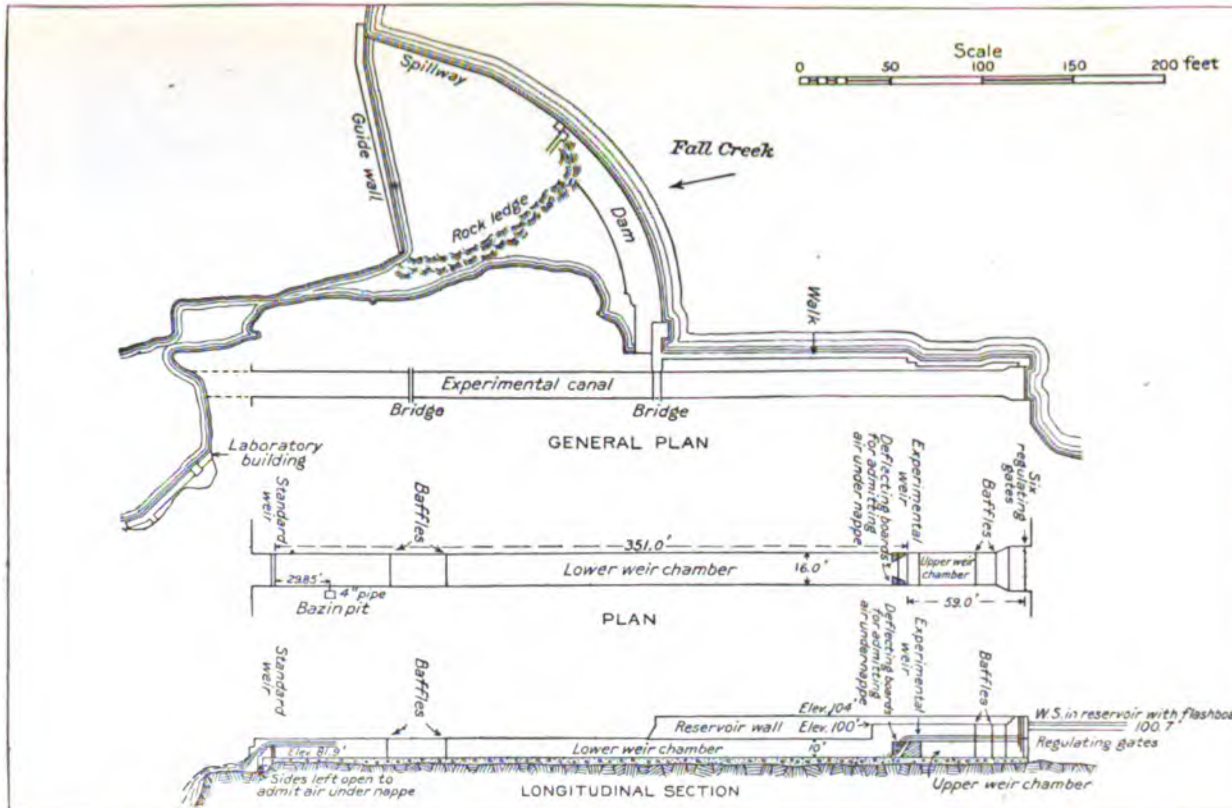
EXPERIMENTS OF UNITED STATES BOARD OF ENGINEERS ON DEEP WATERWAYS.

These experiments were performed at Cornell University hydraulic laboratory in May and June, 1899, for the United States Board of Engineers on Deep Waterways, under the immediate direction of George W. Rafter, engineer for water supply, in conjunction with Prof. Gardner S. Williams. The results of the original computations were published in Trans. Am. Soc. C. E., vol. 44, together with an extended discussion. In the experiments a closely regulated volume of water was passed over a standard thin-edged weir which was placed near the upper end of the experimental canal and had a height of 13.13 feet and a crest length of 16 feet, end contractions suppressed. The nappe was aerated, but was not allowed to expand on downstream side. The water flowed down the experimental canal past a series of screens and baffles and over the experimental weir placed at the lower end of the channel.

The experimental weirs were about 4.5 feet high and 6.56 feet crest length. A leading channel of planed boards, 6.56 feet wide and 48 feet in length, extended upstream from the experimental weir, having at its upper end flaring sides extending 8.3 feet upstream and meeting the sides of the main channel.

The head on both weirs was read by means of open manometers connected to galvanized-iron piezometer pipes, placed horizontally across the bottom of the narrow leading channel, 37 feet upstream from the weir. At the standard weir two piezometers were used, one termed the middle piezometer, placed across the leading channel, 8 inches above the bottom and 10 feet upstream from the standard weir. A second or upstream piezometer was placed 25 feet upstream from the standard weir. Readings of both piezometers were taken. It was decided, however, to use the middle piezometer as the basis of calculation of discharge over the standard weir. Near the close of the experiments it was found that this did not give results agreeing with those which would have been obtained from a piezometer placed flush with the bottom of the channel, as is shown to be necessary from the experiments of H. F. Mills[a] and others. A correction curve was accordingly deduced from comparative experiments between the middle piezometer and the flush piezometer, and the readings of the middle piezometer thus corrected were applied in the Bazin formula to calculate the discharge over the standard weir for heads not exceeding the limit of Bazin's experiments. For depths on the standard weir greater than 2 feet the discharge was computed by using coefficients deduced for higher heads on a shorter experimental weir, on the basis of the Francis formula. Owing to the uncertainty as to the piezometers and

[a] Mills, H. F., Experiments upon piezometers used in hydraulic investigations, Boston, 1878.



HYDRAULIC LABORATORY AT CORNELL UNIVERSITY, ITHACA, N. Y.

CORNELL HYDRAULIC LABORATORY, ARRANGED FOR WEIR EXPERIMENTS.

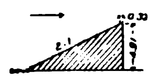
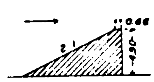
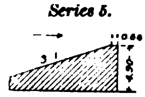
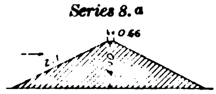
other conditions, the original results of the experiments were credited with a possible error of 5 or 6 per cent.

In connection with the experiments on models of the Croton dam, a very thorough comparison of the so-called upstream piezometer with other methods of obtaining the head on a standard weir was made by Professor Williams. It was found that the upstream piezometer gave the actual head on the standard weir correctly. These results were communicated to the writer, and a recomputation of the Deep Waterways experiments has been made, using readings of the upstream piezometer to calculate the standard weir discharge by Bazin's formula. This method of calculation eliminates the necessity for correcting the piezometer readings at the standard weir, as was necessary in the previous reductions. The discharge over the experimental weir has been calculated from readings of a piezometer placed 38 feet upstream from the weir and 8 inches above channel bottom, corrected to the basis of a flush piezometer.

The United States Deep Waterways experiments included, for each experimental model, a smaller number of heads or periods than either the Croton or United States Geological Survey experiments. They were also the first experiments of the kind conducted at the Cornell laboratory, and the experience gained has probably contributed to the securing of somewhat greater accuracy in the later experiments. It is believed, however, that, as recomputed, the United States Deep Waterways experiments do not differ much in accuracy from those made on models of the Croton dam, which are stated by John R. Freeman to be reliable within about 2 per cent. The coefficients obtained by recomputation, when compared with the original United States Deep Waterways coefficients, show few differences exceeding 2 per cent. The variations are plus and minus in about equal numbers, and it is believed that these experiments are entitled to greater weight than they have hitherto received.

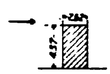
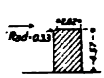
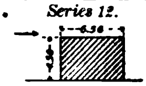
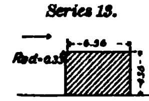
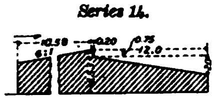
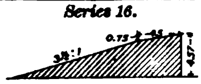
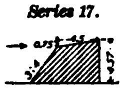
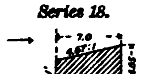
In the accompanying tables a summary of the recomputation is given.

Recomputation of United States Deep Waterways Board experiments on flow of water over model dams, Cornell University hydraulic laboratory, 1899.

Weir model.	Period.	Corrected depth D, experimental weir, centimeters.	D, in feet.	Q= flow per foot, in cubic feet per second.	V	$h=\frac{V^2}{2g}$	H= D+h	C_1	Number of observations of head.
1	2	3	4	5	6	7	8	9	10
Series 1.	1	....	4.972	39.73	....	....	4.972	3.584	....
	2	....	4.872	39.44	....	....	4.872	3.668	....
	3	....	4.853	39.31	....	....	4.853	3.940	....
	4	....	4.138	31.47	....	....	4.138	3.739	....
	5	....	3.368	22.81	....	....	3.368	3.690	....
P=4.91. L=6.58.	6	....	1.725	8.71	....	....	1.725	3.844	....
	7	....	1.190	4.88	....	....	1.190	3.759	....
Series 2.	1	....	5.05	42.13	....	....	5.050	3.712	41
	2	....	4.15	31.17	....	....	4.150	3.758	23
	3	....	3.35	22.89	....	....	3.350	3.733	15
	4	....	2.55	15.03	....	....	2.550	3.691	21
	5	....	1.75	8.39	....	....	1.750	3.633	15
P=4.90. L=6.58.	6	....	.923	3.02	....	....	.923	3.406	18
	7	....	.34	.82	....	....	.340	4.120	7
Series 3. 	1	....	....	....	....	....	5.28	3.393	21
	2	....	....	....	....	....	3.49	3.383	15
P=4.91. L=6.58.	3	....	....	....	....	....	1.75	3.382	21
Series 4.	1	....	....	....	....	....	5.11	3.547	21
	2	....	....	....	....	....	4.28	3.373	27
	3	....	....	....	....	....	3.43	3.484	27
	4	....	....	....	....	....	2.57	3.485	27
P=4.91. L=6.58.	5	....	....	....	....	....	1.73	3.485	23
	6	....	....	....	....	....	....	....	....
Series 5.	1	146.70	4.812	41.04	4.23	0.2782	5.0902	3.574	25
	2	123.00	4.034	31.22	3.50	.1959	4.2299	3.598	31
	3	98.80	3.242	21.49	2.65	.1092	3.3512	3.503	27
	4	75.72	2.484	13.27	1.80	.0504	2.5344	3.289	25
P=4.90. L=6.58.	5	50.65	1.662	8.21	1.24	.0239	1.6859	3.751	21
Series 7.	1	142.75	4.682	41.40	4.30	.2875	4.9695	3.737	29
	2	119.20	3.969	30.72	3.47	.1872	4.1562	3.626	29
	3	96.72	3.173	21.64	2.67	.1108	3.2838	3.637	27
P=4.895. L=6.58.	4	74.50	2.444	14.07	1.92	.0573	2.5013	3.557	27
Series 8.[a]	1	144.00	4.723	41.22	4.28	.2848	5.0078	3.678	18
	2	120.50	3.953	30.50	3.45	.1850	4.1380	3.623	26
	3	97.27	3.192	21.53	2.62	.1067	3.299	3.591	23
	4	74.35	2.439	14.18	1.92	.0573	2.4963	3.595	27
P=4.90. L=6.58.	5	49.77	1.633	7.63	1.17	.0213	1.6543	3.585	15
Series 9.	1	147.10	4.825	41.16	4.20	.2742	5.0992	3.575	22
	2	123.00	4.034	30.47	3.38	.1776	4.2116	3.525	29
	3	99.62	3.268	21.48	2.62	.1067	3.3927	3.437	25
	4	76.22	2.500	14.24	1.92	.0573	2.5573	3.482	
P=4.94. L=6.58.	5	51.00	1.673	7.85	1.19	.0220	1.6950	3.557	24

[a] Same as series 7, but upstream face covered with ¼-inch mesh galvanized wire netting.

Recomputation of United States Deep Waterways Board experiments on flow of water over model dams, Cornell University hydraulic laboratory, 1899—Continued.

Weir model.	Period.	Corrected depth D, experimental weir, centimeters.	D, in feet.	Q = flow per foot, in cubic feet per second.	V	$h=\frac{V^2}{2g}$	$H=D+h$	C_1	Number of observations of head.
1	2	3	4	5	6	7	8	9	10
Series 10.	1	153.82	5.046	42.01	4.37	0.2969	5.3429	3.402	21
	2	132.94	4.361	31.27	3.50	.1904	4.5514	3.220	21
	3	110.98	3.641	22.33	2.72	.1150	3.7560	3.068	20
	4	87.99	2.887	14.77	1.99	.0616	2.9486	2.917	17
$P=4.57$. $L=6.58$.	5	61.70	2.024	8.12	1.24	.0239	2.0479	2.771	17
Series 11.	1	149.15	4.892	42.06	4.45	.3079	5.1999	3.547	....
	2	127.70	4.189	31.66	3.60	.2015	4.3905	3.418	....
	3	105.60	3.464	22.02	2.72	.1150	3.5790	3.252	....
	4	82.52	2.707	14.34	1.97	.0603	2.7673	3.115	....
$P=4.57$. $L=6.58$.	5	58.80	1.930	8.26	1.26	.0247	1.9547	3.022	....
Series 12.	1	154.55	5.069	30.69	3.19	.1582	5.219	2.574	....
	2	126.80	4.160	21.58	2.48	.0956	4.2556	2.458	....
	3	96.75	3.174	14.12	1.82	.0515	3.2255	2.438	....
$P=4.56$. $L=6.58$.	4	66.30	2.174	7.85	1.17	.0213	2.1953	2.413	....
Series 13.	1	144.70	4.747	30.69	3.28	.1673	4.9143	2.817	....
	2	116.60	3.825	21.75	2.60	.1051	3.9301	2.790	....
	3	88.52	2.904	14.07	1.90	.0561	2.9601	2.763	....
	4	60.02	1.969	7.85	1.19	.0220	1.9910	2.859	....
$P=4.56$. $L=6.58$.	5	30.80	1.010	2.89	.51	.0040	1.0140	2.830	....
Series 14.	1	157.05	5.151	41.16	4.23	.2782	5.4292	3.254	....
	2	131.60	4.317	30.64	3.45	.1850	4.5020	3.208	....
	3	105.40	3.458	21.68	2.72	.1150	3.5730	3.212	....
	4	80.25	2.633	14.07	1.97	.0603	2.6933	3.188	....
	5	54.25	1.780	7.78	1.24	.0239	1.8039	3.219	....
$P=4.53$. $L=6.58$.	6	28.05	.920	2.75	.51	.0040	.9240	3.096	....
Series 15.	1	129.55	4.250	31.12	3.55	.1959	4.4459	3.320	....
	2	105.00	3.444	21.87	2.72	.1150	3.5590	3.257	....
	3	79.52	2.608	14.02	1.97	.0603	2.6683	3.217	....
$P=4.53$. $L=6.58$.	4	53.65	1.760	7.95	1.26	.0247	1.7847	3.333	....
Series 16.	1	126.75	4.157	30.69	3.52	.1926	4.3496	3.383	....
	2	102.55	3.364	21.87	2.75	.1176	3.4816	3.366	....
	3	78.02	2.559	14.02	1.97	.0603	2.6193	3.307	....
$P=4.57$. $L=6.58$.	4	52.00	1.706	7.73	1.24	.0239	1.7299	3.397	....
Series 17.	1	127.30	4.175	31.08	3.55	.1959	4.3709	3.401	....
	2	103.32	3.389	21.87	2.75	.1176	3.5066	3.331	....
	3	78.39	2.571	14.34	2.02	.0634	2.6344	3.354	....
	4	51.57	1.691	7.73	1.24	.0239	1.7149	3.442	....
$P=4.57$. $L=6.58$.	5	32.40	1.063	3.82	.68	.0072	1.0702	3.450	....
Series 18.	1	38.523	1.264	5.47	.92	.0132	1.2772	3.790	....
	2	149.362	4.899	40.19	4.23	.2782	5.1772	3.412	....
	3	125.693	4.123	30.25	3.45	.1850	4.3080	3.383	....
	4	100.766	3.305	21.38	2.70	.1133	3.4183	3.383	....
$P=4.65$ $L=6.58$.	5	75.427	2.474	13.85	1.94	.0585	2.5325	3.437	....

Recomputation of United States Deep Waterways Board experiments on flow of water over model dams, Cornell University hydraulic laboratory, 1899—Continued.

Weir model.	Period.	Corrected depth D, experimental weir, centimeters.	D, in feet.	Q= flow per foot, in cubic feet per second.	V	$h=\frac{V^2}{2g}$	$H=D+h$	C_1	Number of observations of head.
1	2	3	4	5	6	7	8	9	10
	1	27.04	0.8869	2.75	0.44	0.0030	0.8899	3.276	....
Series 19.	2	51.36	1.685	7.46	1.07	.0178	1.7028	3.357	....
	3	142.128	4.662	40.04	4.20	.2742	4.9362	3.651	....
	4	119.442	3.918	29.62	3.23	.1622	4.0802	3.594	....
	5	97.858	3.210	20.78	2.45	.0933	3.3033	3.461	....
P=5.28. *L*=6.58.	6	77.246	2.534	14.17	1.82	.0515	2.5855	3.401	....
	7	53.42	1.752	7.68	1.09	.0185	1.7705	3.260	....

Column 5 shows the discharge over the experimental weir per foot of crest, deduced from the readings of the upstream piezometer at the standard weir, by Bazin's formula, and corrected for slight leakage.

Column 3 shows the head on the experimental weir, in centimeters, taken by a piezometer 38 feet upstream and 8 inches above channel bottom, corrected to reduce it to the equivalent reading of the flush piezometer.

Column 4 shows the equivalent head in feet.

Column 6 shows the absolute velocity of approach.

Column 7 shows the velocity head.

Column 8 shows the head corrected for velocity of approach; the correction being made by the simple addition of the velocity head to the measured head, which is assumed to be a sufficiently precise equivalent to the Francis correction formula for this purpose.

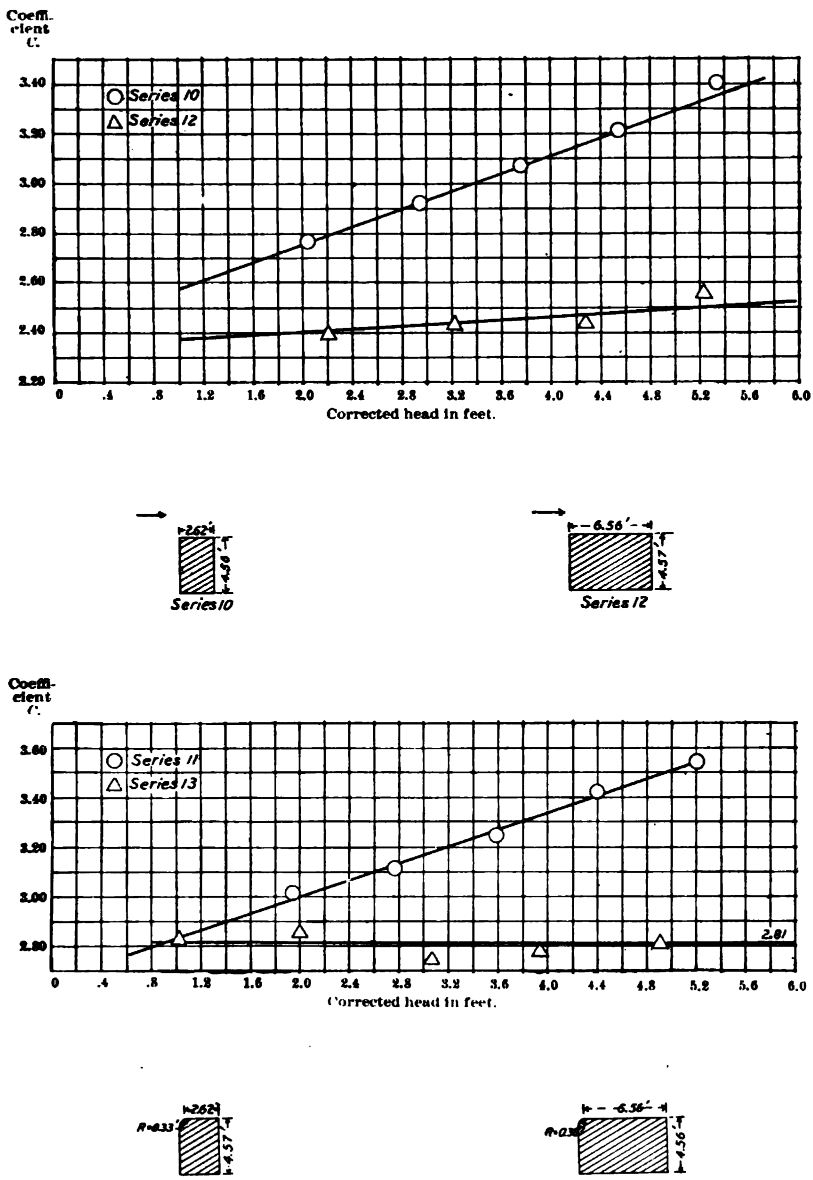
Column 9 gives the coefficient C_1, deduced from the foregoing.

The resulting coefficient diagrams are shown on Pls. XV to XVIII, inclusive.

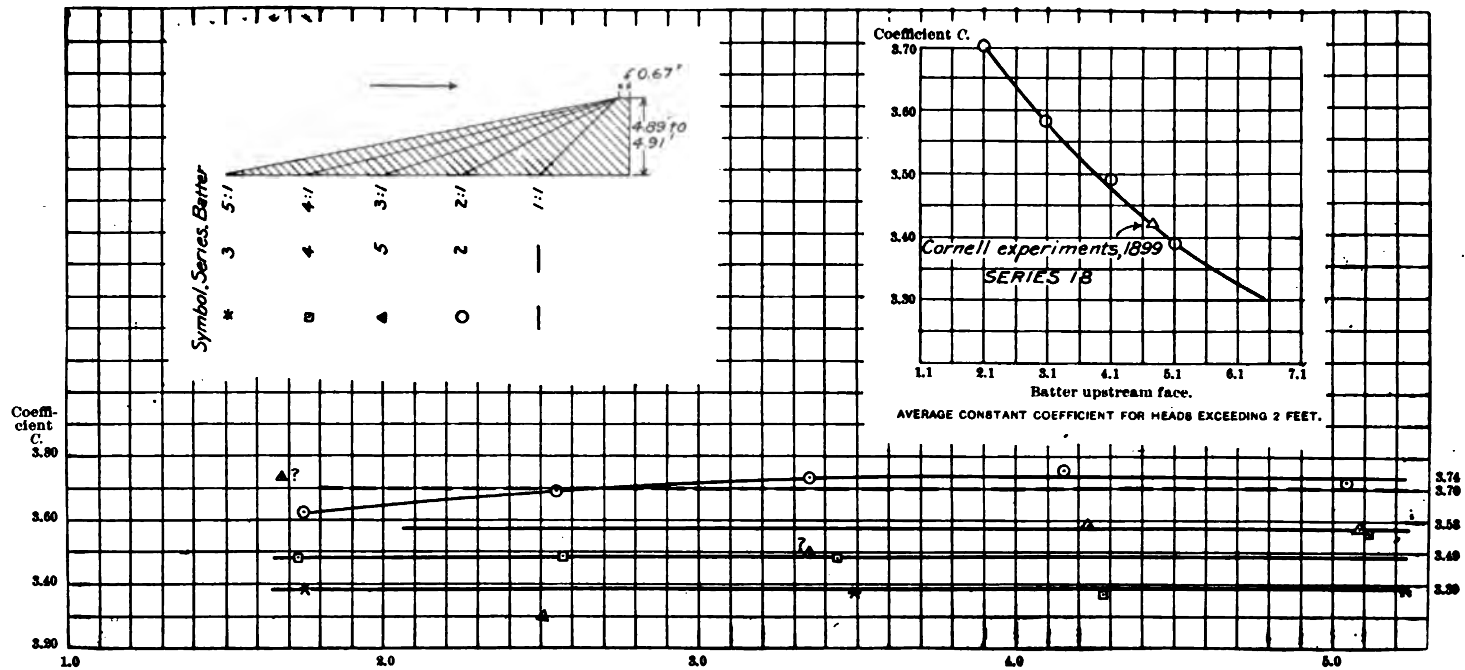
EXPERIMENTS AT CORNELL UNIVERSITY HYDRAULIC LABORATORY ON MODELS OF OLD CROTON DAM.[a]

These experiments were made in November and December, 1899, by Prof. Gardner S. Williams, under the direction of John R. Freeman. The standard weir used was located near the head of the experimental canal, water being admitted and regulated by head-gates in the usual manner. The standard weir was 11.25 feet high and 16 feet long on the crest. The experimental weir was placed 232.5 feet farther downstream, and also occupied the full width of the experimental canal. The models of the Croton dam were constructed of framed timber and were 6 to 9 feet high.

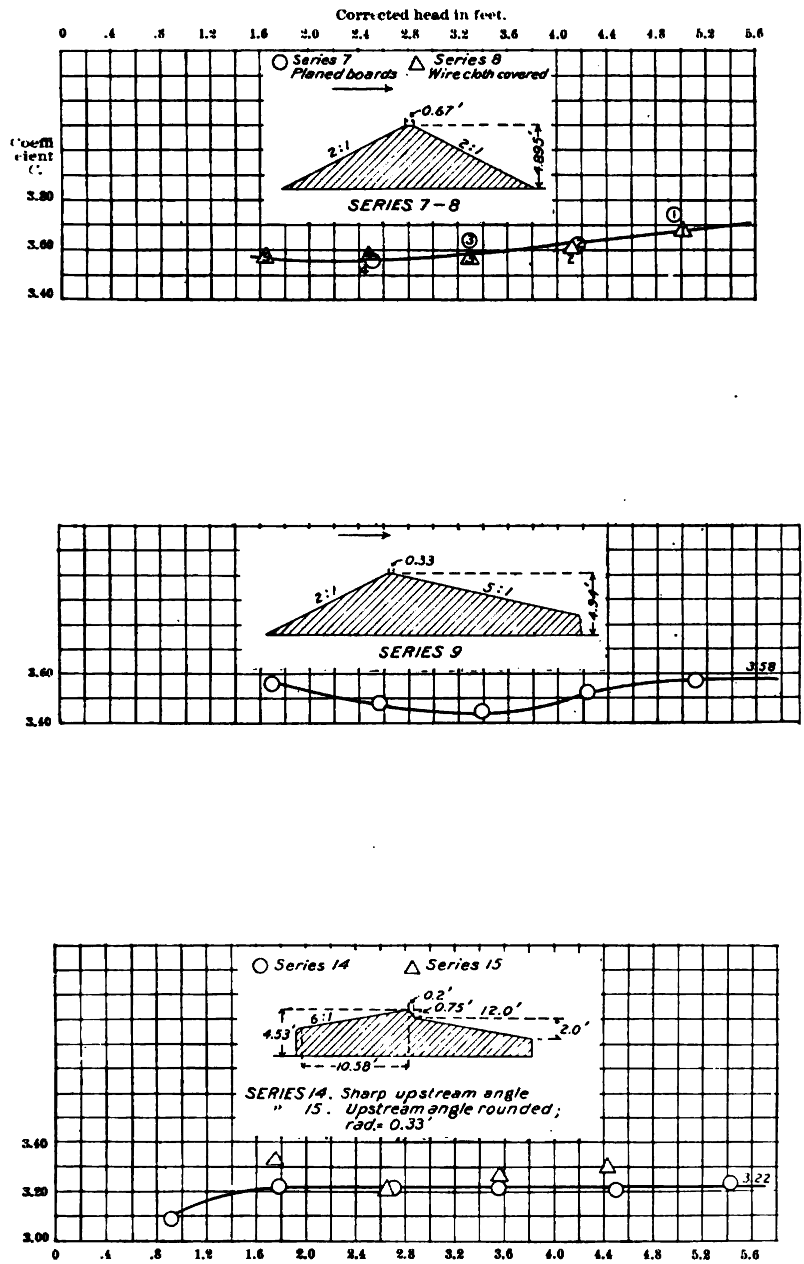
[a] Report on New York's water supply, Freeman, 1900, pp. 139-141.



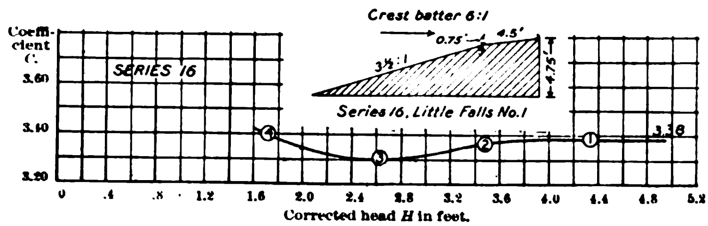
EXPERIMENTS OF UNITED STATES DEEP WATERWAYS BOARD AT CORNELL UNIVERSITY, 1899.

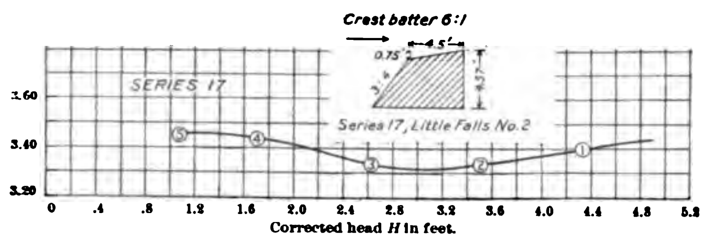


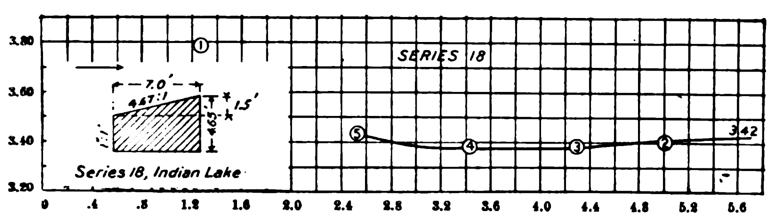
EXPERIMENTS OF UNITED STATES DEEP WATERWAYS BOARD AT CORNELL UNIVERSITY, 1899.

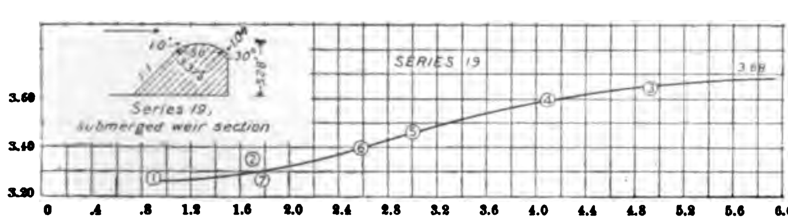


EXPERIMENTS OF UNITED STATES DEEP WATERWAYS BOARD AT CORNELL UNIVERSITY, 1899.









EXPERIMENTS OF UNITED STATES DEEP WATERWAYS BOARD AT CORNELL UNIVERSITY, 1899.

The head on the weirs was measured by means of open glass manometers connected to piezometer tubes in the channel above each weir. The piezometer tubes were made of 1-inch galvanized-iron pipe with small holes drilled along the sides, the ends being plugged. At the standard weir three piezometers were used, placed parallel to the current, at about mid-depth of the channel, one being near each side and one at mid-width of the channel, the mid-length of the pipes being 26.5 feet upstream from the standard weir. A hook gage in the same section was used to check the observed head.

Experiments on volume of flow over models of old Croton dam, Cornell University hydraulic laboratory, 1899.

	Period No.	Observed depth on model dam, in feet.	Mean velocity of approach, in feet per second.	Correction for velocity of approach, in feet.	Corrected head on model dam, in feet.	Discharge over model dam per foot of length, in cubic feet per second.	C_1
1	2	3	4	5	6	7	8
Series 1—Model A. Round crest, old Croton dam, smooth pine, crest and slope 16 feet long. Nov. 28-29, 1899.	1	2.7229	1.685	0.0439	2.7668	14.762	3.208
	2	2.1857	1.283	.0259	2.2116	10.562	3.211
	3	1.4388	.749	.0087	1.4475	5.604	3.218
	4	.9830	.449	.0031	.9861	3.154	3.222
	5	.5907	.219	.0008	.5915	1.451	3.190
	6	.1230	.024	.0000	.1230	.147	3.408
Series 1a—Model A. Round crest, old Croton dam, unplaned plank, crest 16 feet long, smooth slope. Nov. 6, 1899.	1	2.0897	.978	.0149	2.1046	9.578	3.137
	2	1.8293	.810	.0102	1.8395	7.883	3.160
	3	1.5878	.661	.0068	1.5946	6.284	3.121
	4	1.2562	.467	.0034	1.2596	4.287	3.032
	5	.9929	.338	.0018	.9947	3.006	3.030
	6	.6301	.175	.0004	.6305	1.494	2.988
	7	.4871	.111	.0002	.4873	.991	2.913
Series 2—Model A. Round crest, old Croton dam, 16-foot smooth crest, rough slope formed of cleats and stone to simulate concrete and riprap. Dec. 4, 1899.	1	2.9227	1.839	.0526	2.9753	16.475	3.240
	2	2.8591	1.794	.0500	2.9091	15.969	3.218
	3	2.4948	1.516	.0357	2.5305	12.933	3.213
	4	2.1420	1.248	.0241	2.1661	10.211	3.203
	5	1.6238	.880	.0120	1.6358	6.740	3.222
	6	1.2597	.623	.0060	1.2657	4.548	3.194
	7	1.1419	.545	.0046	1.1465	3.913	3.188
	8	.7196	.288	.0013	.7209	1.945	3.178
	9	.4873	.166	.0004	.4877	1.087	3.192
Series 3—Model A. Round crest, old Croton dam, 16-foot crest, covered with wire cloth of No. 18 wire, ¼-inch mesh,[a] rough slope, as in series 2. Nov. 28, 1899.	1	2.0080	1.124	.0197	2.0187	9.037	3.148
	2	1.4091	.712	.0078	1.4129	5.308	3.161
	3	.8675	.366	.0021	.8656	2.527	3.138
	4	.4288	.133	.0003	.4251	.861	3.099
	5	.1184	.020	.0000	.1144	.124	3.205

[a] In experiments with wire cloth over crest, 0.004 foot is deducted from observed depth to compensate for thickness of wire.

Experiments on volume of flow over models of old Croton dam, Cornell University hydraulic laboratory, 1899—Continued.

	Period No.	Observed depth on model dam, in feet.	Mean velocity of approach, in feet per second.	Correction for velocity of approach, in feet.	Corrected head on model dam, in feet.	Discharge over model dam per foot of length, in cubic feet per second.	C_1
1	2	3	4	5	6	7	8
Series 1—Model B. Angular crest, old Croton dam, 16-foot crest, all unplaned plank. Nov. 15, 1899.	1	1.8635	0.973	0.0147	1.8782	9.506	3.693
	2	.9246	.370	.0021	.9267	3.272	3.668
	3	.6419	.219	.0008	.6427	1.870	3.630
	4	.3481	.090	.0001	.3482	.741	3.606
	5	.1787	.034	.0000	.1787	.272	3.601
Series 2—Model B. Angular crest, old Croton dam, 16-foot unplaned plank, crest slope roughened with cleats and stone. Nov. 28, 1899.	1	2.4126	1.298	.0262	2.4388	13.478	3.539
	2	1.5251	.736	.0084	1.5335	6.945	3.657
	3	.9611	.391	.0024	.9635	3.466	3.665
	4	.5157	.162	.0004	.5161	1.369	3.692
	5	.3051	.077	.0001	.3052	.631	3.742
	6	.0890	.012	.0000	.0890	.094	3.540
Series 2—Continued model B. Conditions as in preceding. Nov. 16, 1899.	1	1.8930	.988	.0151	1.9081	9.683	3.674
	2	.9605	.391	.0024	.9629	3.465	3.667
	3	.7028	.251	.0010	.7038	2.165	3.667
	4	.3941	.108	.0002	.3943	.900	3.635
	5	.1952	.039	.0000	.1952	.314	3.641
Series 3—Model B. Angular crest, old Croton dam, wire cloth on crest, rough slope. Nov. 16, 1899.	1	2.0053	1.047	.0170	a2.0183	10.385	3.622
	2	.9787	.389	.0024	.9771	3.463	3.586
	3	.7391	.259	.0011	.7362	2.241	3.548
	4	.1785	.032	.0000	.1745	.260	3.567
Series 1—Model C. Round crest, old Croton dam, 12-inch timber on crest, 16 feet long, rough slope. Dec. 1, 1899.	1	1.9941	1.097	.0187	2.0128	9.904	3.468
	2	1.1817	.512	.0040	1.1857	4.211	3.262
	3	.8832	.328	.0017	.8849	2.594	3.116
	4	.6873	.222	.0008	.6881	1.722	3.017
	5	.4986	.141	.0003	.4989	1.065	3.022
	6	.2992	.071	.0001	.2993	.522	3.188
	7	.1177	.019	.0000	.1177	.139	3.450
	8	.0846	.009	.0000	.0846	.067	2.723
Series 1—Continued model C. Conditions as in preceding. Dec. 4, 1899.	1	2.7146	1.632	.0414	2.7560	15.917	3.479
	2	2.4519	1.436	.0320	2.4839	13.629	3.482
	3	1.5566	.774	.0093	1.5659	6.660	3.399
	4	1.1046	.016	.0000	.1046	.112	3.311
	5	.1070	.0165	.0000	.1070	.118	3.371
Series 1—Model D. Angular crest, old Croton dam, 12-inch timber, on 16-foot crest, rough slope. Nov. 16, 1899.	1	1.2390	.495	.0038	1.2428	5.026	3.628
	2	.7885	.249	.0010	.7865	2.415	3.443
	3	.4448	.113	.0002	.4450	1.054	3.551

a In experiments with wire cloth over crest, 0.004 foot is deducted from observed depth to compensate for thickness of wire.

Experiments on volume of flow over models of old Croton dam, Cornell University hydraulic laboratory, 1899—Continued.

	Period No.	Observed depth on model dam, in feet.	Mean velocity of approach, in feet per second.	Correction for velocity of approach, in feet.	Corrected head on model dam, in feet.	Discharge over model dam per foot of length, in cubic feet per second.	C_1
1	2	3	4	5	6	7	8
	1	2.3051	1.154	0.0207	2.3258	11.778	3.321
	2	1.8125	.845	.0111	1.8236	8.214	3.336
Series 1—Model E.	3	1.2278	.507	.0040	1.2318	4.632	3.388
16-foot angular crest, old Croton dam, without timber, but with obstructed channel, with sharp contraction. Nov. 18, 1899.	4	.8598	.313	.0015	.8613	2.744	3.433
	5	.5745	.179	.0005	.5750	1.516	3.477
	6	.3245	.078	.0001	.3246	.641	3.466
	7	.1120	.017	.0000	.1120	.133	3.548
	8	.1102	.017	.0000	.1102	.140	3.827
	1	1.4569	.636	.0063	1.4632	5.957	3.366
	2	.9168	.338	.0018	.9186	2.982	3.387
Series 1—Model E, repeated.	3	.6866	.237	.0009	.6875	2.087	3.578
Conditions as in preceding. Nov. 25, 1899.	4	.4820	.148	.0003	.4823	1.240	3.702
	5	.2811	.071	.0001	.2812	.579	3.883
	6	.1416	.028	.0000	.1416	.226	4.208
	1	2.2927	1.139	.0202	2.3129	[a]11.613	3.302
Series 2—Model E.	2	2.2914	1.138	.0202	2.3116	[b]11.606	3.302
	3	1.1395	.473	.0034	1.1429	4.278	3.501
Angular crest, old Croton dam, 16 feet long without timber, and with slope instead of sharp edge to upstream end of obstruction. Nov. 27, 1899.	4	1.1403	.453	.0031	1.1434	[a]4.097	3.351
	5	1.1006	.448	.0031	1.1037	[b]4.034	3.479
	6	1.1099	.457	.0032	1.1131	4.119	3.507
	7	.4763	.141	.0003	.4766	1.180	3.586
	8	.0233	.031	.0000	.0233	.025	7.029

[a] Trap open. [b] Trap closed.

At the experimental weir two similar piezometers, each about one third of the width of the channel from the side, were used. Owing to the long back slope of some of the model dams, the head was measured 69.75 feet upstream from the crest of the experimental weirs. Readings of all the piezometers were taken at half-minute intervals, two and sometimes three observers working at each weir. The mean of ten to twenty observations was used to determine the head for each period in the experiment. Freeman states that he considers the results of these experiments for heads up to 2.5 feet, including all sources of errors, as certainly correct within 2 per cent, and probably much closer. In reducing the experiments, the head on the experimental weir is corrected by a method comparable with that of Francis. Freeman does not give the resulting coefficients for the weir formula, but presents the results in the form of diagrams showing the discharge per foot of crest for the various models. In the accompanying tables the computations have been carried out to show the coefficients, some errors in the original data having been omitted.

Column 3 shows the observed head on the experimental dam, in feet.

Column 7 shows the computed discharge over the experimental dam, per foot of crest. This was determined by calculating the discharge over the standard weir by means of both the Francis and Bazin formulas, the mean of the two having been used. The result corrected for slight leakage, divided by 16 (the length in feet of the experimental weir model), appears in column 7.

Columns 4 and 5 show the velocity of approach and the corresponding velocity head at the experimental weir. The velocity of approach correction was made by adding directly the velocity head as given to the observed depth on the model dam, this being considered a sufficiently close approximation to the Francis method of correction.

Columns 1 to 7 are taken from the original computations. The coefficient C_1 has been computed from the data in columns 6 and 7 by the formula

$$C_1 = \frac{Q}{H^{\frac{3}{2}}}.$$

Pls. XIX to XXII show the resulting coefficients applicable in the formula here adopted,

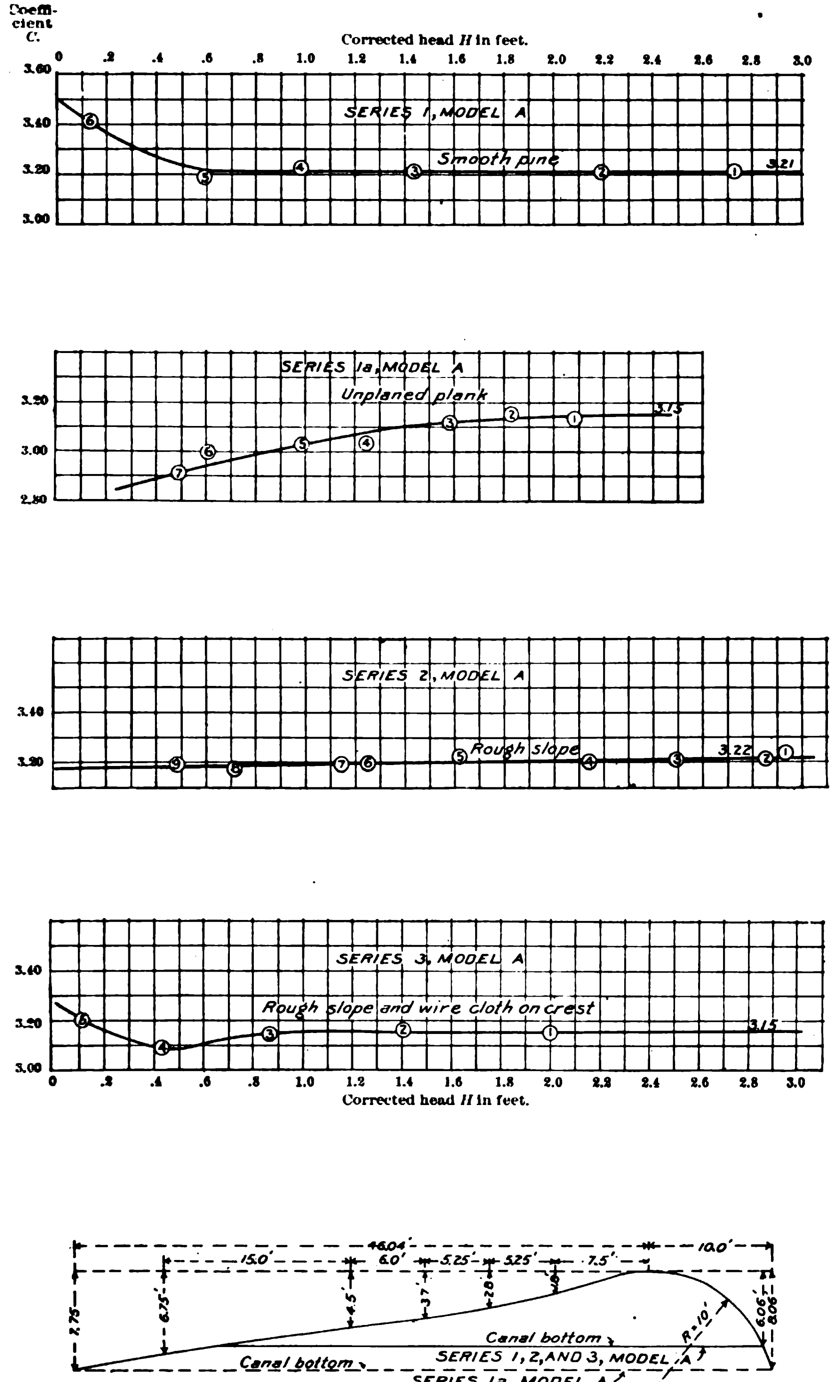
$$Q = C_1 L H^{\frac{3}{2}},$$

correction for velocity of approach being made by the Francis correction formula or an equivalent method.

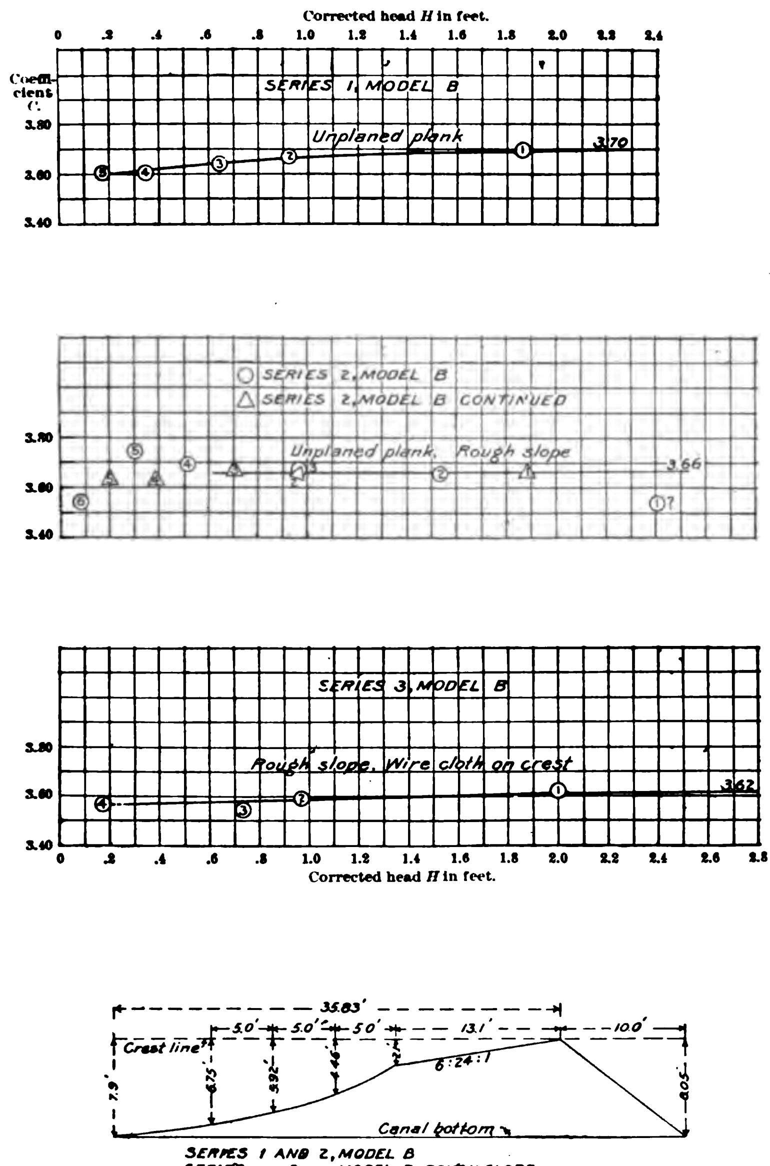
These experiments were performed for the specific purpose of determining the discharge over the old Croton dam. They include two main groups: (1) Experiments on round-crested portion of the dam; (2) experiments on the angular-crested portion of the dam. Each group includes series of experiments on: (*a*) Model of smooth-planed pine; (*b*) model of unplaned plank; (*c*) model with cleats and fragments of stone on the upstream slope to simulate the natural back filling; (*d*) model with rough slope and with ¼-inch-mesh wire cloth on crest to simulate cut stone; (*e*) model surmounted by 12-inch-square timber on crest. Experiments were added with a construction to simulate a natural rock ledge lying upstream from the angular portion of the dam.

The experiments were abbreviated owing to lateness of season and trouble from air in the gage pipes.

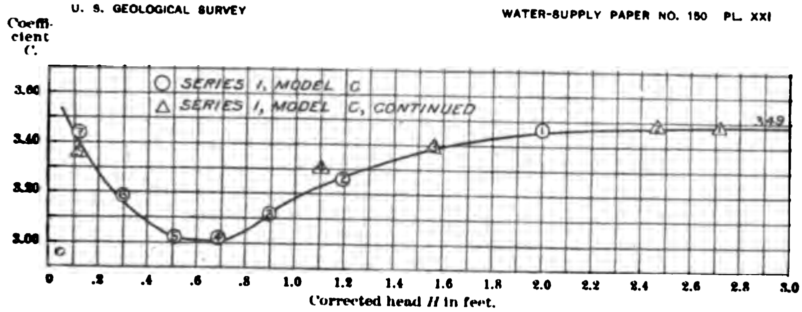
The value of the results is limited by the narrow range of heads covered. The models were of unusual forms, and show some peculiar differences when an attempt is made to compare the results with those of other weirs of similar slopes. The data are of value as showing the effect of various degrees of roughness on the discharge.

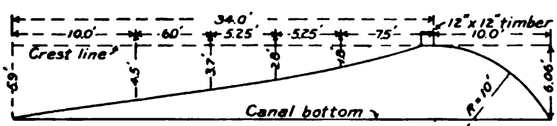


EXPERIMENTS ON ROUND-CRESTED MODELS OF OLD CROTON DAM.

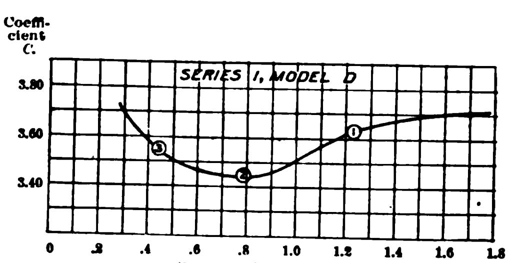


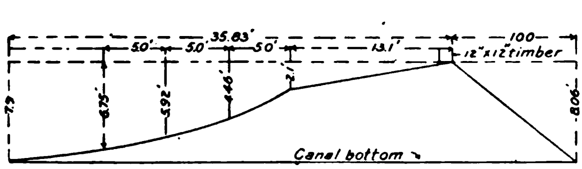
EXPERIMENTS ON ANGULAR-CRESTED MODELS OF OLD CROTON DAM.





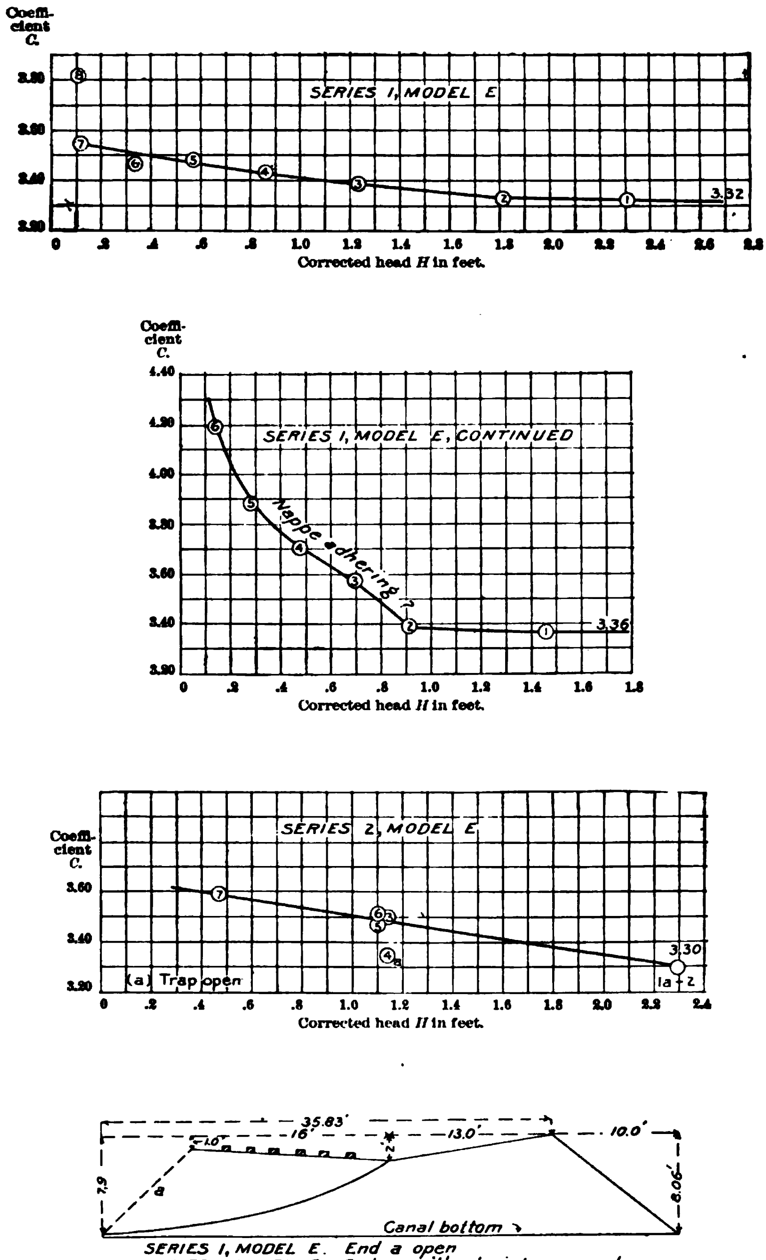
SERIES 1, MODEL C



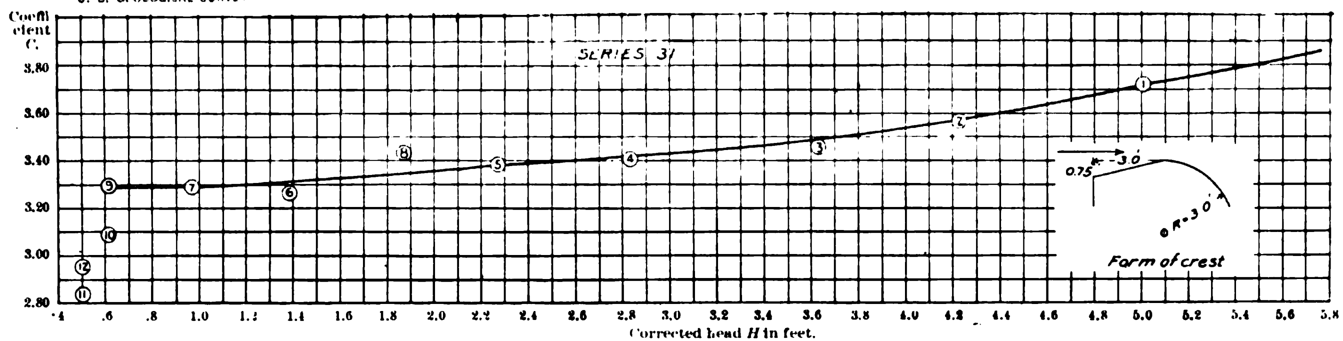


SERIES 1, MODEL D

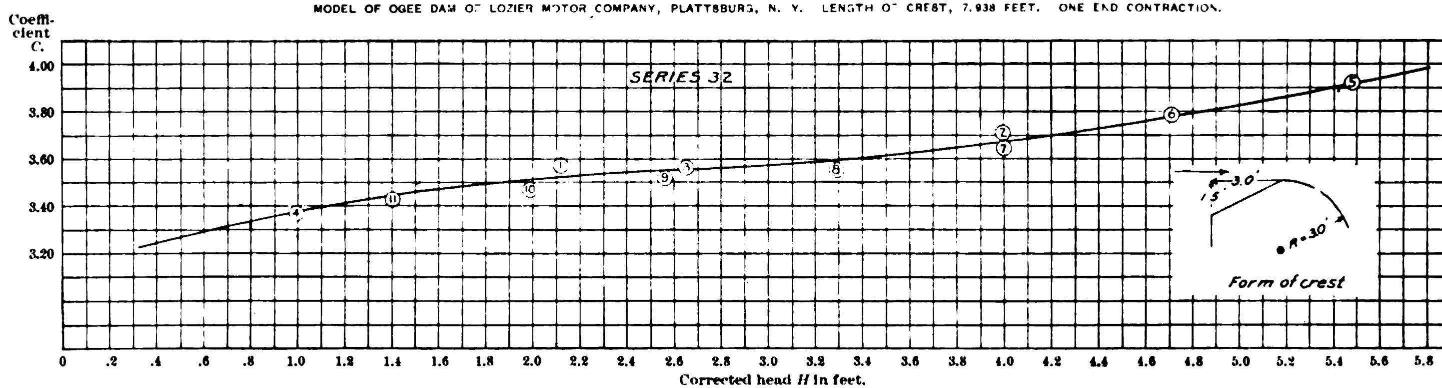
OLD CROTON DAM MODELS WITH CREST TIMBER.



ANGULAR CROTON DAM MODEL, WITH CONSTRUCTION TO SIMULATE ROCK LEDGE.

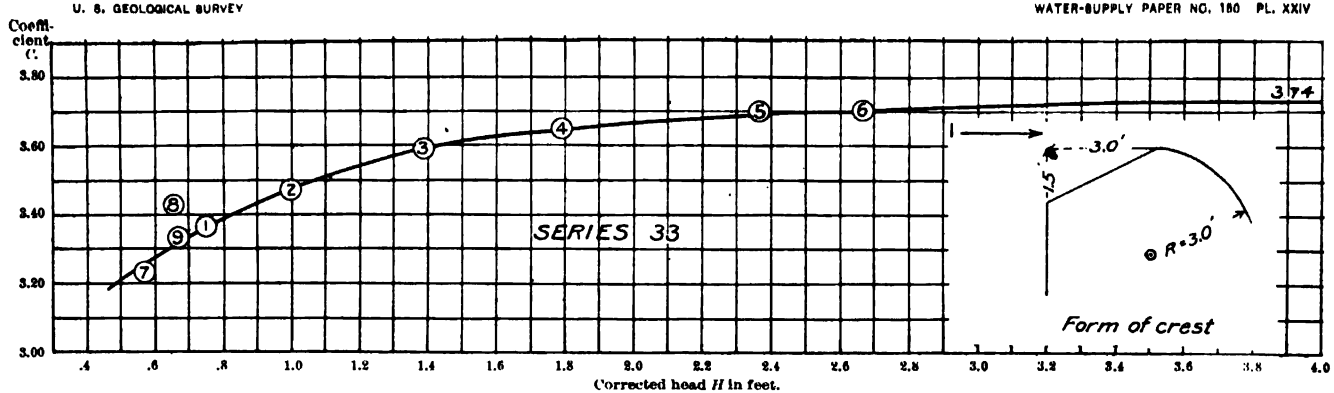


MODEL OF OGEE DAM OF LOZIER MOTOR COMPANY, PLATTSBURG, N. Y. LENGTH OF CREST, 7.938 FEET. ONE END CONTRACTION.

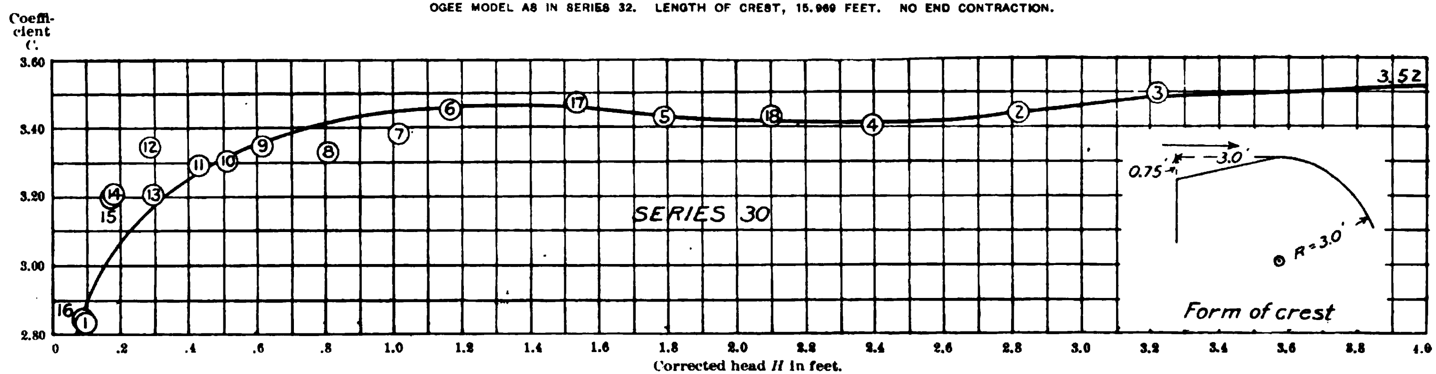


MODEL SIMILAR TO PLATTSBURG DAM, WITH BACK SLOPE MODIFIED. LENGTH OF CREST, 7.979 FEET. ONE END CONTRACTION.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.



OGEE MODEL AS IN SERIES 32. LENGTH OF CREST, 15.969 FEET. NO END CONTRACTION.



MODEL OF OGEE DAM OF LOZIER MOTOR COMPANY, PLATTSBURG, N. Y. LENGTH OF CREST, 15.969 FEET.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY AT CORNELL UNIVERSITY HYDRAULIC LABORATORY.

In April, 1903, the writer was instructed to plan and execute a series of experiments on models of dams similar to those in use at gaging stations of the Geological Survey in New York, Michigan, and elsewhere.

The experiments were performed at the hydraulic laboratory of Cornell University, mainly during the months of May and June, 1903, and were conducted, under the supervision of the writer, by Prof. Gardner S. Williams, director of the laboratory.

The various types of dams most commonly occurring were grouped as follows:

1. Weirs with broad horizontal or slightly inclined crests.
2. Weirs with vertical downstream faces and inclined upstream slopes.
3. Weirs having compound slopes, including those with inclined upstream faces and with either broad crests or with sloping aprons.
4. Completely or partially curved weir sections, including those of ogee profile.

It was found impossible to include in the experiments all the forms of section desired, and it was accordingly determined to limit the experiments to the thorough study of two classes—weirs with broad crests and weirs with ogee sections—and to extend, if possible, the measurements to include dams with vertical downstream faces and sloping upstream approaches. The order of operation used in previous experiments was transposed, the experimental models being built on a bulkhead forming the standard weir hitherto used and located near the head of the experimental canal.

The quantity of water passing over the experimental weir was measured on a standard weir below, 6.65 feet high and having a crest length of 15.93 feet. The head on the standard weir was measured in a Bazin pit, 3 by 4 feet in section, reaching to the depth of the bottom of the canal, and communicating therewith through a pipe 4 inches in diameter and about 3.5 feet long, opening at the bottom of the channel of approach, 29.88 feet upstream from the weir. The head on the standard weir was observed in the gage pit by means of a hook gage reading to millimeters and estimated to about one-fifth millimeter. The conditions at the standard weir were thus closely comparable to those obtained in Bazin's experiments, and his formula for this height and length of weir was applied to determine the discharge. Observations to determine the leakage between the experimental and standard weirs were made, and corrections were applied for whatever leakage was indicated, the amount being usually less than 0.01 cubic foot per second per foot of crest. The discharge over the standard weir was computed in cubic meters per second and has been reduced to cubic feet per second, the discharge table being as follows:

Discharge over standard weir at different heads.

Head, in meters.	Q in cubic meters, per second.	Head, in meters.	Q in cubic meters, per second.
0.05	0.111863	0.60	4.21730
.10	.296230	.70	5.34459
.15	.53207	.80	6.57096
.20	.81166	.90	7.89078
.25	1.12871	1.00	9.30650
.30	1.48032	1.10	10.81066
.40	2.27850	1.20	12.40420
.50	3.19350		

The discharge curve for the standard weir has also been carefully checked by comparing the depth flowing over with that on a similar weir, using the formula and method of determining the head adopted by Fteley and Stearns; it has also been checked by float and current-meter measurements, and for lower heads by means of volumetric measurement of the discharge in the gaging channel, so that it is believed that the discharge in these experiments is known within 1 or 2 per cent of error as a maximum.

The work of calibrating the standard weir had been accomplished by Professor Williams and his assistants before the experiments of the United States Geological Survey were taken up, so that somewhat more certainty attaches to the results of these later experiments than to earlier experiments made before the standard-weir discharge had been thoroughly checked.

It was the wish of the Geological Survey that the conditions at the experimental weirs should conform to those actually existing at dams which are utilized as weirs, in connection with the stream-gaging operations. In such cases it is often impracticable to utilize gage pits of the form adopted by Bazin or to use piezometer or hook gages. The usual method is to read the depth directly on a graduated vertical scale or measure the distance to water surface from a suitable bench mark. The method adopted in the weir experiments consisted of reading directly the distance to water surface from bench marks located above the central line of the channel. The readings were taken by means of a needle-pointed plumb bob attached to a steel tape forming a point gage, readings being taken to thousandths of a foot.

Two gages were used, one located 10.3 feet upstream from the crest and another 16.059 feet upstream. In series XXXV and following, for the higher heads, the readings of the upstream tape were used. For heads where no general difference was apparent the average of the readings of the two tapes was taken. In general, the surface curve did not perceptibly affect the reading of the gage nearest the weir for depths below 3 feet. The readings of the tapes were checked from time to time by observations with hook gages, thus practically eliminating the effect of temperature on the tapes. Observations of the head were usually taken at intervals of thirty seconds. Great care was used to maintain a uniform regimen of flow during each experimental period, and the variations of head were very slight. The character of the observations is illustrated by the following data taken from the experiments:

Readings of tapes to determine head at experimental weir.

Series XL. Period 10. Date 6, 22, 08.				Series XLI. Period 5. Date 6, 23, 08.			
Time.			Readings.	Time.			Readings.
h.	*m.*	*s.*		*h.*	*m.*	*s.*	
12	37	40	42.681	1	34	30	43.633
			.681				.632
			.680				.633
			.681				.630
12	39	00	.682				.630
12	52	30	42.680				.633
			.681				.635
			.680				.635
			.679				.634
12	54	30	.680				.633
Mean			42.6805				.630
							.635
Series XLIII. Period 3. Date 6, 26, 08.							.630
							.635
							.635
Time.			Readings.				.633
							.630
h.	*m.*	*s.*					.630
12	36	20	2.305				.630
			.302				.630
			.302				.630
			.302				.634
			.301				.630
			.302				.628
			.301				.630
			.300				.632
12	42	10	.300				.628
Mean			2.3015	1	51	20	.631
				Mean			43.6317

For the lower heads the discharge over the experimental weir was volumetrically determined by measuring the rise of water in the canal, as follows:

List of experimental periods for which the discharge was volumetrically determined.

Series.	Periods.	Series.	Periods.	Series.	Periods.
30	1, 13, 14, 16	39	1	44	1, 2, 3
31	10, 11	40	1, 2	45	1, 2, 3
34	1, 9, 10	41	13, 14	46	1, 2, 3
37	5 *a*	43	1, 2, 3	47	1, 2, 3
38	5, 6	43 *a*	1, 2, 3, 4		

United States Geological Survey experiments at Cornell University hydraulic laboratory on model of Plattsburg dam.

[Series No. XXX. Height of weir $=P$, 11.25 feet; length of weir crest $=L$, 15.969 feet; width of channel $=b$, 15.970 feet; height of upstream crest corner, 10.50 feet; crest width, 3 feet.]

No.	Measured head on experimental weir, in feet.				$A=$ area of section per foot of crest.	$v=$ mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet.		$Q=$ discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
	Number of observations.	Maximum.	Minimum.	Mean $=D$.			$H^{\frac{3}{2}}$	H		
1	2	3	4	5	6	7	8	9	10	11
1	92	0.0993	0.0818	0.096	11.347	0.007	0.030	0.096	0.084	2.835
2	31	.7993	.7863	2.790	14.041	1.155	4.710	2.810	16.218	3.443
3	19	3.1993	3.1473	3.187	14.438	1.396	5.766	3.216	20.152	3.495
4	26	2.3993	2.3173	2.384	13.635	.926	3.710	2.396	12.631	3.405
5	29	1.7973	1.7853	1.793	13.043	.636	2.412	1.799	8.282	3.433
6	27	1.1803	1.1723	1.174	12.425	.354	1.274	1.176	4.399	3.454
7	18	1.0133	1.0073	1.010	12.260	.280	1.016	1.011	3.435	3.381
8	16	.8043	.8003	.802	12.053	.200	.719	.803	2.409	3.348
9	34	.6143	.6113	.613	11.864	.136	.481	.614	1.608	3.345
10	24	.5083	.5073	.508	11.758	.102	.362	.508	1.195	3.302
11	15	.4283	.4263	.427	11.678	.079	.280	.428	.921	3.296
12	20	.2943	.2903	.291	11.542	.046	.157	.291	.526	3.344
13	32	.2933	.2903	.292	11.543	.044	.158	.292	.506	3.209
14	21	.1783	.1763	.178	11.427	.021	.075	.178	.240	3.202
15	24	.1793	.1733	.179	11.429	.020	.076	.179	.226	2.976
16	40	.0893	.0893	.089	11.340	.007	.027	.089	.076	2.841
17	26	1.5373	1.5303	1.532	12.783	.517	1.904	1.535	6.607	3.471
18	25	2.1033	2.0843	2.094	13.345	.793	3.051	2.104	10.576	3.466

United States Geological Survey experiments at Cornell University hydraulic laboratory on model of Plattsburg dam—Continued.

[Series No. XXXI. Height of weir=*P*, 11.25 feet; length of weir crest=*L*, 7.938 feet; width of channel=*b*, 15.970 feet; height of upstream crest corner, 10.50 feet; width of crest, 3 feet.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean =*D*.	*A*= area of section per foot of crest.	*v*= mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	*H*	*Q*=discharge per foot of crest, in cubic feet per second.	Discharge coefficient *C*.	*L*=effective length of crest weir.
1	2	3	4	5	6	7	8	9	10	11	12
1	25	5.1003	4.9273	5.0014	259.535	1.201	11.2[illegible]7	5.023	41.925	3.724	7.436
2	15	4.2873	4.1693	4.2214	247.078	.945	8.714	4.235	31.086	3.567	7.514
3	44	3.6893	3.5253	3.6191	507.460	.356	6.890	3.621	23.868	3.464	7.580
4	21	2.8213	2.8003	2.8185	224.674	.551	4.743	2.823	16.170	3.409	7.656
5	39	2.2713	2.2623	2.2696	215.780	.412	3.425	2.273	11.545	3.377	7.711
6	40	1.3903	1.3813	1.3861	201.998	.206	1.633	1.387	5.338	3.269	7.799
7	26	.9673	.9653	.9663	195.094	.125	.950	.966	3.120	3.284	7.841
8	25	1.8793	1.8723	1.8749	209.605	.326	2.570	1.876	8.814	3.429	7.750
9	30	.6093	.6083	.6087	189.383	.065	.475	.609	1.562	3.288	7.877
10	18	.6073	.6063	.6063	189.345	.060	.472	.606	1.456	3.083	7.877
11	40	.3023	.2993	.3017	184.471	.020	.165	.301	.469	2.838	7.908
12	19	.3003	.2993	.3017	184.481	.021	.166	.302	.490	2.956	7.908

[Series No. XXXII. Height of weir=*P*, 11.25 feet; length of weir crest=*L*, 7.979 feet; width of channel=*b*, 15.970 feet; height of upstream crest corner, 9.75 feet; width of crest, 3 feet.]

1	10	2.1193	2.0943	2.1173	213.476	0.004	3.081	2.117	11.029	3.580	7.767
2	23	4.0343	3.9883	4.0053	243.627	.928	8.055	4.018	29.846	3.706	7.577
3	26	2.6693	2.6333	2.6469	221.934	.535	4.317	2.651	15.398	3.567	7.714
4	34	.9953	.9923	.9942	195.540	.135	.992	.994	3.342	3.370	7.880
5	32	5.5543	5.2983	5.4712	267.038	1.404	12.855	5.487	50.440	3.924	7.430
6	27	4.7343	4.6443	4.6947	254.637	1.141	10.235	4.714	38.701	3.782	7.508
7	27	4.0173	3.9883	4.0007	240.808	.925	8.040	4.013	29.395	3.656	7.578
8	28	3.3053	3.2673	3.2890	232.188	.701	5.984	3.296	21.278	3.556	7.649
9	31	2.5863	2.5193	2.5596	220.539	.506	4.104	2.564	14.460	3.524	7.723
10	28	1.9913	1.9693	1.9801	211.285	.357	2.790	1.982	9.686	3.471	7.781
11	53	1.4113	1.4023	1.4015	202.044	.221	1.661	1.402	5.704	3.434	7.889

United States Geological Survey experiments at Cornell University hydraulic laboratory on model of Plattsburg dam—Continued.

[Series No. XXXIII. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel= b, 15.970 feet; height of upstream crest corner, 9.79 feet; width of crest, 3 feet.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean = D.	A = area of section per foot of crest.	v = mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	15	0.7563	0.7493	0.752	12.003	0.183	0.653	0.753	2.197	3.363
2	43	.9973	.9873	.992	12.243	.281	.990	.993	3.439	3.473
3	29	1.3963	1.3903	1.394	12.644	.468	1.651	1.397	5.913	3.580
4	33	1.7893	1.7803	1.784	13.034	.671	2.396	1.790	8.747	3.651
5	23	2.3733	2.3368	2.352	13.603	.990	3.641	2.367	13.464	3.698
6	24	2.6683	2.6503	2.660	13.910	1.170	4.387	2.680	16.272	3.709
7	52	.5593	.5493	.553	11.803	1.183	.432	.572	1.397	3.232
8	42	.6603	.6493	.653	11.904	.152	.528	.654	1.808	3.422
9	12	.6804	.6783	.679	11.930	.156	.560	.680	1.866	3.330

[Series No. XXXIV. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; height of upstream crest corner, 8.37 feet; width of crest, 3 feet.]

1	16	0.6453	0.6383	0.641	11.891	0.137	0.513	0.641	1.632	3.182
2	16	.6423	.6383	.640	11.891	.141	.513	.640	1.680	3.276
3	21	2.0253	2.0043	2.015	13.266	.816	2.882	2.025	10.826	3.756
4	19	1.6303	1.6203	1.628	12.878	.592	2.086	1.633	7.627	3.656
5	19	1.2333	1.2293	1.232	12.483	.391	1.371	1.235	4.877	3.555
6	14	.9523	.9483	.950	12.201	.261	.927	.951	3.185	3.434
7	20	.4013	.3983	.400	11.650	.070	.253	.400	.815	3.225
8	10	.6243	.6243	.624	11.875	.136	.493	.625	1.613	3.269
9	32	.2243	.2203	.222	11.473	.029	.105	.222	.328	3.133
10	39	.1103	.1093	.110	11.360	.009	.036	.110	.100	2.777
11	14	5.0963	5.0793	3.089	14.340	1.509	5.515	3.122	21.636	3.923
12	25	2.7763	2.7263	2.749	14.000	1.258	4.615	2.772	17.612	3.816
13	31	2.4383	2.3963	2.421	13.672	1.051	3.805	2.437	14.374	3.778
14	25	2.8013	2.7963	.800	12.050	.194	.716	.800	2.341	3.271

United States Geological Survey experiments at Cornell University hydraulic laboratory on model of Chambly dam.

[Series No. XXXV. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; height of upstream crest corner, 10.25 feet; width of crest 4.5 feet.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean $=D$.	A = area of section per foot of crest.	v= mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	64	2.5503	2.5463	0.549	11.799	0.110	0.407	0.549	1.297	3.189
2	55	1.0103	1.0063	1.008	12.259	.272	1.014	1.009	3.331	3.285
3	40	1.5643	1.5523	1.559	12.810	.511	1.954	1.563	6.548	3.352
4	40	2.0273	2.0133	2.021	13.272	.744	2.890	2.029	9.874	3.416
5	38	1.7513	1.7303	1.739	12.990	.602	2.304	1.744	7.816	3.392
6	45	1.2673	1.2583	1.262	12.513	.379	1.422	1.265	4.747	3.337
7	41	.7593	.7543	.757	12.008	.176	.660	.758	2.114	3.205
8	43	.4453	.4373	.441	11.692	.079	.293	.441	.919	3.132
9	20	.3043	.3003	.302	11.553	.045	.166	.302	.525	3.159
11	18	3.7303	3.7063	3.755	15.006	1.740	7.403	3.798	26.114	3.528
12	17	3.2193	3.1813	3.195	14.446	1.406	5.789	3.224	20.319	3.510
13	17	3.0023	2.9873	2.987	14.238	1.283	5.224	3.011	18.267	3.496
14	18	2.6743	2.6643	2.642	13.892	1.077	4.334	2.658	14.963	3.452
15	23	2.3533	2.3283	2.317	13.568	.901	3.555	2.330	12.221	3.438
16	22	1.4923	1.4873	1.463	12.713	.478	1.775	1.466	6.072	3.420
17	37	.2103	.2093	.184	11.435	.023	.079	.184	.258	3.260
18	25	.3913	.3903	.391	11.642	.065	.245	.391	.758	3.096
19	17	.3313	.3313	.331	11.582	.053	.191	.331	.609	3.194
20	24	.2523	.2523	.253	11.504	.035	.127	.253	.404	3.258
21	21	.2203	.2193	.221	11.472	.028	.104	.221	.325	3.120
22	24	.1843	.1833	.183	11.434	.021	.078	.183	.238	3.039
23	19	.1323	.1313	.132	11.383	.012	.048	.132	.136	2.826
24	20	.0823	.0823	.083	11.334	.007	.024	.083	.079	3.321

[Series No. XXXVI. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; height of upstream crest corner, 10.50 feet; width of crest, 4.5 feet, with 4 inches radius quarter round.]

1	18	2.7653	2.6913	2.741	13.991	1.185	4.594	2.764	16.586	3.610
2	18	2.3513	2.2913	2.316	13.567	.936	3.558	2.330	12.698	3.569
3	30	2.9373	2.8923	1.915	13.166	.702	2.666	1.923	9.243	3.467
4	23	1.5173	1.4993	1.507	12.758	.501	1.857	1.511	6.390	3.441
5	21	1.1143	1.1013	1.111	12.361	.320	1.173	1.112	3.950	3.367
6	19	.7553	.7493	.753	12.004	.177	.654	.753	2.123	3.248
7	22	.4943	.4903	.492	11.743	.095	.345	.492	1.113	3.221

United States Geological Survey experiments at Cornell hydraulic laboratory on model of Dolgeville dam with injured apron.

[Series No. XXXVII. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; height of upstream crest corner, 10.25 feet; width of crest, 6 feet.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean=D.	A=area of section per foot of crest.	v=mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	30	0.9203	0.9113	0.916	12.166	0.251	0.878	0.917	3.049	3.474
2	35	3.5613	3.4233	3.565	14.816	1.535	6.828	3.599	22.744	3.331
3	28	2.9123	2.8573	2.927	14.178	1.170	5.060	2.947	16.593	3.279
4	52	2.3293	2.2903	2.324	13.574	.906	3.570	2.336	12.297	3.444
5	59	1.6413	1.6213	1.635	12.885	.585	2.100	1.640	7.544	3.593
5a	41	1.3973	1.3923	.415	11.666	.069	.267	.415	.810	3.029

United States Geological Survey experiments at Cornell hydraulic laboratory on model of Dolgeville dam.

[Series No. XXXVIII. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; height of upstream crest corner, 10.25 feet; width of crest, 600 feet.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean=D.	A=area of section per foot of crest.	v=mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	32	0.3973	0.3913	0.395	11.646	0.068	0.249	0.396	0.790	3.176
2	36	1.6843	1.6713	1.689	12.940	.605	2.206	1.695	7.826	3.548
3	25	1.1143	1.1093	1.112	12.362	.333	1.175	1.114	4.113	3.501
4	32	.7513	.7473	.749	12.000	.185	.649	.750	2.223	3.426
5	34	.5093	.5083	.504	11.754	.101	.358	.504	1.186	3.310
6	24	.2173	.2113	.209	11.460	.026	.096	.209	.294	3.061
7	27	3.4726	3.4586	3.470	14.721	1.466	6.551	3.501	21.583	3.295
8	34	3.1176	3.0806	3.100	14.351	1.271	5.521	3.124	18.238	3.303
9	29	2.6176	2.5926	2.605	13.856	1.019	4.243	2.621	14.124	3.329
10	30	2.2096	2.1806	2.198	13.449	.846	3.283	2.209	11.381	3.467
11	28	1.9306	1.8156	1.920	13.171	.726	2.677	1.928	9.558	3.570
12	28	1.5286	1.5106	1.517	12.768	.514	1.872	1.519	6.557	3.503

[Series No. XXXIX. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.700 feet; height of upstream crest corner, 10.25 feet; width of crest, 6 feet.]

1	26	0.6883	0.6793	0.683	11.934	0.170	0.565	0.684	2.027	3.586
2				.683	11.934	.167	.565	.684	1.995	3.530
3	22	.4183	.4153	.418	11.668	.081	.270	.418	.944	3.497
4	33	3.9806	3.9206	3.943	15.194	1.721	7.956	3.985	26.150	3.287
5	24	3.1506	3.1366	3.145	14.396	1.280	5.642	3.169	18.431	3.266
6	23	2.5066	2.4806	2.488	13.739	.966	3.958	2.502	13.268	3.352
7	36	1.8986	1.8756	1.884	13.135	.699	2.601	1.891	9.186	3.532
8	32	1.2716	1.2626	1.268	12.518	.401	1.432	1.270	5.025	3.509
9	39	.8286	.8206	.826	12.077	.218	.752	.827	2.639	3.494
10	32	.5626	.5606	.561	11.811	.125	.420	.561	1.481	3.526

United States Geological Survey experiments at Cornell hydraulic laboratory on model of flat-top weirs with vertical faces.

[Series No. XL. Height of weir = P, 11.25 feet; length of weir crest = L, 15.969 feet; width of channel = b, 15.970 feet; width of broad crest, 0.479 foot; nappe aerated.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean = D.	A = area of section per foot of crest.	v = mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q = discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	19	0.6343	0.6293	0.631	11.882	0.133	0.502	0.632	1.580	3.148
2	17	.2643	.2593	.260	11.511	.030	.133	.260	.343	2.584
3	18	.2613	.2513	.264	11.515	.033	.136	.264	.380	2.794
4	30	.1216	.1206	.124	11.375	.011	.044	.124	.129	2.943
5	33	1.9916	1.9816	1.989	13.238	.710	2.821	1.996	9.401	3.332
6	36	1.6256	1.6106	1.618	12.868	.530	2.065	1.622	6.819	3.301
7	26	1.2556	1.2496	1.256	12.507	.377	1.412	1.259	4.713	3.338
8	21	.9756	.9706	.977	12.228	.262	.967	.978	3.209	3.318
9	21	.8213	.8163	.820	12.070	.204	.743	.820	2.469	3.325
10	10	.6513	.6493	.650	11.900	.139	.524	.650	1.654	3.154
11	10	.4503	.4483	.449	11.700	.074	.301	.449	.867	2.881

[Series No. XLI. Height of weir = P, 11.25 feet; length of weir crest = L, 15.969 feet; width of channel = b, 15.970 feet; width of broad crest, 1.646 feet; nappe partly aerated.]

1	25	3.8606	3.8256	3.842	15.092	1.692	7.651	3.883	25.531	3.337
2	25	3.1906	3.1666	3.177	14.428	1.317	5.730	3.202	18.995	3.315
3	26	2.6906	2.6656	2.674	13.925	1.050	4.413	2.690	14.624	3.314
4	31	2.0306	2.0116	2.022	13.272	.680	2.889	2.028	9.021	3.123
5	28	1.6043	1.5973	1.601	12.852	.462	2.032	1.604	5.936	2.922
6	27	1.2373	1.2293	1.233	12.484	.307	1.372	1.234	3.835	2.796
7	25	.9443	.9393	.942	12.192	.203	.915	.942	2.476	2.706
8	34	.6733	.6693	.671	11.922	.123	.575	.692	1.472	2.560
9	30	.4893	.4873	.488	11.739	.078	.341	.488	.910	2.669
10	18	.3303	.3293	.330	11.581	.045	.190	.330	.520	2.742
11	34	.2113	.2093	.210	11.461	.024	.096	.210	.272	2.827
12	....	....	....	.122	11.373	.011	.043	.122	.130	3.047
13	22	.1253	.1213	.788	12.038	.153	.699	.788	1.840	2.631
14	20	.4206	.4136	.417	11.668	.064	.270	.417	.750	2.782
15	44	.4206	.4126	.417	11.668	.065	.270	.417	.759	2.815

[Series No. XLII. Height of weir = P, 11.25 feet; length of weir crest = L, 15.969 feet; width of channel = b, 15.970 feet; width of broad crest, 12.239 feet; nappe partly aerated.]

1	33	0.1706	0.1626	0.168	11.418	0.016	0.069	0.168	0.180	2.611
2	32	4.3706	4.3416	4.353	15.604	1.584	9.196	4.389	24.716	2.688
3	26	3.8316	3.8006	3.809	15.060	1.321	7.510	3.835	19.896	2.649
4	38	3.0446	3.0256	3.032	14.283	.971	5.317	3.046	13.876	2.610
5	32	3.7356	3.7186	1.728	12.979	.467	2.278	1.732	6.066	2.663
6	29	3.5856	3.5776	2.580	13.831	.787	4.166	2.589	10.882	2.612
7	38	3.3966	3.3806	3.387	14.638	1.130	6.285	3.406	16.535	2.631
8	27	2.2506	2.2406	2.243	13.494	.657	3.373	2.249	8.870	2.629
9	34	1.4706	1.4526	1.449	12.700	.369	1.748	1.451	4.682	2.701
10	22	1.0986	1.0906	1.096	12.347	.253	1.149	1.097	3.129	2.723
11	36	.6406	.6376	.639	11.890	.116	.511	.639	1.375	2.689
12	28	.6126	.6106	.611	11.862	.109	.478	.611	1.290	2.700

United States Geological Survey experiments at Cornell hydraulic laboratory on model of flat-top weirs with vertical faces—Continued.

[Series No. XLIII. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 16.302 feet; surface somewhat rough; nappe partly aerated.]

No.	Measured head on experimental weir, in feet. Number of observations.	Maximum.	Minimum.	Mean=D.	A=area of section per foot of crest.	v=mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet. $H^{\frac{3}{2}}$	H	Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
1	2	3	4	5	6	7	8	9	10	11
1	12	0.8536	0.8506	0.851	12.102	0.168	0.786	0.852	2.027	2.579
2	12	.4496	.4426	.447	11.698	.069	.299	.447	.811	2.713
3	10	.3246	.3106	.312	11.563	.040	.174	.312	.468	2.684
4	26	.6936	.6856	.689	11.940	.129	.573	.690	1.544	2.696
5	33	4.4506	4.4176	4.432	15.683	1.595	9.449	4.469	25.011	2.647
6	27	3.6806	3.6306	3.661	14.912	1.251	7.076	3.686	18.657	2.637
7	34	2.9426	2.9306	2.935	14.186	.930	5.062	2.948	13.200	2.608
8	30	2.3626	2.3586	2.360	13.611	.706	3.644	2.368	9.607	2.637
9	23	1.8956	1.8866	1.890	13.141	.520	2.607	1.894	6.841	2.624
10	25	1.4826	1.4756	1.480	12.731	.374	1.804	1.482	4.767	2.531
11	26	1.2086	1.2026	1.206	12.457	.282	1.327	1.208	3.520	2.652

[Series No. XLIII*a*. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 16.302 feet; smooth planed surface.]

1	19	0.3626	0.3596	0.361	11.612	0.050	0.217	0.361	0.576	2.653
2	20	.2496	.2426	.246	11.497	.026	.122	.246	.305	2.494
3	44	.1686	.1646	.167	11.418	.013	.068	.167	.153	2.240
4	16	.9906	.9846	.986	12.236	.214	.980	.986	2.618	2.673
5	28	.9856	.9776	.981	12.232	.210	.973	.982	2.568	2.638
6	23	.7886	.7826	.786	12.036	.153	.697	.786	1.847	2.651
7	31	.6226	.6206	.621	11.872	.110	.490	.622	1.309	2.670
8	21	.4976	.4946	.496	11.747	.080	.350	.496	.945	2.704
9	25	.3926	.3906	.392	11.643	.058	.246	.392	.670	2.728
10	31	.2806	.2786	.280	11.530	.036	.148	.280	.421	2.848
11	28	.1606	.1606	.161	11.411	.016	.064	.161	.184	2.851
12	24	.0776	.0756	.077	11.327	.006	.021	.077	.066	3.127

[Series No. XLIV. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 8.980 feet.]

1	27	0.3136	0.3116	0.312	11.563	0.040	0.174	0.312	0.469	2.691
2	20	.1596	.1576	.159	11.410	.014	.064	.159	.163	2.570
3	32	.4196	.4176	.419	11.670	.058	.271	.419	.679	2.504
4	20	3.0666	3.0506	3.058	14.309	.986	5.386	3.073	14.105	2.619
5	19	2.3306	2.3116	2.319	13.569	.686	3.547	2.326	9.307	2.624
6	17	2.8656	2.8486	1.856	13.107	.512	2.537	1.860	6.712	2.646
7	31	1.5166	1.5106	1.513	12.763	.386	1.864	1.515	4.928	2.642
8	28	1.2556	1.2506	1.252	12.503	.297	1.404	1.254	3.718	2.649
9	31	1.0396	1.0356	1.038	12.289	.228	1.059	1.039	2.803	2.646
10	29	.8996	.8916	.897	12.148	.186	.850	.897	2.254	2.652
11	26	.7326	.7306	.732	11.983	.140	.627	.732	1.676	2.676
12	32	.5046	.5006	.502	11.753	.083	.356	.502	.971	2.727

United States Geological Survey experiments at Cornell hydraulic laboratory on model of flat-top weirs with vertical faces—Continued.

[Series No. XLV. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 5.875 feet; nappe partly aerated.]

No.	Measured head on experimental weir, in feet.				A= area of section per foot of crest.	v= mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet.		Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
	Number of observations.	Maximum.	Minimum.	Mean=D.			$H^{\frac{3}{2}}$	H		
1	2	3	4	5	6	7	8	9	10	11
1	24	0.1766	0.1726	0.174	11.424	0.018	0.072	0.174	0.207	2.857
2	32	.2556	.2526	.253	11.504	.029	.127	.253	.337	2.647
3	38	.3906	.3886	.390	11.640	.055	.243	.390	.641	2.635
4	31	.9906	.9786	.982	12.233	.209	.975	.983	2.557	2.624
5	31	1.2456	1.2396	1.242	12.492	.293	1.386	1.243	3.666	2.645
6	32	.9126	.9066	.908	12.159	.189	.867	.909	2.294	2.646
7	42	.7346	.7306	.733	11.983	.139	.627	.733	1.670	2.663
8	26	1.0006	.9916	1.996	13.247	.564	2.830	2.001	7.469	2.639
9	33	.5906	.5806	1.585	12.836	.410	2.000	1.587	5.264	2.632
10	23	.5916	.5896	.590	11.841	.103	.454	.590	1.220	2.689
11	36	.5216	.5116	.520	11.771	.087	.376	.521	1.022	2.722

[Series No. XLVI. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 3.174 feet; nappe partly aerated.]

1	26	0.2526	0.2446	0.250	11.501	0.029	0.125	0.250	0.333	2.665
2	41	.1916	.1896	.191	11.441	.019	.083	.191	.221	2.660
3	34	.4186	.4156	.417	11.668	.066	.269	.417	.766	2.845
4	23	2.9686	2.9416	2.965	14.216	1.048	5.147	2.981	14.901	2.895
5	30	2.4956	2.4806	2.486	13.737	.803	3.943	2.496	11.032	2.798
6	32	2.0376	2.0126	2.030	13.280	.594	2.903	2.035	7.895	2.720
7	33	1.6006	1.5906	1.597	12.847	.417	2.022	1.599	5.360	2.650
8	32	1.2326	1.2286	1.232	12.483	.291	1.370	1.234	3.628	2.647
9	32	.9726	.9706	.972	12.222	.208	.959	.972	2.549	2.658
10	31	.7866	.7816	.784	12.035	.154	.695	.785	1.856	2.670
11	38	.6026	.6006	.602	11.852	.106	.467	.602	1.254	2.686
12	33	.5056	.5026	.503	11.754	.082	.357	.503	.967	2.706

[Series No. XLVII. Height of weir=P, 11.25 feet; length of weir crest=L, 15.969 feet; width of channel=b, 15.970 feet; width of broad crest, 0.927 foot; nappe partly aerated.]

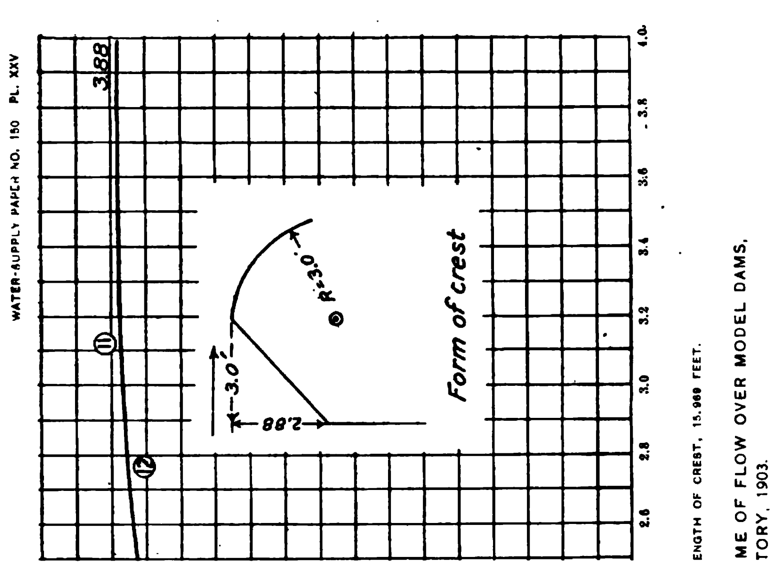
1	27	0.1666	0.1636	0.165	11.416	0.016	0.067	0.165	0.180	2.690
2	29	.2816	.2726	.278	11.529	.033	.147	.278	.377	2.563
3	31	.4156	.4106	.412	11.663	.060	.265	.412	.700	2.644
4	29	2.9446	2.9206	2.933	14.184	1.187	5.076	2.954	16.840	3.318
5	29	2.5306	2.5116	2.522	13.772	.970	4.037	2.535	13.360	3.314
6	26	2.0196	2.0106	2.014	13.264	.722	2.874	2.021	9.572	3.331
7	29	1.5786	1.5706	1.592	12.842	.512	2.015	1.596	6.582	3.266
8	30	1.2296	1.2236	1.226	12.477	.345	1.361	1.228	4.308	3.166
9	27	1.0096	1.0046	1.007	12.258	.248	1.012	1.008	3.046	3.008
10	34	.7786	.7756	.777	12.027	.163	.685	.777	1.965	2.869
11	27	.6296	.6276	.629	11.879	.117	.499	.629	1.389	2.786
12	30	.4616	.4606	.461	11.712	.073	.313	.461	.859	2.744

United States Geological Survey experiments at Cornell hydraulic laboratory on model of Merrimac River dam, at Lawrence, Mass.

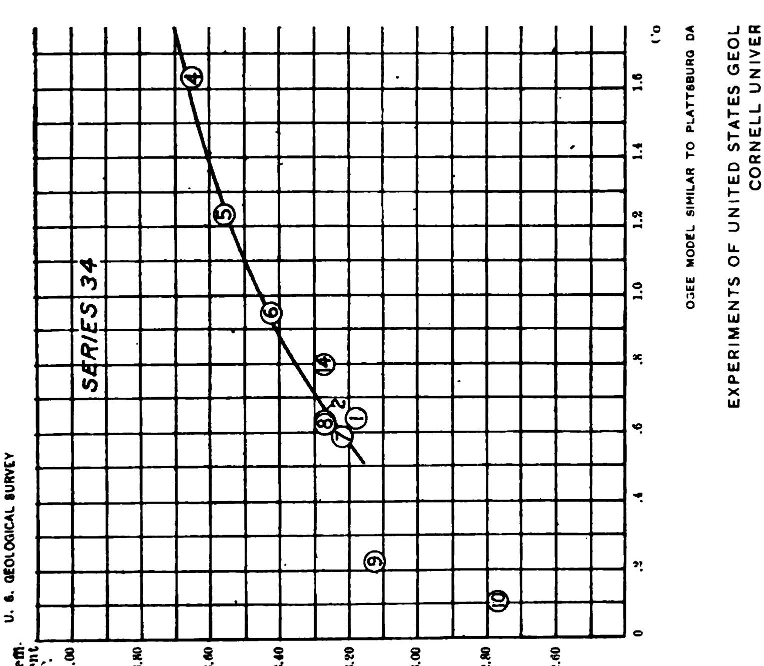
[Height of weir=P, 6.65 feet; length of weir crest=L, 9.999 feet; width of channel=b, 15.97 feet.]

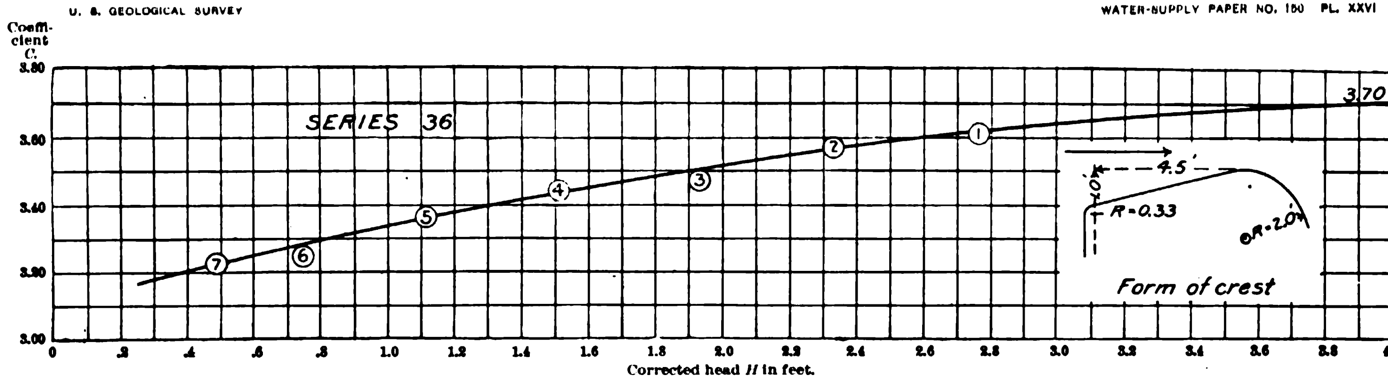
No.	Measured head on experimental weir, in feet.				A= area of section per foot of crest.	v= mean velocity of approach, in feet per second.	Head corrected for velocity of approach, in feet.		Q=discharge per foot of crest, in cubic feet per second.	Discharge coefficient C_1.
	Number of observations.	Maximum.	Minimum.	Mean= D.			$H^{\frac{3}{2}}$	H		
1	2	3	4	5	6	7	8	9	10	11
1a				4.001	10.651	2.618	8.288	4.094	27.893	3.365
2a				3.930	10.580	2.563	8.054	4.018	27.120	3.367
3a				3.630	10.280	2.309	7.029	3.670	23.740	3.377
21				3.630	10.280	2.319	7.029	3.670	23.738	3.377
10				3.166	9.816	1.939	5.769	3.216	19.039	3.300
11				2.815	9.465	1.654	4.837	2.860	15.660	3.237
22				2.510	9.160	1.424	4.067	2.548	13.049	3.208
23				2.223	8.873	1.200	3.361	2.244	10.652	3.169
12				2.130	8.780	1.127	3.150	2.149	9.898	3.142
1				2.041	8.691	1.066	2.933	2.049	9.265	3.158
24				1.850	8.500	.932	2.542	1.868	7.929	3.113
2				1.746	8.396	.860	2.327	1.756	7.227	3.105
3				1.645	8.295	.802	2.302	1.743	6.651	2.889
13				1.496	8.146	.691	1.832	1.497	5.631	3.074
4				1.322	7.972	.600	1.528	1.327	4.791	3.135
25				1.268	7.918	.556	1.434	1.272	4.410	3.075
5				1.089	7.739	.462	1.141	1.092	3.581	3.138
6				.764	7.414	.284	.669	.765	2.108	3.151
7				.584	7.234	.195	.447	.585	1.412	3.158
9				.583	7.233	.192	.446	.584	1.389	3.114
19				.198	6.848	.039	.088	.198	0.270	3.067

In the accompanying tables (pp. 98–106), columns 2, 3, and 4 show, respectively, the number of observations of head and the maximum and minimum readings in each experimental period. In column 5 is given the mean head on the experimental weir deduced from the tape observations above described. Column 6 shows the area of cross section of the channel of approach per foot of crest. For suppressed weirs this quantity equals the sum of the height of weir plus the measured depth on crest. For weirs with one end contraction the quantity A is obtained by dividing the total area of the water section, where D is measured, by the net length of the weir crest corrected for the end contraction. For those series where the depth on the experimental weir was increased by contracting the weir to about one-half of the channel width and introducing one end contraction, the net length of crest has been determined by the method of Francis, by deducting one tenth the head from the measured length of crest. The discharge per foot of crest of the experimental weir given in column 10 has been deduced from the discharge over the standard weir,

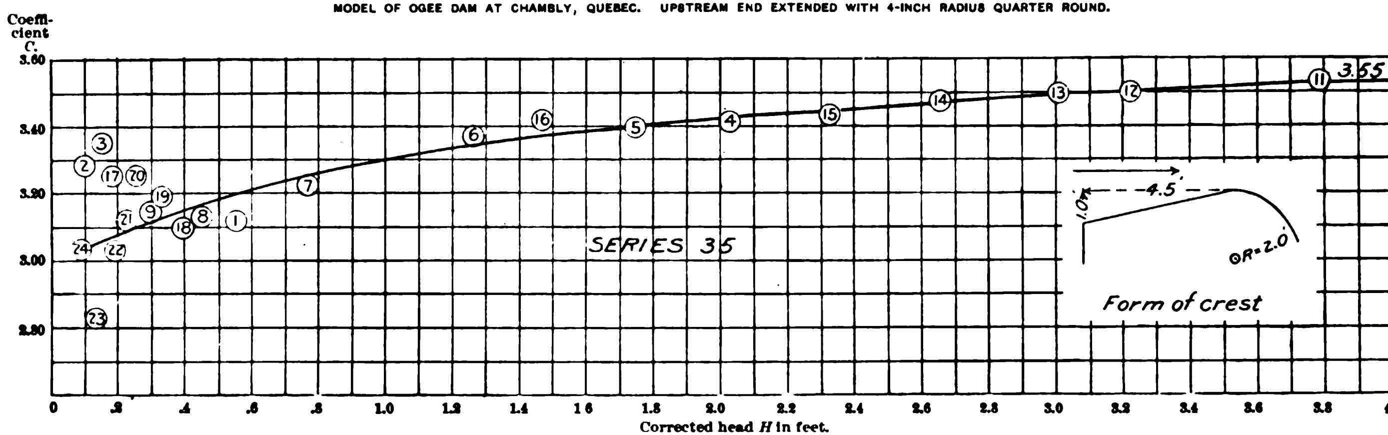


Plates XXIII and XXIV will be found immediately preceding page 95.



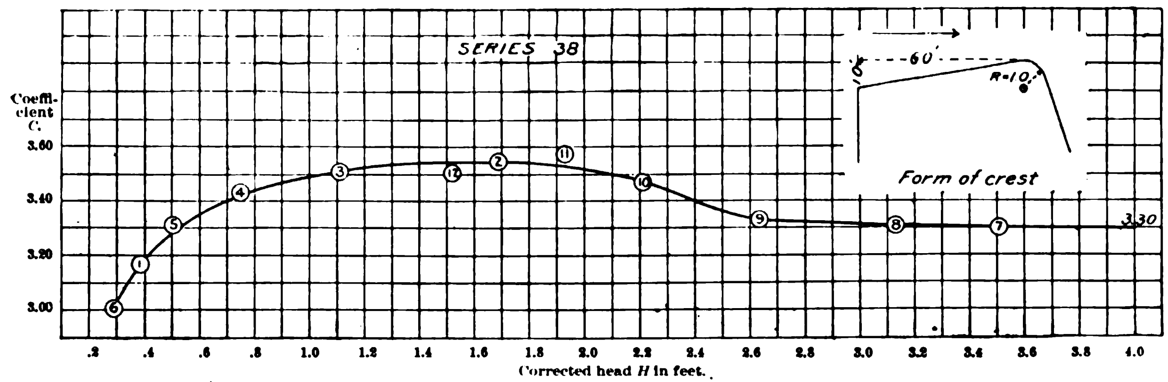


MODEL OF OGEE DAM AT CHAMBLY, QUEBEC. UPSTREAM END EXTENDED WITH 4-INCH RADIUS QUARTER ROUND.

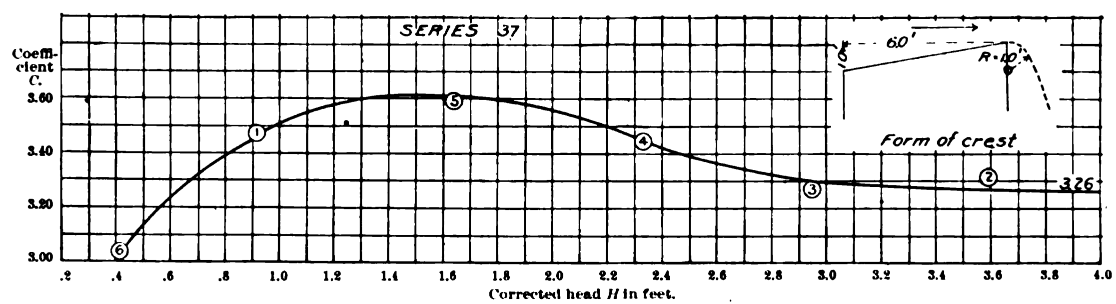


MODEL OF OGEE DAM AT CHAMBLY, QUEBEC. LENGTH OF CREST, 15.969 FEET.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

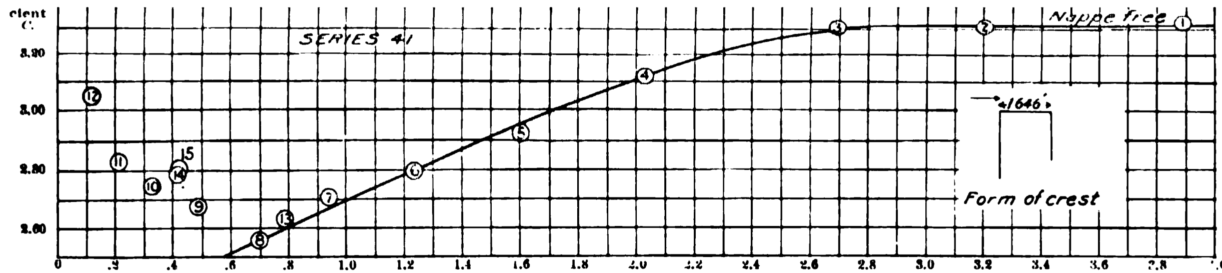


MODEL OF OGEE DAM AT DOLGEVILLE, N. Y. LENGTH OF CREST, 15.969 FEET.

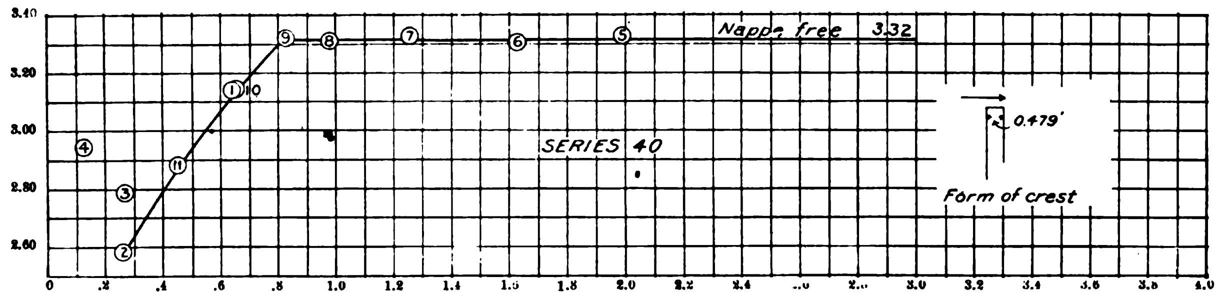


MODEL OF OGEE DAM AT DOLGEVILLE, N. Y. PART OF APRON CARRIED AWAY IN SECOND EXPERIMENT. LENGTH OF CREST, 15.969 FEET.

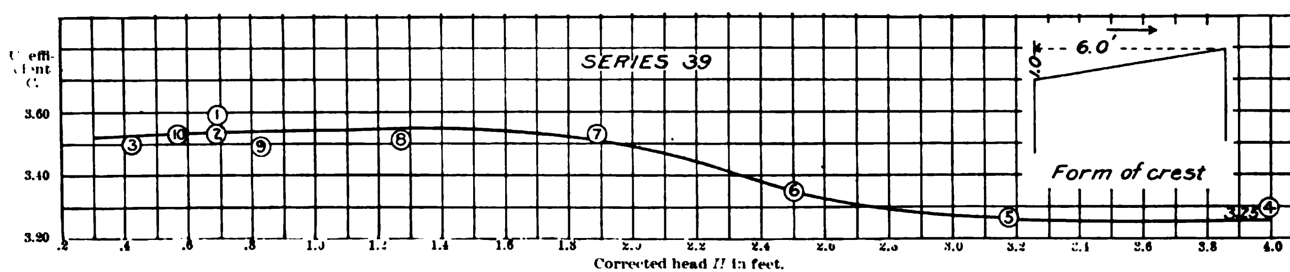
EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.



BROAD-CREST WEIR. WIDTH, 1.640 FEET; LENGTH, 15.009 FEET. NAPPE PARTLY AERATED.

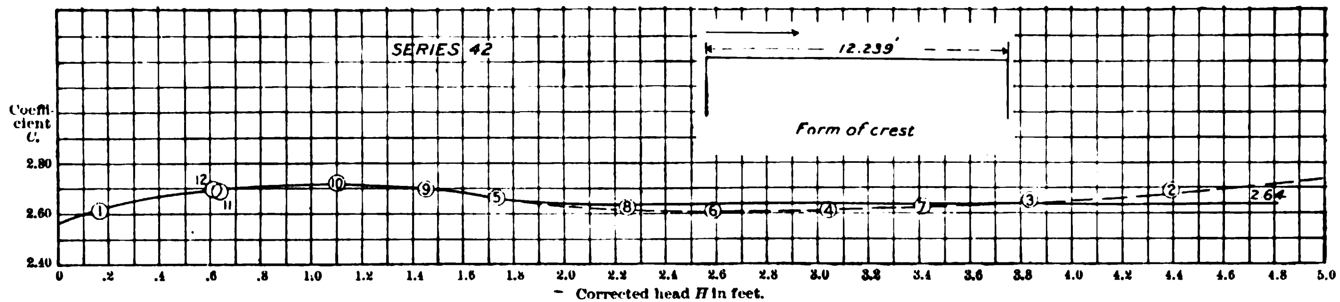


BROAD-CREST WEIR. WIDTH, 0.479 FOOT; LENGTH, 15.960 FEET. NAPPE AERATED.

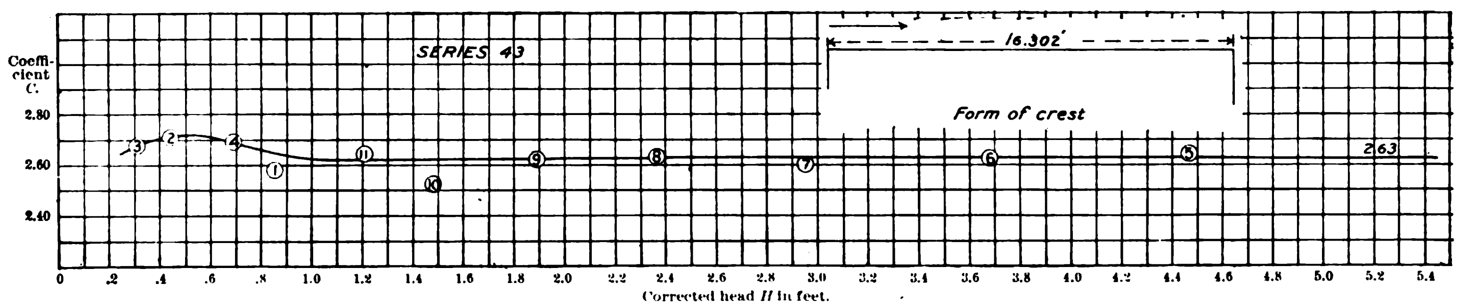


MODEL OF DOLGEVILLE DAM WITH APRON REMOVED, LEAVING TRAPEZOID. LENGTH OF CREST, 15.969 FEET. NAPPE AERATED.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

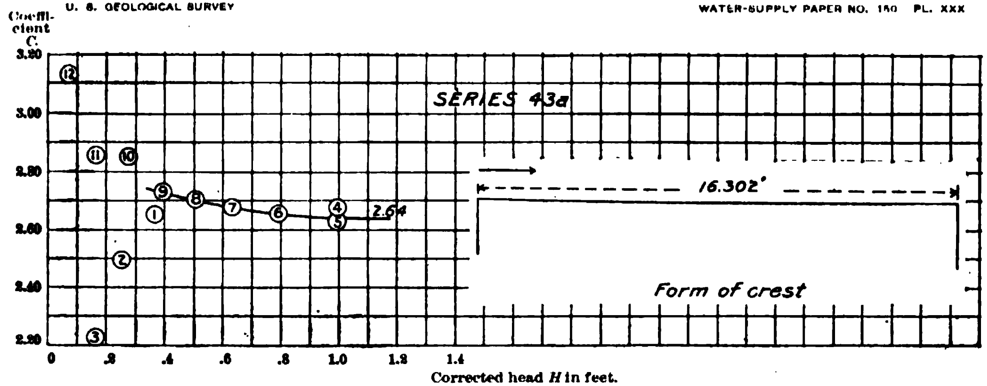


BROAD-CREST WEIR. WIDTH, 12.239 FEET; LENGTH, 15.989 FEET. NAPPE PARTLY AERATED.

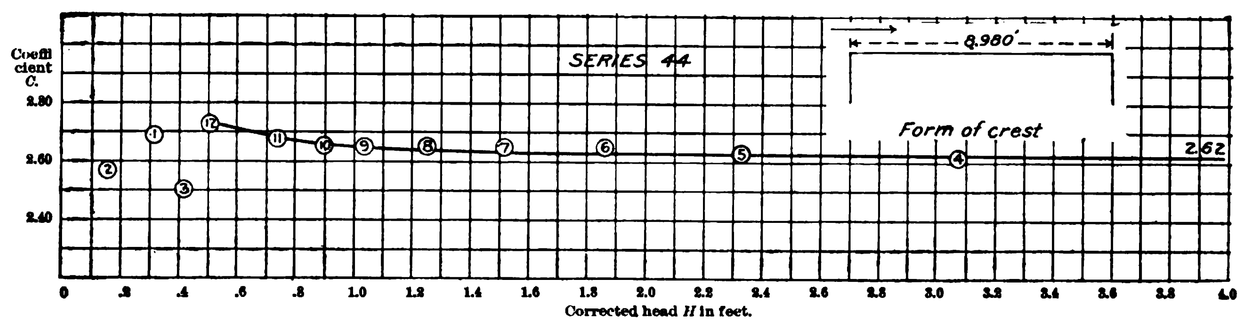


BROAD-CREST WEIR. WIDTH, 16.302 FEET; LENGTH, 15.969 FEET. EDGES OF TRANSVERSE BOARDS NOT FLUSH; NAPPE PARTLY AERATED.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

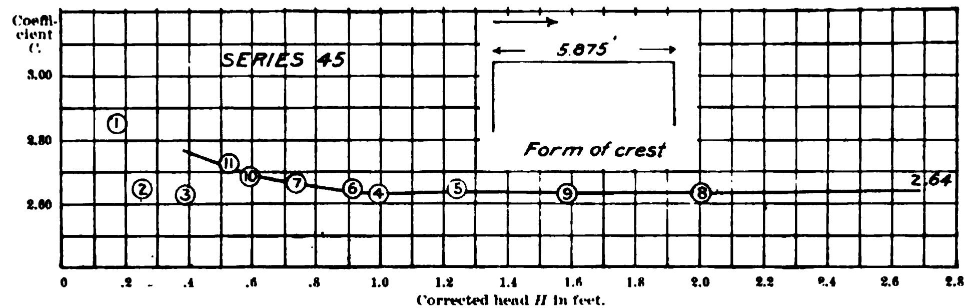


BROAD-CREST WEIR. WIDTH, 16.302 FEET; LENGTH, 15.969 FEET. SMOOTH CREST SURFACE; NAPPE PARTLY AERATED.

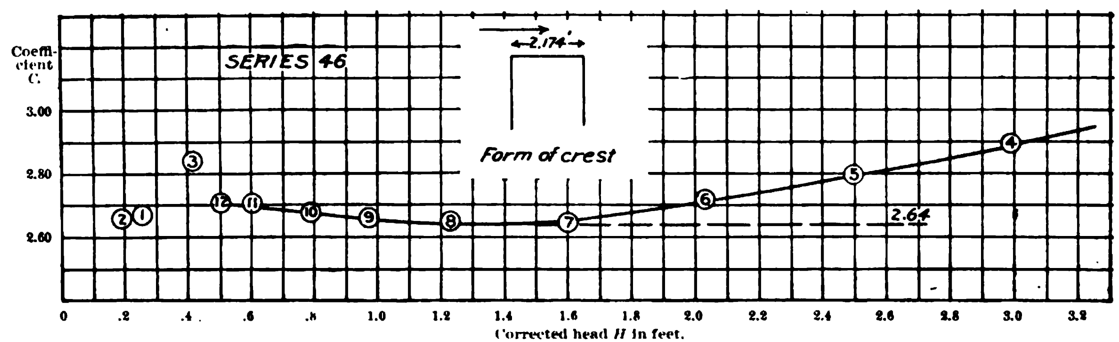


BROAD-CREST WEIR. WIDTH, 8.980 FEET; LENGTH, 15.969 FEET. NAPPE PARTLY AERATED.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

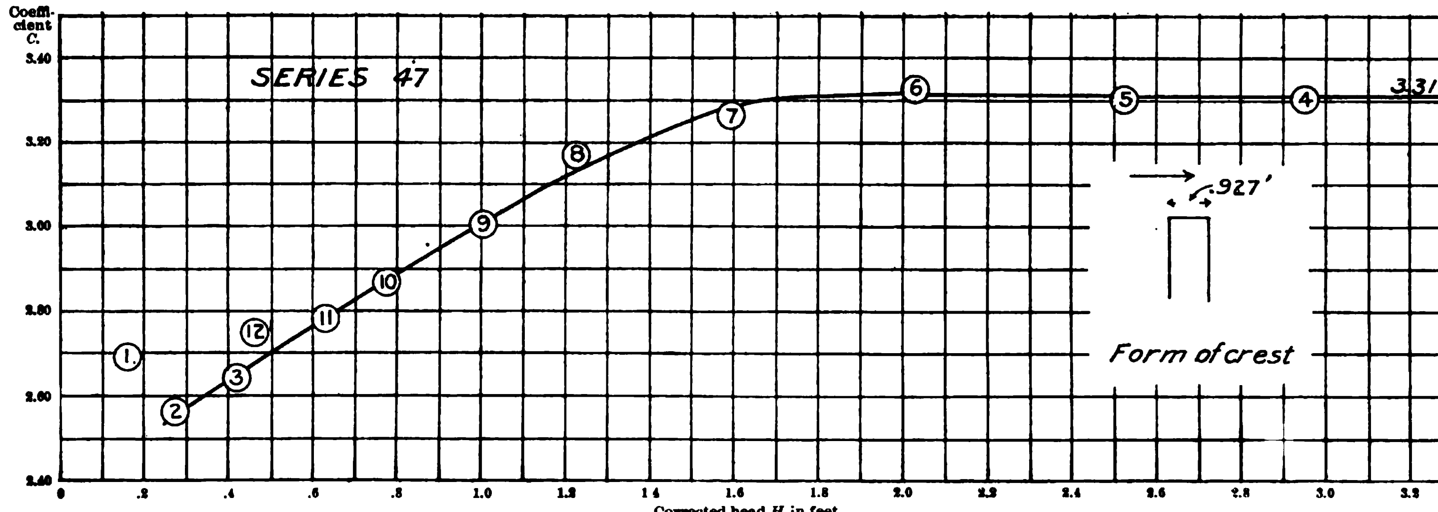


BROAD-CREST WEIR. WIDTH, 5.875 FEET; LENGTH, 15.969 FEET. NAPPE PARTLY AERATED.



BROAD-CREST WEIR. WIDTH, 2.174 FEET; LENGTH, 15.969 FEET. NAPPE PARTLY AERATED.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.



BROAD-CREST WEIR. WIDTH, 0.927 FOOT; LENGTH, 15.960 FEET. NAPPE PARTLY AERATED.

EXPERIMENTS OF UNITED STATES GEOLOGICAL SURVEY ON VOLUME OF FLOW OVER MODEL DAMS, CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

obtained, as described above, by dividing the total discharge by the net length of the experimental weir. The mean velocity of approach v, given in column 7, has been obtained by the formula

$$v=\frac{Q}{A}.$$

The correction for velocity of approach has been carefully computed by the Francis formula

$$H^{\frac{3}{2}}=(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}},$$

where

$$h=\frac{v^2}{2g}.$$

The resulting values of $H^{\frac{3}{2}}$ are given in column 8. The corresponding values of H, given in column 9, have been obtained by interpolation from a table of three-halves powers. The discharge coefficient C_1 given in column 11 has been obtained by the formula

$$C_1=\frac{Q}{H^{\frac{3}{2}}}.$$

This coefficient represents the discharge per linear foot of crest, if the head is 1 foot, with no velocity of approach, it being the coefficient in a weir formula of the same form as that used by J. B. Francis for a thin-edged weir.

Pls. XXIII to XXXII show the coefficient diagrams deduced from these experiments.

EXPERIMENTS ON MODEL OF DAM OF THE ESSEX COMPANY, MERRIMAC RIVER, AT LAWRENCE, MASS.[a]

A series of experiments covering five different depths on crest was made by James B. Francis at lower locks, Lowell, Mass., November, 1852. The model had a crest length of 9.999 feet, with end contractions suppressed. Height of water was measured by hook gage in a chamber at one side of the channel, 6 feet upstream from crest, so arranged as to give substantially the height of the still-water surface above the crest without correction for velocity of approach. The discharge was volumetrically determined as in Francis's thin-edged weir experiments.

The experiments of Francis covered depths on crest ranging from 0.5872 foot to 1.6338 feet. From these experiments he deduced tne formula for discharge,

$$Q=3.01208LH^{1.53}.$$

[a] Francis, J. B., Lowell Hydraulic Experiments, pp. 136–137.

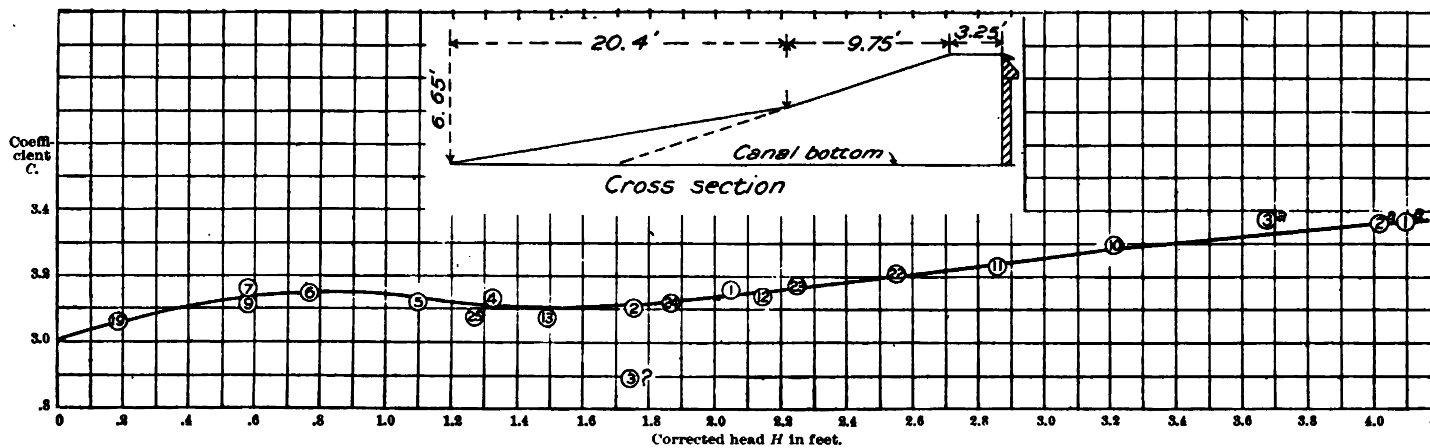
If the discharge were expressed in terms of the usual formula, $Q=C_1LH^{\frac{3}{2}}$, with a varying coefficient C_1, we should have a continuously increasing coefficient.

A series of experiments on a similar model dam, 6.65 feet high, with crest length of 15.932 feet, was made at Cornell University hydraulic laboratory in 1903. The model there used differed from that shown by Francis only in the substitution of a flatter upstream slope near the bottom of the canal, as shown in Pl. XXXIII. The end contractions were suppressed and the depth on crest was measured with steel tape and plumb bob suspended over center of channel at points 14.67 feet and 29.82 feet, respectively, upstream from crest of experimental weir. Discharge was previously measured over the standard weir, calibrated by Bazin's and Fteley and Stearns's formulas, located at head of experimental canal.

The experiments covered a range of heads varying from 0.198 foot to 4.94 feet. In the majority of the experiments the head was observed at both points. The upper point of measuring depth was at the upstream end of the inclined approach. The lower point was over the incline, where the area of the section of approach was smaller and the velocity larger than in the deeper channel above. The experiments have been reduced with reference to the heads measured 29.82 feet upstream from crest. By comparison of the depths simultaneously observed at the two points correction factors have been deduced for the reduction of the remaining experiments, in which the head was observed at the downstream point of observation only.

The observed head has been corrected for velocity of approach by the formula of Francis. The resulting mean coefficient curve, based on 19 valid observations, shows a larger coefficient of discharge in the formula $Q=C_1LH^{\frac{3}{2}}$ than does that of Francis.

For a head of 1 foot the formula of Francis for the Merrimac dam gives a discharge of 90.3 per cent of that for a thin-edged weir. The Cornell experiments show 94.5 per cent of the discharge over a thin-edged weir under the same head.



EXPERIMENTS ON MODEL SIMILAR TO MERRIMAC RIVER DAM AT LAWRENCE, MASS., MADE FOR UNITED STATES GEOLOGICAL SURVEY AT CORNELL UNIVERSITY HYDRAULIC LABORATORY, 1903.

Discharge per foot of crest, Francis formula for Merrimac dam, compared with Cornell experiments on similar cross section.

Depth on crest, H.	Q per foot of crest, in cubic feet per second, Francis.	Coefficient C_1 in formula $Q=C_1LH^{\frac{3}{2}}$. Francis's formula.	Coefficient C_1 in formula $Q=C_1LH^{\frac{3}{2}}$. Cornell experiments.	Depth on crest, H.	Q per foot of crest, in cubic feet per second, Francis.	Coefficient C_1 in formula $Q=C_1LH^{\frac{3}{2}}$. Francis's formula.	Coefficient C_1 in formula $Q=C_1LH^{\frac{3}{2}}$. Cornell experiments.
0.15	0.1653	2.845	3.05	0.85	2.3490	2.997	....
.20	.2567	2.871	3.06	.90	2.5636	3.002	3.15
.25	.3611	2.889	3.07	.95	2.7846	3.007	3.15
.30	.4774	2.905	3.08	1.00	3.0121	3.012	3.15
.35	.6043	2.913	3.09	1.15	3.7500	3.041	3.13
.40	.7431	2.937	3.11	1.25	4.2378	3.033	3.12
.45	.8877	2.940	3.12	1.50	5.6012	3.048	3.10
.50	1.0430	2.940	3.13	1.75	....	....	3.12
.55	1.206	2.956	3.135	2.00	8.6975	3.075	3.14
.60	1.379	2.966	3.14	2.50	....	....	3.20
.65	1.5581	2.973	3.14	3.00	16.1750	3.113	3.26
.70	1.7452	2.980	3.14	3.50	....	....	3.31
.75	1.9395	2.986	3.14	4.00	25.1200	3.140	3.36
.80	2.1408	2.992	3.15				

Aside from Blackwell's experiments the Francis formula for the Merrimac dam was until recently the only one available for a large dam of irregular section, and for want of more appropriate data it has been used for the calculation of discharge over many forms of weirs of irregular section, and in spite of Francis's explicit caution, it has been applied where the heads differed widely from those used in the original experiments.

Considering the limited experiments on which it is based, Francis's Merrimac dam formula gives good agreement with the much more extended experiments on a similar section made at Cornell hydraulic laboratory.

FLOW OVER WEIRS WITH BROAD CRESTS.

THEORETICAL FORMULA OF UNWIN AND FRIZELL.[a]

Consider a weir of such breadth that the nappe becomes of sensibly uniform depth in the portion BC, fig. 8, the upstream corner of the weir being rounded to prevent vertical contraction and the surface slightly inclined downstream so that it becomes parallel with the surface of the nappe BC.

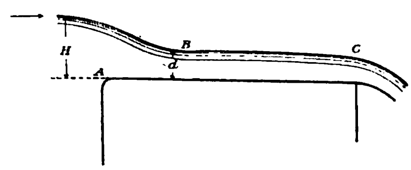


FIG. 8.—Broad-crested weir.

The fall causing the velocity V in the section BC is $H-d$. It follows that if v is the mean velocity in BC

$$v=\sqrt{2g(H-d)} \qquad Q=Ldv=Ld\sqrt{2g(H-d)}$$

In this equation Q is 0 when $d=0$ or $d=H$. There must, therefore, be an intermediate value of d for which Q will be a maximum. Differentiating we find for the condition of a maximum,

$$\frac{dQ}{dd}=0=L\sqrt{2g}\left[\sqrt{H-d}-\frac{1}{2}\frac{d}{\sqrt{H-d}}\right]$$

Giving $H-d=\frac{d}{2}$ and $d=\frac{2}{3}H$, or, for maximum discharge, one-third the head would be expended in producing the velocity of flow. With this value of d the expression for discharge becomes

$$\left.\begin{array}{ll} & Q=\frac{2}{3}\frac{LH}{\sqrt{3}}\sqrt{2gH}=0.38490LH\sqrt{2gH} \\ \text{or if} & \sqrt{2g}=8.02, \\ & Q=3.087LH^{\frac{3}{2}} \end{array}\right\} \quad . \quad . \quad . \quad . \quad (58)$$

In this formula frictional resistance has been neglected. The discharge given is the maximum for the conditions, and would result only if the stream discharges itself in accordance with the "principle of least energy."

Blackwell's experiments, given elsewhere, show a considerably larger coefficient for weirs 3 feet broad, slightly inclined downward, than for those with horizontal crests.

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[a] Given by W. C. Unwin, in article 1 on Hydrodynamics in Ency. Brit. Independently derived by J. P. Frizell. See his Water Power, pp. 198–200.

Let $d=KH$, then from the formula first given

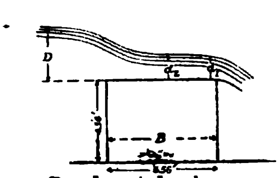
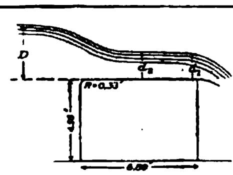
$$Q=LKH\sqrt{2gH(1-K)}$$
$$=K\sqrt{2g(1-K)}LH^{\frac{3}{2}}$$

$$C_1=8.02K\sqrt{1-K} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (59)$$

The theoretical coefficient C_1 can be computed from this equation if K has been determined experimentally.

From profiles taken in connection with United States Deep Waterways experiments at Cornell University hydraulic laboratory in 1899 the following values of D and d for broad-crested weirs have been scaled and the ratio $d : D$ computed. D was taken 4 feet upstream from the upper face of the weir, and does not include velocity of approach correction; values of d_1 and d_2 were taken at the lower-crest lip and center of crest, respectively. The value of d_2 at center of crest has been used in the computations.

Values of D and d for broad-crested weirs.

		D	d_1	d_2	$K=\frac{d_2}{D}$
	1	0.90	0.35	0.52	0.58
	2	1.15	.45	.68	.59
	3	1.80	.75	1.14	.63
	4	2.60	1.20	1.75	.67
	5	3.55	1.72	2.52	.71
Broad-crested weir.	6	5.15	2.20	3.15	.61
	1	1.00	.35	.50	.50
	2	1.32	.53	.70	.50
	3	1.98	.75	.98	.50
	4	2.85	1.08	1.70	.60
	5	3.90	1.50	2.50	.64
Broad-crested weir.	6	4.65	2.10	3.10	.61

For low heads a sudden drop begins near the upstream crest corner and terminates at a distance 1.5 to 2 D below the upstream corner. From this point to within a distance about equal to D from the downstream crest corner the surface is nearly parallel with the crest. If the width of crest is not greater than 2.5 to 3 D the nappe passes over the broad crest in a continuous surface curve, becoming more nearly convex as the ratio $D : B$ increases.

For low heads Cornell experiment 13, crest 6.56 feet wide, with rounded upstream corner, complies very well with the theory of discharge in accordance of the principle of least energy. The coefficient computed as above is

$$\begin{aligned} C_1 &= 8.02 \times 0.585 \sqrt{1-0.585} \\ &= 8.02 \times 0.585 \times 0.6442 \\ &= 3.02 \end{aligned}$$

The experimental coefficient with head corrected for velocity of approach is 2.82.

The following additional data may be cited:

Trautwine[a] quotes data of Elwood Morris, C. E., for Clegg's dam, Cape Fear River, North Carolina. Horizontal crest 8.42 feet wide, vertical faces. $H=1.25$ feet. d (throughout central portion of crest) $=0.50$ foot. $d/H=0.40$.

Thos. T. Johnston[b] gives data of elaborate profiles of the nappe for Desplaines River dam, Illinois. Horizontal planed stone coping, vertical downstream face; upstream face batter, 1/2:1. $H=0.587$ foot. $d=0.315$ to 0.307 foot in central, nearly level portion at distances 1.5 to 4 feet from upstream edge of crest. Johnston and Cooley deduce the coefficient $C=1.69$ for this case.

BLACKWELL'S EXPERIMENTS ON DISCHARGE OF WATER OVER BROAD-CRESTED WEIRS.

Experiments made by Thomas E. Blackwell,[c] M. Inst. C. E., are of interest as being probably the first recorded for weirs with broad crests. The discharge was volumetrically measured, and the conditions were generally favorable to accuracy. The experiments were made on a side pond of the Kennet and Avon Canal, 106,200 square feet surface area, closed by a lock at each end, the water being admitted from time to time as required, the relation between area of reservoir and volume of discharge being such that there was no sensible variation in water level during an experiment.

The weir was constructed in a dock to which the water had access through an irregularly shaped channel 40 feet in width, cut off from the main pond by a submerged masonry wall 9 feet wide, situated 25 feet upstream from the weir, having its top 18 inches to 20 inches below water surface.

The water level in the pond being constant when outflow took place, the weir, which had a crest adjustable in a vertical plane, was set with its crest level at the depth below water surface desired for an experiment, by means of adjusting screws at the ends of the weir; the water

[a] Engineers' Pocket Book.

[b] Johnston, T. T., and Cooley, E. L., New experimental data for flow over a broad-crest dam: Jour. Western Soc. Engrs., vol. 1, Jan., 1896, pp. 30–51.

[c] Original paper before Institution of Civil Engineers of London, reprinted in the Journal of the Franklin Institute, Philadelphia, March and April, 1852.

was then allowed to waste through the weir until a uniform regimen of flow was established.

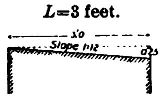
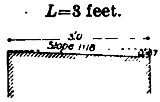
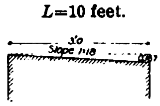
A gaging tank having a floor of brick laid in cement, with plank sides, and 449.39 cubic feet capacity, was erected at the foot of the weir. At a given signal the lid of this tank was raised, the time noted, and the rate of filling of the tank recorded by several observers. Such leakage from the tank as occurred was separately measured and allowed for. There was no correction for velocity of approach or for end contractions.

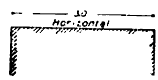
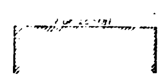
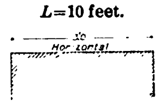
The wind was so slight as to be negligible, except during one series when there was a brisk wind blowing downstream. The experimenter states that parallel experiments on a quiet day indicated an increase of about 5 per cent in discharge due to this wind.

The crest of the thin-edged weir consisted of an iron plate barely one-sixteenth inch thick. A square-top plank 2 inches thick was attached to the weir, and an apron of deal boards, roughly planed so as to form an uninterrupted continuation downstream, constituted the wide-crested weir used in the experiments.

The coefficient C_1 from Blackwell's experiments has been worked out and is given in the following table. The measured depths taken in inches have also been reduced to feet.

Blackwell's experiments on broad-crested weirs, Kennet and Avon Canal, England, 1850.

Weir.	Measured head, in inches.	Head, in feet.	Q per foot, in cub. ft. per sec.	M	$C_1=M\sqrt{2g}$
II. Thin plate, 10 feet long.	1	0.083	0.104	0.539	4.32
	2	.167	.292	.535	4.29
	3	.25	.429	.428	3.43
	4	.333	.675	.437	3.50
	5	.417	.935	.433	3.47
	8	.667	1.691	.387	3.10
	9	.750	1.842	.353	2.83
VII. L=3 feet. 	1	.083	.060	.311	2.49
	2	.167	.194	.355	2.85
	3	.250	.360	.359	2.88
	4	.333	.468	.303	2.43
	6	.500	1.005	.354	2.84
	7	.583	1.254	.351	2.82
	9	.750	1.729	.332	2.66
Series VII, average.	.....	.....	.....	.....	2.71
VIII. L=3 feet. 	1	.083	.070	.363	2.91
	2	.167	.199	.364	2.92
	3	.250	.359	.358	2.87
	4	.333	.443	.287	2.30
	5	.417	.743	.344	2.76
	7	.583	1.222	.342	2.74
	8	.667	1.426	.327	2.62
	9	.750	1.709	.328	2.63
IX. L=10 feet. 	1 to $\frac{7}{8}$	.073	.060	.311	2.49
	2	.167	.181	.330	2.65
	4	.333	.530	.343	2.75
	6	.500	1.028	.362	2.90
	8	.667	1.476	.338	2.71
Series VIII-IX, average	.....	.....	.....	.....	2.71

Weir.	Measured head, in inches.	Head, in feet.	Q per foot, in cub. ft. per sec.	M	$C_1=M\sqrt{2g}$
X. L=3 feet. 	1	0.083	0.058	0.301	2.41
	2	.167	.175	.321	2.57
	3	.250	.295	.294	2.36
	4	.333	.431	.279	2.24
	5	.417	.689	.319	2.56
	6	.500	.947	.334	2.68
	7	.583	1.162	.325	2.61
	8	.667	1.369	.313	2.51
	9	.750	1.642	.317	2.54
XI. L=6 feet. 	1to1¼	.093	.071	.....	.....
	3	.250	.329	.328	2.63
	4	.333	.511	.331	2.65
	6	.500	.963	.....	.....
	7	.583	1.191	.331	2.65
	9	.750	1.670	.....	.....
	10	.883	1.895	.310	2.49
	12	1.0	2.495	.311	2.49
XII. L=10 feet. 	1	.083	.049	.254	2.04
	2	.167	.174	.319	2.56
	5	.417	.745	.345	2.77
	6	.500	.972	.342	2.74
	8	.667	1.362	.312	2.50
	9	.750	1.688	.324	2.60
	10	.833	1.847	.303	2.43
					52.08
Series X, XI, XII, average	.....	.....	.....	.....	2.48

EAST INDIAN ENGINEERS' FORMULA FOR BROAD-CRESTED WEIRS.[a]

This formula is

$$Q=\frac{2}{3}M'LH\sqrt{2gH}\text{, or if }\sqrt{2g}=8.02,\ Q=5.35M'LH^{\frac{3}{2}}=C_1LH^{\frac{3}{2}} \quad (60)$$

Where M'=coefficient for thin-edged weir[b]$=0.654-0.01H$,

$$M'=M-\frac{0.025M(B+1)}{H+1} \quad . \quad . \quad . \quad . \quad . \quad (61)$$

[a] Mullins, Gen. Joseph, Irrigation Manual, Madras, 1890.
[b] See table giving values of M and equivalent values of C, p. 22.

Experimental data not given. This formula gives values of M' or C_1 decreasing as breadth of crest B increases, and for low heads increasing to a maximum for a head of about 1 foot, then slowly decreasing.

The formula reduces to

$$C_1=C\left(\frac{H+1-0.025\,(B+1)}{H+1}\right)=RC \quad . \quad . \quad . \quad (62)$$

For $B=0$

$$C_1=C\frac{H+0.975}{H+1}=RC$$

which differs by the ratio R from the equivalent value of C' for a thin-edged weir.

Values of coefficient C_1 of discharge over a broad-crested weir and of R, the ratio of the former to C the coefficient of discharge over a thin-edged weir, by Mullins's formula.

H feet. \ B	1 foot.		2 feet.		3 feet.		4 feet.	
	R	C_1	R	C_1	R	C_1	R	C_1
1	0.975	3.359	0.962	3.316	0.95	3.319	0.938	3.230
2	.983	3.335	.975	3.307	.967	3.279	.958	3.251
3	.988	3.296	.981	3.275	.975	3.255	.969	3.234
4	.990	3.253	.985	3.237	.98	3.220	.975	3.204
5	.992	3.204	.988	3.191	.983	3.177	.979	3.164
6	.993	3.155	.989	3.144	.986	3.132	.982	3.121
7	.994	3.104	.991	3.095	.988	3.085	.984	3.075
8	.994	3.054	.992	3.045	.989	3.037	.986	3.028
9	.995	3.003	.992	2.995	.99	2.988	.988	3.042
10	.995	2.950	.993	2.944	.991	2.937	.989	2.930

H feet. \ B	5 feet.		6 feet.		7 feet.		8 feet.	
	R	C_1	R	C_1	R	C_1	R	C_1
1	0.925	3.182	0.912	3.144	0.9	3.100	0.888	3.057
2	.95	3.22	.942	3.194	.933	3.166	.925	3.138
3	.962	3.213	.956	3.192	.95	3.171	.944	3.150
4	.97	3.193	.965	3.171	.96	3.154	.955	3.138
5	.975	3.149	.971	3 137	.967	3.123	.962	3.110
6	.978	3.11	.975	3.098	.971	3.080	.968	3.076
7	.981	3.06	.978	3.057	.975	3.046	.972	3.036
8	.983	3.017	.980	3.012	.977	2.999	.975	2.994
9	.985	2.97	.982	2.966	.98	3.019	.978	2.950
10	.986	2.92	.984	2.916	.982	2.910	.979	2.093

The values of C_1 given in the above table have been deduced from the corresponding values of C for a thin-edged weir by Mullins's formula. The ratio R may, if desired, be applied approximately to correct values of C derived from other standard weir formulas.

FTELEY AND STEARNS EXPERIMENTS ON BROAD-CRESTED WEIRS.[a]

The formula of Fteley and Stearns is based on five series of experiments made in the Sudbury River conduit, Boston, 1877, on weirs 2, 3, 4, 6, and 10 inches wide, respectively. Suppressed weirs 5 feet long were used, the depths being as follows:

Fteley and Stearns experiments.

Width of crest, in inches.	Number of experiments.	Range of depth observed on broad crests, in feet. From—	To—
2	7	0.1158	0.2926
3	21	.1307	.4619
4	25	.1318	.6484
6	22	.1320	.8075
10	17	.1352	.8941

The results are given by the authors in the form of a table of corrections to be added algebraically to the measured head for the broad-crested weir to obtain the head on a thin-edged weir that would give the same discharge.

Fteley and Stearns's correction c may be found approximately from the formula

$$c=0.2016\sqrt{[(0.807\,B-H)^2+0.2146\,B^2]}-0.1876\,B \quad . \quad . \quad (63)$$

or if $k=0.2016$, $m=0.1876$, $n=0.2146$, $O=0.807$, then

$$Q=CL[H-mB+k\sqrt{(OB-H)^2+nB^2}]^{\frac{3}{2}} \quad . \quad . \quad . \quad (64)$$

If the head on a broad-crested weir is H, the discharge will be

$$Q=CL(H+c)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (65)$$

C being the coefficient of discharge for thin-edged weirs.

If C_1 is the coefficient for the broad weir, then we may also write

$$Q=C_1LH^{\frac{3}{2}}.$$

Hence

$$\frac{C_1}{C}=\left(\frac{H+c}{H}\right)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (66)$$

From formula (66) have been calculated Fteley and Stearns's coefficients for weirs with nappe adhering to crest for use in the formula

$$Q=CLH^{\frac{3}{2}},$$

[a] Fteley and Stearns, Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, pp. 86–96.

correction for velocity of approach being made by adding $1.5 \frac{v^2}{2g}$ to the measured head to obtain H.[a]

Values of the ratio $\frac{C_1}{C}$ of the coefficient of discharge for a broad-crested weir, by Fteley and Stearns's experiments, to that for a thin-edged weir.

H	Width of crest, in inches.					
	3	4	6	8	10	12
0.0	....	....	....	....	....	....
.1	0.7466	0.74798	0.7562	0.7589	0.7576	0.7576
.2	.8234	.7878	.7740	.7679	.7644	.7624
.3	.9172	.8524	.8003	.7809	.7727	.7685
.4	.9963	.9201	.8353	.8003	.7850	.7768
.5	....	.9806	.8781	.8255	.8003	.7865
.6	....	1.0080	.9230	.8567	.8199	.7989
.7	....	....	.9891	.8911	.8424	.8150
.8	....	....	.9997	.9245	.8695	.8339
.9	....	....	1.0317	.9558	.8983	.8552
1.0	....	....	....	.9835	.9406	.8824
1.1	....	....	....	1.0090	.9508	.9027
1.2	....	....	....	1.0317	.9732	.9252
1.3	....	....	....	1.0499	.9948	.9465
1.4	....	....	....	....	1.0148	.9657
1.5	....	....	....	....	1.0317	.9850

BAZIN'S FORMULA AND EXPERIMENTS ON BROAD-CRESTED WEIRS.

These included series of about 20 periods each for depths not exceeding 1.4 feet on weirs of 0.164, 0.328, 0.656, 1.315, 2.62, and 6.56 feet breadth of crest. The coefficient C_1 in the formula $Q = C_1 L H^{\frac{3}{2}}$, deduced from a recomputation of the experiments on weirs 2.46 feet high, using the Francis velocity of approach correction, is given on Pl. IV.

Other experiments were made for the four narrower weirs with heights 1.148 and 1.64 feet, to determine the comparative velocity of approach effect.

Bazin shows that if the nappe is free from the downstream face of the weir it may assume two forms: (1) It may adhere to the horizontal crest surface; (2) it may become detached at the upstream edge in such a manner as to flow over the crest without touching the downstream edge. In the second case the influence of the flat crest evidently disappears and the discharge is like that over a thin-edged weir. The nappe usually assumes this form when the depth D exceeds twice the breadth of crest B, but it may occur whenever the depth exceeds $\frac{3}{2} B$. Between these limits the nappe is in a state of instability; it tends to detach itself from the crest, and may do so under the

[a] Fteley and Stearns's formula for a thin-edged weir has been used to calculate Q in deriving these coefficients, the experiments having been made under conditions similar to those under which their formula was derived.

influence of any external disturbance, as, for example, the entrance of air or the passage of a floating object over the weir.

When the nappe adheres to the crest, the coefficient C_1 depends chiefly on the ratio D/B and may be represented by the formula

$$C_1 = C\,(0.70 + 0.185\ D/B) \quad . \quad . \quad . \quad . \quad . \quad (67)$$

in which C is the coefficient for a thin-edged weir.

When $D/B = 1.50$ to 2, $C_1/C = 0.98$ to 1.07 if the nappe adheres to crest, or $C_1/C = 1.00$ if nappe is detached, and for $D/B > 2$, $C_1/C = 1.00$. Between the limits $D = 1.5B$ and $D = 2B$ the value which the coefficient C_1 will assume in a particular case is uncertain. Bazin considers that his formula gives accurate results for adhering nappes with breadth of crests up to 2 or 3 feet. For a crest 6.56 feet wide and $D = 1.476$ feet he finds the result by formula (67) 93.4 per cent of that given directly by the experiment.

Values of the ratio C_1/C, for a broad-crested weir, with adhering nappe, by Bazin's formula.[a]

D/B	$C_1/C = 0.700 + 0.185\ D/B$.									
	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.700	0.7018	0.7037	0.7056	0.7074	0.7092	0.7111	0.7130	0.7148	0.7166
.1	.7185	.7204	.7222	.7240	.7259	.7278	.7296	.7314	.7333	.7352
.2	.7370	.7388	.7407	.7426	.7444	.7462	.7481	.7500	.7518	.7536
.3	.7555	.7574	.7592	.7610	.7629	.7648	.7666	.7684	.7703	.7722
.4	.7740	.7758	.7777	.7796	.7814	.7832	.7851	.7870	.7888	.7906
.5	.7925	.7944	.7962	.7980	.7999	.8018	.8036	.8054	.8073	.8092
.6	.8110	.8128	.8147	.8166	.8184	.8202	.8221	.8240	.8258	.8276
.7	.8295	.8314	.8332	.8350	.8369	.8388	.8406	.8424	.8443	.8462
.8	.8480	.8498	.8517	.8536	.8554	.8572	.8591	.8610	.8628	.8646
.9	.8665	.8684	.8702	.8720	.8739	.8758	.8776	.8794	.8813	.8832
1.0	.8850	.8868	.8887	.8906	.8924	.8942	.8961	.8980	.8998	.9016
1.1	.9035	.9054	.9072	.9090	.9109	.9128	.9146	.9164	.9183	.9202
1.2	.9220	.9238	.9257	.9276	.9294	.9312	.9331	.9350	.9368	.9386
1.3	.9405	.9424	.9442	.9460	.9479	.9498	.9516	.9534	.9553	.9572
1.4	.9590	.9608	.9627	.9646	.9664	.9682	.9701	.9719	.9738	.9756
1.5	.9775	.9794	.9812	.9830	.9849	.9868	.9886	.9904	.9923	.9942

[a] If there is velocity of approach, the value of D/B, not H/B, should be used as an argument. The ratio C_1/C may be applied in a formula which includes the velocity of approach correction, either in the head H or in the coefficient.

Bazin's formula gives ratios which continually increase as H increases, B remaining constant, and which continually decrease as B increases, H remaining constant. It gives, however, a constant ratio for all widths or heads where the ratio H/B is unchanged.

Compared with their respective standard weir formulas, Mullins's formula gives for a broad-crested weir a continuously decreasing ratio of discharge as B increases from zero, H remaining constant, and a continuously increasing discharge as H increases from zero, B remaining constant; Fteley and Stearns's experiments give a discharge ratio which is less than unity, but which varies in an irregular manner, depending on the head and breadth of weir.

On referring to Pl. IV, in which the Bazin coefficients are given in a form comparable with the experiments of the United States Geological Survey, it will be noticed that, except for the lowest heads, the coefficient curves are simple linear functions of the head. The rate of increase of the coefficients as the head increases grows rapidly less as the breadth of the weir increases, indicating that for a very broad weir the coefficient would be sensibly constant throughout the range of stability of the nappe.

For the narrower weirs the coefficients tend to increase rapidly almost from the start toward the value for a thin-edged weir or detached nappe. For the weirs 2.62 and 6.56 feet breadth of crest the total variation in the coefficient for the range of heads covered by the experiments is comparatively small. The average coefficients are as follows:

Average Bazin coefficients, broad-crested weirs.

Bazin series No.	Crest width, in feet.	Range of head, in feet. From—	To—	Average constant coefficient, C_1.
113	1.312	Lowest.	0.60	2.64
114	2.624	0.35	.85	2.59
			1.32	2.62
115	6.56	.55	Highest.	[a] 2.58

[a] Coefficient increases slowly throughout.

The average coefficients show a fair agreement with the constant coefficient for broad-crested weirs with stable nappe deduced from the experiments of the United States Geological Survey (page 120).

EXPERIMENTS OF THE UNITED STATES GEOLOGICAL SURVEY ON BROAD-CRESTED WEIRS.

The method of conducting these experiments and the detailed results are given on pages 95–107. The coefficient curves are presented on Pls. XXVIII to XXXII. It may be remarked here that the models were larger and the range of breadth of crest and depth of flow experimented upon was greater than in the earlier experiments described. In general, the laws of behavior of the nappe pointed out by Fteley and Stearns and Bazin were confirmed.

The following table presents a résumé of the results:

Résumé of United States Geological Survey experiments on broad-crested weirs.

Series.	Breadth of crest, in feet.	Nappe unstable for heads less than values below, in feet.	Nappe detached from crest at head, in feet.	Coefficient C_1 varies between the limits— Head, in feet. From—	To—	Coefficient. From—	To—	Coefficient constant. Above head, in feet.	C_1
40	0.479	0.3	0.8	0.3	0.8	2.64	3.32	0.8	3.32
47	.927	.3	1.8	.3	1.8	2.57	3.31	1.8	3.31
41	1.646	.7	2.8	.7	2.8	2.56	3.32	2.8	3.32
46	3.174	.5	....	.5	1.3	2.70	2.64	1.3	Increases.
45	5.875	.5	....	.5	.9	2.72	2.64	.9	2.64
44	8.980	.5	....	.5	2.0	2.73	2.62	2.0	2.62
42	12.239	....	....	.2	2.0	2.62 / 2.73	2.64	2.0	[a]2.64
43	16.302	....	....	.3	1.1	2.68	2.72 / 2.63	1.1	[b]2.63
43a	16.302	.4	....	.4	1.0	2.72	2.64	1.0	2.64
Average.	....	....	....	....	....	....	....	....	2.634

[a] Coefficient shows tendency to increase slowly with head.
[b] Edges of planed and matched boards not flush. Crest smoothed in series 43a.

The deductions that follow have been based on a consideration of earlier experiments as well as those here given for the first time.

1. For depths below 0.3 to 0.5 foot the nappe is very unstable, owing probably to magnified effect of crest friction and to the varying aeration or adhesion of the nappe to the downstream weir face.

2. For heads from 0.5 foot to 1 or 2 feet for very broad weirs, or from 0.5 foot to the point of detachment for narrower weirs, the coefficient is somewhat variable and changes in an uncertain manner. For the broader weirs, the range of variation of C_1 between the depths indicated is narrow, from 2.73 to 2.62.

3. When the nappe becomes detached the coefficient remains nearly identical with that for a thin-edged weir. For the narrower weirs the coefficient increases rapidly within the range of tendency to detachment indicated by Bazin, i. e., for heads between D and $2D$.

4. On the broader weirs for depths exceeding 1 to 2 feet up to the limit of the experiments (about 5 feet), the experiments indicate a sensibly constant coefficient for all depths. Where there is any tendency to variation within the range indicated there is a gradual increase in C_1.

For weirs of 5 to 16 feet breadth the experiments show no conspicuous tendency for the coefficient C_1 to change with variation in either H or B, the range of value of C_1 being from 2.62 to 2.64.

The line of detachment of the nappe for a weir of 5 feet breadth would be 7.5 to 10 feet head or perhaps more, and a higher head for broader crests. If this depth were ever reached it may be surmised that the coefficient C_1 would increase to about 3.33 at the point of detachment. It would also appear, as is in fact indicated in Bazin's formula, that the coefficient should very slowly increase with H and decrease as B increases, independent of the tendency to detachment of the nappe, and owing to the decreased relative effect of crest friction and contraction.

The United States Geological Survey experiments indicate that this effect is of relatively little significance for large heads and broad weirs, and hence a constant coefficient covering a wide range may be safely adopted.

The average coefficient, 2.64, which we have tentatively chosen for weirs exceeding 3 feet in breadth under heads exceeding 2 feet, may apparently be applied for considerably lower heads for weirs of 5 feet or more crest breadth with but small error.

TABLE OF DISCHARGE OVER BROAD-CRESTED WEIRS WITH STABLE NAPPE.

A table has been calculated, using $C_1=2.64$ and covering heads varying by 0.1 foot increment from zero to 10 feet (p. 177). It is considered applicable for weirs of 3 feet or more crest breadth when H/B lies between the general limits 0.25 to 1.5. The coefficient 2.64 gives a discharge 79.2 per cent of that for a thin-edged weir by the Francis formula. The relative discharge obtained by other formulas and experimenters is shown in the following table:

Comparison of broad-crested weir formulas and experiments giving percentage of discharge over a thin-edged weir.[a]

Formula or experiments.	1 foot width.			2.62 feet width.			6.56 feet width.			
	H/B=0.5	1.0	1.5	0.5	1.0	1.5	0.25	0.5	1.0	1.5
	H=0.5	1.0	1.5	1.31	2.62	3.93	1.64	3.28	6.56	9.84
Mullins[b]	96.7	97.5	98.0	......	......	......	93.2	95.5	97.5	98.2
Fteley and Stearns	78.6	88.2	98.5	......	......	......	......	......	......	......
Bazin formula	79.2	88.5	97.8	79.2	88.5	97.8	74.6	79.2	88.5	97.8
U. S. Deep Waterways experiments	......	......	......	82.8	93.3	114.1	72.0	71.1	72.3	73.2
U. S. Geological Survey experiments	81.0	90.3	97.5	[c]79.5	[c]81.3	[c]86.7	79.2	[d]79.2	[d]79.2	[d]79.2

[a] No velocity of approach.
[b] East Indian engineers' formula, given in Mullins's Irrigation Manual, Madras Presidency.
[c] Weir 2.17 feet broad.
[d] Weir 5.88 feet broad.

Considering the low heads used, it may be noted that before Bazin's experiments only those of Blackwell included a weir breadth sufficient to eliminate the early tendency to detachment and permit the existence of the stable period for which a constant coefficient applies.

Blackwell's experiments on weirs 3 feet broad indicate a maximum coefficient C_1 of 2.65 to 2.77 for a head of about 0.5 foot, decreasing as the head increased.

The experiments of the United States Deep Waterways Board on models with 2.62 and 6.56 foot crest width are shown on Pl. XV. For the narrower weir the coefficient increased uniformly with the head. The nappe did not leave the crest, although the experiments were continued to the limit $H/B=2$, at which stage the coefficient exceeded that for a thin-edged weir. For the broader weir the coefficients are much less variable and the curves indicate that the coefficients approach a constant as the breadth of crest is increased.

It will be noted that considerable care must be exercised in determining the condition of the nappe for broad-crested weirs of inconsiderable width, while for those of greater breadth the wind may exert considerable influence on the nappe on the broad crest under lower heads. The constant coefficient 2.64 has been deduced from experiments on weirs with smooth, planed crests and sharp upstream crest angles. The effect of crest roughness on weir discharge is discussed on page 133.

EFFECT OF ROUNDING UPSTREAM CREST EDGE.

Experiments by Fteley and Stearns[a] indicate that the effect of rounding the upstream crest corner is to virtually lower the weir, by allowing the water to pass over with less vertical contraction. To determine the discharge over a thin-edged weir, with upstream crest corner rounded to a radius R, add to the measured head the quantity

$$K=0.70R \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (68)$$

The above formula was deduced by Fteley and Stearns from experiments on weirs with crest radii of one-fourth, one-half, and 1 inch. For heads not exceeding 0.17, 0.26, and 0.45 foot, respectively, the nappe adhered to the crest, and the formula does not apply.

The correction formula (68) is equivalent to increasing the discharge coefficient in the ratio

$$\left(\frac{H+0.7R}{H}\right)^{\frac{3}{2}}$$

or nearly in the ratio

$$\frac{H+R}{H}.$$

A second series of experiments was made with rounded upstream edges of similar radii applied to a crest 4 inches wide, giving the correction formula for this case,

$$K=0.41R \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (69)$$

[a] Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, pp. 97–101.

where K is a correction to be added to the measured head before applying the formula for discharge over the broad-crested weir. This formula is applicable for depths of not less than 0.17 and 0.26 foot, respectively, on weirs with radii of one-fourth and one-half inch. Fteley and Stearns's formulas show the effect to decrease with the breadth of crest. It also decreases, when expressed as a percentage, with the head. These formulas are probably applicable to weirs with smaller, though not to those with greatly larger, radii than those of the experimental weirs.

Bazin experimented upon two weirs, duplicated in the United States Deep Waterways experiments, having crest widths of 2.624 and 6.56 feet, respectively, with an upstream crest radius of 0.328 foot (Pl. IV).

Broad-crested weirs with rounded upstream corner.

Head, in feet.	Coefficient C_1, Bazin's experiments.			
	Crest width, 2.62 feet.		Crest width, 6.56 feet.	
	With angle crest.	With rounded crest.	With angle crest.	With rounded crest.
0.25	2.52	2.85	2.40	2.58
.50	2.59	2.95	2.515	2.76
1.00	2.64	3.00	2.575	2.89
1.50	2.69	3.04	2.635	2.92
	Coefficient C_1, United States Deep Waterways experiments.			
1.50	2.67	2.92	2.39	2.81
2.00	2.75	3.00	2.41	2.81
3.00	2.93	3.17	2.44	2.81
4.00	3.11	3.34	2.47	2.81
5.00	3.30	3.51	2.50	2.81
6.00	Nappe free.	3.00	2.53	2.81

United States Deep Waterways series 14 and 15, Pl. XV, show the effect of rounding the upstream crest corner, radius 0.33 foot, on a model of the Rexford flats, New York, dam. In this case, with a weir 22 feet broad with 6:1 slope on each face, the effect of rounding becomes comparatively slight, the average increase being about 2 per cent.

United States Geological Survey experiments, series Nos. XXXV and XXXVI, Pl. XXVI, show the effect of the addition of a 4-inch radius (0.33 foot), quarter-round extension to the upstream face of the model of an ogee-section dam, having 4.5 feet crest width, 4.5 :1 slope.

Effect of rounded upstream crest corner on an ogee dam.

Head, in feet.	Chambly model, series 35.	Same, with rounded upstream crest corner.	Difference per cent of Francis's coefficient.
0.50	3.18	3.22	+1.5
1.00	3.30	3.34	+1.2
2.00	3.42	3.51	+2.7
3.00	3.49	3.64	+4.5

EXPERIMENTS ON WEIRS WITH DOWNSTREAM SLOPE, OR APRON, OF VARYING INCLINATION.

Aside from the experiments of Blackwell on weirs with very slight inclination and a few series by other experimenters on weirs of irregular section involving aprons, the data on this subject are limited to those of Bazin's experiments.

Bazin selected a number of weir types, each having a constant top width, height, and upstream inclination and applied to each a number of different downstream slopes.[a]

TRIANGULAR WEIRS WITH VERTICAL UPSTREAM FACE AND SLOPING APRONS.

Such weirs are occasionally used, as where the apron slopes to the stream bed in log slides. A similar form in which the downstream slope terminates at a greater or less distance from the vertical upstream face is not uncommon, and to this form the Bazin experiments may probably be applied, provided the breadth of the sloping apron is considerable. The experiments are of special interest, however, as showing the effect of attaching a sloping apron to the downstream face of a thin-edged weir, and by inference affording an indication of the effect of a similar apron attached to any form of cross section. The results of Bazin's experiments recomputed on the basis of the Francis formula are shown on Pl. V.

Four series of experiments on weirs 2.46 feet high are included. For all these series the coefficient C tends to remain nearly constant for the range of heads covered, 0.2 foot to 1.5 feet, there being a slight increase in C with the lower heads only.

Two series on weirs 1.64 feet high are also given. In series 145, slope of apron 3:1, there is a general increase in coefficient with head below 0.9 foot. Series 138, for a weir 1.64 feet high, is duplicated on a weir 2.46 feet high, and the latter series is given preference in the general curve. The lower weirs indicate in both cases slightly higher coefficients, possibly owing to the incomplete elimination of the effect of excessive velocity of approach.

[a] Bazin did not attempt to collate the results extensively. His general résumé has been translated by the writer, and may be found in Rept. U. S. Board of Engineers on Deep Waterways, pt. 2, 1900, pp. 646–658.

The average constant coefficients for the several series are shown in the following table:

Mean coefficients, triangular weirs with varying apron slope.

Series.	Height.	Slope.	Range of head.		Range of C.		Average C.
			From—	To—	From—	To—	
136	2.46	1:1	0.3	1.40	3.84	3.88	3.85
137	2.46	2:1	.3	1.6	3.48	3.52	3.50
138	1.64	2:1	.7	1.5	3.56	3.58	3.57
145	1.64	3:1	.9	1.5	3.39	3.41	3.40
141	2.46	5:1	.6	1.5	3.08	3.14	3.13
142	2.46	10:1	.75	1.5	2.90	2.93	2.91

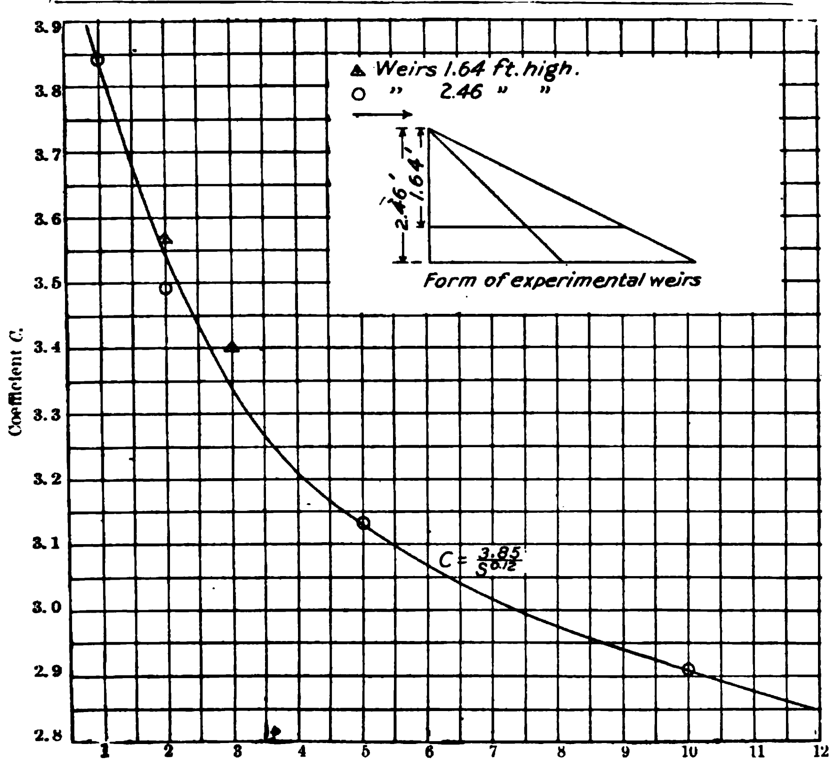


FIG. 9.—Coefficient curve for triangular weirs.

The mean coefficients have also been plotted on fig. 9 and a general curve drawn. This curve becomes approximately a straight line when plotted on logarithmic cross-section paper. Its equation expressed in logarithmic form is

$$C=\frac{3.85}{S^{0.12}} \qquad (70)$$

where S is the batter or slope of apron.

If $S=6$, then, solving by logarithms,

$$\log 6=0.7781513$$
$$\log 6^{0.12}=0.0933782$$
$$\log \frac{1}{6^{0.12}}=9.9066218$$
$$\log 3.85=0.5854607$$
$$\log C=0.4920825$$
$$C=3.105$$

Fig. 9 gives C=3.07; the difference is 1 per cent.

The following conclusions deduced from the recomputed data conform in general with those of Bazin:

1. For steep apron slopes where the nappe tends to break free, the apron materially increases the discharge by permitting a partial vacuum to be formed underneath the nappe.

2. For flat apron slopes the conditions approach those for a horizontal crest.

3. For an apron slope of about 3:1, the discharge is nearly the same as for a thin-edged weir.

4. For slopes greater than 3:1 the apron diminishes the discharge the amount of diminution increasing as the slope becomes flatter.

TRIANGULAR WEIRS WITH UPSTREAM BATTER 1:1 AND VARYING SLOPE OF APRON.

Three series of experiments by Bazin are included (Pl. IX), all made from weirs 1.64 feet high. The results are comparable among themselves, but owing to the high velocity of approach their general applicability is less certain.

Series No. 161, downstream slope 1:1, shows a generally decreasing coefficient with an apparent tendency to become constant through a narrow range of heads, from 0.5 to 0.9 foot, with C=about 4.11.

Series No. 163 and 165, with apron slopes of 2:1 and 5:1, give coefficient lines, which may be fairly represented by the constants 3.82 and 3.47, respectively. These coefficients compare with those for vertical weirs with the same apron slopes as follows:

Comparative coefficients.

Batter of apron.	Vertical face. C	Face inclined 1:1. C	Difference, per cent, Francis's coefficient.
1:1	3.85	4.11	+ 7.8
2:1	3.53	3.82	+ 8.7
5:1	3.13	3.47	+10.2

EXPERIMENTS ON WEIRS OF TRAPEZOIDAL SECTION WITH UPSTREAM SLOPE OF ½:1, HORIZONTAL CREST, AND VARYING DOWNSTREAM SLOPES.

Five series of Bazin's experiments on weirs 2.46 feet high, with crest width of 0.66 foot, are shown on Pls. VI and VII. The curves indicate coefficients increasing with the head, the rate of increase being more rapid for the steeper apron slopes. There is a tendency to depression at about 0.4 foot head, representing, possibly, the point at which the nappe changes from adhering to depressed condition on the downstream face. The curves are all convex, and apparently approach a constant, which was not, however, reached within the limit of experiments, except, perhaps, for the flattest slope of 5:1. The coefficients increase in value as the steepness of the apron slope increases.

Three series of experiments on weirs similar to those above described, but with flat crests 1.317 feet wide, are shown on Pl. VIII. The coefficient curves are of uncertain form for heads below 0.6 foot. For greater heads they may be represented by inclined straight lines. The coefficients increase uniformly with the head, the initial values for 0.6 foot head being nearly the same for the several slopes, the increase being more rapid for the steeper downstream slopes.

It may be seen from the following table that increased width of the flat crest, as compared with that of the preceding weir, causes a decrease in the discharge.

Comparative coefficients at 1-foot head, weirs with flat crests and ½:1 upstream slope.

Slope of apron.	Crest width, in feet.	
	0.66	1.317
1:1	3.52	....
2:1	3.38	2.985
3:1	3.265	....
4:1	3.205	2.94
5:1	3.195	....
6:1	....	2.93

COMBINATION OF COEFFICIENTS FOR WEIRS WITH COMPOUND SLOPES.

Series 163 for an apron slope 2:1 represents a weir form which would be produced by placing, vertical face to vertical face, a weir with back slope 1:1 and a weir with apron slope 2:1. For the former, Bazin's experiments indicate 10 per cent excess discharge over that for a thin-edged weir, and for the latter (from Pl. V) $C=3.50$, equivalent to 5 per cent excess over a thin-edged weir. If the discharge over the 1:1 upstream slope was similarly increased by the addition of an apron, C would be $3.66\times1.05=3.84$. Pl. IX indicates $C=3.82$.

The above method of determining the coefficient for weir of irregular cross section by combining the coefficients for two principal elements of which it is composed, as separate weirs, is restricted in its application and may lead to inconsistencies.

WEIRS WITH VARYING SLOPE OF UPSTREAM FACE.

Experiments were made by Bazin on thin-edged weirs inclined at various angles. Bazin found the ratio of the coefficient of discharge to that for a vertical thin-edged weir to be sensibly constant for all heads within the limits of his experiments, 0.0 to 1.5 feet. Bazin's results were expressed in the form of a modulus by which to multiply the coefficient for a vertical weir to obtain that for an inclined weir. Assuming the Francis coefficient 3.33 to apply to a vertical weir, the coefficients for weirs of various inclinations would be as follows:

Coefficients for inclined weirs, Bazin's experiments.

		Bazin's modulus.	C
Upstream inclination of the weir	1 horizontal to 1 vertical	0.93	3.097
	2 horizontal to 3 vertical	.94	3.130
	1 horizontal to 3 vertical	.96	3.197
Vertical weir		1.00	3.330
Downstream inclination of the weir	1 horizontal to 3 vertical	1.04	3.463
	2 horizontal to 3 vertical	1.07	3.563
	1 horizontal to 1 vertical	1.10	3.663
	2 horizontal to 1 vertical	1.12	3.996
	4 horizontal to 1 vertical	1.09	3.630

On Pl. XVI are shown the results of United States Deep Waterways experiments on weirs 4.9 feet high, having horizontal crests 0.67 foot broad, and with various inclinations of the upstream slope. The experiments cover heads from 1.75 to 5.2 feet, but only 3 or 4 points are given on each coefficient curve. The results indicate in a general way, however, nearly constant coefficients for each inclination of the upstream face. The values of the coefficients are considerably smaller than those obtained by Bazin, whose experiments were on weirs 2.46 feet high with sharp crests.

Pls. X, XI, and XII show the results of experiments of Bazin on weirs of irregular section, with various upstream slopes. Pl. X includes 5 series of experiments on weirs 1.64 feet high, with sharp crest angles, and 2 : 1 downstream slopes. The coefficient curves show a depression period at from 0.3 to 0.7 foot head, beyond which the coefficients may be fairly represented by constants up to 1.5 foot head (the limit of the experiment). A general curve showing the constant coefficient in terms of a downstream slope or batter has been added. This indicates a maximum coefficient of discharge for an upstream slope of about 2.6 : 1. Bazin found, for thin-edged weirs, with inclined downstream slopes, a maximum coefficient for an inclination of 30°, or 1¾ : 1.

Pls. XI and XII show coefficient curves for weirs having the same upstream slopes as in Pl. X, but 2.46 feet high, and with flat crests 0.67 foot wide. The coefficient curves are convex outward, indicating that they may approach constant values at some point beyond the limits of the experiments. The marked difference in character of these coefficient curves, as compared with those in the preceding group, is notable. For weirs with flat crests 0.67 foot wide the coefficients for a given head uniformly increase as the slope becomes flatter up to a batter of about 1¾ : 1. They are also greater for all heads within the limit of the experiments than the coefficients for weirs with sharp crest angles. The comparative values are indicated in the following table:

Comparative coefficients, weirs with varying upstream slope.

Upstream slope.	Pl. X, sharp crest, 2:1 downstream slope; average constant coefficient.	Pls. XI and XII, 0.67 feet crest width, 2:1 downstream slope. Head, in feet. 0.5	1.0	1.5
	C	C	C	C
Vert.	3.58	2.78	3.26	3.51
⅓:1	3.68	2.87	3.34	3.56
½:1	3.72	2.92	3.38	3.62
1:1	3.83	3.03	3.42	3.65
2:1	3.87	3.13	3.43	3.61

It will be seen that the addition of the flat crest has an effect in this case similar to that observed in Pls. VI and VIII, showing the results of experiments by Bazin on weirs with various downstream slopes.

United States Deep Waterways series No. 7, Pl. XVII, may be compared with Bazin's series No. 178, shown on Pl. XI. The former gives a coefficient of 3.55 for a head of 2 feet on a weir 4.895 feet high, the coefficient slowly increasing with the head. The latter gives a coefficient of 3.6 for a head of 1.5 feet, decreasing rapidly as the head decreases.

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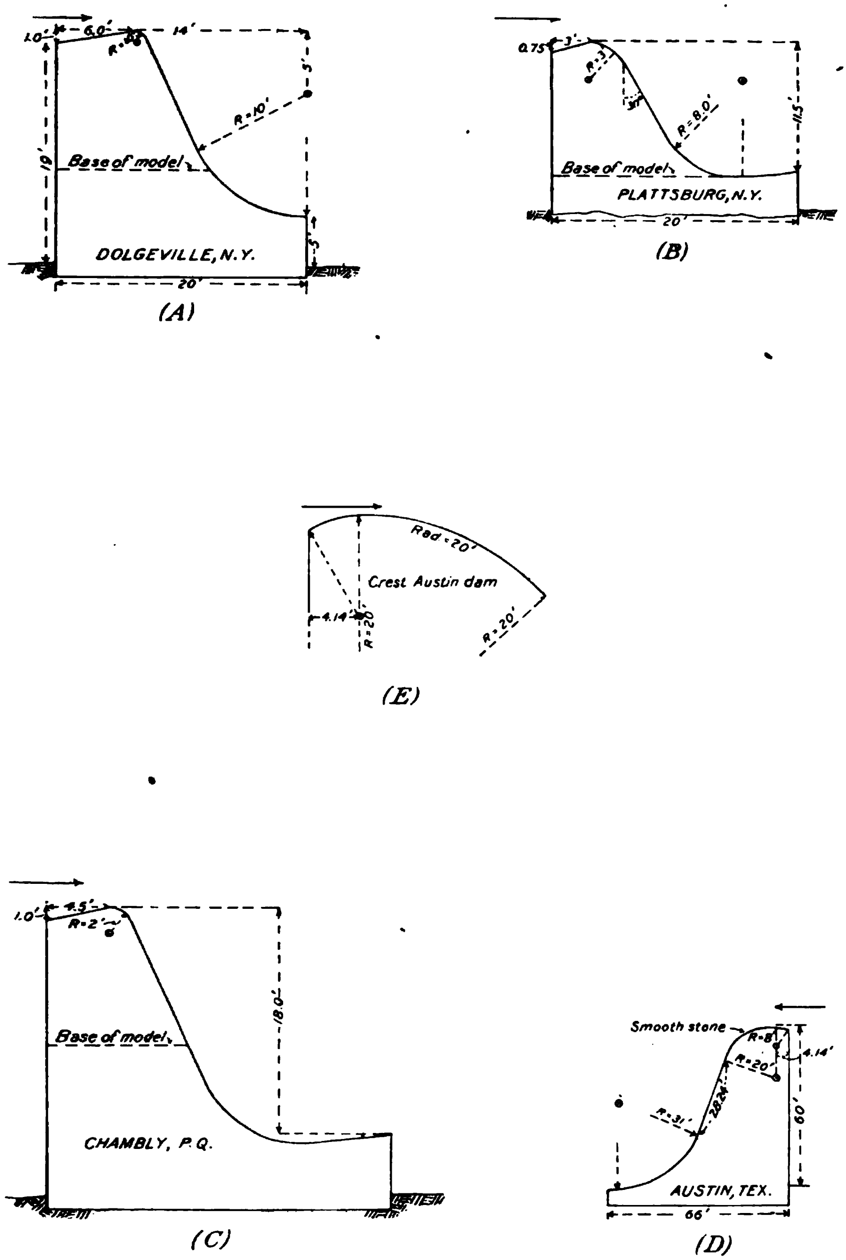
... SECTION. PLATTSBURG-CHAMBLY TYPE.

... experiments on dams of this ... to XXVII. Cross sections of the ... the comparative size of the models ... Survey experiments, are shown ... other ogee dams used as weirs are

... stream crest radius sufficiently ... in contact, yet not so large as to sim- ... We thus exclude the Dolgeville section on ... the nappe as observed in the existing dam par- ... breaks free near the crest for other than very ... other hand, the Austin dam, with a crest ... appears, from the meager data available, to ...

... arranged the available data in order ... width and increased inclination of sloping ... for various depths are as follows:



COMPARATIVE SIZE OF MODELS AND SECTIONS OF OGEE DAMS.

United States Geological Survey series No. XXXIX, Pl. XXVIII, and United States Deep Waterways series No. 18, Pl. XVIII, represent weirs with vertical downstream faces and inclined crests. The upstream slope does not, however, extend back to the bottom of the channel of approach, but is cut off abruptly by a vertical upstream face. The average coefficients deduced from these series have been plotted on a general curve on Pl. XVI, the coefficients agreeing closely with those of the United States Deep Waterways experiments on weirs of similar upstream slope, extending to the channel bottom. United States Geological Survey series No. XXX represents the Dolgeville dam, with rounded crest removed, leaving a trapezoid with crest 6 feet broad and 1 foot lower at upstream than at downstream edge. The coefficient is not constant, but apparently approaches a constant value of about 3.25 for heads exceeding 3 feet. United States Deep Waterways series No. 18 represents a model of the spillway of the Indian Lake dam, having a crest 7 feet wide, 1.5 feet lower at upstream than at downstream edge, which gives an average constant coefficient of 3.42.

It is suggested that if the upstream slope of an inclined weir is continued back 6 feet or more and terminates in a vertical upstream face, the discharge coefficient will not differ materially from that for an upstream slope extending to the channel bottom.

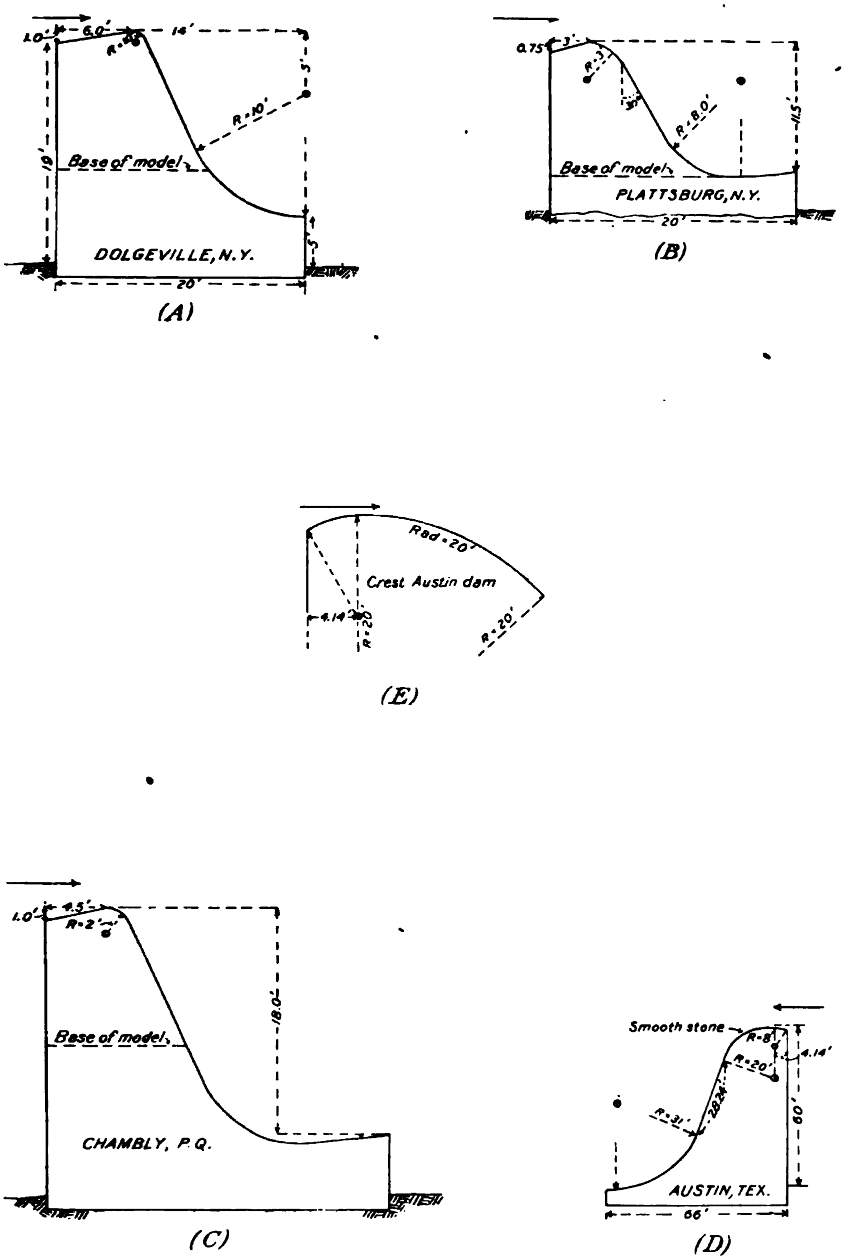
DAMS OF OGEE CROSS SECTION, PLATTSBURG-CHAMBLY TYPE.

The United States Geological Survey experiments on dams of this type are shown on Pls. XXIII to XXVII. Cross sections of the various dams, with lines indicating the comparative size of the models used in the United States Geological Survey experiments, are shown on Pl. XXXIV. Cross sections of other ogee dams used as weirs are shown on Pl. XXXV.

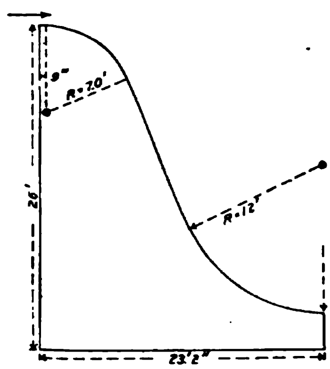
This class includes dams with downstream crest radius sufficiently large to retain the nappe always in contact, yet not so large as to simulate a broad flat crest. We thus exclude the Dolgeville section on the one hand, in which the nappe as observed in the existing dam partially or completely breaks free near the crest for other than very low stages, and on the other hand, the Austin dam, with a crest radius of 20 feet, which appears, from the meager data available, to lie outside this class.

We have arranged the available data in order, advancing with decreased breadth and increased inclination of sloping upstream face.

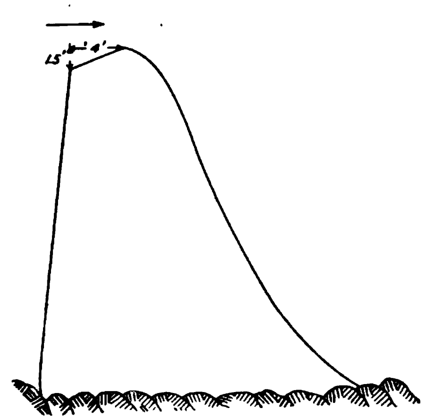
The coefficients for various depths are as follows:



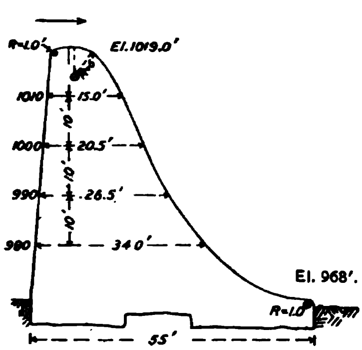
COMPARATIVE SIZE OF MODELS AND SECTIONS OF OGEE DAMS.



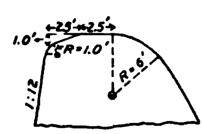
BEAVER RIVER, N.Y.



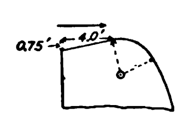
HANNAWA FALLS, N.Y.



TRENTON FALLS, N.Y.

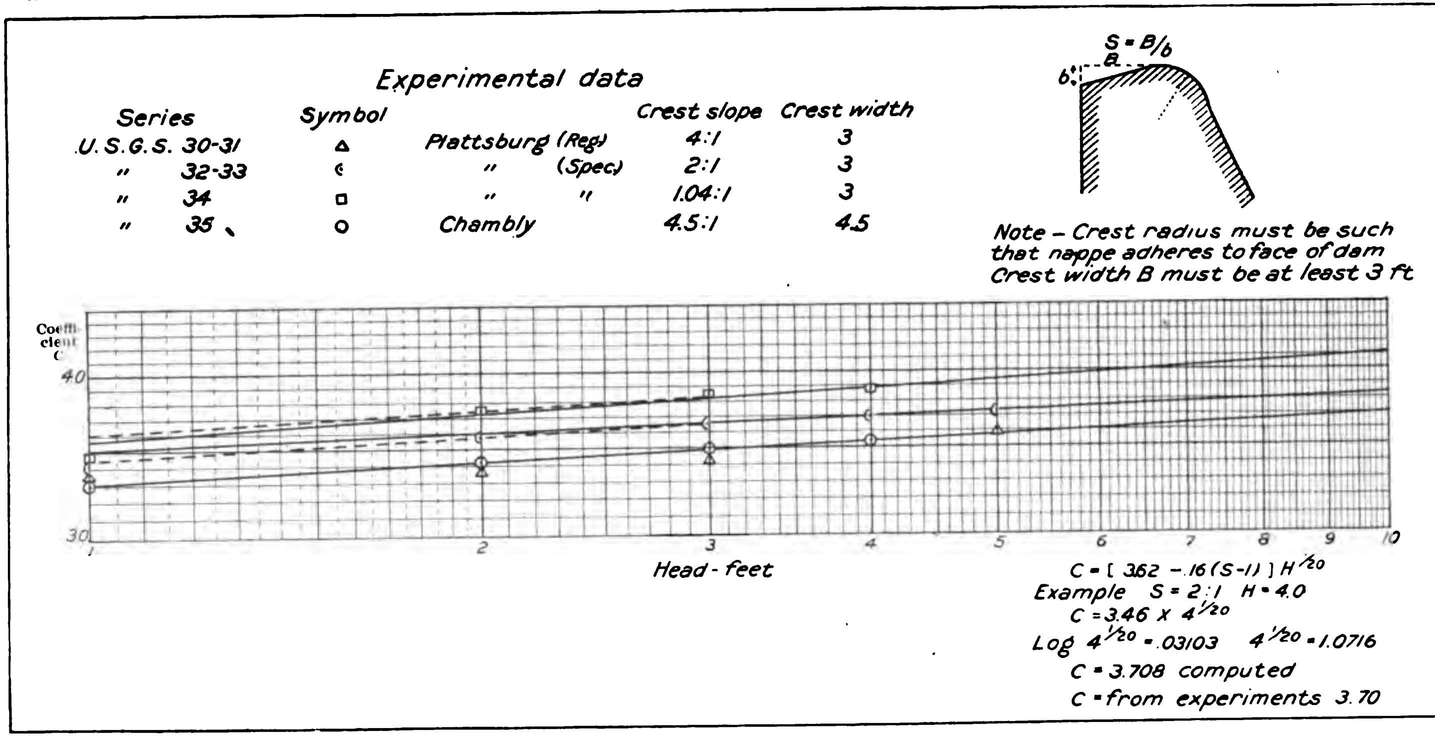


Crest, Trenton Falls



Honk Falls, N.Y.
Crest section

CROSS SECTIONS OF OGEE DAMS.



COEFFICIENT DIAGRAM FOR OGEE DAMS.

Comparative coefficients, dams of ogee cross section.

Dam	Chambly.	Plattsburg.	Modified Plattsburg.					
Approx. constant coefficient	3.435	3.48	3.48	3.48	3.70	3.70	3.70	....
Breadth of slope, in feet	4.5	3	3	3	3	3	3	3
Batter of slope	4½:1	4:1	4:1	4:1	2:1	2:1	2:1	1.04:1
Crest radius, in feet	2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Experiment	....	U.S.G.S.	U.S.G.S.	Mean of	U.S.G.S.	U.S.G.S.	Mean of	U.S.G.S.
Series	....	[a]30	[b]31	30–31	[b]32	[a]33	32–33	34
Head 0.5 foot	....	3.22	....	....	3.29	3.22	3.255	....
1.0 foot	3.30	3.43	3.29	3.36	3.37	3.44	3.405	3.46
2.0 feet	3.42	3.42	3.35	3.385	3.51	3.67	3.59	3.75
3.0 feet	3.49	3.47	3.43	3.45	3.57	3.72	3.645	3.87
4.0 feet	3.53	3.52	3.54	3.53	3.67	3.74	3.705	3.88
5.0 feet	....	....	3.62	....	3.73	....	....	....
6.0 feet	....	....	....	....	....	....	....	....

[a] 15.969 feet crest length, without end contraction.
[b] 7.979 feet crest length, one end contraction.

It appears that the rounded crest changes the character of the law of coefficient from a value tending toward a constant for each back slope to a slowly increasing function of the head. Compared with the constant coefficients for weirs with similar upstream slopes extending back to canal bottom, and with vertical faces, we find that the constant values deduced for these cases correspond with the values of the varying coefficients for ogee sections at a medium head of 2 to 4 feet.

By plotting the data for weirs of ogee section on logarithmic cross-section paper the following convenient approximate formula has been deduced, applicable for weirs with 2 or 3 feet crest radius and upstream slopes 3 to 4.5 feet broad. S indicates the batter ratio of the slope, $\frac{\text{horizontal run}}{\text{vertical rise}}$.

$$C=[3.62-0.16\,(S-1)]\,H^{\frac{1}{20}} \quad . \quad . \quad . \quad . \quad . \quad (71)$$

If $S=2:1$ $\quad H=4.0 \quad C=3.46\times 4^{\frac{1}{20}}$

$$\log 4^{\frac{1}{20}}=0.030103 \qquad C=3.46\times 1.0716=3.70.$$

The experiments give $C=3.74$.

EXPERIMENTS ON DISCHARGE OVER ACTUAL DAMS.

On Pl. XXXVII are shown the results of a number of experiments made by measuring the discharge over existing dams by means of floats or current meters. Aside from those for the Austin, Tex., dam, the data have been collected by Mr. George T. Nelles.[a]

[a] Discussion of paper by G. W. Rafter on the flow of water over dams: Trans. Am. Soc. C. E., vol. 44, pp. 359–362.

BLACKSTONE RIVER AT ALBION, MASS.

This is a timber dam 217 feet long, with horizontal crest 1 foot wide, vertical downstream face, and upstream slope covered with riprap. Discharge was measured by current meter 500 feet below dam, and the depth was measured by hook gage 20 feet upstream from crest. Coefficients have not been corrected to eliminate velocity of approach. They illustrate the uncertainty of discharge for broad-crested weirs of small width under low heads.

MUSKINGUM RIVER, OHIO.

Discharge was measured by rod floats in a cross section 500 feet above the dams, which are constructed of timber cribs filled with stone. Data by Maj. W. H. Bixby, U. S. Army.

Discharge data for Muskingum River dams.

Number of dam.	Length on crest, in feet.	Mean height, in feet.	Area of discharge section, in square feet.	Discharge, in cubic feet per second.	Mean velocity, in feet per second.	Fall over dam, in feet.	Observed depth on crest, in feet.	Coefficient C.
3	848	12.6	7,765	18,118	2.333	8.00	2.86	4.419
4	535	15.9	8,360	25,559	3.045	6.70	4.66	4.723
7	472	14.2	8,230	21,015	2.553	7.00	4.40	4.812
8	515	16.0	7,330	22,310	3.044	5.16	5.90	3.015

The depth on crest has not been corrected to eliminate velocity of approach.

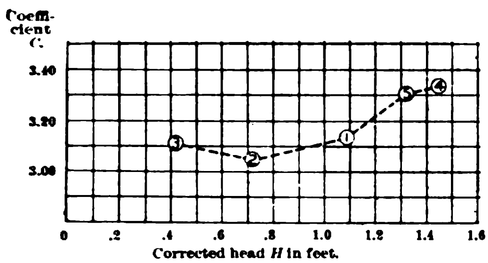
OTTAWA RIVER DAM, CANADA.

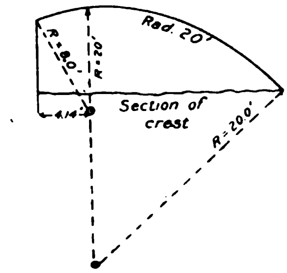
Data by T. C. Clark, C. E. Dam 30 feet high, with upstream and downstream faces planked and sloping 3:1, forming sharp crest angle at junction.

Discharge data for Ottawa River dam.

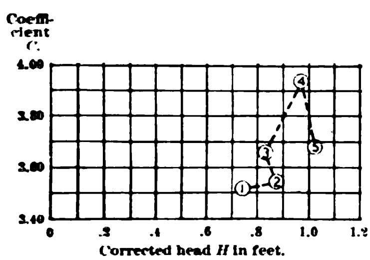
Length of dam, in feet.	Depth on crest, in feet.	Discharge, in cubic feet per second.	Discharge coefficient C.
1,600	2.5	26,000	4.106
1,760	10.0	190,000	3.408

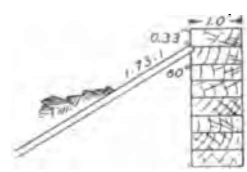
These data are notable as giving the only authentic value of discharge over a dam under so great a head as 10 feet. The high coefficient found for a head of 2.5 feet renders the results somewhat doubtful.



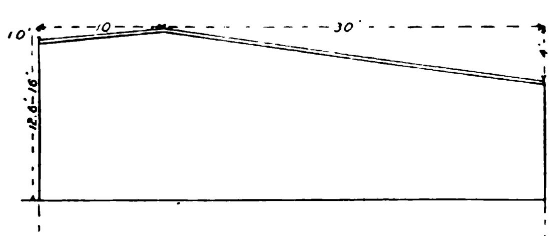


TAYLOR-HOWARD EXPERIMENTS ON DAM AT AUSTIN, TEX

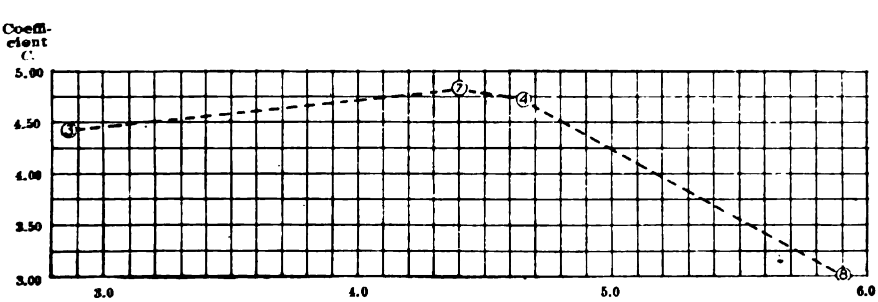




EXPERIMENTS OF DWIGHT PORTER, BLACKSTONE RIVER, ALBION, MASS.



Timber cribs, stone filled



MUSKINGUM RIVER DAM. DATA BY MAJ. W. H. BIXBY, U. S. A.

EXPERIMENTS TO DETERMINE COEFFICIENT C FROM ACTUAL DAMS.

AUSTIN, TEX., DAM.[a]

A series of current-meter measurements of the discharge over this dam were made in January and March, 1900. Several observations at each depth have been combined. The resulting mean coefficients are given in the following table:

Discharge coefficients for the Austin, Tex., dam.

Date.	Number.	H	H=depth at crest of dam.	Range of variation of C.		Number of determinations.	Average value of C.
				From—	To—		
1900.							
Jan. 15	1	1.09	0.838	3.09	3.14	4	3.132
Jan. 18	2	.72	.625	3.00	3.11	11	3.053
Jan. 26	3	.42	.33	3.06	3.13	4	3.112
Mar. 28	4	1.44	1.04	3.32	3.36	3	3.333
Mar. 28	5	1.32	.96	3.26	3.33	5	3.302
Average	......	......	....	......	......	....	3.186

ROUGHNESS OF CREST.

The models used in weir experiments have usually been constructed of planed and matched timber. In actual dams a wide variety of conditions exist, including, in the order of roughness, sheet-steel crests, boards smoothed by wear and rendered slippery by water soaking and fungus growths, unplaned boards, dressed masonry, formed concrete, rubble and undressed ashlar, with earth, cobble, or broken-stone approaches. For the determination of the extent, if any, to which the coefficient applying for a smooth-crested dam must be modified to apply to any of these conditions, the following data are available.

UNITED STATES DEEP WATERWAYS, SERIES 7 AND 8 (PL. XVII).

Model dams, 4.9 feet high, 2:1 slope on both faces. The mean coefficients are about 1 per cent greater for crest of planed boards than for crest covered with one-fourth-inch mesh wire cloth.

[a] Taylor, T. U., the Austin dam: Water-Sup. and Irr. Paper No. 40, U. S. Geol. Survey, 1900, p. 33.

CROTON DAM, ROUND-CREST SECTION, MODEL A (PL. XIX).

Crest rounded, radius 10 feet. Upstream slope about 6:1.

Comparative coefficients with varying roughness, Croton round crest.

Series.....	1	1a	2	3
Head, in feet.	Smooth-pine crest.	Unplaned-plank crest and slope.[a]	Broken-stone slope, unplaned crest.	Broken-stone slope, wire cloth on crest.
0.25	3.34	2.84	3.18	3.16
.50	3.24	2.91	3.18	3.09
1.00	3.21	3.04	3.19	3.15
1.50	3.21	3.12	3.20	3.15
2.00	3.21	3.15	3.21	3.15
2.50	3.21	3.15	3.22	3.15
3.00	3.21	....	3.22	3.15

[a] This series appears doubtful.—R. E. H.

CROTON DAM, ANGULAR SECTION, MODEL B (PL. XX).

Apron slope 1.25:1, upstream slope 6.24:1 for 13 feet, then rough, and slope about 4:1 to bottom.

Comparative coefficients, varying roughness, Croton angular crest.

Series. / Head.	Unplaned plank.	Unplaned plank, rough-stone approach.	Rough-stone approach, wire cloth on crest.
0.25	3.61	....	3.56
.50	3.63	3.66	3.57
1.00	3.67	3.66	3.58
1.50	3.68	3.66	3.60
2.00	3.70	3.66	3.61
2.50	3.70	3.66	3.62

The data given above are somewhat discordant, but indicate that in general the decrease in discharge resulting from the roughness of the various materials forming the crests and approaches of dams will not exceed from 1 to 2 per cent for low heads, and usually decreases as the depth of overflow increases.

FALLS.

Bellasis[a] presents the following analysis for a fall in which there is neither a raised weir nor a lateral reduction in section. If v is the mean velocity at CD, near to AB, then v is both the velocity of approach and the velocity in the weir formula

$$v=\frac{2}{3}c\sqrt{2g\left(D+\alpha\frac{v^2}{2g}\right)},$$

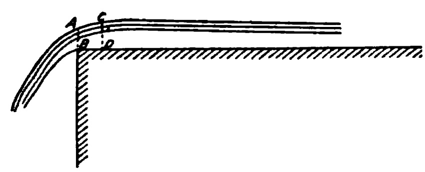


FIG. 10.—Fall.

where c is a coefficient of velocity.

$$v^2=\frac{4}{9}c^2\,2gD+\frac{4}{9}\alpha c^2v^2$$

$$\left(1-\frac{4}{9}\alpha c^2\right)v^2=\frac{4}{9}c^2\,2gD$$

$$v=\frac{\frac{2}{3}c\sqrt{2gD}}{\sqrt{1-\frac{4}{9}\alpha c^2}}$$

Making $\alpha=1.00$ and $c=0.79$.

$$v=\frac{3.424\sqrt{D}}{\sqrt{1-0.277}}=4.74\sqrt{D} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (72)$$

$$Q=vDL=4.74\,LD^{\frac{3}{2}}$$

The depth D is to be measured so near AB that the water shall have acquired its velocity of efflux. The depth will, of course, be affected by the surface curve, the upstream extension of which will be longer according as the slope of the leading channel is flatter, being very great for a horizontal channel. The formula needs experimental verification, but affords a convenient basis of approximation of the flow through troughs and sluices and over aprons and falls.

Experimental data for c are needed.

[a] Hydraulics, p. 99.

WEIR CURVED IN PLAN.

Milldams of both wood and masonry are often constructed to bow upstream, sometimes to secure the added strength of arched form, or to secure additional spillway length, or to follow the crest of a favorable rock ledge, or to throw the ice-bearing current away from intake gates. The dam may follow the arc of a circle, or, as is common with timber dams, there may be an abrupt angle in the plan of the dam. Fig. 11 shows a graphical comparison of curved and angle dams with a straight dam across the same channel, the former being each 13.5 per cent longer than the straight crested dam.

If such an arched spillway opens out of a broad, deep pond, the discharge over it would be greater than for a straight overfall very nearly in proportion to the excess in length of the arc as compared with the length of its chord.

When the stream is confined in a restricted channel, the increased velocity of approach above the longer spillway will become a factor. Thus if two dams—one straight, the other arched—were placed in the same straight, uniform channel, and the depth on crest measured at the same distance upstream from each, then, with the same measured head on both, the velocity of approach to the arched dam would be

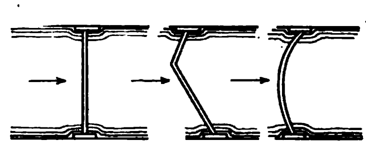


FIG. 11.—Weir curved or angular in plan.

greater nearly in the same proportion that its length of crest and discharge are greater than for the straight crest. Properly corrected for velocity of approach, the arched dam will give a correct measurement of the discharge, the length of the arc being used as the crest length. When the length of the arc greatly exceeds the channel width, the velocity of approach may become excessive, introducing uncertainty as to the proper correction coefficient, difficulty in measuring the head, and an uplifting of the central swifter-flowing portion of the stream surface.

The circular overflow lip of a vertical artesian-well casing is sometimes used to approximate the flow, the measured depth of water above the lip of the pipe, together with its circumference, being used in the weir formula.[a]

[a] Experiments showing the discharge over a circular weir to be proportional to the length of the arc were made by Simpson at Chew Magna, Somersetshire, England, 1850, not recorded in detail.

SUBMERGED WEIRS.

THEORETICAL FORMULA.

In a "submerged," "drowned," "incomplete," or "partial" weir the water on the downstream side stands above the crest level.

The submerged weir is not extensively used as a device for stream gaging. A knowledge of the relations of head, rise, and discharge of such weirs is, however, of great importance in works of river improvement, canals, etc., and the leading formulas are here presented.

It may be added that for situations where head can not be sacrificed, precluding the use of an ordinary weir, and where the velocity is not a continuous function of the depth, as in race ways, making a channel-rating curve inapplicable, the use of submerged weirs to measure or control the discharge merits consideration. Their use for such purposes as the equable division and distribution of water in power canals has hitherto been very restricted, owing to the lack of experimental coefficients.

Let H = Head on upstream side, corrected for velocity of approach.

D = Measured head, upstream side of weir.

d = Measured head, downstream side of weir, or the depth of drowning, taken below the ressault.

Z = Difference of elevation, upstream and downstream sides $= H - d$.

P = Height of weir above channel bottom.

L = Length of weir crest, feet.

v = Mean velocity of approach.

$\triangle$ = Head on a thin-edged weir that would give the same discharge.

M' and C'' coefficients of discharge for a submerged weir.

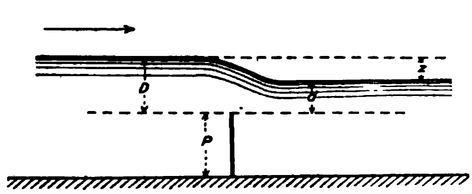


FIG. 12.—Submerged weir.

The theoretical formula of Dubuat for discharge is obtained by regarding the overflow as composed of two portions, one through the upper part $D-d$, treated as free discharge, the other through the lower part d, treated as flow through a submerged orifice.

Combining the two discharges,

$$Q = Q_1 + Q_2 = \frac{2}{3}\sqrt{2g}\,L(D-d)^{\frac{3}{2}} + Ld\sqrt{2g(D-d)}$$

By reducing, including a coefficient, and using the head H corrected for velocity of approach, we have the general formula for a submerged weir.

$$Q=\frac{2}{3}M'L\sqrt{2gZ}\left(H+\frac{d}{2}\right)=C'L\left(H+\frac{d}{2}\right)\sqrt{Z}\quad . \quad . \quad . \quad (73)$$

The head due to the velocity of retreat should in strictness be subtracted from the depth of submergence d. This is not commonly done, however, in the experiments, where the usual method of producing the submergence is by damming and retarding the water below. In practice, if the velocity of retreat is large, the correction should be made.

The theory of formula (73) makes $C''=1.5$ times the value of the coefficient C in the free portion of the discharge.[a] This value is adopted by Dubuat and Weisbach.

D'Aubuisson gives $C'=1.43\,C$.

Francis's early experiments make $C''=1.38\,C$.

From gage records of large rock-filled crib dams on Kentucky River, having planked upstream slope 3:1 and vertical steps below crest—height of dams about 20 feet, heads 4 to 7.5 feet, mean 5.3 feet—Nelles found results as follows:

Dam No. 3, water falling slowly 4 days, $C''=1.5\,C$.
Dam No. 2, water falling slowly 3 days, $C''=1.53\,C$.
Dam No. 1, water rising and falling slowly 5 days, $C''=1.46\,C$.

FTELEY AND STEARNS SUBMERGED-WEIR FORMULA.[b]

Fteley and Stearns use the base formula

$$Q=CL\left(H+\frac{d}{2}\right)\sqrt{Z}\quad . \quad . \quad . \quad . \quad . \quad . \quad (74)$$

Coefficients for the above formula were derived from experiments on thin-edged weirs, by Fteley and Stearns and by J. B. Francis, and give correct results for weirs for which the free discharge would be correctly calculated by the Francis formula.

The head on upstream side varied from 0.3251 to 0.9704 foot, and $\frac{d}{H}$ varied from -0.063 to 0.081 with air under nappe, and from 0.077 to 0.975 with no air under nappe, and in applying the formula the same conditions should be complied with. The authors comment that where sufficient head can not be obtained for a weir of the usual free-discharge type, a submerged weir may be used, provided that the head does not vary greatly.

[a] See valuable discussion of submerged weirs by Geo. T. Nelles in Trans. Am. Soc. C. E., vol. 44, pp. 359–383.

[b] Fteley and Stearns, Experiments on the flow of water, etc.: Trans. Am. Soc. C. E., vol. 12, pp. 101–108.

From a large-scale curve Fteley and Stearns derive the following table of coefficient C, for formula (74):

Fteley and Stearns's coefficients for submerged weirs.

$\frac{d}{H}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	......	3.330	3.331	3.335	3.343	3.360	3.368	3.371	3.372	3.370
.1	3.365	3.359	3.352	3.343	3.335	3.327	3.318	3.310	3.302	3.294
.2	3.286	3.278	3.271	3.264	3.256	3.249	3.241	3.234	3.227	3.220
.3	3.214	3.207	3.201	3.194	3.188	3.182	3.176	3.170	3.165	3.159
.4	3.155	3.150	3.145	3.140	3.135	3.131	3.127	3.123	3.119	3.116
.5	3.113	3.110	3.107	3.104	3.102	3.100	3.098	3.096	3.095	3.093
.6	3.092	3.091	3.090	3.090	3.089	3.089	3.089	3.090	3.090	3.091
.7	3.092	3.093	3.095	3.097	3.099	3.102	3.105	3.109	3.113	3.117
.8	3.122	3.127	3.131	3.137	3.143	3.150	3.156	3.164	3.172	3.181
.9	3.190	3.200	3.209	3.221	3.233	3.247	3.262	3.280	3.300	3.325

Where $\frac{d}{H}$ is less than 0.15 Q is not sensibly affected by submergence.

Where $\frac{d}{H}$ is from 0.5 to 0.8 C may be taken at 3.10.

Correction for velocity of approach was made by the formula $H=D+\frac{v^2}{2g}$. No correction was made for velocity of retreat.

The formula is probably applicable to larger dams and greater depths by selecting proper values of C, $\frac{d}{H}$ being a relative quantity.

A number of empirical formulæ for submerged-weir discharge are also used.

CLEMENS HERSCHEL'S FORMULA.[a]

Herschel's formula, based on experiments of J. B. Francis, 1848, Fteley and Stearns, 1877, and J. B. Francis, 1883, is

$$Q=3.33\ L(NH)^{\frac{3}{2}}=3.33\ L\triangle^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad (75)$$

In this formula the measured head[b] is reduced to an equivalent head that would give the same discharge over a free overflow. The value of the coefficient $N=\frac{\triangle}{H}$ depends on the proportional submergence $\frac{d}{H}$.

[a] Hershel, Clemen, The problem of the submerged weir: Trans. Am. Soc. C. E., vol. 14, May, 1885, pp. 190–196.

[b] Corrected for velocity of approach by method for Francis's formula before applying in above formula.

The values of this ratio, together with their probable error, are given below.

Coefficient N, Herschel's submerged-weir formula.[a]

$\frac{d}{D}$	0.0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.000	1.004	1.006	1.006	1.007	1.007	1.007	1.006	1.006	1.005
.1	1.005	1.003	1.002	1.000	.998	.996	.994	.992	.989	.987
.2	.985	.982	.980	.977	.975	.972	.970	.967	.964	.961
.3	.959	.956	.953	.950	.947	.944	.941	.938	.935	.932
.4	.929	.926	.922	.919	.915	.912	.908	.904	.900	.896
.5	.892	.888	.884	.880	.875	.871	.866	.861	.856	.851
.6	.846	.841	.836	.830	.824	.818	.813	.806	.800	.794
.7	.787	.780	.773	.766	.758	.750	.742	.732	.723	.714
.8	.703	.692	.681	.669	.656	.644	.631	.618	.604	.590
.9	.574	.557	.539	.520	.498	.471	.441	.402	.352	.275

[a] Values for $\frac{d}{D}$ exceeding 0.80 less accurately determined.

$\frac{d}{D}=0.02$ to 0.14, variation of $N=\pm0.005$ to 0.007.
$=0.15$ to 0.22, variation of $N=\pm0.008$ to 0.010.
$=0.24$ to 0,32, variation of $N=\pm0.012$ to 0.014.
$=0.33$ to 0.41, variation of $N=\pm0.015$ to 0.017.
$=0.42$ to 0.59, variation of $N=\pm0.018$.
$=0.60$ to 0.65, variation of $N=\pm0.017$ to 0.015.
$=0.66$ to .071, variation of $N=\pm0.014$ to 0.012.
$=0.72$ to .084, variation of $N=\pm0.011$ to 0.009.

This table indicates that for depths of submergence not exceeding 20 per cent, the head will not ordinarily be increased more than 2 per cent.

The discharge over a submerged weir, according to Herschel's formula, bears the ratio $N^{\frac{3}{2}}$ to that over an unsubmerged weir under the same head.

THE CHANOINE AND MARY FORMULA.

$$Q=M'LH\sqrt{2gZ} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (76)$$

This expression has a form similar to that for the ordinary formula for submerged orifices. It is applicable only under conditions identical with those for which M' has been determined.[a]

[a] Van Nostrand's Eng. Mag., vol. 34, p. 176.

R. H. RHIND'S FORMULA.[a]

$$Q=M'L\sqrt{2g}\left[d\sqrt{Z+0.01v^2}+\frac{2}{3}Z\sqrt{Z+0.035v^2}\right] \quad . \quad . \quad (77)$$

This may be reduced to the theoretical formula (73) by omitting the correction for velocity of approach.

BAZIN'S FORMULAS.[b]

By duplicating, with various depths of submergence, his experiments on thin-edged weirs Bazin deduced the following expressions for the coefficients for submerged weirs to be applied in the discharge formula

$$Q=m'\,LD\,\sqrt{2gD}.$$

Let P represent, as heretofore, the height of weir crest above channel bottom, the coefficient m being that which would apply to the same weir with free discharge.

(1) Accurate formula with small values of d:

$$m'=m\left[1.06+0.16\left(\frac{d}{P}-0.05\right)\frac{P}{D}\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad (78)$$

(2) Accurate formula with large values of d:

$$m'=m\left[\left(1.08+0.18\frac{d}{P}\right)\sqrt[3]{\frac{D-d}{D}}\right] \quad . \quad . \quad . \quad . \quad . \quad (79)$$

(3) Approximate formula for all cases:

$$m'=m\left(1.05+0.21\frac{D}{P}\right)\sqrt[3]{\frac{D-d}{D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (80)$$

The above formulas are for weirs without end contractions.

The coefficient m contains the correction for velocity of approach of the free-discharge weir, and m' contains the necessary factor (if any) for the resulting modification of the velocity of approach effect, when the weir becomes drowned. They are only strictly accurate, therefore, when m' is substituted for m in Bazin's formula.

In Bazin's formulas the height P of the weir enters as a controlling factor in (1), and is present less prominently in (2) and (3).

The modification by drowning is made to depend on $\frac{d}{P}$ in (2), and on this ratio and that of the cube root of $\frac{D-d}{D}$ jointly in formula (3). It is often difficult to determine P or to apply these formulas to a weir fed by a large pond and having end contractions.

[a] Proc. Inst. Civil Engineers, 1896.

[b] Bazin, H., Expériences nouvelles sur l'écoulement en déversoir, 6me art., Ann. Ponts et Chaussées, Mémoires et Documents, 1898.

Assume $P=\infty$

Then (2) becomes

$$m'=m\times 1.08\sqrt[3]{\frac{D-d}{D}}=m\times 1.08\sqrt[3]{\frac{Z}{H}} \quad . \quad . \quad . \quad . \quad (81)$$

and differs from (3) when similarly reduced only in the substitution of 1.05 for 1.08 as a coefficient.

Ex. $D=4'$ $d=2'$ $P=\infty$

If $m=0.425$

$$m'=0.425\times 1.08\sqrt[3]{\frac{2}{4}}=0.364,$$

the discharge being 89.4 per cent of that over an unsubmerged weir under the same head.

Comparison of submerged-weir formulas.[a]

d, feet	.25	.50	.75	.25	.50	.75
H, feet	2.0	2.0	2.0	1.0	1.0	1.0
d/H	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{4}$
	Percentage of unsubmerged-weir discharge.					
Fteley-Stearns	99.91	95.06	89.29	95.01	82.61	64.02
Herschel	100.15	95.83	90.56	95.83	84.24	64.95
Bazin (3)	100.43	95.40	89.78	95.40	83.34	66.15

[a] Weir assumed to be very high so that there is no velocity of approach or of retreat. The coefficient of discharge for a thin-edged weir with free discharge has been taken at 3.33 for the Fteley-Stearns and Herschel formulas.

INCREASE OF HEAD BY SUBMERGED WEIRS.

Any of the submerged-weir formulas may be transformed into expressions giving the rise in water level caused by the construction of a submerged weir in a channel or canal; in this form they are most useful in the design of slack-water navigation works.

RANKINE'S FORMULAS.[a]

Weir not drowned, with flat or slightly rounded crest:

$$H=\triangle=\sqrt[3]{\frac{Q^2}{7L^2}},\text{ approximate} \quad . \quad . \quad . \quad . \quad . \quad (82)$$

Weir drowned:

First approximation—

$$H'=\triangle+d$$

Second approximation—

$$H=H'-d\left(1-\frac{5}{4}\frac{d}{H'-d}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (83)$$

[a] Civil Engineering, p. 689.

COLONEL DYAS'S FORMULA.[a]

This is intended to determine the height of a weir on the crest of a fall in an irrigation or other canal to maintain a desired uniform depth and slope.

D = Depth on weir, feet.
X = Depth of uniform channel, feet.
$P=X-D$ = Height of weir necessary.
A = Area uniform channel section, feet.
R = Hydraulic radius, feet.
S = Slope or fall in feet, per foot.
L = Length of weir crest, feet.

$$D=\left(\frac{900A^2RS}{L^2}\right)^{\frac{1}{3}}-125.8122RS \quad . \quad . \quad . \quad . \quad (84)$$

If $A=1000$ $X=10'$ $R=8.33$ $S=0.001$ $L=100$,

$$D=\left[\frac{900\times1000^2\times8.33\times0.001}{10000}\right]^{\frac{1}{3}}-125.81\times8.33\times0.001$$

$$=9.0856-1.0441=8.04$$

$$P=10-8.04=1.96 \text{ feet.}$$

In this case length of weir equals width of channel, and the velocity of approach would be the mean velocity, which by Kutter's formula will vary, say, from 8 to 10 feet per second under the conditions, depending on the value of the coefficient of roughness n. This would make the flow in the channel 8,000 to 10,000 cubic feet per second.

As a check on the calculated depth D, it will be found that the flow over a weir 100 feet long under a head 8.04 feet (corrected for the large velocity of approach) will also be from, say, 8,000 to 10,000 cubic feet per second, depending upon the coefficient used in the weir formula.

SUBMERGED WEIRS OF IRREGULAR SECTION.

For certain forms of irregular weirs having vertical downstream faces, the discharge when subject to submergence may probably be approximated by applying the ratio of drowned to free discharge for a thin-edged weir similarly submerged as a correction to the coefficient for free discharge over the weir in question. For broad-crested weirs or weirs with aprons this method probably will not be applicable.

BAZIN'S EXPERIMENTS.

For many of the model weirs of irregular section for which free-discharge coefficients were obtained by Bazin, duplicate series of coefficients with various degrees of submergence were also obtained.

[a] Wilson, H. M., Irrigation in India: Twelfth Ann. Rept. U. S. Geol. Survey, 1890-91, pt. 2, p. 482.

Many of these data have been reduced to English units by Nelles.[a] Evidently each form of weir section will require a special formula or table of coefficients, and little more can be done than to refer to the original data for each specific case.

By way of general illustration of the character of submergence effect on weirs of irregular section, the writer has deduced the following roughly approximate formulas from Bazin's experiments on triangular weirs with vertical upstream faces and sloping aprons. The weirs were 2.46 feet high and the end contractions were suppressed. Coefficient curves for free discharge are given on Pl. V.

Three series are included:

Series 195, batter of face 1 : 1.
Series 196, batter of face 2 : 1.
Series 197, batter of face 5 : 1.

Experiments in which the proportional submergence $\frac{d}{D}$ was nearly the same were grouped, and the average values of $\triangle$, D, and d were determined. From these the mean values of $\frac{\triangle}{D}$ and $\frac{d}{D}$ were computed and platted and a straight-line formula deduced.

$$\left.\begin{aligned}\frac{\triangle}{D}&=0.72+b\left(1-\frac{d}{D}\right)\\ b&=0.08+0.17B.\end{aligned}\right\} \quad \ldots\ldots \quad (85)$$

The initial effect occurs when

$$\frac{d}{D}=\frac{0.17B-0.20}{17B+0.08} \quad \ldots\ldots\ldots \quad (86)$$

In the above formulas $\triangle$ is the *measured* head on a weir with free overflow, having the same form of cross section, that would give the same discharge. D is the depth on the submerged weir, d is the depth of submergence, and B is the batter or slope of the apron.

DATA CONCERNING EAST INDIAN WEIRS.

The following data compiled by Nelles[b] are derived from observations on actual dams under heads unusually great. The calculated coefficients in the ordinary weir formula (a)

$$Q=M'LH\sqrt{2gH},$$

in the theoretical submerged-weir formula (b)

$$Q=M'L\sqrt{2gZ}\left(d+\frac{2}{3}Z\right),$$

and in the Rhind formula (77) are given in columns 14, 13, and 12, respectively (p. 145), the observed head being corrected for velocity of approach.

[a] Trans. Am. Soc. C. E., vol. 44, pp. 359–383.
[b] Loc. cit.

Data of submerged flow at certain large masonry dams in India.

Name of river.	Name of weir.	Make-up of weir. Description of parts.	Length of parts, in feet. L	Depth of crest below water surface in— Upper pool, feet. D	Lower pool, feet. d	Observed fall over weir, feet. $D-d=z$	Mean surface velocity of approach, in feet per second. v	Corresponding velocity of head, feet. $\frac{v^2}{2g}$	Calculated discharge, in cubic feet per second. Q	Height of main body of weir, feet. p	Coefficients of discharge in the various submerged-flow formulas. Rhind's (77).	Formula b, observed head. Submerged weir.	Formula a, observed head. Free weir.	Approximate section of weir.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Main body	665	19.21	18.35									
Brahmini	Pattia	First step	60	8.21	7.35	0.86	7.84	0.96	114,000		0.881	0.806	0.241	
		Second step	9	2.21	1.35									Fig. 13.
		Sluices	100	18.10	15.29									
Byturnee	Byturnee	Piers	14	7.68	4.87	2.81			Total, 260,000	7.9	0.953	0.940	0.491	
		Main body	912	10.84	8.03									
		Total overfall	1,026				1.13	8.51	For main body alone, 212,400					
		Seines	100	18.60	15.20					7.9	0.979	0.970	0.511	
Do.	Burrah	Piers	14	8.18	4.78	3.40								
		Main body	412	11.34	7.94									Fig. 14.
		Total overfall	526											
		Sluices	389	18.02	17.55									
Brahmini	Brahmini	Piers	140	8.02	7.55	0.47	6.99	0.76		7.5				
		Main body	3,471	10.52	10.05				260,000		0.878	0.790	0.266	Same as Fig. 13.
		Total overfall	4,000						307,500		0.876	0.786	0.245	
		Sluices	90	30.675	30.15									
Manhanuddy	Kajooree	Breaches	324	25.615	25.09	0.525	11.62	2.10		10.65				Do.
		Main body	3,433	20.115	19.59				680,000		0.893	0.764	0.261	
		Total overfall	3,847						787,000		0.891	0.768	0.355	
		Shore sluices	190	17.45	15.45									
Do.	Manhanuddy	Center sluices	460	17.20	15.20	2.00	7.74	0.93		6.0				Do.
		Main body	5,696	11.45	9.45				488,000		0.595	0.561	0.265	
		Total overfall	6,346						609,000		0.633	0.597	0.274	
		Sluices	255	13.30	12.10									
Do.	Beropa	Piers	118	4.30	3.10	1.20	6.65	0.68		5.0				Do.
		Main body	1,607	8.30	7.10				82,800		0.594	0.570	0.244	
		Total overfall	1,980						112,700		0.648	0.677	0.260	

UNITED STATES DEEP WATERWAYS EXPERIMENTS.[a]

These experiments were made in 1899 at Cornell University hydraulic laboratory on a model having completely rounded profile, being a design for a submerged dam for regulation of Lake Erie.

The coefficient curve for free discharge is given on Pl. XVI. The absolute coefficients and the relative discharge with various degrees of submergence are shown below. The Francis formula is used.

$$Q=C''LH^{\frac{3}{2}}.$$

Absolute coefficients.

D	Submergence from backwater.	C''
Feet.	*Feet.*	
0.0	0.00	3.70
.1	.66	3.67
.2	1.32	3.64
.3	1.98	3.60
.4	2.64	3.54
.5	3.30	3.47
.6	3.99	3.36
.7	4.62	3.17
.8	5.28	2.88
.9	5.94	2.30

Relative coefficients, United States Deep Waterways submerged-weir model.

$\frac{d}{H}$	$\frac{C''}{C}$	$\frac{h}{H}$	$\frac{C''}{C}$
0.0	1.000	0.5	0.937
.1	.991	.6	.907
.2	.983	.7	.856
.3	.972	.8	.778
.4	.956	.9	.621
		1.0	

C is the coefficient for free discharge over a similar weir under the same head.

WEIR DISCHARGE UNDER VARYING HEAD.

Problems of weir discharge under varying head occur in the design of storage reservoirs for river regulation, and in determining the maximum discharge of streams.

[a] Rept. U. S. Board of Engineers on Deep Waterways, pt. 1, p. 291.

An effort has been made in the present chapter to record the various working formulas resulting from the solution of this mathematically difficult portion of the theory of the weir, and to give numerical data to facilitate calculations.

It is assumed that there is no velocity of approach, or, if any, that the head has been corrected therefor. The weir coefficient is also assumed to continue constant through the range of variation of the head.

Notation:

T = Time in seconds required for the head to change between two assigned values.
H_0 = Initial depth on weir, feet.
H_t = Depth on weir at the time t.
S = Reservoir surface area, square feet.
L = Length of overflow weir, feet.
I = Rate of inflow to reservoir, cubic feet per second.
Q = Rate of outflow at time t.

PRISMATIC RESERVOIR, NO INFLOW, TIME REQUIRED TO LOWER WATER SURFACE FROM H_0 TO H_t.[a]

$$dQ = CLH^{\frac{3}{2}}dt = -SdH$$

$$dt = -\frac{S}{CLH^{\frac{3}{2}}}dH$$

$$\int_0^T dt = -\frac{S}{CL}\int_{H_0}^{H_t} H^{-\frac{3}{2}}dH$$

$$T = \frac{2S}{CL}\left(\frac{1}{\sqrt{H_t}} - \frac{1}{\sqrt{H_0}}\right) \quad . \quad . \quad . \quad . \quad . \quad (87)$$

Where $$C = \frac{2}{3}M\sqrt{2g} = 5.35\ M$$

$$T = \infty, \text{ when } H_t = 0.$$

If $S = 1{,}000{,}000$, $H_0 = 4$, $H_t = 0.1$, $C = 3.33$, and $L = 100$,

$$T = \frac{2}{100} \times \frac{3{,}000{,}000}{10}\left(\frac{\sqrt{10}}{1} - \frac{1}{2}\right) = 15{,}972 \text{ seconds} = 4.44 \text{ hours.}$$

To lower the reservoir from $H=4$ to $H=1$ would require 3,000 seconds.

APPROXIMATE TIME OF LOWERING PRISMATIC OR NONPRISMATIC RESERVOIR.

Choosing small successive values of $H_0 - H_t$, we may solve this problem approximately, as shown in the following table:

Time required to lower reservoir from H_0 to $H_t = \dfrac{(H_0 - H_t)\ S}{\text{Mean } Q}$ (88)

[a] Des Ingenieurs Taschenbuch, I, 1902, p. 230.

We may take the mean discharge between the narrow limits H_o and H_t,

$$Q_m = \frac{CL}{2}\left(H_o^{\frac{3}{2}} + H_t^{\frac{3}{2}}\right) \quad . \; . \; . \; . \; . \; . \; . \quad (89)$$

or, using the average head,

$$Q_m = CL\left(\frac{H_o + H_t}{2}\right)^{\frac{3}{2}} \quad . \; . \; . \; . \; . \; . \; . \quad (90)$$

In the following example we have used the latter value, and have made $H_o - H_t = 0.5$ foot. A similar solution may be made for a non-prismatic reservoir, using successive values of $\frac{S_1 + S_2}{2}$ as the reservoir area, and determining the increments of T by formula (88).

Example of varying discharge.

H_o	H_t	Average H	Q per second.	$\frac{1000}{Q}$	T for increment $H_o - H_t$.	Total T, in seconds.
4.0	3.5	3.75	2,417.0	0.4137	207	207
3.5	3.0	3.25	1,951.0	.5126	256	463
3.0	2.5	2.75	1,519.0	.6580	330	793
2.5	2.0	2.25	1,124.0	.8970	448	1,241
2.0	1.5	1.75	771.0	1.2970	650	1,891
1.5	1.0	1.25	465.4	2.1500	1,070	2,961

The total time required in seconds is 2,961, as compared with 3,000 by formula (87).

The time required, using the average Q instead of the average H in the calculation, that is, using formula (89) instead of (90), is 2,933.5 seconds.

The time T is directly proportional to the area of storage surface and inversely proportional to the length of spillway. It is also usually proportional to the value of C in the weir formula.

RESERVOIR PRISMATIC, WITH UNIFORM INFLOW.[a]

GENERAL FORMULAS.

Starting with reservoir full to crest level, $H_o = 0$, to find the time required for the depth of overflow to reach a given stage, H_t.

[a] Mullins, Lieut. Gen. J., Irrigation Manual, Madras Govt., 1890, App. V, pp. 214–223.

When individual values of the increment $H_2 - H_1$ are small, not over 0.5 foot each, if successive values are taken, we have approximately:

$$t = \frac{S\,(H_2 - H_1)}{I - \frac{Q_1 + Q_2}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (91)$$

$$I = \frac{2\,(H_2 - H_1)\,S + Q_1 + Q_2}{2t} \quad . \quad . \quad . \quad . \quad . \quad (92)$$

t = time required to rise through the increment $H_2 - H_1$.

A summation of the successive values of t required for the water to rise each increment will give the total time of rise from H_0 to H_t. Formula (92) will give the maximum run-off from a catchment area tributary to a reservoir if two successive values of H and the corresponding value of t are known.

Formula (92) may also be used to determine T for a nonprismatic reservoir with a variable rate of inflow by choosing such increments, $H_2 - H_1$, that the average values of S, I, and Q will be nearly correct. Variations in the weir coefficient C may also be considered.

FORMULAS FOR TIME OF RISE TO ANY HEAD H, PRISMATIC RESERVOIR WITH UNIFORM INFLOW.

Several analytical solutions of this problem have been made. Starting at spillway level, let H_a equal the depth of overflow corresponding to the quantity of inflow I. The problem is stated by the following differential equation whose primitive is required:

(Rate inflow − rate outflow) $dt = d$ (increase in storage), or

$$(I - CLH^{\frac{3}{2}})\,dt = S\,dH \quad . \quad . \quad . \quad . \quad . \quad . \quad (93)$$

In the solution, mathematical substitutions are necessary in order to render the time-outflow equation integrable in known forms. A very clear demonstration for a special value of C has been given by Frizell.[a] By modifying Frizell's formula to adapt it to the use of any value of C in the weir formula, the following equation is obtained:

$$\frac{3\,CLb}{2S}\,T = \text{nat. log}\,\sqrt{\frac{H + b\sqrt{H} + b^2}{b - \sqrt{H}}} + \sqrt{3}\,\tan^{-1}\sqrt{\frac{1}{3}} - \sqrt{3}\,\tan^{-1}\frac{2\sqrt{H} + b}{b\sqrt{3}} \quad (94)$$

where $b = \sqrt[3]{\frac{I}{CL}}$

When $H = H_a$, the second member becomes the sum of an infinite and two finite quantities, T is then infinite, and the outflow can never

[a] Water Power, pp. 200–203.

become equal to the inflow, or H can never equal H_a, which quantity it approaches as a limit as T increases. Frizell places $H=rH_a$, r having any value less than unity, and, being very nearly unity, $\sqrt{r}$ will be more nearly so, and is taken as equal to unity, without great error, enabling the two inverse trigonometric constants to be evaluated in terms of arc, giving finally:

$$T=\frac{2S}{3(C^2L^2I)^{\frac{1}{3}}}\left(\text{nat. log}\ \frac{\sqrt{1+\sqrt{r}+r}}{1-\sqrt{r}}-0.88625\right)\quad . \quad . \quad (95)$$

Nat. log $N=2.302585 \log_{10} N$

E. Ludlow Gould[a] gives the following formula, identical with the above except in the form of the constant of integration:

$$T=\frac{2S}{3(C^2L^2I)^{\frac{1}{3}}}\Bigg[\text{nat. log}\ \frac{\sqrt{1+\sqrt{r}+r}}{1-\sqrt{r}}$$

$$-\sqrt{3}\left\{\tan^{-1}\frac{2}{\sqrt{3}}\left(\frac{1}{2}+\sqrt{r}\right)-\frac{\pi}{6}\right\}\Bigg]\quad . \quad . \quad . \quad . \quad (96)$$

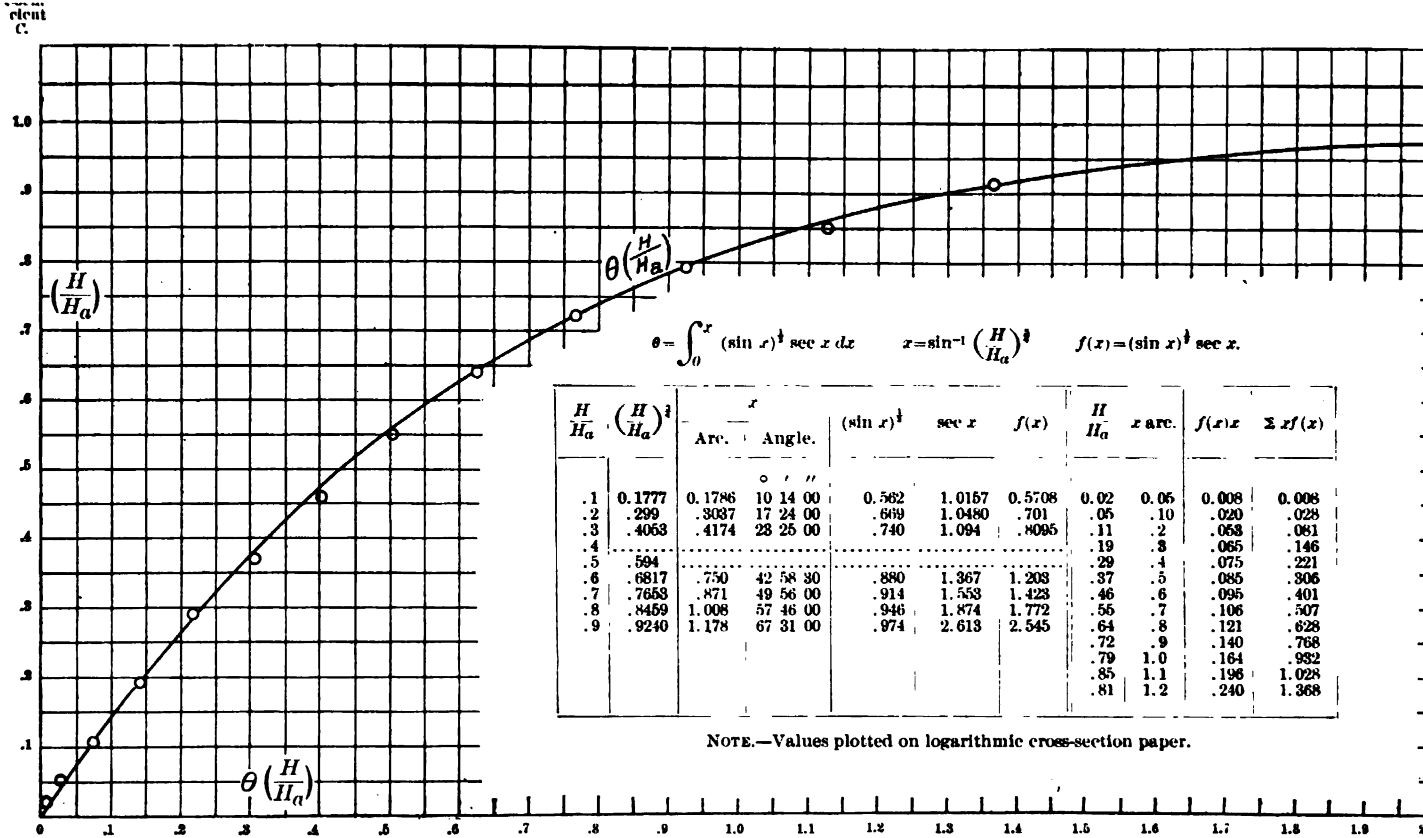
$r=\frac{H}{H_a}$ as before. Gould does not consider $\sqrt{r}$ constant, but derives the values of the function in brackets for various values of r, from which the following table has been derived:

$$\phi=\left[\text{nat. log}\ \frac{\sqrt{1+\sqrt{r}+r}}{1-\sqrt{r}}-\sqrt{3}\left\{\tan^{-1}\frac{2}{\sqrt{3}}\left(\frac{1}{2}+\sqrt{r}\right)-\frac{\pi}{6}\right\}\right]\quad . \quad (97)$$

Values of ϕ, Gould's formula.

$\frac{H}{H_a}$	0	1	2	3	4	5	6	7	8	9
0.0	0.0000	0.0153	0.0306	0.0459	0.0613	0.0766	0.0919	0.1072	0.1226	0.13788
.1	.1532	.16854	..1838	.1992	.2155	.2319	.2483	.2646	.2810	.2973
.2	.3137	.3301	.3464	.3628	.3791	.3955	.4137	.4319	.4501	.4683
.3	.4865	.5047	.5229	.5411	.5593	.5775	.5957	.6139	.6321	.6534
.4	.6747	.6960	.7173	.7386	.7598	.7811	.8024	.8237	.8450	.8663
.5	.8876	.9137	.9399	.9660	.9921	1.0183	1.0444	1.0705	1.0966	1.1128
.6	1.1489	1.1750	1.2012	1.2322	1.2674	1.3027	1.3380	1.3733	1.4086	1.4439
.7	1.4792	1.5145	1.5498	1.5851	1.6203	1.6556	1.7073	1.7590	1.8107	1.8624
.8	1.9141	1.9658	2.0176	2.0715	2.1488	2.2262	2.3035	2.3808	2.4582	2.5355
.9	2.6129	2.7681	2.9233	3.0785	3.2347	3.3889	3.5441	4.0096	4.4750	4.9405

[a] Engineering News, Dec. 5, 1901, pp. 480–481.



$$\theta = \int_0^x (\sin x)^{\frac{1}{3}} \sec x \, dx \qquad x = \sin^{-1}\left(\frac{H}{H_a}\right)^{\frac{3}{4}} \qquad f(x) = (\sin x)^{\frac{1}{3}} \sec x.$$

$\frac{H}{H_a}$	$\left(\frac{H}{H_a}\right)^{\frac{3}{4}}$	x Arc.	x Angle. ° ′ ″	$(\sin x)^{\frac{1}{3}}$	sec x	$f(x)$
.1	0.1777	0.1786	10 14 00	0.562	1.0157	0.5708
.2	.299	.3037	17 24 00	.669	1.0480	.701
.3	.4053	.4174	23 25 00	.740	1.094	.8095
.4						
.5	.594					
.6	.6817	.750	42 58 30	.880	1.367	1.203
.7	.7653	.871	49 56 00	.914	1.553	1.423
.8	.8459	1.008	57 46 00	.946	1.874	1.772
.9	.9240	1.178	67 31 00	.974	2.613	2.545

$\frac{H}{H_a}$	x arc.	$f(x)x$	$\Sigma\, xf(x)$
0.02	0.05	0.008	0.008
.05	.10	.020	.028
.11	.2	.053	.081
.19	.3	.065	.146
.29	.4	.075	.221
.37	.5	.085	.306
.46	.6	.095	.401
.55	.7	.106	.507
.64	.8	.121	.628
.72	.9	.140	.768
.79	1.0	.164	.932
.85	1.1	.196	1.028
.81	1.2	.240	1.368

NOTE.—Values plotted on logarithmic cross-section paper.

Corrected head H in feet.

DIAGRAM OF VARIABLE DISCHARGE.

We may write formula (9)

$$T=\phi\left(\frac{H}{H_a}\right)\times\frac{2S}{3\sqrt[3]{C^2L^2}\,I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (98)$$

R. S. Woodward suggests the formula [a]

$$\left.\begin{aligned} T&=\frac{4S}{3\sqrt[3]{C^2L^2I}}\int_0^X (\sin X)^{\frac{1}{3}}\sec X dX \\ \text{where}\quad X&=\sin^{-1}\sqrt{\frac{CLH^{\frac{3}{2}}}{I}}=\sin^{-1}\left(\frac{H}{H_a}\right)^{\frac{3}{4}} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (99)$$

This, like the preceding expressions, becomes infinity when the integral is carried over the entire range $X=0$ to $X=\frac{\pi}{2}$, conforming with the physical conditions.

The writer has evaluated this function for finite values of $\frac{H}{H_a}$ by mechanical quadrature, as shown in the diagram, Pl. XXXVIII. The diagram illustrates the rapid rise until a head closely approaching H_a is attained, occupying a comparatively short time interval, while for further increments of head the time interval is relatively very great.

E. Sherman Gould [b] gives the same integral developed as an infinite series

$$T=\frac{2SH}{I}\left[\frac{1}{2}+\frac{K}{5}+\frac{K^2}{8}+\cdots+\frac{K^n}{2+3_n}\right] \quad . \quad . \quad (100)$$

where $K=\left(\frac{H}{H_a}\right)^{\frac{3}{2}}$.

If $\vartheta\left(\frac{H}{H_a}\right)=\left[\frac{K^0}{2}+\frac{K^1}{5}+\frac{K^2}{8}+\cdots+\frac{K^n}{2+3_n}\right]$

$$T=\frac{2SH}{I}\times\vartheta\left(\frac{H}{H_a}\right)=\frac{2S}{\sqrt[3]{C^2L^2I}}\vartheta\left(\frac{H}{H_a}\right) \quad . \quad (101)$$

[a] Engineering News, December 5, 1901, p. 431.
[b] Engineering News, November 14, 1901, pp. 362–368.

If we write

$$F=\frac{2S}{3\sqrt[3]{C^2L^2I}},$$

then Frizell's formula may be written $T=F\times\rho\left(\frac{H}{H_a}\right)$

E. L. Gould's formula may be written $T=F\times\phi\left(\frac{H}{H_a}\right)$

Woodward's formula may be written $T=2F\psi\left(\frac{H}{H_a}\right)$

E. Sherman Gould's formula may be written $T=F\times\vartheta\left(\frac{H}{H_a}\right)$. (102)

The formulas are therefore identical, the transcendental factors bearing the relation,

$$\rho\left(\frac{H}{H_a}\right)=\phi\left(\frac{H}{H_a}\right)=2\psi\left(\frac{H}{H_a}\right)=\vartheta\left(\frac{H}{H_a}\right)$$

The E. L. Gould, Woodward, and E. S. Gould formulas are applicable for any value of the ratio $\frac{H}{H_a}$. That of Frizell can be strictly applied only when $\frac{H}{H_a}$ is nearly unity. In the E. S. Gould formula $\vartheta\left(\frac{H}{H_a}\right)$ converges very slowly as the argument approaches unity.

For rough calculations E. S. Gould gives the rule

$$TI-TCL(\mu H)^{\frac{3}{2}}=SH$$

where μ is the coefficient in the weir formula for reducing final head to mean head.

$$T=\frac{SH}{I-CL(\mu H)^{\frac{3}{2}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (103)$$

The ratio μ of the constant mean head which would give the total discharge SH in the time T he finds by trial.

E. S. Gould gives the values

$\mu=0.67$ for small values of H

to $\mu=0.75$ for large values of H.

Comparing the formulas,

Let $S=1,000,000$ square feet

$C=3.33=\frac{10}{3}$

$L=100$

$I=10,000$ cubic feet per second

$H_a=\left(\frac{I}{CL}\right)^{\frac{2}{3}}=30^{\frac{2}{3}}=9.655$ feet.

Required the time to rise to a height $H_a=0.9H=8.6895$ feet.

$$F=\frac{2S}{3\sqrt[3]{C^2L^2I}}=643.5$$

Frizell	(95) $T=1677.6$ seconds.
E. L. Gould	(96) $T=1681.5$ seconds.
Woodward	(99) $T=1660.2$ seconds.
E. S. Gould (approximate)	(103) $T=1488.3$ seconds.

The difference in the value of T by the first three formulas represents the difference in the values of the transcendental portions of the equations as evaluated by different methods.

The time required to rise from H_a to H_b will be the difference of the times T_1 and T_2 by the above formulas.

NONPRISMATIC RESERVOIR, UNIFORM INFLOW.

P. P. L. O'CONNELL.[a]

Representing the reservoir by a cone having its apex at distance A, below plane of the overflow,

$$\left.\begin{array}{ll}\text{Area at overflow level} & =S_0=\pi(\alpha A)^2 \\ \text{Area at any other level} & =S=\pi[\alpha(A+H)]^2\end{array}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (104)$$

where α is the slope of the sides, or where there is α foot horizontal run to 1 foot vertical rise. From (104) with S_0 and α given, A may be determined.

Where the factor $\qquad I_1=\sqrt[3]{\frac{I}{CL}},$

$$T=\frac{\pi\alpha^2}{CL}\Bigg[-4A\sqrt{H}-\frac{2}{3}H^{\frac{3}{2}}-\frac{2}{3}I_1^3 \text{ nat. log}\frac{I_1^3-H^{\frac{3}{2}}}{I_1^3}$$

$$-\frac{(2AI_1^2+A^2)}{3I_1}\text{ nat. log}\left(\frac{[\sqrt{H}-I_1]^2}{H+I_1\sqrt{H}+I_1^2}\right)$$

$$+\frac{2A^2-4AI_1^2}{3I_1}\sqrt{3}\tan^{-1}\left(\frac{-\sqrt{3H}}{2I_1+\sqrt{H}}\right)\Bigg] \quad . \quad . \quad (105)$$

[a] Mullins's Irrigation Manual.

E. L. GOULD.[a]

Calling i the angle of inclination of the banks, P_o the perimeter, at spillway level, exclusive of overflow,

$$S=S_o+BH+B_1H^2 \text{ where } B=P_o \cot i$$

$$B_1=\pi \cot i^2 \quad r=\frac{H}{H_1}$$

$$T=\frac{2}{3I^{\frac{1}{3}}C^2L^2}\left[\left(S_o(CL)^{\frac{4}{3}}+B(ICL)^{\frac{2}{3}}\right) \text{nat. log } \frac{\sqrt{1+\sqrt{r}+r}}{1-\sqrt{r}} \right.$$

$$-\left(S_o(CL)^{\frac{4}{3}}-B(ICL)^{\frac{2}{3}}\right)\sqrt{3}\left\{\tan^{-1}\frac{1+2\sqrt{r}}{\sqrt{3}}-\frac{\pi}{6}\right\}$$

$$\left.-B_1I^{\frac{4}{3}}\left\{\text{nat. log }(1-r^{\frac{3}{2}})+r^{\frac{3}{2}}\right\}-B(ICL)^{\frac{2}{3}}\sqrt{r}\right] \quad . \quad (106)$$

For $i=90°$ and $B=0$, the above formula reduces to (96). the equation for a prismatic reservoir.

VARIABLE INFLOW, NONPRISMATIC RESERVOIR.

This problem may be solved by dividing the reservoir into successive levels, and solving by the formulas previously given, as if each layer represented a portion of a reservoir with a constant inflow equal to the average rate, or if the formulas for prismatic reservoir are used, then each layer will be supposed to represent a portion of a prismatic reservoir of area equal to the average area of the layer.

Mullins's formula may often be more conveniently used and a better solution be obtained than by attempting to average the area and inflow, as would be necessary to apply the analytical formulas given.

The general differential equation for rise in time T with a variable inflow and reservoir area is

$$(I-Q)\, dT=SdH \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (107)$$

If we can express I as a function of T, and S and Q as functions of H, and integrate between the limits $H=0$, $H=H_1$, we may obtain an equation between H and T similar to those given for prismatic reservoirs with constant inflow.

We may write the ordinary weir formula,

$$Q=CLH^{\frac{3}{2}}.$$

[a] Loc. cit.

The area S can usually be readily expressed in terms of the area at crest level and slope of the reservoir sides (assumed constant within the narrow limits $0, H$); the inflow I often increases nearly as a linear function of T while a stream is rising rapidly; we have, then,

$$Q_t = CLH^{\frac{3}{2}}$$
$$S_t = S_o + 2\alpha\sqrt{S_o}H + \alpha^2 H^2$$
$$I_t = I_o + fT.$$

Substituting in (107)

$$\left(I_o + fT - CLH^{\frac{3}{2}} \right) dT = \left(S_o + 2\alpha\sqrt{S_o}H + \alpha^2 H^2 \right) dH \quad . \quad (108)$$

The complete primitive of this differential equation can be determined only as an infinite series.[a]

Rivers during flood usually rise rapidly and fall slowly. The time-inflow function can sometimes be approximated by a modified sinusoid.

$$I = I_o + I_m \sin (bt)^{\frac{1}{n}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (109)$$

where $n =$ or > 1

T=Total duration of flood.

I_m=Maximum rate of inflow.

T_m=Time elapsed from beginning of rise to maximum.

The constants are so chosen that the arc value of the duration of the flood from stage I_o through to the same stage is π, or,

$$(bt)^{\frac{1}{n}} = \pi \qquad b = \frac{\pi^n}{T} \quad . \quad . \quad . \quad . \quad . \quad . \quad (110)$$

For the maximum we will have, differentiating (109),

$$\cos (bT_m)^{\frac{1}{n}} = 0, \qquad \text{or } (bT_m)^{\frac{1}{n}} = \frac{\pi}{2} \quad . \quad . \quad . \quad (111)$$

$$\text{or } b = \frac{1}{T_m}\left(\frac{\pi}{2}\right)^n = \frac{\pi^n}{T} \qquad \text{or} \qquad \frac{T}{T_m} = 2^n$$

$$n = \log \frac{T}{T_m} \times \frac{1}{\log 2} = 3.32204 \log \left(\frac{T}{T_m}\right) \quad . \quad . \quad . \quad (112)$$

common logarithms being used.

If T=1000 and if T_m=200, then n=3.322 log 5=2.322,

$$b = \frac{\pi^{2.322}}{1000} = 0.0143, \text{ and } \frac{1}{n} = 0.43066$$

---

[a] Seddon, James A., C. E. (Proc. Am. Soc. C. E., vol. 24, June, 1898, pp. 559–598), has solved equation (108) for the Great Lakes reservoir system, assuming an annual cycle following the law $I = I_m + A \sin T$, I_m being the mean inflow and T the time arc on a circle whose circumference represents one year. He also assumes $Q = Q_o + bH$, or a linear function of the height H.

Example of variable flood discharge computed by formula (109).

t, in seconds.	(bt)	$\log (bt)$	$\log (bt)^{\frac{1}{n}}$	$(bt)^{\frac{1}{n}}$	Angle.		$\sin (bt)^{\frac{1}{n}}$
					°	′	
100	1.43	0.155336	0.06695	1.1667	66	55	0.9199
200	2.86	.456366	.196694	1.573	90	00	1.0000
300	4.29	.632457	.27259	1.8732	107	21	.9938
400	5.72	.757396	.32644	2.1206	121	08	.8560
500	7.15	.854306	.36820	2.3346	133	49	.7216
600	8.58	.933487	.40233	2.5254	144	42	.5779
700	10.01	1.000434	.43100	2.6978	155	00	.3907
800	11.44	1.058426	.45618	2.8588	163	52	.2779
900	12.87	1.109578	.47790	3.0054	172	12	.1320

The form of the graph of the flood may be determined by plotting the quantities in the last column of this table in terms of t. The resulting curve rises rapidly to a maximum when $t=200$, after which it descends slowly.

TABLES FOR CALCULATIONS OF WEIR DISCHARGE.

The investigations at Cornell University have greatly extended the limit for which weir coefficients are definitely known. The experiments of Bazin did not reach beyond 1.8 feet head maximum. The tables of Francis for thin-edged weirs extended to a head of 3 feet.

The experiments at Cornell have furnished the coefficients for a variety of weir forms for heads up to 4, 5, and 6 feet. At such heads the nappe form has become stable for nearly all forms of weirs. We may now predict the probable extension of the coefficient curves for higher heads with more confidence than could be done by starting from a lower datum.

Owing to their usefulness in the approximate determination of flood discharges, the weir tables have been carried up to a head of 10 feet.

In the tables here given the head is uniformly expressed in feet. For computing the flow over irrigation modules and other small weirs where the head is measured in inches, weir tables expressed with the inch as the argument of head are convenient. Numerous tables of this character are available. The following may be referred to:

The Emerson weir tables, computed by Charla A. Adams, pages 251–285 of Emerson's Hydrodynamics, published by J. and W. Jolly, Holyoke, Mass. These give discharge in cubic feet per minute for weirs with two end contractions having lengths of 2, 3, 4, 5, 6, 7, 8, 10, 12, 16, and 20 feet. The discharge is computed by the Francis formula for heads from 0.001 foot to 2 feet, advancing by thousandths of a foot, with auxiliary table of decimal equivalents of fractional parts of inches.

The Measurement and Division of Water, Bulletin No. 27, Agricultural Experiment Station, Fort Collins, Colo. This publication gives tables of discharge in cubic feet per second, computed by the Francis formula, for a weir 1 foot long, for heads in inches and sixteenths, from $\frac{1}{16}$ inch to 30 inches, with auxiliary table for end contractions, and for velocity of approach correction by the Fteley and Stearns rule ($H=D+\frac{3}{2}h$). A similar weir table for a weir 1 inch long is given. Also a table of discharge for Cippoletti weirs ($C=3.36\frac{2}{3}$), for lengths of crest sill of 1, 1.5, 2, 3, 4, 5, and 10 feet. Head in inches and decimals with feet equivalents.

Special Instructions to Watermasters as to Measurements of Water, State Engineer's Office, Salt Lake City, Utah, 1896. Table of discharge, in cubic feet per second, for 1-foot crest, based on the Francis formula, with auxiliary table for end contractions and velocity of approach. The head is expressed in inches and thirty-seconds (with equivalents in feet) for $\frac{1}{32}$ inch to 36 inches. A similar table for heads in inches and sixteenths, from $\frac{1}{16}$ to 36 inches, gives the discharge in cubic feet per second by the Francis formula for weirs with two end contractions and for the crest lengths of 1, $1\frac{1}{2}$, 2, $2\frac{1}{2}$, 3, 4, 5, 6, 7, 8, 9, 10, 11, and 12 feet. A table for trapezoidal weirs ($C=3.367$) of various crest lengths is also given.

California Hydrography, by J. B. Lippincott, Water-Supply Paper No. 81, United States Geological Survey. This publication contains a table of weir discharge in cubic feet per second for heads, advancing by sixteenths, from $\frac{1}{16}$ inch to 10 inches (with equivalent decimals of a foot), for weirs with two end contractions having crest lengths as follows: 4, 6, 9, 12, 15, and 18 inches, 2, 2.5, 3, 3.5, 4, 4.5, 5, 6, 7, 8, 9, 10, 12, 14, 16, 18, and 20 feet. Based on the Francis formula. Also published as a circular.

The tables that follow are all original computations, with exception of the "Francis weir tables," page 162, and the table of head due to various velocities, page 158.

TABLE I.—HEAD DUE TO VARIOUS VELOCITIES.[a]

This table gives values of the expression

$$h=\frac{v^2}{2g},$$

based on the constant of gravity for the latitude and altitude of Lowell, Mass.,

$$g=32.1618,\ \frac{1}{2g}=0.01554639.$$

[a] Francis, Lowell Hydraulic Experiments, extended.

TABLE 1.—*Values of* $h=\frac{v^2}{2g}$, *or heads due to velocities from 0 to 4.99 feet per second.*

v	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0001	0.0001	0.0001
.1	.0002	.0002	.0002	.0003	.0003	.0003	.0004	.0004	.0005	.0006
.2	.0006	.0007	.0008	.0008	.0009	.0010	.0011	.0011	.0012	.0013
.3	.0014	.0015	.0016	.0017	.0018	.0019	.0020	.0021	.0022	.0024
.4	.0025	.0026	.0027	.0029	.0030	.0031	.0033	.0034	.0036	.0037
.5	.0039	.0040	.0042	.0044	.0045	.0047	.0049	.0051	.0052	.0054
.6	.0056	.0058	.0060	.0062	.0064	.0066	.0068	.0070	.0072	.0074
.7	.0076	.0078	.0081	.0083	.0085	.0087	.0090	.0092	.0095	.0097
.8	.0099	.0102	.0105	.0107	.0110	.0112	.0115	.0118	.0120	.0123
.9	.0126	.0129	.0132	.0134	.0137	.0140	.0143	.0146	.0149	.0152
1.0	0.0155	0.0159	0.0162	0.0165	0.0168	0.0171	0.0175	0.0178	0.0181	0.0185
.1	.0188	.0192	.0195	.0199	.0202	.0206	.0209	.0213	.0216	.0220
.2	.0224	.0228	.0231	.0235	.0239	.0243	.0247	.0251	.0255	.0259
.3	.0263	.0267	.0271	.0275	.0279	.0283	.0288	.0292	.0296	.0300
.4	.0305	.0309	.0313	.0318	.0322	.0327	.0331	.0336	.0341	.0345
.5	.0350	.0354	.0359	.0364	.0369	.0374	.0378	.0383	.0388	.0393
.6	.0398	.0403	.0408	.0413	.0418	.0423	.0428	.0434	.0439	.0444
.7	.0449	.0455	.0460	.0465	.0471	.0476	.0482	.0487	.0493	.0498
.8	.0504	.0509	.0515	.0521	.0526	.0532	.0538	.0544	.0549	.0555
.9	.0561	.0567	.0573	.0579	.0585	.0591	.0597	.0603	.0609	.0616
2.0	0.0622	0.0628	0.0634	0.0641	0.0647	0.0653	0.0660	0.0666	0.0673	0,0679
.1	.0686	.0692	.0699	.0705	.0712	.0719	.0725	.0732	.0739	.0746
.2	.0752	.0759	.0766	.0773	.0780	.0787	.0794	.0801	.0808	.0815
.3	.0822	.0830	.0837	.0844	.0851	.0859	.0866	.0873	.0881	.0888
.4	.0895	.0903	.0910	.0918	.0926	.0933	.0941	.0948	.0956	.0964
.5	.0972	.0979	.0987	.0995	.1003	.1011	.1019	.1027	.1035	.1043
.6	.1051	.1059	.1067	.1075	.1084	.1092	.1100	.1108	.1117	.1125
.7	.1133	.1142	.1150	.1159	.1167	.1176	.1184	.1193	.1201	.1210
.8	.1219	.1228	.1236	.1245	.1254	.1263	.1272	.1281	.1289	.1298
.9	.1307	.1316	.1326	.1335	.1344	.1353	.1362	.1371	.1381	.1390
3.0	0.1399	0.1409	0.1418	0.1427	0.1437	0.1446	0.1456	0.1465	0.1475	0.1484
.1	.1494	.1504	.1513	.1523	.1533	.1543	.1552	.1562	.1572	.1582
.2	.1592	.1602	.1612	.1622	.1632	.1642	.1652	.1662	.1673	.1683
.3	.1693	.1703	.1714	.1724	.1734	.1745	.1755	.1766	.1776	.1787
.4	.1797	.1808	.1818	.1829	.1840	.1850	.1861	.1872	.1883	.1894
.5	.1904	.1915	.1926	.1937	.1948	.1959	.1970	.1981	.1992	.2004
.6	.2015	.2026	.2037	.2049	.2060	.2071	.2083	.2094	.2105	.2117
.7	.2128	.2140	.2151	.2163	.2175	.2186	.2198	.2210	.2221	.2233
.8	.2245	.2257	.2269	.2280	.2292	.2304	.2316	.2328	.2340	.2352
.9	.2365	.2377	.2389	.2401	.2413	.2426	.2438	.2450	.2463	.2475
4.0	0.2487	0.2500	0.2512	0.2525	0.2537	0.2550	0.2563	0.2575	0.2588	0.2601
.1	.2613	.2626	.2639	.2652	.2665	.2677	.2690	.2703	.2716	.2729
.2	.2742	.2755	.2769	.2782	.2795	.2808	.2821	.2835	.2848	.2861
.3	.2875	.2888	.2901	.2915	.2928	.2942	.2955	.2969	.2982	.2996
.4	.3010	.3023	.3037	.3051	.3065	.3079	.3092	.3106	.3120	.3134
.5	.3148	.3162	.3176	.3190	.3204	.3218	.3233	.3247	.3261	.3275
.6	.3290	.3304	.3318	.3333	.3347	.3362	.3376	.3390	.3405	.3420
.7	.3434	.3449	.3463	.3478	.3493	.3508	.3522	.3537	.3552	.3567
.8	.3582	.3597	.3612	.3627	.3642	.3657	.3672	.3687	.3702	.3717
.9	.3733	.3748	.3763	.3779	.3794	.3809	.3825	.3840	.3856	.3871

This value will suffice in ordinary corrections for velocity of approach for localities in the United States.

Velocity of approach correction.

Francis, and as used in portions of this paper (approximate)	$H=D+h$
Fteley and Stearns, contracted weir	$H=D+1.5h$
Hamilton Smith, suppressed weir	$H=D+1\frac{1}{3}h$
Hamilton Smith, contracted weir	$H=D+1.4h$

TABLE 2.—PERCENTAGE INCREASE IN DISCHARGE BY VARIOUS RATES OF VELOCITY OF APPROACH.

This table has been calculated from the Francis correction formula,

$$H^{\frac{3}{2}}=(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}.$$

The percentage increase in discharge over that at the same measured head with no velocity of approach is

$$\text{Percentage}=100\frac{H^{\frac{3}{2}}-D^{\frac{3}{2}}}{D^{\frac{3}{2}}}=K \quad . \quad . \quad . \quad . \quad (113)$$

TABLE 2.—*Percentage increase in discharge over weirs for various rates of mean velocity of approach.*

D=measured head in feet; H=head corrected for velocity of approach; v=mean velocity of approach, feet per second; $h=\frac{v^2}{2g}$; $H=\left[(D+h)^{\frac{3}{2}}-h^{\frac{3}{2}}\right]^{\frac{2}{3}}$.

v	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2
h	0.0025	0.0039	0.0056	0.0076	0.0099	0.0126	0.0155	0.0188	0.0224	0.0263	0.0305	0.0350	0.0398	0.0449	0.0504	0.0561	0.0622	0.0686	0.0752
$h^{\frac{3}{2}}$	0.0002	0.0003	0.0005	0.0007	0.0010	0.0014	0.0019	0.0025	0.0033	0.0041	0.0051	0.0064	0.0079	0.0095	0.0111	0.0132	0.0154	0.0179	0.0206
D	Percentage increase in discharge=$100\left[1-\frac{H^{\frac{3}{2}}}{D^{\frac{3}{2}}}\right]$																		
0.2	1.36	2.65	3.72	5.02	6.44	8.16	9.77	12.19	14.13	16.34	18.62	20.80	22.51	25.39	27.74	30.78	33.55	36.30	39.06
.4	.68	1.34	1.90	2.58	3.33	4.20	5.11	6.16	7.20	8.39	9.62	10.86	12.16	13.54	14.94	16.53	18.11	19.72	21.34
.6	.42	.90	1.29	1.74	2.22	2.86	3.44	4.14	4.88	5.69	6.56	7.49	8.40	9.33	10.42	11.49	12.57	13.72	14.86
.8	.40	.62	.92	1.26	1.62	2.11	2.66	3.12	3.70	4.33	4.98	5.67	6.45	7.14	7.97	8.88	9.66	10.56	14.76
1.0	.36	.56	.79	1.07	1.40	1.76	2.15	2.59	3.05	3.56	4.10	4.66	5.24	5.86	6.54	7.21	7.93	8.68	9.43
1.5	.24	.37	.53	.72	.94	1.19	1.45	1.75	2.07	2.48	2.79	3.22	3.50	4.01	4.48	4.94	5.45	5.96	6.49
2.0	.19	.29	.41	.59	.72	.91	1.11	1.84	1.58	1.84	2.13	2.42	2.72	3.06	3.42	3.78	4.17	4.57	4.98
2.5	.18	.22	.31	.43	.56	.71	.88	1.06	1.25	1.47	1.70	1.94	2.19	2.45	2.75	3.04	3.36	3.68	3.95
3.0	.12	.19	.27	.36	.47	.60	.74	.89	1.06	1.24	1.43	1.63	1.84	2.07	2.31	2.56	2.83	3.10	3.38
3.5	.10	.16	.28	.31	.40	.47	.63	.76	.90	1.06	1,23	1.40	1.58	1.78	1.99	2.21	2.48	2.67	2.98
4.0	.09	.14	.20	.27	.35	.45	.56	.67	.80	.93	1.08	1.23	1.39	1.57	1.75	1.94	2.14	2.36	2.57
4.5	.08	.12	.18	.24	.31	.40	.49	.60	.71	.83	.96	1.10	1.24	1.40	1.56	1.73	1.92	2.10	2.30
5.0	.07	.11	.16	.22	.27	.35	.44	.54	.65	.75	.88	.98	1.12	1.26	1.42	1.57	1.64	1.90	2.07
5.5	.06	.10	.14	.20	.27	.33	.40	.49	.58	.68	.78	.90	1.00	1.15	1.28	1.43	1.57	1.73	1.90
6.0	.06	.09	.13	.17	.23	.30	.37	.45	.53	.63	.72	.84	.94	1.05	1,18	1.31	1.45	1.59	1.74
6.5	.06	.08	.12	.17	.22	.28	.34	.41	.49	.58	.67	.76	.87	.98	1.12	1.21	1.84	1.47	1.61
7.0	.05	.08	.11	.15	.20	.26	.32	.39	.46	.54	.63	.72	.80	.91	1.02	1.18	1.25	1.87	1.50
7.5	.04	.07	.10	.14	.18	.24	.29	.36	.43	.50	.58	.66	.75	.84	.95	1.05	1.17	1.28	1.40
8.0	.04	.07	.10	.14	.18	.23	.28	.34	.40	.47	.55	.63	.71	.80	.90	.99	1.10	1.21	1.32
8.5	.04	.06	.09	.13	.16	.21	.26	.31	.38	.44	.51	.59	.67	.75	.84	.93	1.03	1.13	1.24
9.0	.04	.06	.09	.12	.16	.20	.25	.30	.36	.42	.48	.55	.63	.71	.80	.88	.98	1.07	1.17
9.5	.04	.06	.08	.11	.15	.19	.23	.28	.34	.40	.46	.53	.60	.67	.75	.84	.93	1.02	1.11
10.0	.04	.05	.08	.10	.14	.19	.23	.27	.31	.37	.44	.50	.57	.64	.71	.79	.84	.90	1.09

v	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4.0
h	0.0822	0.0895	0.0972	0.1051	0.1133	0.1219	0.1307	0.1399	0.1494	0.1592	0.1693	0.1797	0.1904	0.2015	0.2128	0.2128	0.2365	0.2487
$h^{\frac{3}{2}}$	0.0235	0.0268	0.0303	0.0340	0.0381	0.0426	0.0472	0.0524	0.0576	0.0634	0.0696	0.0761	0.0830	0.0904	0.0982	0.1064	0.1150	0.1242
D	Percentage increase in discharge $=100\left[1-\frac{H^{\frac{3}{2}}}{D^{\frac{3}{2}}}\right]$																	
0.2	42.08	44.91	48.00	51.14	54.22	57.34	60.64	63.68	67.34	70.65	73.96	75.58	80.80	84.28	87.66	90.35	93.94	97.27
.4	23.06	24.77	26.59	28.45	30.29	32.19	34.15	36.09	38.15	40.22	42.27	44.37	46.50	48.66	50.79	53.07	55.26	57.39
.6	16.12	17.64	18.68	20.02	37.16	22.80	24.17	25.60	27.12	28.62	30.14	31.75	33.26	34.87	36.47	38.12	39.86	41.49
.8	12.44	13.42	14.46	15.51	16.58	17.77	18.89	19.95	21.16	22.26	23.56	24.80	26.06	27.36	28.64	29.97	31.28	32.73
1.0	10.23	11.04	11.90	12.77	13.66	14.57	15.51	16.46	17.47	18.47	19.48	20.52	21.58	22.66	23.74	24.86	26.00	27.11
1.5	7.06	7.65	8.28	8.84	9.47	10.08	10.78	11.46	12.12	12.88	13.62	14.36	15.12	15.90	16.67	17.48	18.30	18.84
2.0	5.44	5.85	6.32	6.79	7.28	7.78	8.30	8.83	9.39	9.93	10.51	11.10	11.69	12.27	12.92	13.55	14.18	14.83
2.5	4.36	4.73	5.11	5.50	5.90	6.32	6.74	7.17	7.63	8.09	8.56	9.04	9.53	10.04	10.54	11.07	11.60	12.14
3.0	3.68	3.99	4.31	4.64	4.98	5.33	5.70	6.07	6.45	6.84	7.24	7.78	8.00	8.50	8.98	9.38	9.84	10.29
3.5	3.18	3.44	3.73	4.01	4.31	4.61	4.98	5.25	6.04	5.93	6.27	6.63	7.00	7.37	7.75	8.14	8.54	8.86
4.0	2.80	3.04	3.28	3.54	3.80	4.07	4.35	4.63	4.93	5.23	5.54	5.86	6.24	6.52	6.85	7.20	7.56	7.91
4.5	2.50	2.71	2.93	3.16	3.40	3.64	3.89	4.14	4.41	4.68	4.96	5.24	5.5	5.84	6.14	6.45	6.7	7.11
5.0	2.26	2.46	2.66	2.87	3.09	3.30	3.53	3.76	4.00	4.25	4.50	4.75	5.0	5.30	5.57	5.86	6.15	6.44
5.5	2.10	2.24	2.42	2.61	2.80	3.01	3.21	3.43	3.63	3.88	4.11	4.34	4.59	4.84	5.09	5.35	5.63	5.88
6.0	1.90	2.06	2.22	2.40	2.58	2.77	2.96	3.09	3.36	3.57	3.78	4.00	4.2	4.46	4.69	4.93	5.1	5.40
6.5	1.75	1.90	2.06	2.22	2.39	2.56	2.74	2.94	3.17	3.31	3.57	3.71	3.9	4.13	4.35	4.52	4.8	5.01
7.0	1.63	1.78	1.92	2.07	2.23	2.39	2.56	2.67	2.96	3.09	3.27	3.46	3.6	3.86	4.06	4.15	4.4	4.68
7.5	1.53	1.66	1.80	1.94	2.13	2.22	2.39	2.55	2.73	2.88	3.06	3.24	3.4	3.61	3.80	4.00	4.2	4.41
8.0	1.44	1.56	1.69	1.82	1.96	2.10	2.25	2.40	2.56	2.72	2.88	3.04	3.2	3.40	3.58	3.76	3.9	4.13
8.5	1.35	1.47	1.59	1.72	1.85	1.98	2.12	2.26	2.41	2.56	2.72	2.88	3.0	3.20	3.38	3.55	3.7	3.89
9.0	1.28	1.39	1.51	1.62	1.78	1.87	2.0	2.14	2.28	2.43	2.57	2.72	2.9	3.04	3.20	3.36	3.5	3.69
9.5	1.22	1.32	1.43	1.54	1.66	1.78	1.90	2.03	2.17	2.30	2.44	2.59	2.7	2.88	3.04	3.20	3.3	3.51
10.0	1.15	1.25	1.36	1.47	1.57	1.68	1.80	1.94	2.05	2.19	2.32	2.47	2.6	2.74	2.89	3.04	3.2	3.36

To use this table the discharge corresponding to the measured head D may be taken directly from Table 3 or 4 and the quantity so obtained increased by the percentage indicated in Table 2. This table is especially useful where the velocity of approach is measured directly. If the velocity of approach is determined from the approximate discharge by the formula $v=\frac{Q}{A}$, successive approximate corrections may be required.

Table 2 shows directly the relative error introduced by various velocities of approach. The large error introduced by moderate velocities with low heads and the comparatively small error resulting from higher velocities under great heads are conspicuous.

TABLES 3 AND 4.—DISCHARGE OVER A THIN-EDGED WEIR BY THE FRANCIS FORMULA.

These tables give the discharge in cubic feet per second, for a crest length of 1 foot, without contractions, computed by the formula

$$Q=3.33LH^{\frac{3}{2}}.$$

TABLE 3.—*Discharge over a thin-edged weir per foot of crest.*

Head H, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
0.00	0.0000	0.0001	0.0003	0.0005	0.0008	0.0012	0.0015	0.0020	0.0024	0.0028
.01	.0033	.0038	.0044	.0049	.0055	.0061	.0067	.0074	.0080	.0087
.02	.0094	.0101	.0109	.0116	.0124	.0132	.0140	.0148	.0156	.0164
.03	.0173	.0182	.0191	.0200	.0209	.0218	.0227	.0237	.0247	.0256
.04	.0266	.0276	.0287	.0297	.0307	.0318	.0329	.0339	.0350	.0361
.05	.0372	.0384	.0395	.0406	.0418	.0430	.0441	.0453	.0465	.0477
.06	.0489	.0502	.0514	.0527	.0539	.0552	.0565	.0578	.0590	.0604
.07	.0617	.0630	.0643	.0657	.0670	.0684	.0698	.0712	.0725	.0739
.08	.0753	.0768	.0782	.0796	.0811	.0825	.0840	.0855	.0869	.0884
.09	.0899	.0914	.0929	.0944	.0960	.0975	.0990	.1006	.1022	.1037
0.10	0.1053	0.1069	0.1085	0.1101	0.1117	0.1133	0.1149	0.1166	0.1182	0.1198
.11	.1215	.1231	.1248	.1265	.1282	.1299	.1316	.1333	.1350	.1367
.12	.1384	.1402	.1419	.1436	.1454	.1472	.1489	.1507	.1525	.1543
.13	.1561	.1579	.1597	.1615	.1633	.1652	.1670	.1689	.1707	.1726
.14	.1744	.1763	.1782	.1801	.1820	.1839	.1858	.1877	.1896	.1915
.15	.1935	.1954	.1973	.1993	.2012	.2032	.2052	.2072	.2091	.2111
.16	.2131	.2151	.2171	.2191	.2212	.2232	.2252	.2273	.2293	.2314
.17	.2334	.2355	.2375	.2396	.2417	.2438	.2459	.2480	.2501	.2522
.18	.2543	.2564	.2586	.2607	.2628	.2650	.2671	.2693	.2714	.2736
.19	.2758	.2780	.2802	.2823	.2845	.2867	.2890	.2912	.2934	.2956
0.20	0.2978	0.3001	0.3023	0.3046	0.3068	0.3091	0.3113	0.3136	0.3159	0.3182
.21	.3205	.3228	.3250	.3274	.3297	.3320	.3343	.3366	.3389	.3413
.22	.3436	.3460	.3483	.3507	.3530	.3554	.3578	.3601	.3625	.3649
.23	.3673	.3697	.3721	.3745	.3769	.3794	.3818	.3842	.3866	.3891
.24	.3915	.3940	.3964	.3989	.4014	.4038	.4063	.4088	.4113	.4138
.25	.4162	.4187	.4213	.4238	.4263	.4288	.4313	.4339	.4364	.4389
.26	.4415	.4440	.4466	.4491	.4517	.4543	.4568	.4594	.4620	.4646
.27	.4672	.4698	.4724	.4750	.4776	.4802	.4828	.4855	.4881	.4907
.28	.4934	.4960	.4987	.5013	.5040	.5067	.5093	.5120	.5147	.5174
.29	.5200	.5227	.5254	.5281	.5308	.5336	.5363	.5390	.5417	.5444

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
0.30	0.5472	0.5499	0.5527	0.5554	0.5582	0.5609	0.5637	0.5664	0.5692	0.5720
.31	.5748	.5775	.5803	.5831	.5859	.5887	.5915	.5943	.5972	.6000
.32	.6028	.6056	.6085	.6113	.6141	.6170	.6198	.6227	.6255	.6284
.33	.6313	.6341	.6370	.6399	.6428	.6457	.6486	.6515	.6544	.6573
.34	.6602	.6631	.6660	.6689	.6719	.6748	.6777	.6807	.6836	.6866
.35	.6895	.6925	.6954	.6984	.7014	.7043	.7073	.7103	.7133	.7163
.36	.7193	.7223	.7253	.7283	.7313	.7343	.7373	.7404	.7434	.7464
.37	.7495	.7525	.7555	.7586	.7616	.7647	.7678	.7708	.7739	.7770
.38	.7800	.7831	.7862	.7893	.7924	.7955	.7986	.8017	.8048	.8079
.39	.8110	.8142	.8173	.8204	.8235	.8267	.8298	.8330	.8361	.8393
0.40	0.8424	0.8456	0.8488	0.8519	0.8551	0.8583	0.8615	0.8646	0.8678	0.8710
.41	.8742	.8774	.8806	.8838	.8870	.8903	.8935	.8967	.8999	.9032
.42	.9064	.9096	.9129	.9161	.9194	.9226	.9259	.9292	.9324	.9357
.43	.9390	.9422	.9455	.9488	.9521	.9554	.9587	.9620	.9653	.9686
.44	.9719	.9752	.9785	.9819	.9852	.9885	.9919	.9952	.9985	1.0019
.45	1.0052	1.0086	1.0119	1.0153	1.0187	1.0220	1.0254	1.0288	1.0321	1.0355
.46	1.0389	1.0423	1.0457	1.0491	1.0525	1.0559	1.0593	1.0627	1.0661	1.0696
.47	1.0730	1.0764	1.0798	1.0833	1.0867	1.0901	1.0936	1.0970	1.1005	1.1039
.48	1.1074	1.1109	1.1143	1.1178	1.1213	1.1248	1.1282	1.1317	1.1352	1.1387
.49	1.1422	1.1457	1.1492	1.1527	1.1562	1.1597	1.1632	1.1668	1.1703	1.1738
0.50	1.1773	1.1809	1.1844	1.1879	1.1915	1.1950	1.1986	1.2021	1.2057	1.2093
.51	1.2128	1.2164	1.2200	1.2235	1.2271	1.2307	1.2343	1.2379	1.2415	1.2451
.52	1.2487	1.2523	1.2559	1.2595	1.2631	1.2667	1.2703	1.2740	1.2776	1.2812
.53	1.2849	1.2885	1.2921	1.2958	1.2994	1.3031	1.3067	1.3104	1.3141	1.3177
.54	1.3214	1.3251	1.3287	1.3324	1.3361	1.3398	1.3435	1.3472	1.3509	1.3546
.55	1.3583	1.3620	1.3657	1.3694	1.3731	1.3768	1.3806	1.3843	1.3880	1.3918
.56	1.3955	1.3992	1.4030	1.4067	1.4105	1.4142	1.4180	1.4217	1.4255	1.4293
.57	1.4330	1.4368	1.4406	1.4444	1.4481	1.4519	1.4557	1.4595	1.4633	1.4671
.58	1.4709	1.4747	1.4785	1.4823	1.4862	1.4900	1.4938	1.4976	1.5014	1.5053
.59	1.5091	1.5130	1.5168	1.5206	1.5245	1.5283	1.5322	1.5361	1.5399	1.5438
0.60	1.5476	1.5515	1.5554	1.5593	1.5631	1.5670	1.5709	1.5748	1.5787	1.5826
.61	1.5865	1.5904	1.5943	1.5982	1.6021	1.6060	1.6100	1.6139	1.6178	1.6217
.62	1.6257	1.6296	1.6335	1.6375	1.6414	1.6454	1.6493	1.6533	1.6572	1.6612
.63	1.6652	1.6691	1.6731	1.6771	1.6810	1.6850	1.6890	1.6930	1.6970	1.7010
.64	1.7050	1.7090	1.7130	1.7170	1.7210	1.7250	1.7290	1.7330	1.7370	1.7410
.65	1.7451	1.7491	1.7531	1.7572	1.7612	1.7652	1.7693	1.7733	1.7774	1.7814
.66	1.7855	1.7896	1.7936	1.7977	1.8018	1.8058	1.8099	1.8140	1.8181	1.8221
.67	1.8262	1.8303	1.8344	1.8385	1.8426	1.8467	1.8508	1.8549	1.8590	1.8632
.68	1.8673	1.8714	1.8755	1.8796	1.8838	1.8879	1.8920	1.8962	1.9003	1.9045
.69	1.9086	1.9128	1.9169	1.9211	1.9252	1.9294	1.9336	1.9377	1.9419	1.9461
0.70	1.9503	1.9544	1.9586	1.9628	1.9670	1.9712	1.9754	1.9796	1.9838	1.9880
.71	1.9922	1.9964	2.0006	2.0048	2.0091	2.0133	2.0175	2.0217	2.0260	2.0302
.72	2.0344	2.0387	2.0429	2.0472	2.0514	2.0557	2.0599	2.0642	2.0684	2.0727
.73	2.0770	2,0812	2.0855	2.0898	2.0941	2.0983	2.1026	2.1069	2.1112	2.1155
.74	2.1198	2.1241	2.1284	2.1327	2.1370	2.1413	2.1456	2.1499	2.1543	2.1586
.75	2.1629	2.1672	2.1716	2.1759	2.1802	2.1846	2.1889	2.1932	2.1976	2.2019
.76	2.2063	2.2107	2.2150	2.2194	2.2237	2.2281	2.2325	2.2369	2.2412	2.2456
.77	2.2500	2.2544	2.2588	2.2632	2.2675	2.2719	2.2763	2.2807	2.2851	2.2896
.78	2.2940	2.2984	2.3028	2.3072	2.3116	2.3161	2.3205	2.3249	2.3293	2.3338
.79	2.3382	2.3427	2.3471	2.3515	2.3560	2.3604	2.3649	2.3694	2.3738	2.3783

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
0.80	2.3828	2.3872	2.3917	2.3962	2.4006	2.4051	2.4096	2.4141	2.4186	2.4231
.81	2.4276	2.4321	2.4366	2.4411	2.4456	2.4501	2.4546	2.4591	2.4636	2.4681
.82	2.4727	2.4772	2.4817	2.4862	2.4908	2.4953	2.4999	2.5044	2.5089	2.5135
.83	2.5180	2.5226	2.5271	2.5317	2.5363	2.5408	2.5454	2.5500	2.5545	2.5591
.84	2.5637	2.5683	2.5728	2.5774	2.5820	2.5866	2.5912	2.5958	2.6004	2.6050
.85	2.6096	2.6142	2.6188	2.6234	2.6280	2.6327	2.6373	2.6419	2.6465	2.6511
.86	2.6558	2.6604	2.6650	2.6697	2.6743	2.6790	2.6836	2.6883	2.6929	2.6976
.87	2.7022	2.7069	2.7116	2.7162	2.7209	2.7256	2.7302	2.7349	2.7396	2.7443
.88	2.7490	2.7536	2.7583	2.7630	2.7677	2.7724	2.7771	2.7818	2.7865	2.7912
.89	2.7959	2.8007	2.8054	2.8101	2.8148	2.8195	2.8243	2.8290	2.8337	2.8385
0.90	2.8432	2.8479	2.8527	2.8574	2.8622	2.8669	2.8717	2.8764	2.8812	2.8860
.91	2.8907	2.8955	2.9003	2.9050	2.9098	2.9146	2.9194	2.9241	2.9289	2.9337
.92	2.9385	2.9433	2.9481	2.9529	2.9577	2.9625	2.9673	2.9721	2.9769	2.9817
.93	2.9865	2.9914	2.9962	3.0010	3.0058	3.0107	3.0155	3.0203	3.0252	3.0300
.94	3.0348	3.0397	3.0445	3.0494	3.0542	3.0591	3.0639	3.0688	3.0737	3.0785
.95	3.0834	3.0883	3.0931	3.0980	3.1029	3.1078	3.1127	3.1175	3.1224	3.1273
.96	3.1322	3.1371	3.1420	3.1469	3.1518	3.1567	3.1616	3.1665	3.1714	3.1764
.97	3.1813	3.1862	3.1911	3.1960	3.2010	3.2059	3.2108	3.2158	3.2207	3.2257
.98	3.2306	3.2355	3.2405	3.2454	3.2504	3.2554	3.2603	3.2653	3.2702	3.2752
.99	3.2802	3.2851	3.2901	3.2951	3.3001	3.3051	3.3100	3.3150	3.3200	3.3250
1.00	3.3300	3.3350	3.3400	3.3450	3.3500	3.3550	3.3600	3.3650	3.3700	3.3751
.01	3.3801	3.3851	3.3901	3.3951	3.4002	3.4052	3.4102	3.4153	3.4203	3.4254
.02	3.4304	3.4354	3.4405	3.4455	3.4506	3.4557	3.4607	3.4658	3.4708	3.4759
.03	3.4810	3.4860	3.4911	3.4962	3.5013	3.5063	3.5114	3.5165	3.5216	3.5267
.04	3.5318	3.5369	3.5420	3.5471	3.5522	3.5573	3.5624	3.5675	3.5726	3.5777
.05	3.5828	3.5880	3.5931	3.5982	3.6033	3.6085	3.6136	3.6187	3.6239	3.6290
.06	3.6342	3.6393	3.6444	3.6496	3.6547	3.6599	3.6651	3.6702	3.6754	3.6805
.07	3.6857	3.6909	3.6960	3.7012	3.7064	3.7116	3.7167	3.7219	3.7271	3.7323
.08	3.7375	3.7427	3.7479	3.7531	3.7583	3.7635	3.7687	3.7739	3.7791	3.7843
.09	3.7895	3.7947	3.8000	3.8052	3.8104	3.8156	3.8209	3.8261	3.8313	3.8365
1.10	3.8418	3.8470	3.8523	3.8575	3.8628	3.8680	3.8733	3.8785	3.8838	3.8890
.11	3.8943	3.8996	3.9048	3.9101	3.9154	3.9206	3.9259	3.9312	3.9365	3.9418
.12	3.9470	3.9523	3.9576	3.9629	3.9682	3.9735	3.9788	3.9841	3.9894	3.9947
.13	4.0000	4.0053	4.0106	4.0160	4.0213	4.0266	4.0319	4.0372	4.0426	4.0479
.14	4.0532	4.0586	4.0639	4.0692	4.0746	4.0799	4.0853	4.0906	4.0960	4.1013
.15	4.1067	4.1120	4.1174	4.1228	4.1281	4.1335	4.1389	4.1442	4.1496	4.1550
.16	4.1604	4.1657	4.1711	4.1765	4.1819	4.1873	4.1927	4.1981	4.2035	4.2089
.17	4.2143	4.2197	4.2251	4.2305	4.2359	4.2413	4.2467	4.2522	4.2576	4.2630
.18	4.2684	4.2738	4.2793	4.2847	4.2901	4.2956	4.3010	4.3065	4.3119	4.3173
.19	4.3228	4.3282	4.3337	4.3392	4.3446	4.3501	4.3555	4.3610	4.3665	4.3719
1.20	4.3774	4.3829	4.3883	4.3938	4.3993	4.4048	4.4103	4.4158	4.4212	4.4267
.21	4.4322	4.4377	4.4432	4.4487	4.4542	4.4597	4.4652	4.4707	4.4763	4.4818
.22	4.4873	4.4928	4.4983	4.5038	4.5094	4.5149	4.5204	4.5260	4.5315	4.5370
.23	4.5426	4.5481	4.5537	4.5592	4.5647	4.5703	4.5759	4.5814	4.5870	4.5925
.24	4.5981	4.6036	4.6092	4.6148	4.6203	4.6259	4.6315	4.6371	4.6427	4.6482
.25	4.6538	4.6594	4.6650	4.6706	4.6762	4.6818	4.6874	4.6930	4.6986	4.7042
.26	4.7098	4.7154	4.7210	4.7266	4.7322	4.7378	4.7435	4.7491	4.7547	4.7603
.27	4.7660	4.7716	4.7772	4.7829	4.7885	4.7941	4.7998	4.8054	4.8111	4.8167
.28	4.8224	4.8280	4.8337	4.8393	4.8450	4.8506	4.8563	4.8620	4.8676	4.8733
.29	4.8790	4.8847	4.8903	4.8960	4.9017	4.9074	4.9131	4.9187	4.9244	4.9301

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
1.30	4.9358	4.9415	4.9472	4.9529	4.9586	4.9643	4.9700	4.9757	4.9814	4.9872
.31	4.9929	4.9986	5.0043	5.0100	5.0158	5.0215	5.0272	5.0330	5.0387	5.0444
.32	5.0502	5.0559	5.0616	5.0674	5.0731	5.0789	5.0846	5.0904	5.0961	5.1019
.33	5.1077	5.1134	5.1192	5.1249	5.1307	5.1365	5.1423	5.1480	5.1538	5.1596
.34	5.1654	5.1712	5.1769	5.1827	5.1885	5.1943	5.2001	5.0259	5.2117	5.2175
.35	5.2233	5.2291	5.2349	5.2407	5.2465	5.2523	5.2582	5.2640	5.2698	5.2756
.36	5.2814	5.2873	5.2931	5.2989	5.3048	5.3106	5.3164	5.3223	5.3281	5.3340
.37	5.3398	5.3456	5.3515	5.3573	5.3632	5.3691	5.3749	5.3808	5.3866	5.3925
.38	5.3964	5.4142	5.4101	5.4160	5.4219	5.4277	5.4336	5.4395	5.4454	5.4513
.39	5.4572	5.4630	5.4689	5.4748	5.4807	5.4866	5.4925	5.4984	5.5043	5.5102
1.40	5.5162	5.5221	5.5280	5.5339	5.5398	5.5457	5.5516	5.5576	5.5635	5.5694
.41	5.5754	5.5813	5.5872	5.5932	5.5991	5.6050	5.6110	5.6169	5.6229	5.6288
.42	5.6348	5.6407	5.6467	5.6526	5.6586	5.6646	5.6705	5.6765	5.6825	5.6884
.43	5.6944	5.7004	5.7064	5.7123	5.7183	5.7243	5.7303	5.7363	5.7423	5.7482
.44	5.7542	5.7602	5.7662	5.7722	5.7782	5,7842	5.7902	5.7962	5.8023	5.8083
.45	5.8143	5.8203	5.8263	5.8323	5.8384	5.8444	5.8504	5.8564	5.8625	5.8685
.46	5.8745	5.8806	5.8866	5.8926	5.8987	5.9047	5.9108	5.9168	5.9229	5.9289
.47	5.9350	5.9410	5.9471	5.9532	5.9592	5.9653	5.9714	5.9774	5.9835	5.9896
.48	5.9957	6.0017	6.0078	6.0139	6.0200	6.0261	6.0322	6.0382	6.0443	6.0504
.49	6.0565	6.0626	6.0687	6.0748	6.0809	6.0870	6.0931	6.0993	6.1054	6.1115
1.50	6.1176	6.1237	6.1298	6.1360	6.1421	6.1482	6.1543	6.1605	6.1666	6.1727
.51	6.1789	6.1850	6.1912	6.1973	6.2034	6.2096	6.2157	6.2219	6.2280	6.2342
.52	6.2404	6.2465	6.2527	6.2588	6.2650	6.2712	6.2773	6.2835	6.2897	6.2959
.53	6.3020	6.3082	6.3144	6.3206	6.3268	6.3330	6.3391	6.3453	6.3515	6.3577
.54	6.3639	6.3701	6.3763	6.3825	6.3887	6.3949	6.4012	6.4074	6.4136	6.4198
.55	6.4260	6.4322	6.4385	6.4447	6.4509	6.4571	6.4634	6.4696	6.4758	6.4821
.56	6.4883	6.4945	6.5008	6.5070	6.5133	6.5195	6.5258	6.5320	6.5383	6.5445
.57	6.5508	6.5570	6.5633	6.5696	6.5758	6.5821	6.5884	6.5946	6.6009	6.6072
.58	6.6135	6.6198	6.6260	6.6323	6.6386	6.6449	6.6512	6.6575	6.6638	6.6701
.59	6.6764	6.6827	6.6890	6.6953	6.7016	6.7079	6.7142	6.7205	6.7268	6.7331
1.60	6.7394	6.7458	6.7521	6.7584	6.7647	6.7711	6.7774	6.7837	6.7901	6.7964
.61	6.8027	6.8091	6.8154	6.8217	6.8281	6.8344	6.8408	6.8471	6.8535	6.8598
.62	6.8662	6.8726	6.8789	6.8853	6.8916	6.8980	6.9044	6.9108	6.9171	6.9235
.63	6.9299	6.9363	6.9426	6.9490	6.9554	6.9618	6.9682	6.9746	6.9810	6.9874
.64	6.9937	7.0001	7.0065	7.0129	7.0193	7.0258	7.0322	7.0386	7.0450	7.0514
.65	7.0578	7.0642	7.0706	7.0771	7.0835	7.0899	7.0963	7.1028	7.1092	7.1156
.66	7.1221	7.1285	7.1349	7.1414	7.1478	7.1543	7.1607	7.1672	7.1736	7.1801
.67	7.1865	7.1930	7.1994	7.2059	7.2124	7.2188	7.2253	7.2318	7.2382	7.2447
.68	7.2512	7.2576	7.2641	7.2706	7.2771	7.2836	7.2901	7.2965	7.3030	7.3095
.69	7.3160	7.3225	7.3290	7.3355	7.3420	7.3485	7.3550	7.3615	7.3680	7.3745
1.70	7.3810	7.3876	7.3941	7.4006	7.4071	7.4136	7.4201	7.4267	7.4332	7.4397
.71	7.4463	7.4528	7.4593	7.4659	7.4724	7.4789	7.4855	7.4920	7.4986	7.5051
.72	7.5117	7.5182	7.5248	7.5313	7.5379	7.5445	7.5510	7.5576	7.5641	7.5707
.73	7.5773	7.5839	7.5904	7.5970	7.6036	7.6102	7.6167	7.6233	7.6299	7.6365
.74	7.6431	7.6497	7.6563	7.6628	7.6694	7.6760	7.6826	7.6892	7.6958	7.7024
.75	7.7091	7.7157	7.7223	7.7289	7.7355	7.7421	7.7487	7.7554	7.7620	7.7686
.76	7.7752	7.7819	7.7885	7.7951	7.8018	7.8084	7.8150	7.8217	7.8283	7.8349
.77	7.8416	7.8482	7.8549	7.8615	7.8682	7.8748	7.8815	7.8882	7.8948	7.9015
.78	7.9081	7.9148	7.9215	7.9281	7.9348	7.9415	7.9482	7.9548	7.9615	7.9682
.79	7.9749	7.9816	7.9882	7.9949	8.0016	8.0083	8.0150	8.0217	8.0284	8.0351

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
1.80	8.0418	8.0485	8.0552	8.0619	8.0686	8.0753	8.0820	8.0888	8.0965	8.1022
.81	8.1089	8.1156	8.1223	8.1291	8.1358	8.1425	8.1493	8.1560	8.1627	8.1695
.82	8.1762	8.1829	8.1897	8.1964	8.2032	8.2099	8.2167	8.2234	8.2302	8.2369
.83	8.2437	8.2504	8.2572	8.2640	8.2707	8.2775	8.2842	8.2910	8.2978	8.3046
.84	8.3113	8.3181	8.3249	8.3317	8.3385	8.3452	8.3520	8.3588	8.3656	8.3724
.85	8.3792	8.3860	8.3928	8.3996	8.4064	8.4132	8.4200	8.4268	8.4336	8.4404
.86	8.4472	8.4540	8.4608	8.4677	8.4745	8.4813	8.4881	8.4949	8.5018	8.5086
.87	8.5154	8.5223	8.5291	8.5359	8.5428	8.5496	8.5564	8.5633	8.5701	8.5770
.88	8.5838	8.5907	8.5975	8.6044	8.6112	8.6181	8.6250	8.6318	8.6387	8.6456
.89	8.6524	8.6593	8.6661	8.6730	8.6799	8.6868	8.6936	8.7005	8.7074	8.7143
1.90	8.7212	8.7281	8.7349	8.7418	8.7487	8.7556	8.7625	8.7694	8.7763	8.7882
.91	8.7901	8.7970	8.8039	8.8108	8.8177	8.8246	8.8316	8.8385	8.8454	8.8523
.92	8.8592	8.8662	8.8731	8.8800	8.8869	8.8939	8.9008	8.9077	8.9147	8.9216
.93	8.9285	8.9355	8.9424	8.9494	8.9563	8.9633	8.9702	8.9772	8.9841	8.9911
.94	8.9980	9.0050	9.0119	9.0189	9.0259	9.0328	9.0398	9.0468	9.0537	9.0607
.95	9.0677	9.0747	9.0816	9.0886	9.0956	9.1026	9.1096	9.1165	9.1235	9.1305
.96	9.1375	9.1445	9.1515	9.1585	9.1655	9.1725	9.1795	9.1865	9.1935	9.2005
.97	9.2075	9.2145	9.2216	9.2286	9.2356	9.2426	9.2496	9.2567	9.2637	9.2707
.98	9.2777	9.2848	9.2918	9.2988	9.3059	9.3129	9.3199	9.3270	9.3340	9.3411
.99	9.3481	9.3552	9.3622	9.3693	9.3763	9.3834	9.3904	9.3975	9.4045	9.4116
2.00	9.4187	9.4257	9.4328	9.4399	9.4469	9.4540	9.4611	9.4682	9.4752	9.4823
.01	9.4894	9.4965	9.5036	9.5106	9.5177	9.5248	9.5319	9.5390	9.5461	9.5532
.02	9.5603	9.5674	9.5745	9.5816	9.5887	9.5958	9.6029	9.6100	9.6171	9.6243
.03	9.6314	9.6385	9.6456	9.6527	9.6599	9.6670	9.6741	9.6812	9.6884	9.6955
.04	9.7026	9.7098	9.7169	9.7240	9.7312	9.7383	9.7455	9.7526	9.7598	9.7669
.05	9.7741	9.7812	9.7884	9.7955	9.8027	9.8098	9.8170	9.8242	9.8313	9.8385
.06	9.8457	9.8528	9.8600	9.8672	9.8744	9.8815	9.8887	9.8959	9.9031	9.9103
.07	9.9174	9.9246	9.9318	9.9390	9.9462	9.9534	9.9606	9.9678	9.9750	9.9822
.08	9.9894	9.9966	10.004	10.011	10.018	10.025	10.033	10.040	10.047	10.054
.09	10.062	10.069	10.076	10.083	10.090	10.098	10.105	10.112	10.119	10.127
2.10	10.134	10.141	10.148	10.156	10.163	10.170	10.177	10.185	10.192	10.189
.11	10.206	10.214	10.221	10.228	10.235	10.248	10.250	10.257	10.264	10.272
.12	10.279	10.286	10.293	10.301	10.308	10.315	10.323	10.330	10.337	10.344
.13	10.352	10.359	10.366	10.374	10.381	10.388	10.396	10.403	10.410	10.417
.14	10.425	10.432	10.439	10.447	10.454	10.461	10.469	10.476	10.483	10.491
.15	10.498	10.505	10.513	10.520	10.527	10.535	10.542	10.549	10.557	10.564
.16	10.571	10.579	10.586	10.593	10.601	10.608	10.615	10.623	10.630	10.637
.17	10.645	10.652	10.659	10.667	10.674	10.682	10.689	10.696	10.704	10.711
.18	10.718	10.726	10.733	10.741	10.748	10.755	10.763	10.770	10.777	10.785
.19	10.792	10.800	10.807	10.814	10.822	10.829	10.837	10.844	10.851	10.859
2.20	10.866	10.874	10.881	10.888	10.896	10.903	10.911	10.918	10.926	10.933
.21	10.940	10.948	10,955	10.963	10.970	10.978	10.985	10.992	11.000	11.007
.22	11.015	11.022	11.030	11.037	11.045	11.052	11.059	11.067	11.074	11.082
.23	11.089	11.097	11.104	11.112	11.119	11.127	11.134	11.141	11.149	11.156
.24	11.164	11.171	11.179	11.186	11.194	11.201	11.209	11.216	11.224	11.231
.25	11.239	11.246	11.254	11.261	11.269	11.276	11.284	11.291	11.299	11.306
.26	11.314	11.321	11.329	11.336	11.344	11.351	11.359	11.366	11.374	11.381
.27	11.389	11.396	11.404	11.412	11.419	11.427	11.434	11.442	11.449	11.457
.28	11.464	11.472	11.479	11.487	11.494	11.502	11.510	11.517	11.525	11.532
.29	11.540	11.547	11.555	11.562	11.570	11.578	11.585	11.598	11.600	11.608

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
2.30	11.615	11.623	11.631	11.638	11.646	11.653	11.661	11.669	11.676	11.684
.31	11.691	11.699	11.706	11.714	11.722	11.729	11.737	11.744	11.752	11.760
.32	11.767	11.775	11.783	11.790	11.798	11.805	11.813	11.821	11.828	11.836
.33	11.843	11.851	11.859	11.866	11.874	11.882	11.889	11.897	11.904	11.912
.34	11.920	11.927	11.935	11.943	11.950	11.958	11.966	11.973	11.981	11.989
.35	11.996	12.004	12.012	12.019	12.027	12.035	12.042	12.050	12.058	12.065
.36	12.073	12.081	12.088	12.096	12.104	12.111	12.119	12.127	12.134	12.142
.37	12.150	12.157	12.165	12.173	12.181	12.188	12.196	12.204	12.211	12.219
.38	12.227	12.234	12.242	12.250	12.258	12.265	12.273	12.281	12.288	12.296
.39	12.304	12.312	12.319	12.327	12.335	12.342	12.350	12.358	12.366	12.373
2.40	12.381	12.389	12.397	12.404	12.412	12.420	12.428	12.435	12.443	12.451
.41	12.459	12.466	12.474	12.482	12.490	12.497	12.505	12.513	12.521	12.528
.42	12.536	12.544	12.552	12.560	12.567	12.575	12.583	12.591	12.598	12.606
.43	12.614	12.622	12.630	12.637	12.645	12.653	12.661	12.669	12.676	12.684
.44	12.692	12.700	12.708	12.715	12.723	12.731	12.739	12.747	12.754	12.672
.45	12.770	12.778	12.786	12.794	12.801	12.809	12.817	12.825	12.833	12.840
.46	12.848	12.856	12.864	12.872	12.880	12.888	12.895	12.903	12.911	12.919
.47	12.927	12.935	12.942	12.950	12.958	12.966	12.974	12.982	12.990	12.997
.48	13.005	13.013	13.021	13.029	13.037	13.045	13.053	13.060	13.068	13.076
.49	13.084	13.092	13.100	13.108	13.116	13.124	13.131	13.139	13.147	13.155
2.50	13.163	13.171	13.179	13.187	13.195	13.202	13.210	13.218	13.226	13.234
.51	13.242	13.250	13.258	13.266	13.274	13.282	13.290	13.297	13.305	13.313
.52	13.321	13.329	13.337	13.345	13.353	13.361	13.369	13.377	13.385	13.393
.53	13.401	13.409	13.417	13.424	13.432	13.440	13.448	13.456	13.464	13.472
.54	13.480	13.488	13.496	13.504	13.512	13.520	13.528	13.536	13.544	13.552
.55	13.560	13.568	13.576	13.584	13.592	13.600	13.608	13.616	13.624	13.632
.56	13.640	13.648	13.656	13.664	13.672	13.680	13.688	13.696	13.704	13.712
.57	13.720	13.728	13.736	13.744	13.752	13.760	13.768	13.776	13.784	13.792
.58	13.800	13.808	13.816	13.824	13.832	13.840	13.848	13.856	13.864	13.872
.59	13.880	13.888	13.896	13.904	13.912	13.920	13.928	13.936	13.944	13.953
2.60	13.961	13.969	13.977	13.985	13.993	14.001	14.009	14.017	14.025	14.033
.61	14.041	14.049	14.057	14.065	14.074	14.082	14.090	14.098	14.106	14.114
.62	14.122	14.130	14.138	14.146	14.154	14.162	14.171	14.179	14.187	14.195
.63	14.203	14.211	14.219	14.227	14.235	14.243	14.252	14.260	14.268	14.276
.64	14.284	14.292	14.300	14.308	14.316	14.325	14.333	14.341	14.349	14.357
.65	14.356	14.373	14.382	14.390	14.398	14.406	14.414	14.422	14.430	14.438
.66	14.447	14.455	14.463	14.471	14.479	14.487	14.496	14.504	14.512	14.520
.67	14.528	14.536	14.545	14.553	14.561	14.569	14.577	14.585	14.594	14.602
.68	14.610	14.618	14.626	14.634	14.643	14.651	14.659	14.667	14.675	14.684
.69	14.692	14.700	14.708	14.716	14.725	14.733	14.741	14.749	14.757	14.766
2.70	14.774	14.782	14.790	14.798	14.807	14.815	14.823	14.831	14.839	14.848
.71	14.856	14.864	14.872	14.881	14.889	14.897	14.905	14.913	14.922	14.930
.72	14.938	14.946	14.955	14.963	14.971	14.979	14.988	14.996	15.004	15.012
.73	15.021	15.029	15.037	15.045	15.054	15.062	15.070	15.078	15.087	15.095
.74	15.103	15.112	15.120	15.128	15.136	15.145	15.153	15.161	15.169	15.178
.75	15.186	15.194	15.203	15.211	15.219	15.227	15.236	15.244	15.252	15.261
.76	15.269	15.277	15.285	15.294	15.302	15.310	15.319	15.327	15.335	15.344
.77	15.352	15.360	15.369	15.377	15.385	15.394	15.402	15.410	15.419	15.427
.78	15.435	15.443	15.452	15.460	15.468	15.477	15.485	15.494	15.502	15.510
.79	15.519	15.527	15.535	15.544	15.552	15.560	15.569	15.577	15.585	15.594

TABLE 3.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*, feet.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
2.80	15.602	15.610	15.619	15.627	15.635	15.644	15.652	15.661	15.669	15.677
.81	15.686	15.694	15.702	15.711	15.719	15.728	15.736	15.744	15.753	15.761
.82	15.769	15.778	15.786	15.795	15.803	15.811	15.820	15.828	15.837	15.845
.83	15.853	15.862	15.870	15.879	15.887	15.895	15,904	15.912	15.921	15.929
.84	15.938	15.946	15.954	15.963	15.971	15.980	15.988	15.997	16.005	16.013
.85	16.022	16.030	16.039	16.047	16.056	16.064	16.072	16.081	16.089	16.098
.86	16.106	16.115	16.123	16.132	16.140	16.148	16.157	16.165	16.174	16.182
.87	16.191	16.199	16.208	16.216	16.225	16.233	16.242	16.250	16.258	16.267
.88	16.275	16.284	16.292	16.301	16.309	16.318	16.326	16.335	16.343	16.352
.89	16.360	16.369	16.377	16.386	16.394	16.403	16.411	16.420	16.428	16.437
2.90	16.445	16.454	16.462	16.471	16.479	16.488	16.496	16.505	16.513	16.522
.91	16.530	16.539	16.547	16.556	16.565	16.573	16.582	16.590	16.599	16.607
.92	16.616	16.624	16.633	16.641	16.650	16.658	16.667	16.675	16.684	16.693
.93	16.701	16.710	16.718	16.727	16.735	16.744	16.752	16.761	16.770	16.778
.94	16.787	16.795	16.804	16.812	16.821	16.830	16.838	16.847	16.855	16.864
.95	16.872	16.881	16.890	16.898	16.907	16.915	16.924	16.932	16.941	16.950
.96	16.958	16.967	16.975	16.984	16.993	17.001	17.010	17.018	17.027	17.036
.97	17.044	17.053	17.062	17.070	17.079	17.087	17.096	17.105	17.113	17.122
.98	17.130	17.139	17.148	17.156	17.165	17.174	17.182	17.191	17.199	17.208
.99	17.217	17.225	17.234	17.243	17.251	17.260	17.269	17.277	17.286	17.295
3.00	17.3033	....	....	....	....	....	....	....	....	....

TABLE 4.—*Discharge over a thin-edged weir per foot of crest.*

Head *H*, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.0000	0.0033	0.0094	0.0173	0.0266	0.0372	0.0489	0.0617	0.0753	0.0899
.1	.1053	.1215	.1384	.1561	.1744	.1935	.2131	.2334	.2543	.2758
.2	.2978	.3205	.3436	.3673	.3915	.4162	.4415	.4672	.4934	.5200
.3	.5472	.5748	.6028	.6313	.6602	.6895	.7193	.7495	.7800	.8110
.4	.8424	.8742	.9064	.9390	.9719	1.0052	1.0389	1.0730	1.1074	1.1422
.5	1.1773	1.2128	1.2487	1.2849	1.3214	1.3583	1.3955	1.4330	1.4709	1.5091
.6	1.5476	1.5865	1.6257	1.6652	1.7050	1.7451	1.7855	1.8262	1.8673	1.9086
.7	1.9503	1.9922	2.0344	2.0770	2.1198	2.1629	2.2063	2.2500	2.2940	2.3382
.8	2.3828	2.4276	2.4727	2.5180	2.5637	2.6096	2.6558	2.7022	2.7490	2.7959
.9	2.8432	2.8907	2.9385	2.9865	3.0348	3.0834	3.1322	3.1813	3.2306	3.2802
1.0	3.3300	3.3801	3.4304	3.4810	3.5318	3.5828	3.6342	3.6857	3.7375	3.7895
1.1	3.8418	3.8943	3.9470	4.0000	4.0532	4.1067	4.1604	4.2143	4.2384	4.3228
1.2	4.3774	4.4322	4.4873	4.5426	4.5981	4.6538	4.7098	4.7660	4.8224	4.8790
1.3	4.9358	4.9929	5.0502	5.1077	5.1654	5.2233	5.2814	5.3398	5.3984	5.4572
1.4	5.5162	5.5754	5.6348	5.6944	5.7542	5.8143	5.8745	5.9350	5.9957	6.0565
1.5	6.1176	6.1789	6.2404	6.3020	6.3638	6.4260	6.4883	6.5508	6.6135	6.6764
1.6	6.7394	6.8027	6.8662	6.9299	6.9937	7.0578	7.1221	7.1865	7.2512	7.3160
1.7	7.3810	7.4463	7.5117	7.5773	7.6431	7.7091	7.7752	7.8416	7.9081	7.9749
1.8	8.0418	8.1689	8.1762	8.2437	8.3118	8.3792	8.4472	8.5154	8.5838	8.6524
1.9	8.7212	8.7901	8.8592	8.9285	8.9980	9.0677	9.1375	9.2075	8.2777	9.3481

TABLE 4.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head *H*. feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
2.0	9.4187	9.4894	9.5603	9.6314	9.7026	9.7741	9.8457	9.9174	9.9894	10.0620
2.1	10.1340	10.2060	10.2790	10.3520	10.4250	10.4980	10.5710	10.6450	10.7180	10.7920
2.2	10.8660	10.9400	11.0150	11.0890	11.1640	11.2390	11.3140	11.3890	11.4640	11.5400
2.3	11.6150	11.6910	11.7670	11.8430	11.9200	11.9960	12.0730	12.1500	12.2270	12.3040
2.4	12.3810	12.4590	12.5360	12.6140	12.6920	12.7700	12.8480	12.9270	13.0050	13.0840
2.5	13.1630	13.2480	13.3210	13.4010	13.4800	13.5600	12.6400	13.7200	13.8000	13.8800
2.6	13.9610	14.0410	14.1220	14.2030	14.2840	14.3650	14.4470	14.5280	14.6100	14.6920
2.7	14.7740	14.8560	14.9380	15.0210	15.1030	15.1860	15.2690	15.3520	15.4350	15.5190
2.8	15.6020	15.6860	15.7690	15.8530	15.9380	16.0220	16.1060	16.1910	16.2750	16.3600
2.9	16.4450	16.5300	16.6160	16.7010	16.7870	16.8720	16.9580	17.0440	17.1300	17.2170
3.0	17.3033	17.3899	17.4698	17.5634	17.6503	17.7376	17.8248	17.9124	18.0000	18.0876
3.1	18.1754	18.2634	18.3516	18.4399	18.5285	18.6170	18.7056	18.7945	18.8838	18.9727
3.2	19.0619	19.1515	19.2410	19.3307	19.4206	19.5105	19.6007	19.6910	19.7812	19.8718
3.3	19.9624	20.0538	20.1442	20.2354	20.3267	20.4179	20.5095	20.6011	20.6930	20.7849
3.4	20.8777	20.9690	21.0618	21.1538	21.2464	21.3390	21.4319	21.5248	21.6180	21.7118
3.5	21.8045	21.8980	21.9917	22.0856	22.1795	22.2734	22.3677	22.4618	22.5564	22.6510
3.6	22.7456	22.8405	22.9354	23.0306	23.1259	23.2211	23.3167	23.4122	23.5081	23.6040
3.7	23.6999	23.7962	23.8924	23.9887	24.0852	24.1818	24.2787	24.3756	24.4728	24.5697
3.8	24.6673	24.7645	24.8621	24.9600	25.0576	25.1555	25.2537	25.3520	25.4502	25.5488
3.9	25.6473	25.7459	25.8748	25.9437	26.0429	26.1422	26.2414	26.3410	26.4405	26.5401
4.0	26.6400	26.7399	26.8401	26.9404	27.0406	27.1412	27.2417	27.3423	27.4432	27.5411
4.1	27.6458	27.7466	27.8478	27.9494	28.0509	28.1525	28.2544	28.3563	28.4582	28.5604
4.2	28.6626	28.7652	28.8678	28.9703	29.0732	29.1761	29.2790	29.3823	29.4855	29.5890
4.3	29.6926	29.7962	29.9001	30.0040	30.1079	30.2118	30.3163	30.4205	30.5251	30.6297
4.4	30.7342	30.8391	30.9440	31.0493	31.1545	31.2597	31.3649	31.4705	31.5764	31.6820
4.5	31.7878	31.8941	32.0008	32.1065	32.2128	32.3193	32.4259	32.5324	32.6393	32.7462
4.6	32.8534	32.9607	33.0679	33.1755	33.2830	33.3906	33.4985	33.6064	33.7143	33.8225
4.7	33.9307	34.0373	34.1475	34.2560	34.3646	34.4735	34.5824	34.6913	34.8005	34.9097
4.8	35.0193	35.1288	35.2354	35.3480	35.4578	35.5677	35.6780	35.7882	35.8984	36.0086
4.9	36.1182	36.2297	36.3406	36.4515	36.5624	36.6736	36.7845	36.8961	37.0073	37.1188
5.0	37.2304	37.3423	37.4542	37.5661	37.6783	37.7905	37.9027	38.0153	38.1275	38.2404
5.1	38.3529	38.4658	28.5787	38.6919	38.8052	38.9184	39.0319	39.1455	39.2591	39.3726
5.2	39.4865	39.6004	39.7146	39.8288	39.9430	40.0576	40.1718	40.2867	40.4012	40.5161
5.3	40.6310	40.7462	40.5281	40.9766	41.0919	41.2074	41.3230	41.4386	41.5544	41.6703
5.4	41.7866	41.9024	42.0186	42.1352	42.2517	42.3683	42.4848	42.6017	42.7186	42.8355
5.5	42.9523	43.0700	43.1871	43.3043	43.4219	42.5394	43.6573	43.7752	43.8931	44.0109
5.6	44.1292	44.2474	44.3659	44.4845	44.6030	44.7216	44.8404	44.9593	45.0782	45.1974
5.7	45.3166	45.4359	45.5554	45.6746	45.7945	45.9140	46.0339	46.1538	46.2740	46.3939
5.8	46.5141	46.6347	46.7552	46.8757	46.9963	47.1172	47.2380	47.3589	47.4798	47.6010
5.9	47.7226	47.8438	47.9653	48.0869	48.2084	48.3303	48.4522	48.5744	48.6963	48.8185
6.0	48.9407	49.0632	49.1858	49.3083	48.4312	49.5537	49.6766	49.7999	49.9230	50.0462
6.1	50.1694	50.2930	50.4162	50.5401	50.6637	50.7875	50.9114	51.0356	51.1595	51.2837
6.2	51.4082	51.5324	51.6570	51.7818	51.9034	52.0318	52.1531	52.2818	52.4062	52.5314
6.3	52.6570	52.7822	52.9077	53.0336	53.1591	53.2850	53.4109	53.5371	53.6630	53.7892
6.4	53.9157	54.0419	54.1684	54.2950	54.4219	54.5487	54.6756	54.8025	54.9297	55.0569
6.5	55.1832	55.3116	55.4392	55.5667	55.6943	55.8221	55.9500	56.0779	56.2061	56.3343
6.6	56.4625	56.5910	56.7192	56.8478	56.9766	57.1055	57.2340	57.3233	57.4921	57.6213
6.7	57.7505	57.8801	58.0093	58.1388	58.2687	58.3982	58.5281	58.6580	58.7882	58.9180
6.8	59.0482	59.1788	59.3090	59.4428	59.5700	59.7009	59.8314	59.9623	60.0935	60.2244
6.9	60.3556	60.4868	60.6183	60.7499	60.8814	61.0129	61.1445	61.2763	61.4082	61.5404

TABLE 4.—*Discharge over a thin-edged weir per foot of crest*—Continued.

Head H, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
7.0	61.6736	61.8048	61.9370	62.0692	62.2017	62.3343	62.4671	62.6000	62.7329	62.8657
7.1	62.9986	63.1318	63.2650	63.3992	63.5317	63.6653	63.7991	63.9327	64.0665	64.2004
7.2	64.3343	64.4685	64.6027	64.7369	64.8711	65.0056	65.1268	65.2750	65.4096	65.5444
7.3	65.6793	65.8145	65.9493	66.0845	66.2197	66.3552	66.4908	66.6263	66.7618	66.8977
7.4	67.0336	67.1694	67.3053	67.4415	67.5777	67.7139	67.8504	67.9869	68.1235	68.2600
7.5	68.3969	68.5337	68.6706	68.8078	68.9447	69.0818	69.2794	69.3566	69.4941	69.6316
7.6	69.7695	69.9070	70.0449	70.1827	70.3209	70.4591	70.5973	70.7355	70.8737	71.0123
7.7	71.1508	71.2896	71.4282	71.5670	71.7059	71.8451	71.9843	72.1235	72.2627	72.4743
7.8	72.5414	72.6809	72.8208	72.9603	73.1002	73.2400	73.3802	73.5201	73.6603	73.8005
7.9	73.9410	74.0815	74.2220	74.3626	74.5031	74.6439	74.7848	74.9260	75.0669	75.2081
8.0	75.3492	75.4908	75.6320	75.7735	75.9150	76.0569	76.1987	76.3406	76.4824	76.6243
8.1	76.7665	76.9087	77.0509	77.1934	77.3360	77.4784	77.6210	77.7638	77.9067	78.0496
8.2	78.1924	78.3356	78.4788	78.6220	78.7655	78.9087	79.0522	79.1957	79.3396	79.4834
8.3	79.6278	79.7711	79.9153	80.0592	80.2034	80.3479	80.4921	80.6366	80.7811	80.9260
8.4	81.0705	81.2154	81.3602	81.5054	81.6503	81.7955	81.9406	82.0862	82.2314	82.3769
8.5	82.5224	82.6682	82.8141	82.9600	83.1058	83.2517	83.3979	83.5440	83.6902	83.8367
8.6	83.9833	84.1298	84.2763	84.4228	84.5697	84.7165	84.8634	85.0106	85.1578	85.3049
8.7	85.4521	85.5996	85.7472	85.8947	86.0455	86.1897	86.3376	86.4854	86.6336	86.7815
8.8	86.9297	87.0778	87.2264	87.3745	87.5231	87.6716	87.8204	87.9689	88.1178	88.2666
8.9	88.4192	88.5647	88.7139	88.8630	89.0126	89.1617	89.3113	89.4608	89.6103	89.7602
9.0	89.9100	90.0599	90.2064	90.3599	90.5101	90.6602	90.4778	90.9609	91.1115	91.2620
9.1	91.4125	91.5633	91.7142	91.8650	92.0159	92.1671	92.3183	92.4694	92.6206	92.7721
9.2	92.9237	93.0782	93.2267	93.3785	93.5304	93.6822	93.8341	93.9863	94.1384	94.2906
9.3	94.4428	94.5950	94.7475	94.9000	95.0529	95.2054	95.3582	95.5111	95.6639	95.8171
9.4	95.9703	96.1234	96.2766	96.4298	96.5833	96.7368	96.8903	97.0442	97.1977	97.3516
9.5	97.5057	97.6596	97.8138	97.9679	98.1021	98.2763	98.4308	98.5853	98.7398	98.8943
9.6	99.0492	99.2040	99.3589	99.5141	99.6689	99.8241	99.9793	100.1344	100.2899	100.4455
9.7	100.6010	100.7565	100.9123	101.0678	101.2237	101.3799	101.5857	101.6919	101.8481	102.0042
9.8	102.1607	102.3169	102.4734	102.6299	102.7868	102.9433	103.1001	103.2570	103.4141	103.5710
9.9	103.7282	103.8853	104.0429	104.2000	104.3575	104.5121	104.6726	104.8304	104.9882	105.1461
10.0	105.3039	105.4618	105.6199	105.7781	105.9363	106.0945	106.2530	106.4115	106.5700	106.7285

When applied to a weir with N end contractions, the measured crest length L' should be reduced by the formula

$$L = L' - 0.1\,NH$$

When applied to a weir having appreciable velocity of approach, the measured head should be corrected by the correction formula of Francis (see p. 15), or by one of the simpler approximate equivalents; or the correction may be applied as a percentage to the discharge, by the use of Table 2.

Table 3, taken from Lowell Hydraulic Experiments, by James B. Francis, gives the discharge for heads from zero to 3 feet, advancing by thousandths.

Table 4 is original and gives the discharge for heads from zero to 10.09 feet, advancing by hundredths.

By increasing the quantities from either table 1 per cent, the discharge by the Cippoletti formula will be obtained,

$$Q=3.36\tfrac{2}{3}\ LH^{\frac{3}{2}}.$$

In calculating discharge by this formula, the head should be corrected for velocity of approach by the formula

$$H=D+1.5h.$$

TABLES 5 AND 6.—THREE-HALVES POWERS.

These tables of three-halves powers (cubes of the square roots) were prepared by the writer to facilitate the calculation of discharge over weirs of various forms, by the use of coefficients taken from the diagrams that accompany this paper and the base formula

$$Q=CLH^{\frac{3}{2}}.$$

TABLE 5.—*Three-halves powers for numbers 0 to 1.49.*

Numbers.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009	.010
0.00	0.0000	0.0001	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010
.01	.0010	.00118	.00136	.00154	.00172	.00190	.00208	.00236	.00244	.00262	.0028
.02	.0028	.00304	.00328	.00352	.00376	.00400	.00424	.00448	.00472	.00496	.0052
.03	.0052	.00548	.00576	.00604	.00632	.00660	.00688	.00716	.00744	.00772	.0080
.04	.0080	.00832	.00864	.00896	.00928	.00960	.00992	.01024	.01056	.01088	.0112
.05	.0112	.01155	.01190	.01225	.01260	.01295	.01330	.01365	.01400	.01435	.0147
.06	.0147	.01508	.01546	.01584	.01622	.01660	.01698	.01736	.01774	.01812	.0185
.07	.0185	.01891	.01932	.01973	.02014	.02055	.02096	.02137	.02178	.02219	.0226
.08	.0226	.02304	.02348	.02392	.02436	.02480	.02524	.02568	.02612	.02656	.0270
.09	.0270	.02746	.02792	.02838	.02884	.02930	.02976	.03022	.03068	.03114	.0316
0.10	0.0316	0.03209	0.03258	0.03307	0.03356	0.03405	0.03454	0.03503	00355.2	0.03601	0.0365
.11	.0365	.03701	.03752	.03803	.03854	.03905	.03956	.04007	.04058	.04109	.0416
.12	.0416	.04213	.04266	.04319	.04372	.04425	.04478	.04531	.04584	.04637	.0469
.13	.0469	.04745	.04800	.04855	.04910	.04965	.05020	.05075	.05130	.05185	.0524
.14	.0524	.05297	.05354	.05411	.05468	.05525	.05582	.05639	.05696	.05753	.0581
.15	.0581	.05869	.05928	.05987	.06046	.06105	.06164	.06223	.06282	.06341	.0640
.16	.0640	.06451	.06522	.06583	.06644	.06705	.06766	.06827	.06888	.06949	.0701
.17	.0701	.07073	.07136	.07199	.07262	.07325	.07388	.07451	.07514	.07577	.0764
.18	.0764	.07704	.07768	.07832	.07896	.07960	.08024	.08088	.08152	.08216	.0828
.19	.0828	.08346	.08412	.08478	.08544	.08610	.08676	.08742	.08808	.08874	.0894
0.20	0.0894	0.09008	0.09076	0.09144	0.09212	0.09280	0.09348	0.09416	0.09484	009552	0.0962
.21	.0962	.09690	.09760	.09830	.09900	.09970	.10040	.10110	.1018	.1025	.1032
.22	.1032	.10391	.10462	.10533	.10604	.10675	.10746	.10817	.10888	.10959	.1103
.23	.1103	.11103	.11176	.11249	.11322	.11395	.11468	.11541	.11614	.11687	.1176
.24	.1176	.11834	.11908	.11982	.12506	.12130	.12204	.12278	.12352	.12426	.1250
.25	.1250	.12576	.12652	.12728	.12804	.12880	.12956	.13032	.13108	.13184	.1326
.26	.1326	.13337	.13414	.13491	.13568	.13645	.13722	.13799	.13876	.13953	.1403
.27	.1403	.14109	.14188	.14267	.14346	.14425	.14504	.14583	.14662	.14741	.1482
.28	.1482	.1490	.1498	.1506	.1514	.1522	1530	.1538	.1546	.1554	.1562
.29	.1562	.15701	.15782	.15863	.15944	.16025	.16106	.16187	.16268	.16349	.1643

TABLE 5.—*Three-halves powers for numbers 0 to 1.49*—Continued.

Numbers.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009	.010
0.30	0.1643	0.16513	0.16596	0.16679	0.16762	0.16845	0.16928	0.17011	0.17094	0.17177	0.1726
.31	.1726	.17344	.17428	.17512	.17596	.17680	.17764	.17848	.17932	.18016	.1810
.32	.1810	.18186	.18272	.18358	.18444	.18530	.18616	.18702	.18788	.18874	.1896
.33	.1896	.19047	.19134	.19221	.19308	.19395	.19482	.19569	.19656	.19743	.1983
.34	.1983	.19918	.20006	.20094	.20182	.20270	.20358	.20446	.20534	.20622	.2071
.35	.2071	.20799	.20888	.20977	.21066	.21155	.21244	.21333	.21422	.21511	.2160
.36	.2160	.21691	.21782	.21873	.21964	.22055	.22146	.22237	.22328	.22419	.2251
.37	.2251	.22601	.22692	.22783	.22874	.22965	.23056	.2317	.23238	.23329	.2342
.38	.2342	.23514	.23608	.23702	.23796	.23890	.23984	.24078	.24172	.24266	.2436
.39	.2436	.24454	.24548	.24642	.24736	.24830	.24924	.25018	.25112	.25206	.2530
0.40	0.2530	0.25395	0.25490	0.25585	0.25680	0.25775	0.25870	0.25965	0.26060	0.26155	0.2625
.41	.2625	.26347	.26444	.26541	.26638	.26735	.26832	.26929	.27026	.27123	.2722
.42	.2722	.27318	.27416	.27514	.27612	.27710	.27808	.27906	.28004	.28102	.2820
.43	.2820	.28299	.28397	.28497	.28596	.28695	.28794	.28893	.28992	.29091	.2919
.44	.2919	.2929	.2939	.2949	.2959	.2969	.2979	.2989	.2999	.3009	.3019
.45	.3019	.30291	.30392	.30493	.30594	.30695	.30796	.30897	.30998	.31099	.3120
.46	.3120	.31302	.31404	.31506	.31608	.31710	.31812	.31914	.32016	.32118	.3222
.47	.3222	.32323	.32426	.32529	.32632	.32735	.32838	.32941	.33044	.33147	.3325
.48	.3325	.33355	.33460	.33565	.33670	.33775	.33880	.33985	.34090	.34195	.3430
.49	.3430	.34406	.34512	.34618	.34724	.3483	.34936	.35042	.35148	.35254	.3536
0.50	0.3536	0.35466	0.35572	0.35678	0.35784	0.35890	0.35996	0.36102	0.36208	0.36314	0.3642
.51	.3642	.36528	.36636	.36744	.36852	.36960	.37068	.37176	.37284	.37392	.3750
.52	.3750	.37608	.37716	.37824	.37932	.38040	.38148	.38256	.38364	.38472	.3858
.53	.3858	.38690	.38800	.38910	.39020	.39130	.39240	.39350	.39460	.3957	.3968
.54	.3968	.39791	.39902	.40013	.40124	.40235	.40346	.40457	.40568	.40679	.4079
.55	.4079	.40902	.41014	.41126	.41238	.41350	.41462	.41574	.41686	.41798	.4191
.56	.4191	.42022	.42134	.42246	.42358	.42470	.42582	.42694	.42806	.42918	.4303
.57	.4303	.43144	.43258	.43372	.43486	.43600	.43714	.43828	.43942	.44056	.4417
.58	.4417	.44285	.44400	.44515	.44630	.44745	.44860	.44975	.45090	.45205	.4532
.59	.4532	.45436	.45552	.45668	.45784	.45900	.46016	.46132	.46248	.46364	.4648
0.60	0.4648	0.46596	0.46712	0.46828	0.46944	0.47060	0.47176	0.47292	0.47408	0.47524	0.4764
.61	.4764	.47758	.47876	.47994	.48112	.48230	.48348	.48466	.48584	.48702	.4882
.62	.4882	.48938	.49056	.49174	.49292	.49410	.49528	.49646	.49764	.49882	.5000
.63	.5000	.50120	.50240	.5036	.5048	.5060	.5072	.5084	.5096	.5108	.5120
.64	.5120	.5132	.5144	.5156	.5168	.5180	.5192	.5204	.5216	.5228	.5240
.65	.5240	.52522	.52644	.52766	.52888	.53010	.53132	.53254	.53376	.53498	.5362
.66	.5362	.53742	.53864	.53986	.54108	.54230	.54352	.54474	.54596	.54718	.5484
.67	.5484	.54963	.55086	.55209	.55332	.55455	.55578	.55701	.55824	.55947	.5607
.68	.5607	.56195	.56320	.56445	.56570	.56695	.56820	.56945	.57070	.57195	.5732
.69	.5732	.57445	.5757	.57695	.57820	.57945	.58070	.58195	.58320	.58445	.5857
0.70	0.5857	0.58696	0.58822	0.58948	0.59074	0.59200	0.59326	0.59452	0.59578	0.59704	0.5983
.71	.5983	.59956	.60082	.60208	.60334	.60460	.60586	.60712	.60838	.60964	.6109
.72	.6109	.61218	.61346	.61474	.61602	.61730	.61858	.61986	.62114	.62242	.6237
.73	.6237	.62499	.62628	.62757	.62886	.63015	.63144	.63273	.63402	.63531	.6366
.74	.6366	.63789	.63918	.64047	.64176	.64305	.64434	.64563	.64692	.64821	.6495
.75	.6495	.65081	.65212	.65343	.65474	.65605	.65736	.65867	.65998	.66129	.6626
.76	.6626	.66391	.66522	.66653	.66784	.66915	.67046	.67177	.67308	.67439	.6757
.77	.6757	.67702	.67834	.67966	.68098	.68230	.68362	.68494	.68626	.68758	.6889
.78	.6889	.69023	.69156	.69289	.69422	.69555	.69688	.69821	.69954	.70087	.7022
.79	.7022	.70353	.70486	.70619	.70752	.70885	.71018	.71151	.71284	.71417	.7155

TABLE 5.—*Three-halves powers for numbers 0 to 1.49*—Continued.

Numbers.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009	.010
0.80	0.7155	0.71685	0.71820	0.71955	0.72090	0.72225	0.72360	0.72495	0.72630	0.72765	0.7290
.81	.7290	.73035	.7317	.73305	.73440	.73575	.73710	.73845	.73980	.74115	.7425
.82	.7425	.74387	.74524	.74661	.74798	.74935	.75072	.75209	.75346	.75483	.7562
.83	.7562	.75757	.75894	.76031	.76168	.76305	.76443	.76579	.76716	.76853	.7699
.84	.7699	.77128	.77266	.77404	.77542	.77680	.77818	.77956	.78094	.78232	.7837
.85	.7837	.78508	.78646	.78784	.78922	.79060	.99198	.79336	.79474	.79612	.7975
.86	.7975	.79890	.80030	.80170	.8031	.8045	.8059	.8073	.8087	.8101	.8115
.87	.8115	.8129	.8143	.8157	.8171	.8185	.8199	.8213	.8227	.8241	.8255
.88	.8255	.82691	.82832	.82973	.83114	.83255	.83396	.83537	.83678	.83819	.8396
.89	.8396	.84102	.84244	.84386	.84528	.84670	.84812	.84954	.85096	.85238	.8538
0.90	0.8538	0.85523	0.85666	0.85809	0.85952	0.86095	0.86238	0.86381	0.86524	0.86667	0.8681
.91	.8681	.86953	.87096	.87239	.87382	.87525	.87668	.87811	.87954	.88097	.8824
.92	.8824	.88385	.8853	.88675	.88820	.88965	.89110	.89255	.8940	.89545	.8969
.93	.8969	.89835	.89980	.90125	.90270	.90415	.9056	.90705	.9085	.90995	.9114
.94	.9114	.91285	.9143	.91575	.91720	.91865	.92010	.92155	.9230	.92445	.9259
.95	.9259	.92737	.92884	.93081	.93178	.93325	.93472	.93619	.93766	.93913	.9406
.96	.9406	.94207	.94354	.94501	.94648	.94795	.94942	.95089	.95236	.95383	.9553
.97	.9553	.95679	.95828	.95977	.96126	.96275	.96424	.96573	.96722	.96871	.9702
.98	.9702	.97168	.97316	.97464	.97612	.97760	.97908	.98056	.98204	.98352	.9850
.99	.9850	.9865	.9880	.9895	.9910	.9925	.9940	.9955	.9970	.9985	1.0000
1.00	1.0000	1.0015	1.0030	1.0045	1.0060	1.0075	1.0090	1.0105	1.0120	1.0135	1.0150
1.01	1.0150	1.01652	1.01804	1.01956	1.02108	1.02260	1.02412	1.02564	1.02716	1.02868	1.0302
1.02	1.0302	1.03171	1.03322	1.03473	1.03624	1.03775	1.03926	1.04077	1.04282	1.04379	1.0453
1.03	1.0453	1.04683	1.04836	1.04989	1.05142	1.05295	1.05448	1.05601	1.05754	1.05907	1.0606
1.04	1.0606	1.06213	1.06366	1.06519	1.06672	1.06825	1.06978	1.07131	1.07284	1.07437	1.0759
1.05	1.0759	1.07744	1.07898	1.08052	1.08206	1.08360	1.08514	1.08668	1.08822	1.08976	1.0913
1.06	1.0913	1.09285	1.09440	1.09595	1.09750	1.09905	1.10060	1.10215	1.10370	1.10525	1.1068
1.07	1.1068	1.10836	1.10992	1.11148	1.11304	1.11460	1.11616	1.11772	1.11928	1.12084	1.1224
1.08	1.1224	1.12396	1.12552	1.12708	1.12864	1.13020	1.13176	1.13332	1.13488	1.13644	1.1380
1.09	1.1380	1.13957	1.14114	1.14271	1.14428	1.14585	1.14742	1.14899	1.15056	1.15213	1.1537
1.10	1.1537	1.15528	1.15686	1.15844	1.16002	1.16160	1.16318	1.16476	1.16634	1.16792	1.1695
1.11	1.1695	1.17108	1.17266	1.17424	1.17582	1.17740	1.17898	1.18056	1.18214	1.18372	1.1853
1.12	1.1853	1.18689	1.18848	1.19007	1.19166	1.19325	1.19484	1.19643	1.19802	1.19961	1.2012
1.13	1.2012	1.20280	1.20440	1.20600	1.20760	1.20920	1.21080	1.21240	1.21400	1.21560	1.2172
1.14	1.2172	1.21880	1.22040	1.22200	1.22360	1.22520	1.22680	1.22840	1.23000	1.23160	1.2332
1.15	1.2332	1.23482	1.23644	1.23806	1.23968	1.24130	1.24292	1.24454	1.24616	1.24778	1.2494
1.16	1.2494	1.25102	1.25264	1.25426	1.25588	1.25750	1.25912	1.26074	1.26236	1.26398	1.2656
1.17	1.2656	1.26722	1.26884	1.27046	1.27208	1.27370	1.27532	1.27694	1.27856	1.28018	1.2818
1.18	1.2818	1.28343	1.28506	1.28669	1.28832	1.28995	1.29158	1.29321	1.29484	1.29647	1.2981
1.19	1.2981	1.29974	1.30138	1.30302	1.30466	1.30630	1.30794	1.30958	1.31122	1.31286	1.3145
1.20	1.3145	1.31615	1.31780	1.31945	1.32110	1.32275	1.32440	1.32605	1.32770	1.32935	1.3310
1.21	1.3310	1.33265	1.33430	1.33595	1.33760	1.33925	1.34090	1.34255	1.34420	1.34585	1.3475
1.22	1.3475	1.34916	1.35082	1.35248	1.35414	1.35580	1.35746	1.35912	1.36078	1.36244	1.3641
1.23	1.3641	1.36577	1.36744	1.36911	1.37678	1.37245	1.37412	1.37579	1.37746	1.37913	1.3808
1.24	1.3808	1.38247	1.38414	1.38581	1.38748	1.38915	1.39082	1.39249	1.39416	1.39583	1.3975
1.25	1.3975	1.39919	1.40088	1.40257	1.40426	1.40595	1.40764	1.40933	1.41102	1.41271	1.4144
1.26	1.4144	1.41608	1.41776	1.41944	1.42112	1.42280	1.42448	1.42616	1.42784	1.42952	1.4312
1.27	1.4312	1.4329	1.4346	1.4363	1.4380	1.4397	1.4414	1.4431	1.4448	1.4465	1.4482
1.28	1.4482	1.4499	1.4516	1.4533	1.4550	1.4567	1.4584	1.4601	1.4618	1.4635	1.4652
1.29	1.4652	1.4669	1.4686	1.4703	1.4720	1.4737	1.4754	1.4771	1.4788	1.4805	1.4822

TABLE 5.—*Three-halves powers for numbers 0 to 1.49*—Continued.

Numbers.	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009	.010
1.30	1.4822	1.48392	1.48564	1.48736	1.48908	1.49080	1.49252	1.49424	1.49596	1.49768	1.4994
1.31	1.4994	1.50112	1.50284	1.50456	1.50628	1.50800	1.50972	1.51144	1.51316	1.51488	1.5166
1.32	1.5166	1.51832	1.52004	1.52176	1.52348	1.52520	1.52692	1.52864	1.53036	1.53208	1.5338
1.33	1.5338	1.53554	1.53728	1.53902	1.54076	1.54250	1.54424	1.54598	1.54772	1.54946	1.5512
1.34	1.5512	1.55294	1.55468	1.55642	1.55816	1.55990	1.56164	1.56338	1.56512	1.56686	1.5686
1.35	1.5686	1.57034	1.57208	1.57382	1.57556	1.57730	1.57904	1.58078	1.58252	1.58426	1.5860
1.36	1.5860	1.58775	1.58950	1.59125	1.59300	1.59475	1.59650	1.59825	1.60000	1.60175	1.6035
1.37	1.6035	1.60526	1.60702	1.60878	1.61054	1.61230	1.61406	1.61582	1.61758	1.61934	1.6211
1.38	1.6211	1.62287	1.62464	1.62641	1.62818	1.62995	1.63172	1.63349	1.63526	1.63703	1.6388
1.39	1.6388	1.64057	1.64234	1.64411	1.64588	1.64765	1.64942	1.65119	1.65296	1.65473	1.6565
1.40	1.6565	1.65828	1.66006	1.66184	1.66362	1.66540	1.66718	1.66896	1.67075	1.67252	1.6743
1.41	1.6743	1.67608	1.67786	1.67964	1.68142	1.68320	1.68498	1.68676	1.68854	1.69032	1.6921
1.42	1.6921	1.69389	1.69568	1.69747	1.69926	1.70105	1.70284	1.70463	1.70642	1.70821	1.7100
1.43	1.7100	1.7118	1.7136	1.7154	1.7172	1.7190	1.7208	1.7226	1.7244	1.7262	1.7280
1.44	1.7280	1.7298	1.7316	1.7334	1.7352	1.7370	1.7388	1.7406	1.7424	1.7442	1.7460
1.45	1.7460	1.74781	1.74962	1.75143	1.75324	1.75505	1.75686	1.75867	1.76048	1.76229	1.7641
1.46	1.7641	1.76592	1.76774	1.76956	1.77138	1.77320	1.77502	1.77684	1.77866	1.78048	1.7823
1.47	1.7823	1.78412	1.78594	1.78776	1.78958	1.79140	1.79322	1.79504	1.79686	1.79868	1.8005
1.48	1.8005	1.80233	1.80416	1.80599	1.80782	1.80965	1.81148	1.81331	1.81514	1.81697	1.8188
1.49	1.8188	....	....	....	....	....	....	....	....	....	....

TABLE 6.—*Three-halves powers for numbers from 0 to 12.*

Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10	11
0.00	0.0000	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829
.01	.0010	1.0150	2.8497	5.2222	8.0300	11.2139	14.7337	18.5600	22.6699	27.0450	31.6702	36.5326
.02	.0028	1.0302	2.8710	5.2482	8.0601	11.2475	14.7705	18.5997	22.7123	27.0890	31.7177	36.5824
.03	.0052	1.0453	2.8923	5.2743	8.0902	11.2811	14.8073	18.6394	22.7548	27.1351	31.7652	36.6322
.04	.0080	1.0606	2.9137	5.3004	8.1203	11.3148	14.8442	18.6792	22.7973	27.1802	31.8127	36.6820
.05	.0112	1.0759	2.9352	5.3266	8.1505	11.3485	14.8810	18.7190	22.8399	27.2253	31.8602	36.7319
.06	.0147	1.0913	2.9567	5.3528	8.1807	11.3822	14.9179	18.7589	22.8825	27.2705	31.9078	36.7818
.07	.0185	1.1068	2.9782	5.3791	8.2109	11.4160	14.9549	18.7988	22.9251	27.3156	31.9554	36.8317
.08	.0226	1.1224	2.9998	5.4054	8.2412	11.4497	14.9919	18.8387	22.9677	27.3608	32.0030	36.8816
.09	.0270	1.1380	3.0215	5.4317	8.2715	11.4836	15.0289	18.8786	23.0103	27.4060	32.0506	36.9315
0.10	0.0316	1.1537	3.0432	5.4581	8.3019	11.5174	15.0659	18.9185	23.0530	27.4512	32.0983	36.9815
.11	.0365	1.1695	3.0650	5.4845	8.3323	11.5513	15.1030	18.9585	23.0957	27.4965	32.1460	37.0315
.12	.0416	1.1853	3.0868	5.5110	8.3627	11.5852	15.1400	18.9985	23.1384	27.5418	32.1937	37.0815
.13	.0469	1.2012	3.1086	5.5375	8.3932	11.6192	15.1772	19.0386	23.1812	27.5871	32.2414	37.1315
.14	.0524	1.2172	3.1306	5.5641	8.4237	11.6532	15.2143	19.0786	23.2240	27.6324	32.2892	37.1816
.15	.0581	1.2332	3.1525	5.5907	8.4542	11.6872	15.2515	19.1187	23.2668	27.6778	32.3370	37.2317
.16	.0640	1.2494	3.1745	5.6173	8.4848	11.7213	15.2887	19.1589	23.3096	27.7232	32.3848	37.2817
.17	.0701	1.2656	3.1966	5.6440	8.5154	11.7554	15.3260	19.1990	23.3525	27.7686	32.4326	37.3319
.18	.0764	1.2818	3.2187	5.6708	8.5460	11.7895	15.3632	19.2392	23.3954	27.8140	32.4804	37.3820
.19	.0828	1.2981	3.2409	5.6975	8.5767	11.8236	15.4005	19.2794	23.4383	27.8595	32.5283	37.4322

TABLE 6.—*Three-halves powers for numbers from 0 to 12*—Continued.

Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10	11
0.20	0.0894	1.3145	3.2631	5.7243	8.6074	11.8578	15.4379	19.3196	23.4812	27.9050	32.5762	37.4824
.21	.0962	1.3310	3.2854	5.7512	8.6382	11.8920	15.4752	19.3599	23.5242	27.9514	32.6241	37.5326
.22	.1032	1.3475	3.3077	5.7781	8.6690	11.9263	15.5126	19.4002	23.5672	27.9960	32.6720	37.5828
.23	.1103	1.3641	3.3301	5.8050	8.6998	11.9606	15.5501	19.4405	23.6102	28.0416	32.7200	37.6331
.24	.1176	1.3808	3.3525	5.8320	8.7307	11.9949	15.5866	19.4808	23.6533	28.0872	32.7680	37.6833
.25	.1250	1.3975	3.3750	5.8590	8.7616	12.0293	15.6250	19.5212	23.6963	28.1328	32.8160	37.7336
.26	.1326	1.4144	3.3975	5.8861	8.7925	12.0636	15.6616	19.5576	23.7394	28.1784	32.8640	37.7840
.27	.1403	1.4312	3.4201	5.9132	8.8235	12.0981	15.7001	19.6021	23.7825	28.2241	32.9121	37.8343
.28	.1482	1.4482	3.4427	5.9403	8.8545	12.1325	15.7376	19.6425	23.8257	28.2698	32.9600	37.8847
.29	.1562	1.4652	3.4654	5.9675	8.8856	12.1670	15.7752	19.6830	23.8689	28.3155	33.0083	37.9351
0.30	0.1643	1.4822	3.4881	5.9947	8.9167	12.2015	15.8129	19.7235	23.9121	28.3612	33.0564	37.9855
.31	.1726	1.4994	3.5109	6.0220	8.9478	12.2361	15.8505	19.7641	23.9553	28.4069	33.1046	38.0359
.32	.1810	1.5166	3.5337	6.0493	8.9790	12.2706	15.8882	19.8046	23.9986	28.4527	33.1527	38.0864
.33	.1896	1.5338	3.5566	6.0767	9.0102	12.3053	15.9260	19.8452	24.0418	28.4985	33.2009	38.1369
.34	.1983	1.5512	3.5795	6.1041	9.0414	12.3399	15.9637	19.8858	24.0851	28.5444	33.2492	38.1874
.35	.2071	1.5686	3.6025	6.1315	9.0726	12.3746	16.0015	19.9265	24.1285	28.5902	33.2974	38.2379
.36	.2160	1.5860	3.6255	6.1590	9.1040	12.4093	16.0393	19.9672	24.1718	28.6361	33.3457	38.2884
.37	.2251	1.6035	3.6486	6.1865	9.1353	12.4440	16.0772	20.0079	24.2152	28.6820	33.3940	38.3390
.38	.2342	1.6211	3.6717	6.2141	9.1667	12.4788	16.1150	20.0486	24.2586	28.7279	33.4423	38.3896
.39	.2436	1.6388	3.6949	6.2417	9.1981	12.5136	16.1529	20.0894	24.3021	28.7739	33.4906	38.4402
0.40	0.2530	1.6565	3.7181	6.2693	9.2295	12.5485	16.1909	20.1302	24.3455	28.8199	33.5390	38.4908
.41	.2625	1.6743	3.7413	6.2970	9.2610	12.5833	16.2288	20.1710	24.3890	28.8659	33.5874	38.5415
.42	.2722	1.6921	3.7646	6.3247	9.2925	12.6182	16.2668	20.2118	24.4325	28.9119	33.6358	38.5922
.43	.2820	1.7100	3.7880	6.3525	9.3241	12.6532	16.3048	20.2527	24.4761	28.9579	33.6842	38.6429
.44	.2919	1.7280	3.8114	6.3803	9.3557	12.6882	16.3429	20.2936	24.5196	29.0040	33.7327	38.6936
.45	.3019	1.7460	3.8349	6.4081	9.3873	12.7232	16.3810	20.3345	24.5632	29.0501	33.7811	38.7443
.46	.3120	1.7641	3.8584	6.4360	9.4189	12.7582	16.4191	20.3755	24.6068	29.0962	33.8297	38.7951
.47	.3222	1.7823	3.8819	6.4639	9.4506	12.7933	16.4572	20.4165	24.6505	29.1424	33.8782	38.8459
.48	.3325	1.8005	3.9055	6.4919	9.4824	12.8284	16.4954	20.4575	24.6941	29.1885	33.9267	38.8967
.49	.3430	1.8188	3.9292	6.5199	9.5141	12.8635	16.5336	20.4985	24.7378	29.2347	33.9753	38.9475
0.50	0.3536	1.8371	3.9529	6.5479	9.5459	12.8986	16.5718	20.5396	24.7815	29.2810	34.0239	38.9984
.51	.3642	1.8555	3.9766	6.5760	9.5778	12.9338	16.6101	20.5807	24.8253	29.3272	34.0725	39.0493
.52	.3750	1.8740	4.0004	6.6041	9.6097	12.9691	16.6484	20.6218	24.8691	29.3735	34.1211	39.1002
.53	.3858	1.8925	4.0242	6.6323	9.6416	13.0043	16.6867	20.6630	24.9129	29.4198	34.1698	39.1511
.54	.3968	1.9111	4.0481	6.6605	9.6735	13.0396	16.7250	20.7041	24.9567	29.4661	34.2185	39.2020
.55	.4079	1.9297	4.0720	6.6887	9.7055	13.0749	16.7634	20.7453	25.0005	29.5124	34.2672	39.2530
.56	.4191	1.9484	4.0960	6.7170	9.7375	13.1103	16.8018	20.7866	25.0444	29.5588	34.3159	39.3040
.57	.4303	1.9672	4.1200	6.7453	9.7695	13.1457	16.8402	20.8278	25.0883	29.6052	34.3647	39.3550
.58	.4417	1.9860	4.1441	6.7737	9.8016	13.1811	16.8787	20.8691	25.1322	29.6516	34.4135	39.4060
.59	.4532	2.0049	4.1682	6.8021	9.8337	13.2165	16.9172	20.9104	25.1762	29.6980	34.4623	39.4571
0.60	0.4648	2.0238	4.1924	6.8305	9.8659	13.2520	16.9557	20.9518	25.2202	29.7445	34.5111	39.5082
.61	.4764	2.0429	4.2166	6.8590	9.8981	13.2875	16.9943	20.9931	25.2642	29.7910	34.5599	39.5593
.62	.4882	2.0619	4.2408	6.8875	9.9303	13.3231	17.0328	21.0345	25.3082	29.8375	34.6088	39.6104
.63	.5000	2.0810	4.2651	6.9161	9.9626	13.3587	17.0714	21.0759	25.3522	29.8841	34.6577	39.6615
.64	.5120	2.1002	4.2895	6.9447	9.9949	13.3943	17.1101	21.1174	25.3963	29.9306	34.7066	39.7127
.65	.5240	2.1196	4.3139	6.9733	10.0272	13.4299	17.1488	21.1589	25.4404	29.9772	34.7557	39.7639
.66	.5362	2.1388	4.3383	7.0020	10.0596	13.4656	17.1874	21.2004	25.4845	30.0238	34.8045	39.8151
.67	.5484	2.1581	4.3628	7.0307	10.0920	13.5013	17.2172	21.2419	25.5287	30.0704	34.8535	39.8663
.68	.5607	2.1775	4.3874	7.0595	10.1244	13.5370	17.2649	21.2834	25.5729	30.1171	34.9025	39.9176
.69	.5732	2.1970	4.4119	7.0883	10.1569	13.5728	17.3037	21.3250	25.6171	30.1638	34.9516	39.9689

TABLE 6.—*Three-halves powers for numbers from 0 to 12*—Continued.

Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10	11
0.70	0.5857	2.2165	4.4366	7.1171	10.1894	13.6086	17.3425	21.3666	25.6613	30.2105	35.0006	40.0202
.71	.5983	2.2361	4.4612	7.1460	10.2214	13.6444	17.3814	21.4083	25.7056	30.2572	35.0497	40.0715
.72	.6109	2.2558	4.4859	7.1749	10.2545	13.6803	17.4202	21.4499	25.7499	30.3040	35.0988	40.1228
.73	.6237	2.2755	4.5107	7.2038	10.2871	13.7161	17.4591	21.4916	25.7942	30.3507	35.1479	40.1742
.74	.6366	2.2952	4.5355	7.2328	10.3197	13.7521	17.4981	21.5333	25.8395	30.3975	35.1971	40.2256
.75	.6495	2.3150	4.5604	7.2618	10.3524	13.7880	17.5370	21.5751	25.8828	30.4444	35.2462	40.2770
.76	.6626	2.3349	4.5853	7.2909	10.3851	13.8240	17.5760	21.6169	25.9272	30.4912	35.2954	40.3284
.77	.6757	2.3548	4.6102	7.3200	10.4178	13.8600	17.6150	21.6587	25.9716	30.5381	35.3446	40.3798
.78	.6889	2.3748	4.6352	7.3492	10.4506	13.8961	17.6541	21.7005	26.0161	30.5850	35.3939	40.4313
.79	.7022	2.3949	4.6602	7.3783	10.4834	13.9321	17.6931	21.7423	26.0605	30.6319	35.4431	40.4828
0.80	0.7155	2.4150	4.6853	7.4076	10.5163	13.9682	17.7322	21.7842	26.1050	30.6789	35.4924	40.5343
.81	.7290	2.4351	4.7104	7.4368	10.5492	14.0044	17.7714	21.8261	26.1495	30.7258	35.5417	40.5859
.82	.7425	2.4553	4.7356	7.4661	10.5812	14.0406	17.8105	21.8681	26.1941	30.7728	35.5911	40.6374
.83	.7562	2.4756	4.7608	7.4955	10.6150	14.0768	17.8507	21.9100	26.2386	30.8198	35.6404	40.6890
.84	.7699	2.4959	4.7861	7.5248	10.6480	14.1130	17.8889	21.9520	26.2832	30.8669	35.6898	40.7406
.85	.7837	2.5163	4.8114	7.5542	10.6810	14.1493	17.9282	21.9940	26.3278	30.9139	35.7392	40.7922
.86	.7975	2.5367	4.8367	7.5837	10.7141	14.1856	17.9674	22.0361	26.3725	30.9610	35.7886	40.8439
.87	.8115	2.5572	4.8621	7.6132	10.7472	14.2219	18.0067	22.0781	26.4171	31.0081	35.8380	40.8955
.88	.8255	2.5777	4.8875	7.6427	10.7803	14.2582	18.0461	22.1202	26.4618	31.0553	35.8875	40.9472
.89	.8396	2.5983	4.9130	7.6723	10.8134	14.2946	18.0854	22.1623	26.5065	31.1024	35.9370	40.9989
0.90	0.8538	2.6190	4.9385	7.7019	10.8466	14.3311	18.1248	22.2045	26.5523	31.1496	35.9865	41.0507
.91	.8681	2.6397	4.9641	7.7315	10.8798	14.3675	18.1642	22.2467	26.5960	31.1968	36.0360	41.1024
.92	.8824	2.6604	4.9897	7.7702	10.9131	14.4040	18.2037	22.2889	26.6408	31.2441	36.0856	41.1542
.93	.8969	2.6812	5.0154	7.7909	10.9464	14.4405	18.2432	22.3311	26.6856	31.2913	36.1352	41.2060
.94	.9114	2.7021	5.0411	7.8207	10.9797	14.4770	18.2827	22.3733	26.7305	31.3386	36.1848	41.2578
.95	.9259	2.7230	5.0668	7.8505	11.0131	14.5136	18.3222	22.4156	26.7753	31.3850	36.2344	41.3097
.96	.9406	2.7440	5.0926	7.8803	11.0464	14.5502	18.3617	22.4579	26.8202	31.4332	36.2841	41.3615
.97	.9553	2.7650	5.1184	7.9102	11.0799	14.5869	18.4013	22.5008	26.8651	31.4806	36.3337	41.4134
.98	.9702	2.7861	5.1443	7.9401	11.1133	14.6235	18.4409	22.5426	26.9100	31.5280	36.3834	41.4653
.99	.9850	2.8072	5.1702	7.9700	11.1468	14.6602	18.4806	22.5850	26.9550	31.5754	36.4331	41.5173
1.00	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	41.5692

The tables of three-halves powers may conveniently be used in conjunction with Crelle's Rechentafeln, or similar tables of the products of pairs of factors. C will usually be constant, or nearly so. Entering Crelle's tables with C or CL as an argument, the discharge corresponding to values of $H^{\frac{3}{2}}$ read from the tables here given may be taken out directly, and usually with sufficient precision at least for 1 foot length of crest, without any arithmetical computation. Table 5 gives $H^{\frac{3}{2}}$ for values of H from zero to 1.5 feet, advancing by thousandths. In Table 6 the increment is 0.1 foot, and the range zero to 12 feet. Should $H^{\frac{3}{2}}$ be required for larger values of H, it may be found from the three-halves power of $\frac{1}{4}H$, by the formula

$$H^{\frac{3}{2}}=8\left(\frac{H}{4}\right)^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (114)$$

TABLE 7.—FLOW OVER BROAD-CRESTED WEIRS, WITH STABLE NAPPE.

This table gives values of

$$Q = C_1 L H^{\frac{3}{2}},$$

where

$$L = 1$$
$$C_1 = 2.64$$

The derivation of this coefficient is given in connection with discussion of broad-crested weirs (pp. 119–121). It may be applied to broad-crested weirs of any width of cross section exceeding 2 feet within such limiting heads that the nappe does not adhere to the downstream face of the weir for low heads nor tend to become detached with increased head. Under the latter condition the coefficient increases to a limit near the value which applies for a thin-edged weir, a point being finally reached where the nappe breaks entirely free from the broad crest and discharges in the same manner as for a thin-edged weir. The coefficient, 2.64, may often be applied for weirs exceeding 2-feet crest width and for heads from 0.5 foot up to 1.5 or 2 times the breadth of weir crest. If corrections for the velocity of approach are required the Francis correction formula, or its equivalent, should be used.

TABLE 7.—*Weir discharge per foot of crest length.*

[Coefficient C_1=2.64.]

Head H, feet. Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10
0.00	0.000	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5
.01	.003	2.68	7.52	13.8	21.2	29.6	38.9	49.0	59.8	71.4	83.6
.02	.007	2.72	7.58	13.8	21.3	29.7	39.0	49.1	59.9	71.5	83.7
.03	.014	2.76	7.64	13.9	21.4	29.8	39.1	49.2	60.1	71.6	83.9
.04	.021	2.80	7.69	14.0	21.4	29.9	39.2	49.3	60.2	71.7	84.0
.05	.030	2.84	7.75	14.1	21.5	30.0	39.3	49.4	60.3	71.9	84.1
.06	.039	2.88	7.81	14.1	21.6	30.0	39.4	49.5	60.4	72.0	84.2
.07	.049	2.92	7.86	14.2	21.7	30.1	39.5	49.6	60.5	72.1	84.4
.08	.060	2.96	7.92	14.3	21.8	30.2	39.6	49.7	60.6	72.2	84.5
.09	.071	3.00	7.98	14.3	21.8	30.3	39.7	49.8	60.7	72.3	84.6
0.10	0.083	3.04	8.03	14.4	21.9	30.4	39.8	49.9	60.8	72.5	84.7
.11	.096	3.09	8.09	14.5	22.0	30.5	39.9	50.0	61.0	72.6	84.9
.12	.110	3.13	8.15	14.5	22.1	30.6	40.0	50.2	61.1	72.7	85.0
.13	.124	3.17	8.21	14.6	22.2	30.7	40.1	50.3	61.2	72.8	85.1
.14	.138	3.21	8.26	14.7	22.2	30.8	40.2	50.4	61.3	72.9	85.2
.15	.153	3.26	8.32	14.8	22.3	30.8	40.3	50.5	61.4	73.1	85.4
.16	.169	3.30	8.38	14.8	22.4	30.9	40.4	50.6	61.5	73.2	85.5
.17	.185	3.34	8.44	14.9	22.5	31.0	40.5	50.7	61.6	73.3	85.6
.18	.202	3.38	8.50	15.0	22.6	31.1	40.6	50.8	61.8	73.4	85.7
.19	.218	3.43	8.56	15.0	22.6	31.2	40.7	50.9	61.9	73.5	85.9

TABLE 7.—*Weir discharge per foot of crest length*—Continued.

Head *H*, feet. Units / Hundredths.	0	1	2	3	4	5	6	7	8	9	10
0.20	0.236	3.47	8.61	15.1	22.7	31.3	40.8	51.0	62.0	73.7	86.0
.21	.254	3.51	8.67	15.2	22.8	31.4	40.9	51.1	62.1	73.8	86.1
.22	.272	3.56	8.73	15.2	22.9	31.5	41.0	51.2	62.2	73.9	86.2
.23	.291	3.60	8.79	15.3	23.0	31.6	41.0	51.3	62.3	74.0	86.4
.24	.310	3.64	8.85	15.4	23.0	31.7	41.1	51.4	62.4	74.1	86.5
.25	.330	3.69	8.91	16.5	23.1	31.8	41.2	51.5	62.6	74.3	86.6
.26	.350	3.73	8.97	15.5	23.2	31.8	41.3	51.6	62.7	74.4	86.8
.27	.370	3.78	9.03	15.6	23.3	31.9	41.4	51.7	62.8	74.5	86.9
.28	.391	3.82	9.09	15.7	23.4	32.0	41.5	51.9	62.9	74.6	87.0
.29	.412	3.87	9.15	15.8	23.4	32.1	41.6	52.0	63.0	74.8	87.1
0.30	0.434	3.91	9.21	15.8	23.5	32.2	41.7	52.1	63.1	74.9	87.3
.31	.456	3.96	9.27	15.9	23.6	32.3	41.8	52.2	63.2	75.0	87.4
.32	.478	4.00	9.33	16.0	23.7	32.4	41.9	52.3	63.4	75.1	87.5
.33	.500	4.05	9.39	16.0	23.8	32.5	42.0	52.4	63.5	75.2	87.6
.34	.524	4.10	9.45	16.2	23.9	32.6	42.1	52.5	63.6	75.4	87.8
.35	.547	4.14	9.51	16.2	24.0	32.7	42.2	52.6	63.7	75.5	87.9
.36	.570	4.19	9.57	16.3	24.1	32.8	42.3	52.7	63.8	75.6	88.0
.37	.594	4.23	9.63	16.3	24.1	32.8	42.4	52.8	63.9	75.7	88.2
.38	.618	4.28	9.69	16.4	24.2	32.9	42.5	52.9	64.0	75.8	88.3
.39	.643	4.33	9.75	16.5	24.3	33.0	42.6	53.0	64.2	76.0	88.4
0.40	0.668	4.37	9.82	16.6	24.4	33.1	42.7	53.1	64.3	76.1	88.5
.41	.693	4.42	9.88	16.6	24.4	33.2	42.8	53.2	64.4	76.2	88.7
.42	.719	4.47	9.94	16.7	24.5	33.3	42.9	53.4	64.5	76.3	88.8
.43	.744	4.51	10.0	16.8	24.6	33.4	43.0	53.5	64.6	76.4	88.9
.44	.771	4.56	10.1	16.8	24.7	33.5	43.1	53.6	64.7	76.6	89.0
.45	.797	4.61	10.1	16.9	24.8	33.6	43.2	53.7	64.8	76.7	89.2
.46	.824	4.66	10.2	17.0	24.9	33.7	43.3	53.8	65.0	76.8	89.3
.47	.851	4.70	10.2	17.1	24.9	33.8	43.4	53.9	65.1	76.9	89.4
.48	.878	4.75	10.3	17.1	25.0	33.9	43.6	54.0	65.2	77.0	89.6
.49	.905	4.80	10.4	17.2	25.1	34.0	43.6	54.1	65.3	77.2	89.7
0.50	0.934	4.85	10.4	17.3	25.2	34.0	43.7	54.2	65.4	77.3	89.8
.51	.961	4.90	10.5	17.4	25.3	34.1	43.8	54.3	65.5	77.4	90.0
.52	.990	4.95	10.6	17.4	25.4	34.2	44.0	54.4	65.6	77.5	90.1
.53	1.02	5.00	10.6	17.5	25.4	34.3	44.1	54.6	65.8	77.7	90.2
.54	1.05	5.04	10.7	17.6	25.5	34.4	44.2	54.7	65.9	77.8	90.3
.55	1.08	5.09	10.8	17.7	25.6	34.5	44.3	54.8	66.0	77.9	90.5
.56	1.11	5.14	10.8	17.7	25.7	34.6	44.4	54.9	66.1	78.0	90.6
.57	1.14	5.19	10.9	17.8	25.8	34.7	44.5	55.0	66.2	78.2	90.7
.58	1.17	5.24	10.9	17.9	25.9	34.8	44.6	55.1	66.3	78.3	90.8
.59	1.20	5.29	11.0	18.0	26.0	34.9	44.7	55.2	66.5	73.4	91.0
0.60	1.23	5.34	11.1	18.0	26.0	35.0	44.8	55.3	66.6	78.5	91.1
.61	1.26	5.39	11.1	18.1	26.1	35.1	44.9	55.4	66.7	78.6	91.2
.62	1.29	5.44	11.2	18.2	26.2	35.2	45.0	55.5	66.8	78.8	91.4
.63	1.32	5.49	11.2	18.2	26.3	35.3	45.1	55.6	66.9	78.9	91.5
.64	1.35	5.54	11.3	18.3	26.4	35.4	45.2	55.7	67.0	79.0	91.6
.65	1.38	5.60	11.4	18.4	26.5	35.4	45.3	55.9	67.2	79.1	91.8
.66	1.42	5.65	11.4	18.5	26.6	35.5	45.4	56.0	67.3	79.3	91.9
.67	1.45	5.70	11.5	18.6	26.6	35.6	45.5	56.1	67.4	79.4	92.0
.68	1.48	5.75	11.6	18.6	26.7	35.7	45.6	56.2	67.5	79.5	92.1
.69	1.51	5.80	11.6	18.7	26.8	35.8	45.7	56.3	67.6	79.6	92.3

TABLE 7.—*Weir discharge per foot of crest length*—Continued.

Head *H*, feet. Units / Hundredths.	0	1	2	3	4	5	6	7	8	9	10
0.70	1.55	5.85	11.7	18.8	26.9	35.9	45.8	56.4	67.7	79.8	92.4
.71	1.58	5.90	11.8	18.9	27.0	36.0	45.9	56.5	67.9	79.9	92.5
.72	1.61	5.96	11.8	18.9	27.1	36.1	46.0	56.6	68.0	80.0	92.7
.73	1.65	6.01	11.9	19.0	27.2	36.2	46.1	56.7	68.1	80.1	92.8
.74	1.68	6.06	12.0	19.1	27.2	36.3	46.2	56.8	68.2	80.2	92.9
.75	1.71	6.11	12.0	19.2	27.3	36.4	46.3	57.0	68.3	80.4	93.0
.76	1.75	6.16	12.1	19.2	27.4	36.5	46.4	57.1	68.4	80.5	93.2
.77	1.78	6.22	12.2	19.3	27.5	36.6	46.5	57.2	68.6	80.6	93.3
.78	1.82	6.27	12.2	19.4	27.6	36.7	46.6	57.3	68.7	80.7	93.4
.79	1.85	6.32	12.3	19.5	27.7	36.8	46.7	57.4	68.8	80.9	93.6
0.80	1.89	6.38	12.4	19.6	27.8	36.9	46.8	57.5	68.9	81.0	93.7
.81	1.92	6.43	12.4	19.6	27.8	37.0	46.9	57.6	69.0	81.1	93.8
.82	1.96	6.48	12.5	19.7	27.9	37.1	47.0	57.7	69.2	81.2	94.0
.83	2.00	6.54	12.6	19.8	28.0	37.2	47.1	57.8	69.3	81.4	94.1
.84	2.03	6.59	12.6	19.9	28.1	37.3	47.2	58.0	69.4	81.5	94.2
.85	2.07	6.64	12.7	19.9	28.2	37.4	47.3	58.1	69.5	81.6	94.4
.86	2.10	6.70	12.8	20.0	28.3	37.4	47.4	58.2	69.6	81.7	94.5
.87	2.14	6.75	12.8	20.1	28.4	37.5	47.5	58.3	69.7	81.9	94.6
.88	2.18	6.80	12.9	20.2	28.5	37.6	47.6	58.4	69.9	82.0	94.7
.89	2.22	6.86	13.0	20.2	28.5	37.7	47.7	58.5	70.0	82.1	94.9
0.90	2.25	6.91	13.0	20.3	28.6	37.8	47.8	58.6	70.1	82.2	95.0
.91	2.29	6.97	13.1	20.4	28.7	37.9	48.0	58.7	70.2	82.4	95.1
.92	2.33	7.02	13.2	20.5	28.8	38.0	48.1	58.8	70.3	82.5	95.3
.93	2.37	7.08	13.2	20.6	28.9	38.1	48.2	59.0	70.4	82.6	95.4
.94	2.41	7.13	13.3	20.6	29.0	38.2	48.3	59.1	70.6	82.7	95.5
.95	2.44	7.19	13.4	20.7	29.1	38.3	48.4	59.2	70.7	82.9	95.6
.96	2.48	7.24	13.4	20.8	29.2	38.4	48.5	59.3	70.8	83.0	95.8
.97	2.52	7.30	13.5	20.9	29.3	38.5	48.6	59.4	70.9	83.1	95.9
.98	2.56	7.36	13.6	21.0	29.3	38.6	48.7	59.5	71.0	83.2	96.0
.99	2.60	7.41	13.6	21.0	29.4	38.7	48.8	59.6	71.2	83.4	96.2
1.00	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5	96.3

TABLE 8.—BACKWATER CAUSED BY A DAM OR WEIR.

In a channel of uniform depth, width, and slope, let

D=Original uniform depth.

d_2=Depth at the dam or obstruction.

d_1=Depth at a point upstream.

l=Distance upstream to the point d_1.

w=Width of channel.

l_1=Distance upstream to the "hydrostatic limit."

S=Natural uniform slope or inclination of water surface and stream bed, assumed parallel.

g=Acceleration of gravity.

C=Coefficient in the Chezy or slope formula $v=C\sqrt{RS}$,

where R is the hydraulic radius$=\dfrac{\text{area of section}}{\text{wetted perimeter}}$.

The value of C varies for rivers from about 50 to 140.

The distance upstream from the obstruction at which the depth will be d_1 may be found by the formula

$$l=\frac{d_2-d_1}{S}+D\left(\frac{1}{S}-\frac{C^2}{g}\right)\left[F_1-F_2\right] \quad . \quad . \quad . \quad (115)$$

F is a function of $\dfrac{d}{D}$, whose value can be expressed mathematically only as a transcendental equation. The numerical values of this function are given in Table 8. F_1 will be found opposite the argument $\dfrac{D}{d_1}$, and F_2 opposite $\dfrac{D}{d_2}$.

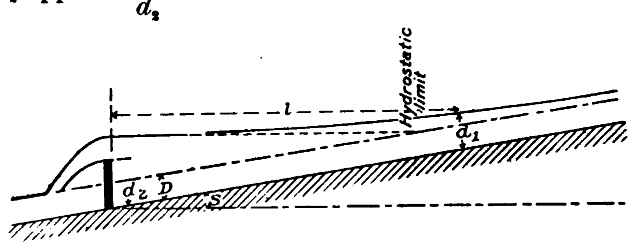


FIG. 15.—Concave backwater surface.

The inverse problem of finding the depth at any given distance l upstream can be solved only by successive trials.

Using the above equation, a series of values of d_1 may be determined giving in tabular form the corresponding values of l. From this data the form of the surface curve may be graphically shown or the depth of back piling at any point may be interpolated.

If $D=5$, $d_2=10$, $C=75$, $S=0.0001$, $\dfrac{D}{d_2}=0.5$, and $F_2=0.1318$,

$$D\left(\frac{1}{S}-\frac{C^2}{g}\right)=5\left(10000-\frac{75^2}{32.16}\right)=49625.$$

Column (2) in the following table gives the values of l for various values of d_1 computed by means of formula (115).

Form of backwater curve above a dam.

d	l	δ	$d_1-\delta$
Depth, feet (1).	Distance from dam to depth d_1, feet (2).	Hydrostatic depth at distance l, feet (3).	Depth of "back piling," feet (4).
9	11,687	8.83	.17
8	24,277	7.57	.43
7	38,526	6.15	.85
6.5	47,024	5.30	1.20
6.4	48,798	5.13	1.27
6.3	50,796	[a] 5.00	1.30
6.2	52,813	[a] 5.00	1.20
6.1	55,010	[a] 5.00	1.10
6	57,240	[a] 5.00	1.00
5.5	67,252	[a] 5.00	.50
5.3	82,940	[a] 5.00	.30
5.1	101,255	[a] 5.00	.10

[a] Above hydrostatic limit.

If the pond formed by the dam were level, the *hydrostatic depth* δ at any distance upstream would be

$$\delta=d_2-l\sin S \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (116)$$

Column (3) in the above table shows this factor for the several values of l. The true "back piling" or rise due to the *surface curvature* is expressed by the difference $d_1-\delta$, as given in column (4).

This quantity has a maximum value at the *hydrostatic limit*, or terminus of the level pond, where $\delta=D$.

Its location is such that if l_1 is the distance upstream from the dam

$$l_1=\frac{d_2-D}{\sin S} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (117)$$

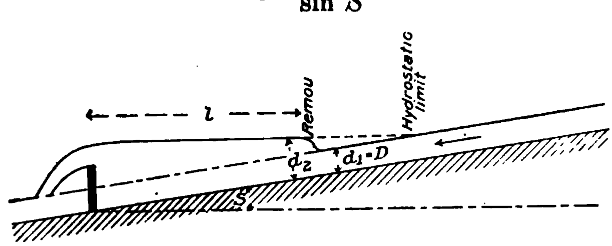


FIG. 16.—Convex backwater surface.

In the example given the hydrostatic limit occurs at a distance $l_1=50{,}000$ feet above the dam, at which point the maximum back piling of about 1.31 feet occurs.

Above the hydrostatic limit the depth of back piling is d_1-D.

When $$S \overline{\gtrless} \frac{g}{C^2} \quad \ldots \ldots \ldots \quad (118)$$

the pond surface will not be concave, but a *remou* or hydraulic jump will occur, having a height

$$d_2 = 2\sqrt{d_1 \frac{v_1^2}{2g}} \quad \ldots \ldots \ldots \quad (119)$$

where v_1 is the mean velocity corresponding to d_1. To find the distance upstream to the point where the jump occurs, solve equation (115) for the value of d_2, found by formula (119).

If $S=0.004 \qquad C=100 \qquad \frac{g}{C^2}=0.003216.$

If $d_1=5 \qquad w=100 \qquad D=5 \qquad R=\frac{500}{110}=4.55.$

$$v_1 = 100\sqrt{4.55 \frac{4}{1000}} = 13.49 \text{ feet per second.}$$

$$d_2 = 2\sqrt{d_1 \frac{v_1^2}{2g}} = 2\sqrt{5 \frac{185}{64.32}} = 7.48.$$

Let the depth at the dam be 10 feet; using d_2 as found above as the terminal depth in formula (115), we obtain

$$l = \frac{10-7.48}{0.004} + 5\left(\frac{1}{0.004} - 311\right)(F_2 - F_1)$$

$$\frac{D}{d_1} = \frac{5}{7.48} = 0.669 \qquad \frac{D}{d_2} = 0.5$$

$$F_1 = 0.2578 \qquad F_2 = 0.1318$$

$$l = 630 + (250 - 311) \times 0.126 = 622.3 \text{ feet.}$$

The hydrostatic limit in this case is

$$l_1 = \frac{10-5}{0.004} = 1{,}250 \text{ feet.}$$

If the channel above an obstruction consists of successive reaches having different slopes or cross sections, the depth at the head of the first reach or level may be found by the method outlined, and using this as the initial depth d_2, a similar solution may be made for the second and succeeding levels.[a]

[a] Table 8 has been extended from Bresse's original table by interpolation. Demonstrations of the formulas here given may be found in Merriman's or Bovey's Hydraulics. In case of a fall a different function must be employed. Its values will be found in the works mentioned.

TABLE 8.—*Backwater function* (F) *for a dam or obstruction.*

[The column headings are hundredths for values of $\frac{D}{d}$ from zero to 0.29, thousandths for values of $\frac{D}{d}$ from 0.30 to 0.899, and ten-thousandths for values of $\frac{D}{d}$ from 0.900 to 0.999.]

$\frac{D}{d}$	0	1	2	3	4	5	6	7	8	9
	F	F	F	F	F	F	F	F	F	F
0.0	0.0000	0.0001	0.0003	0.0005	0.0009	0.0013	0.0018	0.0026	0.0034	0.0042
.1	.0050	.0061	.0072	.0085	.0098	.0113	.0128	.0045	.0162	.0181
.2	.0201	.0221	.0243	.0266	.0290	.0314	.0340	.0367	.0395	.0425
0.30	0.0455	0.0458	0.0461	0.0464	0.0467	0.0471	0.0474	0.0477	0.0480	0.0483
.31	.0486	.0489	.0493	.0496	.0499	.0503	.0506	.0509	.0512	.0516
.32	.0519	.0522	.0526	.0529	.0533	.0536	.0539	.0543	.0546	.0550
.33	.0553	.0556	.0560	.0563	.0567	.0570	.0573	.0577	.0580	.0584
.34	.0587	.0591	.0594	.0598	.0601	.0605	.0609	.0612	.0616	.0619
.35	.0623	.0627	.0630	.0634	.0638	.0642	.0645	.0649	.0653	.0656
.36	.0660	.0664	.0668	.0672	.0676	.0680	.0683	.0687	.0691	.0695
.37	.0699	.0703	.0707	.0711	.0715	.0718	.0722	.0726	.0730	.0734
.38	.0738	.0742	.0746	.0750	.0754	.0758	.0763	.0767	.0771	.0775
.39	.0779	.0783	.0787	.0792	.0796	.0800	.0804	.0808	.0813	.0817
0.40	0.0821	0.0825	0.0830	0.0834	0.0839	0.0843	0.0847	0.0852	0.0856	0.0861
.41	.0865	.0869	.0874	.0878	.0883	.0887	.0891	.0896	.0900	.0905
.42	.0909	.0918	.0926	.0935	.0943	.0952	.0961	.0969	.0978	.0986
.43	.0995	.0996	.0997	.0997	.0998	.0999	.1000	.1001	.1001	.1002
.44	.1003	.1008	.1013	.1018	.1023	.1030	.1032	.1037	.1042	.1047
.45	.1052	.1057	.1062	.1067	.1072	.1077	.1082	.1087	.1092	.1097
.46	.1102	.1107	.1112	.1118	.1123	.1128	.1133	.1138	.1144	.1149
.47	.1154	.1159	.1165	.1170	.1175	.1180	.1186	.1191	.1196	.1202
.48	.1207	.1212	.1218	.1224	.1229	.1234	.1240	.1246	.1215	.1256
.49	.1262	.1268	.1273	.1279	.1284	.1290	.1296	.1301	.1307	.1312
0.50	0.1318	0.1324	0.1330	0.1335	0.1341	0.1347	0.1353	0.1359	0.1364	0.1370
.51	.1376	.1382	.1388	.1394	.1400	.1406	.1411	.1417	.1423	.1429
.52	.1435	.1441	.1447	.1454	.1460	.1466	.1472	.1478	.1485	.1491
.53	.1497	.1503	.1510	.1516	.1522	.1528	.1535	.1541	.1547	.1554
.54	.1560	.1566	.1573	.1580	.1586	.1592	.1599	.1606	.1612	.1618
.55	.1625	.1632	.1638	.1645	.1652	.1658	.1665	.1672	.1679	.1685
.56	.1692	.1699	.1706	.1713	.1720	.1726	.1733	.1740	.1747	.1754
.57	.1761	.1768	.1775	.1782	.1789	.1796	.1804	.1811	.1818	.1825
.58	.1832	.1839	.1847	.1854	.1861	.1868	.1876	.1883	.1890	.1898
.59	.1905	.1912	.1920	.1928	.1935	.1942	.1950	.1958	.1965	.1972
0.60	0.1980	0.1988	0.1996	0.2003	0.2011	0.2019	0.2027	0.2035	0.2042	0.2050
.61	.2058	.2066	.2074	.2082	.2090	.2098	.2106	.2114	.2122	.2130
.62	.2138	.2146	.2155	.2163	.2171	.2180	.2188	.2196	.2204	.2213
.63	.2221	.2230	.2238	.2246	.2255	.2264	.2272	.2280	.2289	.2298
.64	.2306	.2315	.2324	.2333	.2342	.2350	.2359	.2368	.2377	.2386
.65	.2395	.2404	.2418	.2422	.2431	.2440	.2450	.2459	.2468	.2477
.66	.2486	.2495	.2505	.2514	.2524	.2533	.2542	.2552	.2561	.2571
.67	.2580	.2589	.2597	.2606	.2615	.2624	.2632	.2641	.2650	.2658
.68	.2667	.2678	.2689	.2700	.2711	.2722	.2734	.2745	.2756	.2767
.69	.2778	.2788	.2799	.2810	.2820	.2830	.2841	.2852	.2862	.2872

TABLE 8.—*Backwater function* (*F*) *for a dam or obstruction*—Continued.

[The column headings are hundredths for values of $\frac{D}{d}$ from zero to 0.29, thousandths for values of $\frac{D}{d}$ from 0.30 to 0.899, and ten-thousandths for values of $\frac{D}{d}$ from 0.900 to 0.999.]

$\frac{D}{d}$	0	1	2	3	4	5	6	7	8	9
	F	*F*	*F*	*F*	*F*	*F*	*F*	*F*	*F*	*F*
0.70	0.2883	0.2894	0.2905	0.2915	0.2926	0.2937	0.2948	0.2959	0.2969	0.2980
.71	.2991	.3002	.3013	.3025	.3036	.3047	.3058	.3070	.3081	.3093
.72	.3104	.3116	.3127	.3139	.3150	.3162	.3174	.3186	.3197	.3209
.73	.3221	.3233	.3245	.3258	.3270	.3282	.3294	.3306	.3319	.3331
.74	.3343	.3356	.3368	.3381	.3393	.3406	.3419	.3432	.3444	.3457
.75	.3470	.3483	.3496	.3510	.3523	.3536	.3549	.3563	.3576	.3590
.76	.3603	.3617	.3630	.3644	.3657	.3671	.3685	.3699	.3713	.3727
.77	.3741	.3755	.3770	.3784	.3799	.3813	.3828	.3842	.3857	.3871
.78	.3886	.3901	.3916	.3932	.3947	.3962	.3977	.3993	.4008	.4024
.79	.4039	.4055	.4070	.4086	.4101	.4117	.4133	.4149	.4166	.4182
0.80	0.4198	0.4215	0.4231	0.4248	0.4264	0.4281	0.4298	0.4315	0.4333	0.4350
.81	.4367	.4384	.4402	.4419	.4437	.4454	.4472	.4490	.4508	.4526
.82	.4544	.4563	.4581	.4600	.4618	.4637	.4656	.4675	.4695	.4714
.83	.4733	.4753	.4772	.4792	.4811	.4831	.4851	.4871	.4892	.4912
.84	.4932	.4953	.4974	.4995	.5016	.5037	.5059	.5081	.5102	.5124
.85	.5146	.5168	.5191	.5213	.5236	.5258	.5281	.5304	.5328	.5351
.86	.5374	.5398	.5422	.5446	.5470	.5494	.5519	.5544	.5569	.5594
.87	.5619	.5645	.5671	.5697	.5723	.5749	.5776	.5803	.5830	.5857
.88	.5884	.5912	.5940	.5969	.5997	.6025	.6055	.6084	.6114	.6143
.89	.6173	.6204	.6235	.6265	.6296	.6327	.6359	.6392	.6424	.6457
0.900	0.6489	0.6492	0.6496	0.6499	.6502	0.6506	0.6509	0.6512	0.6515	0.6519
.901	.6522	.6525	.6529	.6532	.6536	.6539	.6542	.6546	.6549	.6553
.902	.6556	.6559	.6563	.6566	.6570	.6573	.6576	.6580	.6583	.6587
.903	.6590	.6594	.6597	.6600	.6604	.6608	.6611	.6614	.6618	.6622
.904	.6625	.6629	.6632	.6636	.6639	.6642	.6646	.6650	.6653	.6656
.905	.6660	.6664	.6667	.6670	.6674	.6678	.6681	.6684	.6688	.6692
.906	.6695	.6698	.6702	.6706	.6709	.6712	.6716	.6720	.6723	.6726
.907	.6730	.6734	.6737	.6741	.6744	.6748	.6752	.6755	.6759	.6762
.908	.6766	.6770	.6773	.6777	.6780	.6784	.6788	.6791	.6795	.6798
.909	.6802	.6806	.6809	.6813	.6817	.6820	.6824	.6828	.6832	.6835
0.910	0.6839	0.6843	0.6846	0.6850	0.6854	0.6858	0.6861	0.6865	0.6869	0.6872
.911	.6876	.6880	.6884	.6887	.6891	.6895	.6899	.6903	.6906	.6910
.912	.6914	.6918	.6922	.6925	.6929	.6933	.6937	.6941	.6944	.6948
.913	.6952	.6956	.6960	.6963	.6967	.6971	.6975	.6979	.6982	.6986
.914	.6990	.6994	.6998	.7002	.7006	.7010	.7013	.7017	.7021	.7025
.915	.7029	.7033	.7037	.7041	.7045	.7049	.7053	.7057	.7061	.7065
.916	.7069	.7073	.7077	.7081	.7085	.7089	.7093	.7097	.7101	.7105
.917	.7109	.7113	.7117	.7121	.7125	.7129	.7133	.7137	.7141	.7145
.918	.7149	.7153	.7157	.7161	.7165	.7170	.7174	.7178	.7182	.7186
.919	.7190	.7194	.7198	.7202	.7206	.7210	.7215	.7219	.7223	.7227
0.920	0.7231	0.7235	0.7239	0.7244	0.7248	0.7252	0.7256	0.7260	0.7265	0.7269
.921	.7273	.7277	.7281	.7286	.7290	.7294	.7298	.7302	.7307	.7311
.922	.7315	.7319	.7324	.7328	.7332	.7336	.7341	.7345	.7349	.7354
.923	.7358	.7362	.7367	.7371	.7375	.7380	.7384	.7388	.7392	.7397
.924	.7401	.7405	.7410	.7414	.7419	.7423	.7427	.7432	.7436	.7441
.925	.7445	.7450	.7454	.7458	.7463	.7468	.7472	.7476	.7481	.7486
.926	.7490	.7494	.7499	.7504	.7508	.7512	.7517	.7522	.7526	.7530
.927	.7535	.7540	.7544	.7549	.7553	.7558	.7563	.7567	.7572	.7576
.928	.7581	.7586	.7590	.7595	.7600	.7604	.7609	.7614	.7619	.7623
.929	.7628	.7633	.7637	.7642	.7647	.7652	.7656	.7661	.7666	.7670

TABLE 8.—*Backwater function (F) for a dam or obstruction*—Continued.

[The column headings are hundredths for values of $\frac{D}{d}$ from zero to 0.29, thousandths for values of $\frac{D}{d}$ from 0.30 to 0.899, and ten-thousandths for values of $\frac{D}{d}$ from 0.900 to 0.999.]

$\frac{D}{d}$	0	1	2	3	4	5	6	7	8	9
	F	F	F	F	F	F	F	F	F	F
0.930	0.7675	0.7680	0.7685	0.7689	0.7694	0.7699	0.7704	0.7709	0.7713	0.7718
.931	.7723	.7728	.7724	.7738	.7743	.7748	.7752	.7757	.7762	.7767
.932	.7772	.7777	.7782	.7787	.7792	.7796	.7801	.7806	.7811	.7816
.933	.7821	.7826	.7831	.7836	.7841	.7846	.7851	.7856	.7861	.7866
.934	.7871	.7876	.7881	.7886	.7891	.7896	.7902	.7907	.7912	.7917
.935	.7922	.7927	.7932	.7937	.7942	.7948	.7953	.7958	.7963	.7968
.936	.7973	.7978	.7984	.7989	.7994	.8000	.8005	.8010	.8015	.8021
.937	.8026	.8031	.8037	.8042	.8047	.8052	.8058	.8063	.8068	.8074
.938	.8079	.8084	.8090	.8095	.8101	.8106	.8111	.8117	.8122	.8128
.939	.8133	.8138	.8144	.8150	.8155	.8160	.8166	.8172	.8177	.8182
0.940	0.8188	0.8194	0.8199	0.8205	0.8210	0.8216	0.8222	0.8227	0.8233	0.8238
.941	.8244	.8250	.8256	.8262	.8268	.8274	.8280	.8286	.8292	.8298
.942	.8301	.8307	.8313	.8318	.8324	.8330	.8336	.8342	.8347	.8353
.943	.8359	.8365	.8371	.8377	.8383	.8388	.8394	.8400	.8406	.8412
.944	.8418	.8424	.8430	.8436	.8442	.8448	.8454	.8460	.8466	.8472
.945	.8478	.8484	.8490	.8496	.8502	.8508	.8515	.8521	.8527	.8533
.946	.8539	.8545	.8552	.8558	.8564	.8570	.8577	.8583	.8589	.8596
.947	.8602	.8608	.8615	.8621	.8627	.8634	.8640	.8646	.8652	.8659
.948	.8665	.8672	.8678	.8684	.8691	.8698	.8704	.8710	.8717	.8724
.949	.8730	.8736	.8743	.8750	.8756	.8762	.8769	.8776	.8782	.8788
0.950	0.8795	0.8802	0.8809	.8815	0.8822	0.8829	0.8836	0.8843	0.8849	0.8856
.951	.8863	.8870	.8877	.8883	.8890	.8897	.8904	.8911	.8917	.8924
.952	.8931	.8938	.8945	.8952	.8959	.8966	.8974	.8981	.8988	.8995
.953	.9002	.9009	.9016	.9023	.9030	.9038	.9045	.9052	.9059	.9066
.954	.9073	.9080	.9088	.9095	.9103	.9110	.9117	.9125	.9132	.9140
.955	.9147	.9154	.9162	.9169	.9177	.9184	.9191	.9199	.9206	.9214
.956	.9221	.9229	.9236	.9244	.9252	.9260	.9267	.9275	.9283	.9290
.957	.9298	.9306	.9314	.9321	.9329	.9337	.9345	.9353	.9360	.9368
.958	.9376	.9384	.9392	.9400	.9408	.9416	.9425	.9433	.9441	.9449
.959	.9457	.9465	.9473	.9482	.9490	.9498	.9506	.9514	.9523	.9531
0.960	0.9539	0.9548	0.9556	0.9564	0.9573	0.9582	0.9590	0.9598	0.9607	0.9616
.961	.9624	.9632	.9641	.9650	.9658	.9666	.9675	.9684	.9692	.9700
.962	.9709	.9718	.9727	.9736	.9745	.9754	.9763	.9772	.9781	.9790
.963	.9799	.9808	.9817	.9826	.9835	.9844	.9854	.9863	.9872	.9881
.964	.9890	.9899	.9909	.9918	.9928	.9937	.9947	.9956	.9966	.9975
.965	.9985	.9994	1.0004	1.0018	1.0023	1.0032	1.0042	1.0051	1.0061	1.0070
.966	1.0080	1.0090	1.0100	1.0110	1.0120	1.0130	1.0140	1.0150	1.0160	1.0170
.967	1.0181	1.0191	1.0201	1.0211	1.0221	1.0231	1.0241	1.0251	1.0261	1.0271
.968	1.0282	1.0292	1.0303	1.0314	1.0324	1.0335	1.0346	1.0356	1.0367	1.0378
.969	1.0389	1.0399	1.0410	1.0421	1.0432	1.0443	1.0453	1.0464	1.0475	1.0486
0.970	1.0497	1.0508	1.0519	1.0530	1.0542	1.0553	1.0565	1.0576	1.0587	1.0598
.971	1.0610	1.0622	1.0633	1.0645	1.0657	1.0668	1.0680	1.0692	1.0704	1.0715
.972	1.0727	1.0739	1.0751	1.0763	1.0775	1.0788	1.0800	1.0812	1.0824	1.0836
.973	1.0848	1.0861	1.0873	1.0886	1.0898	1.0911	1.0924	1.0936	1.0949	1.0961
.974	1.0974	1.0987	1.1000	1.1018	1.1026	1.1040	1.1053	1.1066	1.1079	1.1092
.975	1.1105	1.1119	1.1132	1.1146	1.1159	1.1173	1.1187	1.1200	1.1214	1.1227
.976	1.1241	1.1255	1.1269	1.1284	1.1298	1.1312	1.1326	1.1340	1.1355	1.1369
.977	1.1383	1.1398	1.1413	1.1427	1.1442	1.1457	1.1472	1.1487	1.1501	1.1516
.978	1.1531	1.1546	1.1562	1.1578	1.1593	1.1608	1.1624	1.1640	1.1655	1.1670
.979	1.1686	1.1702	1.1718	1.1735	1.1751	1.1767	1.1783	1.1799	1.1816	1.1832

TABLE 8.—*Backwater function (F) for a dam or obstruction*—Continued.

[The column headings are hundredths for values of $\frac{D}{d}$ from zero to 0.29, thousandths for values of $\frac{D}{d}$ from 0.30 to 0.899, and ten-thousandths for values of $\frac{D}{d}$ from 0.900 to 0.999.]

$\frac{D}{d}$	0	1	2	3	4	5	6	7	8	9
	F	*F*	*F*	*F*	*F*	*F*	*F*	*F*	*F*	*F*
0.980	1.1848	1.1865	1.1882	1.1899	1.1916	1.1934	1.1951	1.1968	1.1985	1.2002
.981	1.2019	1.2037	1.2055	1.2073	1.2091	1.2109	1.2127	1.2145	1.2163	1.2181
.982	1.2199	1.2218	1.2237	1.2256	1.2275	1.2294	1.2314	1.2333	1.2352	1.2371
.983	1.2390	1.2410	1.2430	1.2451	1.2471	1.2491	1.2511	1.2531	1.2552	1.2572
.984	1.2592	1.2614	1.2635	1.2656	1.2678	1.2700	1.2721	1.2742	1.2764	1.2786
.985	1.2807	1.2830	1.2853	1.2876	1.2899	1.2922	1.2945	1.2968	1.2991	1.3014
.986	1.3037	1.3062	1.3086	1.3111	1.3136	1.3160	1.3185	1.3210	1.3235	1.3259
.987	1.3284	1.3311	1.3337	1.3364	1.3391	1.3418	1.3444	1.3471	1.3498	1.3524
.988	1.3551	1.3580	1.3609	1.3638	1.3667	1.3696	1.3725	1.3754	1.3783	1.3812
.989	1.3841	1.3873	1.3905	1.3936	1.3968	1.4000	1.4032	1.4064	1.4095	1.4127
0.990	1.4159	1.4194	1.4229	1.4264	1.4299	1.4334	1.4370	1.4405	1.4440	1.4475
.991	1.4510	1.4549	1.4588	1.4628	1.4667	1.4706	1.4745	1.4784	1.4824	1.4863
.992	1.4902	1.4947	1.4991	1.5036	1.5080	1.5125	1.5170	1.5214	1.5259	1.5303
.993	1.5348	1.5399	1.5451	1.5502	1.5553	1.5604	1.5656	1.5707	1.5758	1.5810
.994	1.5861	1.5922	1.5983	1.6043	1.6104	1.6165	1.6226	1.6287	1.6347	1.6408
.995	1.6469	1.6543	1.6618	1.6692	1.6767	1.6841	1.6915	1.6990	1.7064	1.7139
.996	1.7213	1.7309	1.7405	1.7501	1.7597	1.7692	1.7788	1.7884	1.7980	1.8076
.997	1.8172	1.8307	1.8442	1.8577	1.8712	1.8848	1.8982	1.9118	1.9253	1.9388
.998	1.9523	1.9754	1.9985	2.0216	2.0447	2.0678	2.0910	2.1141	2.1378	2.1603
.999	2.1834	....	....	....	....	....	....	....	....	....
1.000	....	....	....	....	....	....	....	....	....	....

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CLASSIFICATION OF THE PUBLICATIONS OF THE UNITED STATES GEOLOGICAL SURVEY.

[Water-Supply Paper No. 150.]

The serial publications of the United States Geological Survey consist of (1) Annual Reports, (2) Monographs, (3) Professional Papers, (4) Bulletins, (5) Mineral Resources, (6) Water-Supply and Irrigation Papers, (7) Topographic Atlas of United States—folios and separate sheets thereof, (8) Geologic Atlas of the United States—folios thereof. The classes numbered 2, 7, and 8 are sold at cost of publication; the others are distributed free. A circular giving complete lists may be had on application.

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1. A limited number are delivered to the Director of the Survey, from whom they may be obtained, free of charge (except classes 2, 7, and 8), on application.

2. A certain number are delivered to Senators and Representatives in Congress, for distribution.

3. Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be had at prices slightly above cost.

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THE DIRECTOR,
UNITED STATES GEOLOGICAL SURVEY,
WASHINGTON, D. C.

JANUARY, 1906.

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Water-Supply and Irrigation Paper No. 151 Series L, Quality of Water, 11

DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

FIELD ASSAY OF WATER

BY

MARSHALL O. LEIGHTON

WASHINGTON

GOVERNMENT PRINTING OFFICE

1905

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DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., June 9, 1905.

SIR: I transmit herewith a manuscript entitled "Field Assay of Water," by Marshall O. Leighton, and request that it be published as one of the series of Water-Supply and Irrigation Papers.

In this manuscript are described and discussed the methods which have for some time been used with success in connection with the investigations into the quality of water in various parts of the United States carried on by the division of hydro-economics. As the methods have proved of value, it is believed that their publication in the form submitted will be of general interest.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

FIELD ASSAY OF WATER

By M. O. Leighton.

INTRODUCTION.

A chemist aims to secure exceeding refinement in analytical methods and results. He seldom considers whether or not a method is sufficiently exact for certain broad purposes. The fact that it is incomplete, approximate, or susceptible of refinement is to him sufficient reason for improving or rejecting it at the first opportunity.

The scrutiny to which chemical methods have been subjected in the endeavor to secure exact results has led in many cases to processes so complicated and expensive that in commercial work the advantages do not compensate for the increased cost and delay which the methods involve. The result has been that the chemical profession distinguishes between two classes of chemical methods which differ in degree of accuracy. The first includes the exact methods, which afford results as nearly perfect as chemical procedure will permit. Such methods are used in all cases where minute differences in analysis would cause errors in interpretation or in subsequent chemical procedure. The second class consists of "commercial methods," so called because the results obtained by them, while departing from the actual truth, are sufficiently accurate to insure the profitable conduct of industrial chemical processes without appreciable error or waste. Methods of the first class are the product of chemistry, while those of the second are used in response to the demands of expediency—they are good enough for the purposes for which they are used.

In no branch of chemistry are approximate results more serviceable than in the analysis of water for hydro-economic surveys, or surveys made to determine the value of water and its applicability for use in domestic supply, boilers, industries, etc. Under the conditions which generally prevail it is necessary to resort to long, tedious, and expensive processes in order to secure a determination of the character and amount of foreign constituents in water. It is the practice in such cases to secure a sample of the water and transport it to a laboratory, where, after conventional delays, it is passed through the usual course of analysis.

There has in the past been surprisingly little discrimination used with reference to the selection of determinations for specific purposes, and as a general rule the same procedure has usually been followed without regard to the object of the particular investigation. If the purpose of the analysis is to determine the incrusting constituents, the course pursued has been to follow the entire analytical procedure. If, on the other hand, it is desired to determine the amount of organic pollution in a water and show its value for domestic use, the chemist forthwith begins the round of nitrogen determinations, and closes with a statement of the oxygen consumed and the number of bacteria per cubic centimeter. In only a few well-known laboratories has this rule been violated, and such is the conservatism in the chemical profession that it will probably be largely followed in future. Conservatism is the safeguard of science and one of the most commendable qualities of a chemist, but an excess is sometimes almost as bad as a deficiency.

SANITARY ANALYSES.

The requisites to be met by a water in almost every line of special development are broad and flexible. In the sanitary analysis certain results receive certain interpretations, which remain generally unchanged if the results are varied by 1, 2, 3, or sometimes even 10 per cent. A strange feature in connection with sanitary analyses of water is that, in addition to insisting upon superrefinement, many chemists persist in making determinations that are admittedly to no purpose. It is a common thing to see an analyst's report of a water containing the results of determinations of albuminoid and free ammonia, nitrates, and nitrites, accompanied by a footnote stating that these results are unworthy of trust and mean very little, except to verify conclusions made from inspection of the territory from which the water was taken. In case such conclusions do not agree with the analytical evidence, the latter is invariably discredited.

It is to be hoped that some day the great and growing swarm of water analysts will awaken to the fact that sanitary analyses, as generally applied and interpreted, are but a succession of unrelated absurdities. Water experts, who encounter real problems, who must use analytical data as a basis for the design and construction of purification plants, and whose varied experience has taught them that in the United States the waters are as diverse in character as the climates, have learned a few things not taught in text-books nor anticipated in the beautiful theory of the oxidation of organic matter.

The occasional isolated sanitary analysis of water is positively without value. There are throughout the country numerous State, municipal, and private laboratories in which sanitary analyses are carried on. The water analyzed to-day may be from a well, to-morrow from a brook, and the next day from a pond. From the results of a single analysis wise and ponderous verdicts are sent broadcast, and the eager, waiting public is duly impressed. No one understands how singularly misleading a sanitary analysis of water can be until he has examined the results of such analyses of samples taken daily or hourly from the same source; then he sees that in general only a few single analyses in the group contain results which would admit of the interpretation that is finally placed upon the series.

If there is at hand a well-defined problem which involves the consideration of nitrogenous matter and the state in which it appears in a water, certain daily nitrogen determinations are of undoubted value; not, however, by reason of the absolute amounts which are revealed in each determination, but by reason of the daily relations and variations which appear in the successive analyses, and upon which interpretations can be placed. This statement, it should be emphasized, refers almost entirely to water slightly or moderately polluted, and does not include sewage. The organic matter normally occurring in a natural water, or what may be more accurately described as a highly dilute sewage, is, after all, practically infinitesimal in amount. The difficulties attendant upon a determination of nitrogen in its various forms and the true interpretation of the results, grow less and less as the amount of organic matter is increased. Yet, even with strong sewages some of the determinations, such as albuminoid ammonia and nitrites, are not usually productive of valuable information.

In an article entitled "The composition of sewage in relation to problems of disposal,"[a] Mr. George W. Fuller discusses in a characteristically clear manner an experiment which illustrates the apparent futility of the albuminoid-ammonia determination, as follows:

Illustrative of the varying relation of nitrogen in the form of albuminoid ammonia to the total organic nitrogen present in raw sewage, there are given below in a table the results of an experiment made in the Lawrence laboratory and published in the 1894 report of the Massachusetts State board of health, page 461. A bottle of fresh sewage was analyzed just after its collection and again at frequent intervals, allowing the natural decomposition processes to take place at room temperature. In this table it is seen that fresh sewage contains dissolved oxygen, coming, of course, from the water supply which forms the principal portion of the sewage. It also contains nitrogen in the form of nitrates, as well as other salts which are completely oxidized. Through the agency of the bacteria and the oxygen dissolved in the water and yielded by the oxidized salts, the carbon of the organic matter is oxidized and the organic nitrogen uniting with the hydrogen forms free ammonia.

[a] Technology Quarterly, June, 1903, pp. 143–144.

Changes in composition occurring in fresh Lawrence sewage upon standing in a bottle in the laboratory.

[Parts per million.]

Date of examination.		Free ammonia.	Nitrogen as—						Nitrites and nitrates.	Total nitrogen.	Oxygen consumed.		Per cent dissolved oxygen.	Bacteria per cubic centimeter.
			Albuminoid ammonia.			Organic (Kjeldahl).								
Day.	Hour.		Dissolved.	Suspended.	Total.	Dissolved.	Suspended.	Total.			Two minutes.	Five minutes.		
March 11	10.30 a. m.	18.5	5.8	2.1	7.9	31.4	9.1	40.3	3.5	62.2	85.0	132.0	57	1,190,000
March 11	12.30 p. m.	20.6	5.8	2.4	8.2	28.9	11.4	40.3	3.1	63.9	81.0	142.0	60	1,065,000
March 11	3.00 p. m.	21.0	5.8	2.5	8.3	28.9	10.7	39.6	2.9	63.4	85.0	136.0	60	1,505,000
March 11	6.00 p. m.	23.4	5.6	2.7	8.3	27.8	9.9	37.7	2.5	63.6	85.0	133.0	30	1,530,000
March 12	8.00 a. m.	40.7	4.9	3.9	8.8	11.6	12.2	23.8	0.3	64.6	74.0	107.0	0	20,475,000
March 12	12.00 m.	41.2	4.9	4.4	9.3	11.6	8.2	19.8	0.2	61.0	69.0	98.0	0	23,100,000
March 12	5.00 p. m.	41.2	4.2	4.2	8.4	11.6	10.1	21.7	0.0	62.7	69.0	98.0	0	20,000,000
March 13	10.30 a. m.	42.0	4.7	3.5	8.2	11.6	8.2	19.8	0.0	61.6	60.0	89.0	0	12,810,000
March 14	10.30 a. m.	41.2	3.3	4.5	7.8	9.4	8.4	17.8	0.0	58.8	55.0	72.0	0	11,235,000
March 15	10.30 a. m.	41.2	3.4	4.3	7.7	9.3	8.3	17.6	0.0	57.6	49.0	68.0	0	6,825,000
March 16	10.30 a. m.	41.2	3.5	4.1	7.6	7.5	8.9	16.4	0.0	57.4	48.0	72.0	0	4,485,000
March 18	10.30 a. m.	42.0	3.0	3.8	6.8	7.4	6.9	14.3	0.0	56.1	51.0	70.0	0	3,420,000
March 19	10.30 a. m.	42.8	3.0	3.9	8.9	5.9	7.8	13.7	0.0	56.3	50.0	71.0	0	2,341,000

Thus it is seen from the results in the table, page [12], that the dissolved oxygen and the nitrates gradually disappear, the bacteria for a time increase, the oxygen consumed (carbonaceous matter) decreases, the nitrogen as free ammonia increases, and the organic nitrogen (Kjeldahl) decreases. The nitrogen as albuminoid ammonia, however, remains approximately constant, notwithstanding that more than 20 parts of organic nitrogen are changed to free ammonia, and some 5 or 6 parts of free nitrogen escape into the atmosphere.

The nitrite determination, which has been regarded by many as one of the most valuable pollution indicators, fluctuates in a stream or reservoir according to the amount of available oxygen rather than the amount of organic matter undergoing oxidation. It will rise and fall in amount when there is a positive certainty that it can not be due to increase or decrease of organic pollution. Nevertheless, many an interpretation has been made largely on the evidence presented by this determination.

By far the greater number of sanitary water analyses reported include the determination of oxygen consumed, a test which is dependent upon so many features that as a whole the great mass of determinations which have been made are valueless for purposes of comparison. The following table, compiled by Mr. George W. Fuller, shows clearly the relative results of the determinations made according to different methods. In short, the only practical value of the "oxygen-consumed" determination is in its application to highly polluted waters of the same general character and origin, and then only for purposes of comparison between the successive determinations made by absolutely the same method. In other words, it is essentially a sewage determination.

Approximate comparison of average amounts of oxygen consumed by sewage and sewage effluents as shown by different methods.

Method.	Temperature of solution.	Period of contact.	Relative results.
Kübel, as practiced at Boston and generally in America.	Boiling	5 minutes	1.00
Kübel, as practiced at Lawrence, Mass	do	2 minutes	.65
Kübel, as practiced in Germany	do	10 minutes	1.25
English official tests	80° F	3 minutes	.20
	do	15 minutes	.35
	do	4 hours	.60
"Absolute" oxygen consumed	Boiling	do	4.00

INORGANIC ANALYSES.

In the determination of inorganic constituents in a water it would make no difference, in the decision to accept or reject that water for boiler use, for irrigation, or for manufacturing, if the harmful constituents were contained, for example, in the proportion of 40, 44, or 48 grains per gallon. The lines dividing good and bad water for boiler purposes are very broad. If a water contains a certain amount of incrusting constituents and a method used is inaccurate to a limit of 5, or even 10, per cent, it would not lead to the acceptance of a bad water or the rejection of a good one. The "good" and "bad" provinces are approached too gradually to admit of such consequences.

Another error arises from the conventional methods of expression of results. If the analyst finds that a water contains certain amounts of calcium, magnesium, sodium, and potassium, and certain equivalents of the carbonate, sulphate, and chloride radicals, he unites these substances according to methods which are apparently not uniform and entitled to little scientific justification. It is a well-known fact that if several chemists, each independent of every other, analyze a certain water, there will almost invariably be wide differences in the expression of results.

Dr. F. W. Clarke, chief chemist of the United States Geological Survey, in a recent communication has given an excellent illustration of the various hypothetical combinations which may be made of the results of a single analysis of water. The statements are set forth in the following table. Each series of combinations is based upon a generally accepted hypothesis, and each represents a water that is totally different from the others.

Analysis of water from artesian well at Macomb, Ill.[a]

[Grams per liter.]

Statement in ionic form.		Statement according to hypothetical combinations.				
SiO_2	0.0105	KCl		0.0452	0.0181	0.0211
Al_2O_3	.0013	NaCl	0.8968	.8384	.7164	.8172
Fe_2O_3	.0023	$CaCl_2$			.0877	.0292
SO_4	.9991	$MgCl_2$		.0154	.0571	.0242
CO_3	.3953	K_2SO_4	.0528		.0211	.0246
Cl	.5418	Na_2SO_4	.4649	1.4781	1.2495	1.0642
Na	.8086	$CaSO_4$	.5450		.1218	.2223
K	.0237	$MgSO_4$	.3360		.0711	.1357
Ca	.1581	K_2CO_3			.0084	.0028
Mg	.0672	Na_2CO_3	.6983		.2793	.3259
		$CaCO_3$		.3952	.2278	.2076
		$MgCO_3$		.2215	.1355	.1190
		Al_2O_3[b]	.0013	.0013	.0013	.0013
		Fe_2O_3[b]	.0020	.0020	.0020	.0020
		SiO_2[b]	.0105	.0105	.0105	.0105
			3.0076	3.0076	3.0076	3.0076

[a] By George Steiger, laboratory of United States Geological Survey.
[b] Al_2O_3, Fe_2O_3, SiO_2, conventionally regarded as colloidal.

The inorganic constituents of a water should invariably be expressed as positive and negative ions, and if so expressed the result determined according to approximate methods is as valuable as the expression of precisely determined constituents united according to the individual ideas of the analyst.

GENERAL OBSERVATIONS.

A practical disadvantage in chemical water surveys arising from the insistence on refined methods of analysis and lack of discrimination in the choice of specific determinations is the delay which arises in securing valuable information with reference to wide areas. Months and even years have been spent upon water surveys covering only a comparatively small portion of the country. Two examples are here cited.

The most important chemical survey in the United States has been carried on since 1888 by the Commonwealth of Massachusetts. This survey work may safely be taken as the standard in this or any other country. During this period of sixteen years the appropriations for the work have been on an average about $30,000 a year, making a total cost of not far from $480,000. A large part of this sum has been used to pay expenses of experimentation and can not be charged to water survey. The work was confined to Massachusetts.

In the year 1898 the Ohio State board of health commenced an examination of the principal streams of the State, making monthly analyses of samples of water taken from numerous points along the various streams. This work was continued five years before the State was covered, and there resulted merely a large number of periodical analyses in sets of twelve, showing the character of the stream water and its variation according to local conditions.

These two cases are typical. No one would claim that all the results could have been reached by the use of field methods. Undoubtedly a large part of them could have been obtained far more cheaply and quickly, and there would have been no loss to the cause of pure water, nor to science generally, had some of the determinations been omitted.

Among water analysts there seems to be a general tendency to attack every water problem as though the object were to prove its fitness or unfitness for drinking purposes. In many investigations it is well known at the outset that the water can not be used for domestic purposes and the problem is of quite another character. The conventional grind of nitrogen determinations has been made to do service in almost every conceivable water problem. It has been used frequently in investigating pollution problems in which organic matter had absolutely no part. A study of the results obtained in many laboratories will show that in the routine work a number of determinations could be omitted without detriment. These facts are mentioned in order to emphasize the point that if care and discrimination were used in the selection of tests the work necessary in carrying on chemical surveys might be decreased and the money available for such work might be distributed over a wider field; results of more immediate use might be secured and the completion of a chemical survey would not be postponed for the benefit of future generations. "Commercial methods" serve useful purposes in manufacturing; the success of enormous industrial plants is dependent upon them, and in water surveys they would at least be businesslike.

It was these considerations, in connection with the knowledge of the vast areas covered by the United States, which led the United States Geological Survey, through its hydro-economic division, to investigate the practicability of employing field methods for the determination of important characteristics of water. It was realized that if there could be provided a few simple tests, the apparatus for which could be taken into the field and used on the spot, large areas might be covered in a short time, and if it were necessary for any reason to make periodical determinations the cost of the work would still be small and the total would not run up into the large sums which have been spent in such investigations. In general, the testimony of a large number of approximate results is far more representative of actual conditions than that of one or two refined analyses.

The idea of testing waters in the field is by no means new. Very successful field tests have been carried on by numerous authorities. Probably one of the most successful systems now in use is that of the Bureau of Soils, Department of Agriculture. Another excellent field outfit for sanitary analysis has been devised and used with satisfaction by Mrs. Ellen H. Richards. Several others are noteworthy. They all involve the use of considerable apparatus and the carrying of standard solutions. Specially equipped wagons are necessary in some cases. In others the variety of determinations is limited or the equipment can not be used in an extensive circuit without renewal of reagents. The difficulty of carrying solutions and complex apparatus into the field is obvious. The ideal equipment is one which can be carried on journeys afoot or on horseback without much fatigue. The sources of useful or desirable water supply are not distributed with reference to the railroad or wagon routes and the field man must often climb mountains or trace obscure trails to accomplish his purpose. The outfit should contain a sufficient supply of reagents to serve for a large number of determinations without renewal. The processes should be rapid and the results fairly accurate and comprehensive. Finally, the equipment should be provided with material for so various a series of determinations that with proper discrimination the essential characteristics of a water may be shown, whether the purposes be domestic or industrial water supply, irrigation, or any other special line of utilization. Volumetric methods requiring the use of burettes are objectionable; gravimetric methods are impossible. Therefore, in the Survey's study of field methods the whole matter developed into a question of choosing the most useful determinations and so modifying the volumetric methods that their use would be practicable, while at the same time they would give a degree of accuracy sufficiently close for all practical purposes.

To the methods hereinafter proposed the term "assay" readily lends itself. There is no attempt at water analysis. The plan contemplates the determination of ingredients which give to water certain well-known characteristics. The methods and the suggestions with reference to their application are only tentative and will be modified as experience may dictate. As they stand they are the result of extended experimentation, and the tests to which they have been put show that they are practicable. They have been found to be more nearly accurate than was at first anticipated, though this fact, it is believed, has not greatly increased their usefulness for the purposes in view. By their use, combined with a fair amount of common sense, the essential characteristics of waters can be ascertained at small expense. In almost every situation in which such determinations are significant they will afford sufficiently satisfactory data. In the case of finely balanced considerations of a purely physical, chemical, or geologic nature, however, they are practically useless. They are intended for practical purposes and have no place in pure science.

FIELD DETERMINATIONS.

The following determinations are described on subsequent pages:

1. Turbidity.
2. Color.
3. Iron.
4. Chlorine or total chlorides.
5. Total hardness.
6. Alkalinity.
7. Normal carbonates.
8. Bicarbonates.
9. Total sulphates.
10. Calcium.

It should be stated at the outset that the successful operation of these methods depends, as in all chemical procedure, upon the manner in which they are applied. A failure to insist upon strict compliance with the rules laid down may result in total failure.

SUSPENDED MATTER.

The turbidity of water is that property which is imparted to it by substance carried in suspension. In many parts of the United States waters are often extremely muddy, and when this condition is maintained during long periods it becomes one of the most serious difficulties with which the water-supply engineer has to deal. Turbid water is objectionable for domestic use. In industrial operations, especially in those in which water enters into manufacturing processes, turbidity is a factor which, if not removed, may exert a harmful influence upon the manufactured products. It is also important in connection with irrigation works. One of the serious troubles in Western reservoirs is loss of storage capacity due to silt deposits. In the construction of irrigation canals the amount of turbidity usually carried by the water often determines the grade of the canal. Such canals must have grade sufficient to cause the flowing water to carry along suspended matter and not allow it to settle. If this is not taken into consideration, the maintenance of the irrigation system becomes extremely expensive, and cases have occurred where the canals have been practically filled by the deposits of suspended matter.

The suspended substances causing turbidity are of various characters. They are found often in a flocculent condition, settling readily when the water which carries them becomes quiescent. On the other hand, the turbid matter is often made up of minute particles of clay, so fine that they pass through certain filtering media. In some cases the problem of removing turbidity from the water is so difficult that the process which may be successful is so radical that it will remove also dissolved organic material and even a large number of the bacteria.

METHODS OF DETERMINATION.

There are several methods of estimating the proportion of suspended matter in a water, all but one of which have their particular fields of usefulness. The first is merely a statement of the observer's opinion of the degree of turbidity, such as "very slight," "slight," "distinct," or "decided." Although this method of estimation has no real value, it is used by many water chemists.

The second method is also based upon the appearance of the water, but differs from the first in that a definite and fairly well-fixed basis of comparison is provided. A water containing no suspended matter is practically transparent, but matter in suspension intercepts the rays of transmitted light. An observer can see objects distinctly through a body of clear water, but as the water becomes more and more muddy the objects can be seen less and less distinctly until they are quite lost to view. Now, it has been found by experiment that there is a fairly definite relation between the proportion of light rays intercepted and the amount of matter in suspension. This relation varies somewhat with the character of the suspended matter and with the size of the particles, but for the purpose to which this method of measurement is applicable the variations do not often seriously affect the interpretations placed upon the results. The details of the method will be explained on later pages under the caption "Turbidity." For the present it will be sufficient to state that it has found its greatest usefulness in connection with the adaptation and operation of water-filtration plants and sewage-disposal works.

The third method of measuring the amount of suspended matter in water consists in separating it from a weighed portion of the fluid by filtration, weighing the filtered water, and stating the difference between the two weights as suspended matter. The form of statement commonly used is parts of suspended matter per million of water, milligrams per liter, or some other comprehensive proportion. This is undoubtedly the best method of determination, as it is relatively accurate and can be used in the study of all water problems. Its practical disadvantage is that the determination requires a large amount of time and can not be economically performed in serial investigations without considerable equipment and tedious labor. It is also true that the processes for which a knowledge of suspended matter is necessary will generally in practical work be as well served by an approximate determination as by a precise one; therefore the cruder methods, based upon photometry, are more often used.

There is, however, no necessarily constant relation between the weight of suspended matter in a given volume of water and the turbidity produced. A certain weight of suspended substance of one kind does not usually produce the same degree of turbidity as a similar weight of another substance. In other words, the turbidity determination takes no account of the character, weight, or volume of the suspended matter. This has been clearly demonstrated by Mr. Robert Spurr Weston in the report on Water-Purification Investigation and on Plans Proposed for Sewerage and Waterworks Systems made to the sewerage and water board of New Orleans, La., pages 27 and 28. To overcome the errors above cited, Mr. Weston has proposed the use of a "turbidity coefficient," as follows:

All optical methods for the determination of turbidity are naturally compared with the gravimetric determination of the suspended matter which produces the turbidity. Equal weights of suspended matter do not necessarily produce the same turbidity. For example, waters which contain suspended silt or sand exhibit less turbidity per unit of suspended matter by weight than do waters containing finely divided clay. Therefore the ratio between silica turbidity, determined optically, and suspended matter, determined gravimetrically, is most important, as it is an index of the character of the suspended matter producing the turbidity. To express this relation most conveniently, the term " turbidity coefficient " has been adopted.

$$\text{Turbidity coefficient equals } \frac{\text{Suspended matter.}}{\text{Silica turbidity.}}$$

Naturally this coefficient varies with different waters, generally increasing with the size of the particles composing the suspended matter. Thus the samples of unsettled river water have the highest turbidity coefficient, while samples from the effluents of the three-day subsiding basins have the lowest, as the following table will show:

Table of average turbidity coefficients.

	Turbidity coefficient.
Mississippi River water	1.68
Mississippi River water, after 6 hours' subsidence	.90
Mississippi River water, after 12 hours' subsidence	.87
Mississippi River water, after 18 hours' subsidence	.86
Mississippi River water, after 24 hours' subsidence	.85
Mississippi River water, after 48 hours' subsidence	.80
Mississippi River water, after 24 hours' subsidence and coagulation	.60

This table is very easy to understand, since the coarser particles of low turbidity-producing power and somewhat higher specific gravity gradually separate out according to their hydraulic values, the finer particles of high turbidity-producing power and somewhat lower specific gravity remaining longest in suspension.

The idea of Mr. Weston above set forth is an admirable one and should be utilized in connection with all water investigations.

The fourth method of determining suspended matter consists in measuring the cubical contents thereof after sedimentation. This method takes no account of the weight of the substance nor of the turbidity produced by it, and its particular value is confined to those highly turbid waters which it is proposed to conserve in storage reservoirs or to conduct in canals. In the preparation of reservoirs for irrigation and domestic uses in the arid and semiarid regions, one of the most troublesome features is the loss of storage capacity in the reservoir by reason of its filling up with matter deposited from suspension, and indeed it is necessary in the construction of these reservoirs to provide means whereby the silt can be removed at proper intervals. The problem is, therefore, one of cubical contents and the observations are usually made by filling a 100 c. c. graduate with the turbid water, allowing the suspended matter to settle, and reading the depth of the sediment and expressing it in percentage terms.

The results of such observations do not bear any more constant relation to the turbidity produced by the suspended matter than do the determinations of actual weight. An interesting series of observations upon this point has recently been compiled by the Geological

Survey, the water being taken from Gila River at San Carlos, Ariz. This river is probably the muddiest in the United States, and the observations represent extreme conditions. Turbidity measurements consume so small an amount of time in comparison with that necessary in the observation of per cent volume of total solids that an endeavor was made to determine whether or not turbidity measurements possess any constant relation to amount of matter. If such were found to be the case the work necessary in preparing plans for storage reservoirs would be considerably shortened.

Parallel determinations were therefore made of turbidity and per cent volume of sediment upon daily samples taken from Gila River from July 21 to October 24, 1904, the results of which are set forth in the following table. It will be noted in this table that the conversion factor, which should be constant if the hypothesis were correct, varies so widely as to indicate unmistakably the entire absence of any constant relation between the two sets of observations:

Parallel observations of per cent volume of sediment and of turbidity, in terms of parts per million, of silica in water from Gila River at San Carlos, Ariz.

Date.	Per cent sediment volume.	Turbidity (silica parts per million).	Conversion factor.	Date.	Per cent sediment volume.	Turbidity (silica parts per million).	Conversion factor.
July 21..	12	27,300	2,280	Aug. 12..	15	63,936	4,260
22..	16.5	37,800	2,290	13..	10	57,600	5,760
23..	19	43,200	2,280	14..	11	33,550	3,050
24..	18.5	43,200	2,340	15..	17	48,000	2,820
25..	21	44,784	2,140	16..	18.5	54,000	2,920
26..	18	41,568	2,310	17..	15	42,000	2,800
27..	16	40,572	2,540	18..	13	39,000	3,000
28..	9	40,800	4,530	19..	9	33,000	3,670
29..	21	53,000	2,520	20..	12	39,000	3,250
30..	19	64,000	3,370	21..	15.5	36,000	2,320
31..	12	64,000	5,330	22..	13	36,000	2,770
Aug. 1..	13	53,312	4,110	23..	10	27,000	2,700
2..	22	73,536	3,340	24..	13	39,000	3,000
3..	20	67,200	3,350	25..	8	30,000	3,750
4..	19	70,368	3,680	26..	9	30,000	3,340
5..	15	63,936	4,260	27..	8	27,000	3,380
6..	15	62,400	4,160	28..	8	24,000	3,000
7..	14	62,400	4,460	29..	8	27,000	3,380
8..	15	67,200	4,480	30..	8	27,000	3,380
9..	14	63,936	4,560	31..	8	21,000	2,630
10..	14	60,768	4,340	Sept. 1..	8.5	28,500	3,360
11..	13	62,400	4,800	2..	8.5	30,000	3,520

Parallel observations of per cent volume of sediment and of turbidity, in terms of parts per million, of silica in water from Gila River, etc.—Continued.

Date.	Per cent sediment volume.	Turbidity (silica parts per million).	Conversion factor.	Date.	Per cent sediment volume.	Turbidity (silica parts per million).	Conversion factor.
Sept. 3..	9	33,960	3,770	Sept. 29..	0.5	650	1,300
4..	8	36,000	4,500	30..	.5	600	1,200
5..	13	40,980	3,150	Oct. 1..	Trace.	650	....
6..	15	42,000	2,800	2..	2.5	350	....
7..	11	40,980	3,730	3..	Trace.	650	....
8..	10	31,500	3,150	4..	Trace.	280	....
9..	6.5	25,200	3,880	5..	Trace.	220	....
10..	3.5	9,200	2,630	6..	Trace.	6,400	....
11..	1.5	4,200	2,800	7..	2	42,000	....
12..	1.5	3,150	2,090	8..	10	42,000	4,200
13..	5	14,000	2,800	9..	13	39,000	3,000
14..	7.5	20,000	2,670	10..	12	36,000	3,000
15..	4	13,000	3,250	11..	13	27,000	2,080
16..	6	16,000	2,670	12..	9	18,000	2,000
17..	6	16,000	2,670	13..	5.5	12,000	2,180
18..	5	13,000	2,600	14..	3.5	8,000	2,280
19..	5	11,320	2,260	15..	2.5	4,400	1,760
20..	4	12,000	3,000	16..	2	3,800	1,900
21..	2	6,300	3,000	17..	1	3,750	3,750
22..	1	5,500	5,500	18..	7	26,000	3,720
23..	3.5	8,000	2,280	19..	2	6,000	3,000
24..	14	30,000	2,140	20..	1.5	3,000	2,000
25..	17	30,000	1,760	21..	.5	1,000	2,000
26..	3	7,000	2,330	22..	.3	900	3,000
27..	2	8,000	4,000	23..	.5	800	1,600
28..	1	1,000	1,000	24..	.5	766	1,530

TURBIDITY.

As all the usual methods for the determination of turbidity are fairly familiar, having been repeatedly described in numerous scientific journals, no further statements are necessary here. It is customary at the present time to adopt as a basis for the scale of each an absolute turbidity produced by a definite amount of finely divided silica in a certain volume of water. The scale has been described by its originators, Messrs. George C. Whipple and Daniel D. Jackson, in Technology Quarterly, Vol. XII, No. 4, December, 1899, pages 283–287.

UNITED STATES GEOLOGICAL SURVEY TURBIDITY ROD.

DESCRIPTION.

This rod, devised by Messrs. Allen Hazen and George C. Whipple, is a modification of the original Hazen rod, and is described in the following extract from circular No. 9 of the division of hydrography, United States Geological Survey:

Proposed turbidity standard.—The standard of turbidity shall be a water which contains 100 parts of silica per million in such a state of fineness that a bright platinum wire 1 millimeter in diameter can just be seen when the center of the wire is 100 millimeters below the surface of the water and the eye of the observer is 1.2 meters above the wire, the observation being made in the middle of the day, in the open air, but not in sunlight, and in a vessel so large that the sides do not shut out the light so as to influence the results. The turbidity of such water shall be 100.

The turbidity of waters more turbid than the standard shall be computed as follows: The ratio of the turbidity of the water to 100 shall be as the extended volume is to the original volume when the water is diluted with a clear water until the mixture is of standard turbidity.

The turbidities of waters lower than the standard should be computed as follows: The ratio of the turbidity of the water to 100 shall be as the ratio of the original volume of water of standard turbidity is to the extended volume when such water is diluted with clear water until its turbidity is equal to that of the water under examination.

This standard can be used in both field and laboratory. In the field the wire method will be employed as at present, except for a new graduation, while in the laboratory the methods of dilution and comparison now in use for the silica standard will be employed.

Method of application to the platinum-wire process.—A rod with a platinum wire inserted in it at a fixed point and projecting from it at a right angle will be used, as at present. The graduation shall be as follows: The graduation mark of 100 shall be placed on the head of the rod at a distance of 100 millimeters from the center of the wire. Other graduations will be made, based on the best obtainable data, in such a way that when a water is diluted the readings will decrease in the same proportion as the percentage of the original water in the mixture. Such a rod, having the graduation shown in the table below, shall be known as the United States Geological Survey turbidity rod of 1902. When this rod is immersed in water, the visibility of the projecting platinum wire at the depth from the surface shown in the second column will determine the degree of turbidity, as indicated in the first column.

Graduation of turbidity rod of 1902.

Turbidity.	Depth of wire.	Corresponding value on reciprocal scale.	Turbidity.	Depth of wire.	Corresponding value on reciprocal scale.
	mm.			*mm.*	
7	1,095	0.023	70	138	0.184
8	971	.026	75	130	.196
9	873	.029	80	122	.208
10	794	.032	85	116	.219
11	729	.035	90	110	.230
12	674	.038	95	105	.242
13	627	.041	100	100	.254
14	587	.043	110	93	.273
15	551	.046	120	86	.295
16	520	.049	130	81	.314
17	493	.052	140	76	.334
18	468	.054	150	72	.35
19	446	.057	160	68.7	.37
20	426	.060	180	62.4	.41
22	391	.065	200	57.4	.44
24	361	.070	250	49.1	.52
26	336	.076	300	43.2	.59
28	314	.081	350	38.8	.65
30	296	.086	400	35.4	.72
35	257	.099	500	30.9	.82
40	228	.111	600	27.7	.92
45	205	.124	800	23.4	1.09
50	187	.136	1,000	20.9	1.21
55	171	.148	1,500	17.1	1.49
60	158	.160	2,000	14.8	1.72
65	147	.172	3,000	12.1	2.10

This table is compiled from observations made at Cincinnati, St. Louis, New Orleans, Pittsburg, Brooklyn, Philadelphia, and Boston, for records of which we are indebted to several observers. The values of the turbidities by the reciprocal scale are included in the table for convenience, but they do not form a part of the standard.

This graduation is subject to revision whenever additional data shall make it necessary and revised rods shall be designated by the same name, but with the year of revision substituted for 1902. The revisions shall have as their basis the 100 mark, 100 millimeters from the wire.

Near the end of the rod, at a distance of 1.2 meters from the platinum wire, a wire ring shall be placed directly above the wire, through which the observer will look, the object of the ring being to control the distance from the wire to the eye.

When the turbidity is greater than 500 the water should be diluted before the observation is made. When the turbidity is below 7 this method can not be used, and comparison should be made with the silica standard properly diluted in bottles or tubes, as described by Whipple and Jackson in Technology Quarterly, Vol. XII, No. 4, December, 1899.

The number obtained by dividing the weight of suspended matter in parts per million by the turbidity as obtained above shall be called the coefficient of fineness. If greater than unity it indicates that the matter in suspension in the water is coarser than the standard; if less than unity, that it is finer than the standard.

This standard is proposed with the idea of combining the best features of the platinum-wire and silica methods of measuring turbidities as commonly used, and of avoiding, as far as possible, the objections to each.

OBJECTIONS TO ROD METHOD.

The method of turbidity determination above outlined answers all purposes demanded in ordinary use. In field determinations it has many objections which are not easily overcome. It was readily observed in practice that the method is largely a test of the individual and that the point at which the wire disappears from view varies according to the eyesight of the observer. Under ordinary conditions this variation is not sufficient to influence the interpretations placed upon the results, but there are some conditions under which the variation would be large enough to cause considerable error.

Again, the method was found to be inaccurate and unsafe in determining turbidity above 100. It is also difficult to select the conditions prescribed in the directions above set forth. A person in the field is governed absolutely by the conditions which he meets, and it is exceptional when he is able to be at a desired point at a given time. Therefore the observation, which must be made "in the middle of the day, in the open air, but not in sunlight, and in a vessel so large that the sides do not shut out the light," is in most cases an undertaking of extreme difficulty. Another observation is more important; it is necessary for the field man to take observations in the running stream. Obviously it would be impracticable to carry about a container large enough to meet these prescribed conditions, and in the majority of cases a turbidity reading must be taken at long distances from points at which such containers can be borrowed. It is well known that in many cases the suspended matter in running streams occurs in clouds. In a certain section of the stream the turbidity at one moment may be high and at the next moment much reduced, or vice versa. Often the observer, after fixing the point at which the platinum wire disappears, finds that before he is able to read the scale the wire is either plainly in sight or has become submerged below the point of correct turbidity reading.

All these objections make the use of the turbidity rod undesirable in general field work. While its value at selected stations is acknowledged, it has been found to be impracticable under less favorable circumstances; consequently a new method was sought.

JACKSON'S TURBIDIMETER.

DESCRIPTION.

The needs of the Survey were found to be met in a satisfactory manner by the use of a turbidimeter devised by Mr. Daniel D. Jackson, chemist in charge of the Mount Prospect laboratory, department of water supply, gas, and electricity, city of New York. The following is a report by Mr. Jackson with reference to this instrument:

The suspended matter or turbidity in natural waters is the most important physical characteristic in many sections of the country. In such sections the selection of new water supplies, as well as the improvement of existing supplies, rests, to a very great extent, upon a consideration of this particular feature. These milky or muddy waters are often quite variable in the amount and nature of their suspended matter, and, in case they are to be purified, require considerable study to determine the proper treatment.

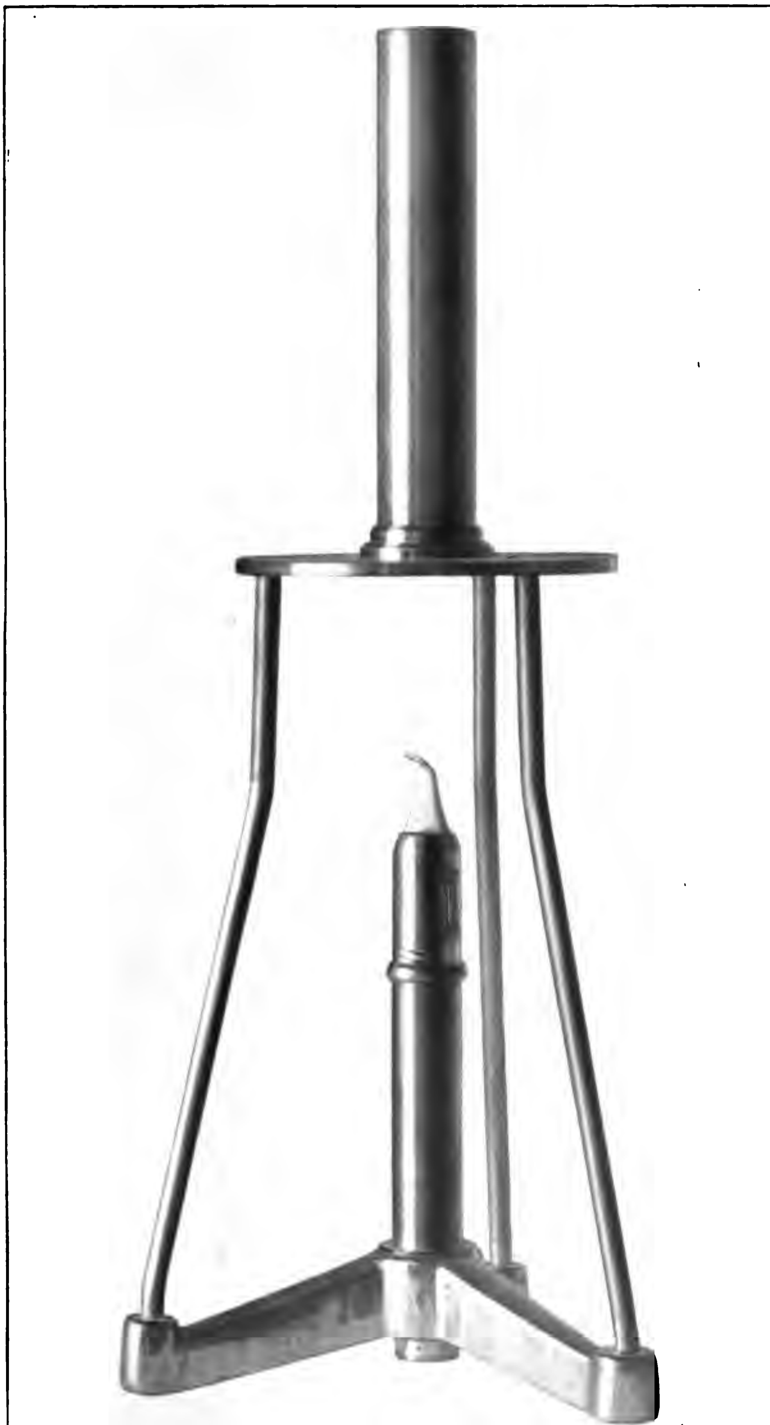
When the maximum and the average turbidity in a water are known, questions may be solved relating to the nature, size, and construction of settling basins, filter plants, and clear-water reservoirs, and, finally, in determining the efficiency of the removal of suspended matter in such filter plants we must know the turbidity of the water before and after filtration.

The hydrographic branch of the United States Geological Survey is particularly interested in developing accurate and rapid methods for the determination of turbidity, both for data relating to water supplies as well as relating to the erosion and the carrying power of suspended matter by rivers and streams. It is necessary that the field methods should be comparable with those of the laboratory, that the work should be rapidly accomplished, and that the results should express, as nearly as possible, the actual weight of the suspended matter present.

If we determine the total solids in a water before and after filtration through a Berkfeld filter, the difference in the results obtained will give the weight of the suspended matter present, but this method is tedious in the laboratory and impossible in the field. It is evident that some photometric standard of comparison must be used, and extensive studies have shown that whatever the instrument employed for this purpose it should be graduated by a standard turbid water. The standard now employed is known as the "silica standard," and is made from diatomaceous earth.[a]

This standard is preferable to all others that have been used in that it is absolutely insoluble, has a very uniform size of particle, and, unlike clay, does not cake together on standing. The diatomaceous earth (infusorial earth) is found in natural deposits in many parts of the country. To prepare the standard this material is first washed and ignited to free it from organic matter. It is then ground to an impalpable powder in an agate mortar, put through a 200-mesh sieve to break up the lumps produced in grinding, treated with dilute hydrochloric acid, and the finest portion decanted. This fine portion is then dried at 100° C., cooled in a desiccator, and kept in a tightly stoppered bottle.

[a] Whipple, G. C., and Jackson, D. D., Silica standards for the determination of the turbidity in water: Techn. Quart., vol. 12, No. 4, Dec., 1899.



JACKSON'S CANDLE TURBIDIMETER.

One gram of this material is weighed out and put into 1 liter of distilled water. The mixture represents a standard of 1,000 parts per million of silica turbidity, and dilutions may be made from this for comparison with natural waters. Readings made with this standard compare very well with the actual weight of the suspended matter in water, but it has been found that the standard as prepared varies slightly when made by different analysts. The author now proposes to make the standard absolute by making readings on the candle turbidimeter and so adjusting the mixture that the standard of 1,000 parts per million will always read 2.3 centimeters on the instrument.

THE CANDLE TURBIDIMETER.

The original form of this instrument was first described by the writer in the Journal of the American Chemical Society, November, 1901, but since that time it has been considerably improved upon. The accompanying illustration gives a good idea of the present form of the instrument and its use. [See Pl. I.] The apparatus consists of a glass tube, closed at the bottom and graduated in centimeters and millimeters depth. This is surrounded by a brass holder, open at the bottom and supported by a stand, in the center of which is a standard English candle, so adjusted by means of a spring below that its top rim is always just 3 inches below the bottom of the glass tube.

The water to be determined for turbidity is poured into the glass tube until the image of the lighted candle below just disappears.[a] The depth of the water in the tube is then read (using the bottom of the meniscus), and this depth is compared with a table which gives the turbidity of the water in parts per million of silica. The tube itself may be graduated in turbidity as well as in millimeters depth, thus dispensing with the use of the table. Between 5,000 and 100 parts per million of silica a tube 25 centimeters in length is necessary, or a comparison with silica standards in tubes or bottles may be substituted. The candle instrument is very convenient in the laboratory, and as its source of light is the standard candle it is ready for use at all times. The candle must always be properly trimmed, and the determination must be made rapidly, so as not to heat the liquid to any extent. The most accurate work is obtained in a dark room, and the candle should be so placed as not to be subjected to a draft of air. The latter necessity renders the instrument absolutely impossible for use in the field.

Several forms of field apparatus in which the candle was employed as a source of light were attempted, but were entirely unsuccessful, and it was found necessary to resort to the electric light for field use.

THE ELECTRIC TURBIDIMETER.

This instrument was designed by the author for the use of the hydrographic branch of the United States Geological Survey, and is intended for field use only. Its construction is so regulated as to be exactly comparable with the candle turbidimeter, and the measuring tubes for each have been made interchangeable.

The electric turbidimeter as shown in fig. 1 consists of the same graduated glass tube as described for the candle turbidimeter, inclosed in a similar manner

[a] It has been found in actual field work that the end point in the electric turbidimeter, viz, the disappearance of the cross of light, is generally sharper and less subject to personal errors than the end point above designated. This is especially true when the two instruments are used by the same person, i. e., a common end point is more satisfactory. The Geological Survey has therefore placed the glass plate and cross disk in the candle turbidimeter.

by a brass holder (A) open at the bottom. This holder is attached to the end of a brass cylinder (m) containing a 2.5-volt dry battery (C) and a 2.5-volt electric light (d). Above the electric-light bulb, at a distance of 1 centimeter, is a disk of glass (e) which is ground on the under side. Immediately above this is a brass disk (b) 1 millimeter thick, through the center of which a cross is cut (B). The lines in this cross are 0.5 millimeter wide. From the top of the brass plate to the bottom of the graduated glass tube the distance is just 1 centimeter.

To make a determination with the electric turbidimeter, first pour the turbid water to be tested back and forth from the glass tube to another vessel until it is thoroughly mixed, and then turn on the light by adjusting the screw (f) at the bottom of the instrument. Place the graduated glass tube in the holder, which has been screwed into place above the light, and pour the turbid water into the tube until the cross of light just disappears. If the tube is not graduated directly in parts per million of silica, read the depth in millimeters of the water in the tube and refer to the table given later. In reading use the bottom of the meniscus as the reading point. In the lower part of the tube read past the disappearance of the sharp cross of light to the disappearance of the hazy cross of light. In this way the end point is the same as in the candle turbidimeter. Higher up in the tube there is only the sharp cross of light for an end point.

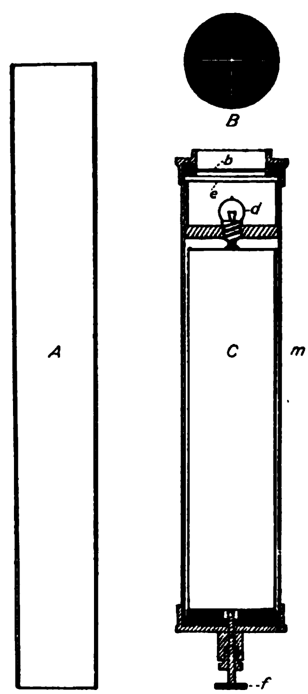


FIG. 1.—Jackson's electric turbidimeter.

If the turbidity is above 100 parts per million use the short tube (25 centimeters long). If the turbidity is between 100 parts and 25 parts per million the long tube (75 centimeters) may be employed, but at any point below 100 the glass tube and the holder may be removed and the instrument lowered directly into the turbid water by means of a steel millimeter tape. Any degree of turbidity may be read in this manner provided the water is sufficiently deep.

If the water is shallow and below 25 turbidity, close estimations may be made by holding a bottle of the water toward the light and comparing it with the remembered appearance of standards of 5, 10, 15, and 20 parts per million in bottles of the same size.

In the determination of turbidity with Jackson's turbidimeter many of the objections to the use of the United States Geological Survey turbidity rod are avoided. As the standard illumination is a part of the apparatus itself rather than the sun, none of the limitations which apply to the use of the rod, such as time of day, shade, etc., are necessary considerations. The instrument may be used at night if desired. As the sample to be tested is collected from the body of water under observation, inaccuracies due to moving water and variations in turbidity caused thereby are avoided, and it is not necessary to consider the depth of water in the river or lake under observation. In the use of the rod this is often a very troublesome feature, because, in case of low turbidity, there may not be water of sufficient depth to allow the rod to be submerged to the point of disappearance of the platinum wire. Another advantage is that the end point in Jackson's turbidimeter is approached more sharply and the use of the instrument is not practically a test of the observer's eyesight, for the measurement of turbidity depends upon the obliteration of a beam of light, and not upon the definition of a certain object.

TESTS OF ELECTRIC TURBIDIMETER.

The Jackson electric turbidimeter is made up of several parts which it was necessary to test in order to determine their effect upon the accuracy of the instrument. These tests, made by Mr. R. B. Dole, assistant engineer, United States Geological Survey, under the direction of Mr. Daniel D. Jackson, are classified as follows:

1. Tests of the battery for current, electromotive force, and durability.
2. Tests of the electric bulb for intensity of light.
3. Test of the ground-glass plate for opacity.
4. Calibration of the tube with silica standard.
5. Determination of the probable error.
6. Calibration of the tube with a sulphate standard. This will be treated under the heading "Sulphates," on page 69.

Battery test.—The "Reliable" 2-cell battery, 6 inches long, was selected and tested, first for constant current and then for recuperation, by running it for one minute, alternating with a rest of five minutes. These tests, applied to three cartridges selected at random, resulted as follows:

Results of tests of 6-inch 2-cell "Reliable" battery.

BATTERY NO. 1.

At end of—	Current.	Loss.	E. M. F.	Light.
	Ampere.	*Ampere.*	*Volts.*	
0 minute	0.250		2.66	Bright.
1 minute	.245	0.005		
2 minutes	.243	.002		
3 minutes	.242	.001		
4 minutes	.240	.002		
5 minutes	.240	.000		
6 minutes	.238	.002		
7 minutes	.236	.002		
8 minutes	.235	.001		
9 minutes	.235	.000		
10 minutes	.235	.000	2.50	Bright.
		.015		

Results of tests of 6-inch 2-cell "Reliable" battery—Continued.

BATTERY NO. 1 AFTER A REST OF 90 MINUTES.

At end of—	Current.	Loss.	E. M. F.	Light.
	Ampere.	*Ampere.*	*Volts.*	
0 minute	0.250		2.66	Bright.
1 minute	.245	0.005		
2 minutes	.243	.002		
3 minutes	.241	.002		
4 minutes	.239	.002		
5 minutes	.237	.002		
6 minutes	.236	.001		
7 minutes	.235	.001		
8 minutes	.235	.000		
9 minutes	.234	.001		
10 minutes	.234	.000	2.50	Bright.
		.016		

BATTERY NO. 2.

At end of—	Current.	Loss.	E. M. F.	Light.
0 minute	0.272		2.80	Bright.
1 minute	.268	0.004		
2 minutes	.265	.003		
3 minutes	.263	.002		
4 minutes	.261	.002		
5 minutes	.258	.003		
6 minutes	.255	.003		
7 minutes	.253	.002		
8 minutes	.251	.002		
9 minutes	.250	.001		
10 minutes	.249	.001	2.63	Bright.
		.023		

BATTERY NO. 2 AFTER A REST OF 90 MINUTES.

At end of—	Current.	Loss.	E. M. F.	Light.
0 minute	0.263		2.70	Bright.
1 minute	.258	0.005		
2 minutes	.253	.005		
3 minutes	.252	.001		
4 minutes	.250	.002		
5 minutes	.248	.002		
6 minutes	.246	.002		
7 minutes	.245	.001		
8 minutes	.244	.001		
9 minutes	.243	.001		
10 minutes	.243	.000	2.52	Bright.
		.020		

Results of tests of 6-inch 2-cell "Reliable" battery—Continued.

BATTERY NO. 3.

At end of—	Current.	Loss.	E. M. F.	Light.
	Ampere.	*Ampere.*	*Volts.*	
0 minute	0.273		2.82	Bright.
1 minute	.270	0.003		
2 minutes	.265	.005		
3 minutes	.262	.003		
4 minutes	.260	.002		
5 minutes	.258	.002		
6 minutes	.255	.003		
7 minutes	.253	.002		
8 minutes	.252	.001		
9 minutes	.251	.001		
10 minutes	.250	.001	2.60	Bright.
		.023		

BATTERY NO. 3 AFTER REST OF 90 MINUTES.

0 minute	0.256		2.70	Bright.
1 minute	.252	0.004		
2 minutes	.249	.003		
3 minutes	.246	.003		
4 minutes	.244	.002		
5 minutes	.243	.001		
6 minutes	.242	.001		
7 minutes	.241	.001		
8 minutes	.240	.001		
9 minutes	.239	.001		
10 minutes	.238	.001	2.50	Bright.
		.018		

The batteries were next tested for recuperation by alternating one minute of use with five minutes of rest, as follows:

BATTERY NO. 1.

Period.	Initial current.	Final current.	Drop in current.	Initial voltage.	Final voltage.	Drop in voltage.
	Ampere.	*Ampere.*	*Ampere.*			
First minute	0.243	0.238	0.005	2.60	2.54	0.06
Second minute	.242	.237	.005	2.56	2.51	.05
Third minute	.241	.237	.004	2.56	2.50	.06
Fourth minute	.240	.238	.002	2.56	2.50	.06
Fifth minute	.240	.236	.004	2.57	2.50	.07

BATTERY NO. 2.

Period.	Initial current.	Final current.	Drop in current.	Initial voltage.	Final voltage.	Drop in voltage.
	Ampere.	*Ampere.*	*Ampere.*			
First minute	0.249	0.245	0.004	2.63	2.56	0.07
Second minute	.249	.245	.004	2.63	2.56	.07
Third minute	.248	.245	.003	2.63	2.56	.07
Fourth minute	.247	.243	.004	2.62	2.56	.06
Fifth minute	.247	.243	.004	2.63	2.60	.03

BATTERY NO. 3.

First minute	0.260	0.255	0.005	2.76	2.66	0.10
Second minute	.257	.253	.004	2.72	2.66	.06
Third minute	.255	.251	.004	2.70	2.63	.07
Fourth minute	.254	.250	.004	2.70	2.63	.07
Fifth minute	.253	.249	.004	2.66	2.60	.06

	Volts.
Highest voltage observed	2.82
Lowest voltage observed	2.50
Extreme variation	.32

Reckoned on an average voltage of 2.66 volts, this is a variation of 12 per cent.

	Ampere.
Highest amperage observed	0.273
Lowest amperage observed	.234
Maximum variation	.039

Reckoned on an average amperage of 0.248 ampere this is a variation of 16 per cent in current. The drop in current averages 0.002 ampere per minute, or about 0.8 per cent.

It will be seen from these results that the battery is quick in recovery and that while in use the reduction in electromotive force is comparatively small. The change in current observed in the batteries tested does not cause any error in a turbidity estimation. Readings were made at different times with a standard of turbidity corresponding to 250 parts per million of silica, and in every case the variation in depth of liquid read in the graduated tube came within the probable deviation occurring in reading. The ordinary variation of current in the battery does not affect the accuracy of the instrument to a measurable degree.

This battery will remain effective under ordinary conditions from fifty to sixty days, at the end of which time it is advisable to change the cartridge.

Electric-bulb test.—Several lights were tested to see if there were a noticeable deviation in the intensity of light produced. The test of four sample lights is here given:

Battery.	Lamp No. 1.	Lamp No. 2.	Lamp No. 3.	Lamp No. 4.
No. 1	8.7	8.5	8.4	7.7
No. 2	8.6	8.6	8.2	8.0
No. 3	8.9	8.5	8.3	7.9

The numbers given are the depths in centimeters produced by using a standard turbidity of 250 with different batteries and lights. The mean of these observations is 8.4, while the average deviation from the mean is 0.3, which brings three of the lamps within the limit allowed on individual readings under constant conditions; the fourth light, however, falls without the limit of error. The lamps were chosen at random from a stock of 2.5-volt lights. It is evident that here is a variation which must be overcome. It may be done by buying a large stock of lamps and selecting only such as come within the standard conditions, or by buying lamps of guaranteed candle power. In conclusion it may be said that it is well to test a new lamp with silica standard before using it in the field.

Glass-plate test.—The glass diaphragm placed over the lamp is ground on one side in order to tone and diffuse the rays from the electric light. It also makes possible the use of a much shorter glass tube than would otherwise be necessary, and it reduces variation in candlepower in the effect thereof on turbidity determinations. It appears to be possible to procure glasses which are ground to the same opacity. Different glasses were tried in the instrument without any apparent effect on the depth of turbid liquid required to shut off the light. It may be said in connection with the ground glass that the cross slit of brass above it should be constant in width of aperture. As this offers no mechanical difficulties, no experiments were made to determine the effect of variation in the width of the slit.

Calibration for turbidity.—When work was begun on the calibration of the instrument it was necessary to prepare a standard silica solution. The standard heretofore used has been very difficult to match on account of the difficulty of grinding the silica fine enough to reach the required turbidity. The standard is such that it gives a reading of 500 parts per million at a depth of 4.5 centimeters, while the standard prepared by ordinary grinding gives a reading of 500 parts per million at a depth of about 5.7 centimeters. Several careful grindings failed to give the desired reading of 4.5 centimeters. It was therefore decided to make some very careful grindings and to select as a standard the one giving the lowest reading in depth with the turbidimeter. It was found that the particles of silica need to be rubbed apart with the finger after being ground, in order to secure the maximum turbidity. It is of interest to note the various readings with the four standard solutions prepared.

Variations in turbidity readings with different degrees of fineness of silica.

DEPTH, IN CENTIMETERS, PRODUCED WITH 500 STANDARD.

Standard No. 1.	Standard No. 2.	Standard No. 3.	Standard No. 4.
5.7	5.8	4.4	6.1
5.6	5.4	4.6	6.0
5.6	5.3	4.5	6.1
5.7	5.2	4.4	6.1
[a] 5.6	[a] 5.3	[a] 4.5	[a] 6.1

DEPTH, IN CENTIMETERS, PRODUCED WITH 250 STANDARD.

10.2	10.2	8.6	11.0
10.2	9.7	8.5	11.1
10.1	9.9	8.6	11.0
10.2	10.0	8.7	11.0
[a] 10.2	[a] 10.0	[a] 8.6	[a] 11.0

DEPTH, IN CENTIMETERS, PRODUCED WITH 125 STANDARD.

20.0	19.8	16.7	21.7
20.3	19.6	16.9	21.5
20.5	19.9	17.0	22.0
20.5	20.1	17.1	21.8
[a] 20.3	[a] 19.8	[a] 16.9	[a] 21.8

[a] Average.

According to these readings it was found that solution No. 3 practically coincides with the old standard and was therefore used as standard. It is believed that this choice will result in less confusion in the future when a new standard solution is desired, because this chosen turbidity represents the limit in grinding.

The work of calibration consisted in taking readings with different dilutions of the silica standard. After thoroughly shaking the standard it was poured into the graduated tube until the depth was reached at which the cross of light disappeared. Precautions were taken to secure uniform conditions of light, and the battery was tested for current at frequent intervals. In the following tables the actual readings of the tube are given, after which the average and average deviation are stated. Observations deviating by more than the average deviation are then discarded and the average of the remainder is taken to determine the resultant point on the curve which represents the turbidity scale.

Calibration of turbidimeter for standard of 250 parts per million turbidity.

[Centimeters. Excess of average deviation indicated by italic figures.]

Reading.	Deviation.	Reading.	Deviation.
9.2	*0.5*	8.6	0.1
8.4	*.3*	8.4	*.3*
8.7	.0	8.8	.1
8.9	.2	8.4	*.3*
8.5	.2	8.4	*.3*
9.2	*.5*	9.2	*.5*
8.4	*.3*	8.4	*.3*
8.7	.0	8.7	.0
8.9	.2	8.9	.2
8.5	.2	8.5	.2
8.6	.1	8.8	.1
8.8	.1	8.9	.2
8.9	.2	8.7	.0
9.0	*.3*	8.5	.2
8.7	.0	269.9	6.2
8.8	.1		
8.5	.2		

Mean=269.9÷31=8.7. Average deviation=6.2÷31=0.2.

It will be noted that 10 of 31 readings differ from the mean by an amount greater than the average deviation (0.2). Only 3 readings have a deviation greater than 0.3. It is therefore assumed that under ordinary conditions a variation of 0.3 centimeter at 250 standard should be allowed.

Calibration of turbidimeter for standard of 200 parts per million turbidity.

[Centimeters. Excess of average deviation indicated by italic figures.]

Reading.	Deviation.	Reading.	Deviation.
11.3	*0.3*	10.6	*0.4*
11.1	.1	11.3	*.3*
11.1	.1	11.1	.1
11.0	.0	11.1	.1
11.0	.0	11.0	.0
10.9	.1	11.0	.0
11.5	*.5*	10.9	.1
10.6	*.4*	187.3	2.9
11.1	.1		
10.7	*.3*		

Mean=187.3÷17=11.0. Average deviation=2.9÷17=0.2.

The deviation of 6 of the 17 observations exceeds 0.2 centimeter, the average deviation. Only 3 observations exceed 0.3 centimeter in deviation. Under ordinary circumstances we may consider 0.3 centimeter as the average deviation.

Calibration of turbidimeter for standard of 100 parts per million turbidity.

[Centimeters. Excess of average deviation indicated by italic figures.]

Reading.	Deviation.	Reading.	Deviation.
21.9	0.2	21.9	0.2
21.5	.2	22.2	*.5*
21.6	.1	21.0	*.7*
21.1	*.6*	21.9	.2
22.2	*.5*	21.5	.2
21.0	*.7*	21.9	.2
21.9	.2	303.1	4.7
21.5	.2		

Mean=303.1÷14=21.7. Average deviation=4.7÷14=0.3.

The deviation of 5 of the 14 observations exceeds 0.3 centimeter, the average deviation. Probably 0.4 centimeter would be the ordinary deviation. If we reckon 0.4 as the average deviation, mean = 21.8. Probably 21.7 is correct.

Calibration of turbidimeter for standard of 125 parts per million turbidity.

[Centimeters. Excess of average deviation indicated by italic figures.]

Reading.	Deviation.	Reading.	Deviation.
17.3	0.0	17.2	.1
16.7	*.6*	17.5	.2
17.2	.1	17.4	.1
17.4	.1	155.6	1.5
17.3	.0		
17.6	*.3*		

Mean = 155.6 ÷ 9 = 17.3. Average deviation = 1.5 ÷ 9 = 0.2.

The average deviation from 9 readings is 0.2 centimeter and is exceeded by only 2 readings. Probably more readings would give greater deviations and the average deviation would be increased.

Calibration of turbidimeter for standard of 500 parts per million turbidity.

[Centimeters. Excess of average deviation indicated by italic figures.]

Reading.	Deviation.	Reading.	Deviation.
4.4	0.1	4.6	.1
4.6	.1	4.5	.0
4.5	.0	31.4	.5
4.4	.1		
4.4	.1		

Mean=31.4÷7=4.5. Average deviation=0.5÷7=0.1.

Though only 7 readings are here given, many more were taken without getting anomalous results. The probable deviation is 0.1 centimeter and will not be exceeded. Mean=4.5 centimeters.

Turbidity of 1,000 parts per million.—From many observations at different times, 2.3 centimeters is the reading for 1,000 standard.

Average deviation=0.1 centimeter.
Mean =2.3 centimeters.

We have, then, determined by actual experiment the depth corresponding to 6 turbidities:

Turbidity	100	125	200	250	500	1,000
Depth (centimeters)	21.7	17.3	11.0	8.7	4.5	2.3

These points are then plotted on logarithmic cross-section paper (fig. 2) and intermediate points determined by measurement on the plot.

Below 100, depths have been determined at 50 and at 25 by using a longer tube, with which the effect will be the same as lowering the light into the standard by means of a tape. The observations made

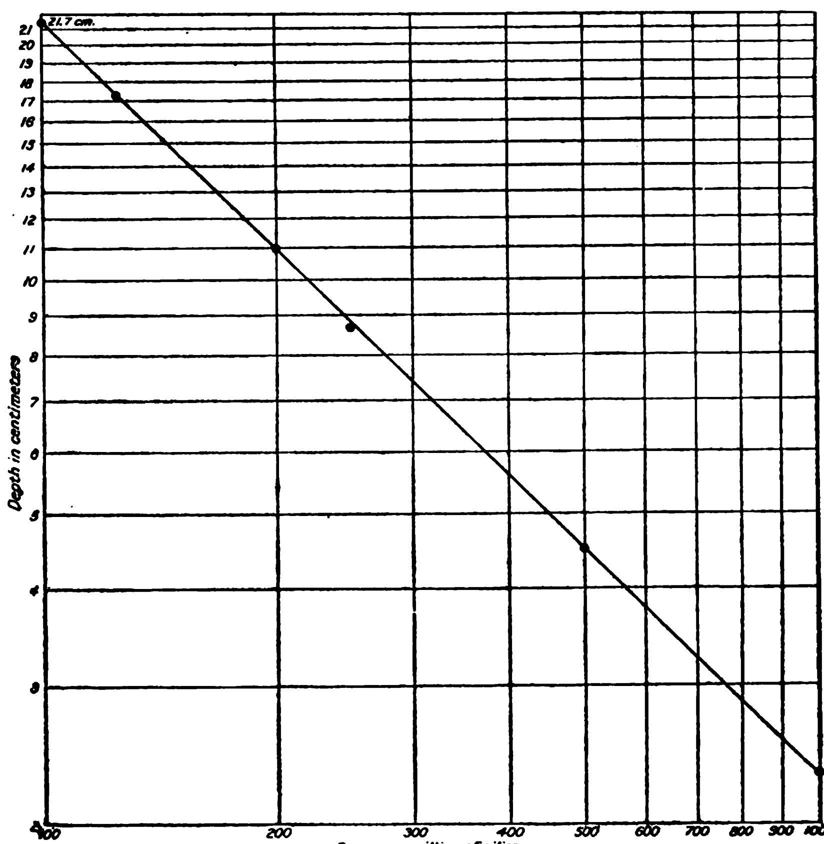


FIG. 2.—Logarithmic scale of turbidity.

at 50 and 25 seem to indicate that the curve begins to swing away from its course at 100. This change may be due to the fact that the distance between light and eye is increased. From 1,000 to 100 the light is 25 centimeters from the eye; at 50 it is 40 centimeters away, while at 25 the distance is 65 centimeters. The readings are as follows:

Calibration of turbidimeter for standard of 50 and 25 parts per million turbidity.

[Centimeters.]

50 parts.
36.1
34.0
34.5
34.4
35.3
35.0
34.3
34.1
33.9
35.2
346.8

Mean=346.8 ÷ 10=34.7.

[Centimeters.]

25 parts.
64
61
56
61
63
60
65
430

Mean=430 ÷ 7=61.

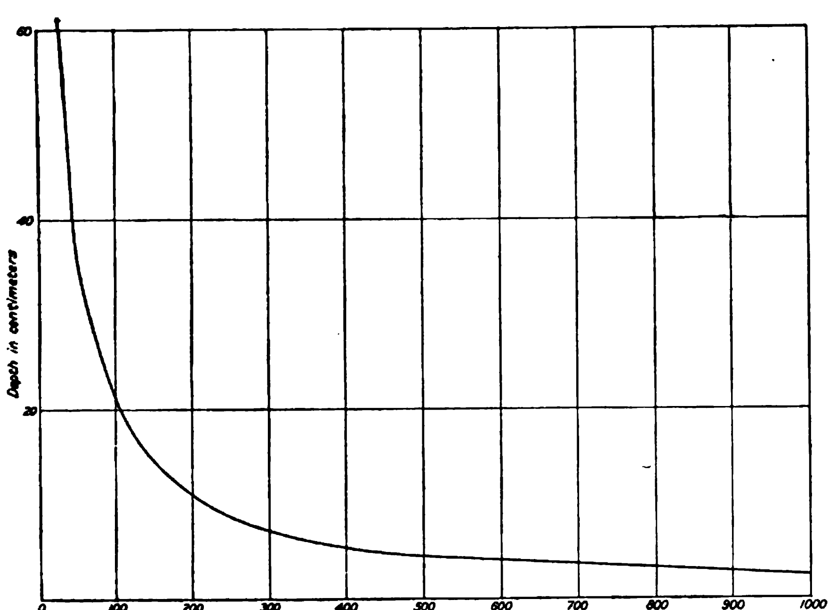


FIG. 3.—Turbidity curve.

These readings may not be accurate. Further experiments may show that they are too low. At most, however, they are within 10 parts per million of silica in their relation to the real values (fig. 3). The limit of accuracy between 100 and 1,000 is well defined from the points determined.

From the values heretofore determined the depths of liquid in the turbidimeter corresponding to a silica standard of turbidity are set forth in the following table:

Turbidity determinations.

Depth.	Silica.	Depth.	Silica.
Cm.	*Parts per million.*	*Cm.*	*Parts per million.*
2.3	1,000	10.5	210
2.6	900	11.0	200
2.9	800	11.5	190
3.2	700	12.1	180
3.5	650	12.8	170
3.8	600	13.6	160
4.1	550	14.4	150
4.5	500	15.4	140
4.9	450	16.6	130
5.6	400	18.0	120
6.3	350	19.6	110
7.3	300	21.7	100
7.6	290	23.0	90
7.8	280	25.0	80
8.1	270	28.0	70
8.5	260	31.0	60
8.7	250	35.0	50
9.1	240	42.0	40
9.5	230	52.0	30
10.0	220	70.0	20

DETERMINATION OF THE PROBABLE ERROR.

Readings on the same standard solution with the same battery and light by the same person will vary within narrow limits. These limits have been determined for several points and calculated as average deviation.

Limits of accuracy on duplicate readings.

Standard.	Average deviation in centimeters.	Limit of accuracy in parts per million.
1,000	0.1	Reading correct within 35 parts.
500	.1	Reading correct within 15 parts.
250	.3	Reading correct within 10 parts.
200	.3	Reading correct within 8 parts.
100	.4	Reading correct within 5 parts.
50	....	Reading correct within 5 parts.
30	....	Reading correct within 5 parts.

In other words, a turbidity between 500 and 1,000 parts can be measured accurately within 35 parts. Between 200 and 500 parts measurement can be made within 10 to 15 parts, and between 50 and 200 within 5 to 8 parts per million.

The limit of accuracy is not changed by change of observers. Since the thing seen is a ray of light, it appears to be visible to any eye and appears to be shut off at the same depth for different observers.

The limit is greater than that change in depth caused by normal variations in the current. Therefore the limit of accuracy is not increased by variations in the battery within ordinary limits.

Changes in the electric bulb can introduce a constant error greater than the probable error in determination. Therefore only such lamps should be used as have been tested with a standard silica solution.

In the general field work of the hydrographic branch the field assistants and those cooperating are instructed to use the Jackson turbidimeter in connection with all waters having a turbidity of more than 100, while the turbidity rod could be used in waters having a lower turbidity. The objections mentioned in connection with the use of this rod are not so serious in the determination of low turbidities.

COLOR.

OCCURRENCE.

The term "color" as used in water chemistry should not be confounded with the term as ordinarily used. The streams of the Mississippi Valley, and indeed the great river itself, appear highly colored. One will find rivers which are habitually red, yellow, brown, or black in appearance. This color is not due to the water itself, but to the character of the matter which is carried in suspension. It is a factor of the turbidity, and reveals the nature of the geologic formations eroded by the flowing water. On the other hand, waters may have a color due to dissolved substances, and this is the feature referred to by the term as used in water analysis.

In those parts of the United States where the underlying rock is resistant—that is, where it does not readily break up and disintegrate under the forces of erosion—we usually find colored water. At first sight it seems paradoxical that the clear waters of New England, many of which drain from granitic formations and hills of gravel, are colored, while those of the Central West, which carry large amounts of suspended matter eroded from the surface, are, when freed from turbidity, nearly colorless. In many cases this is due to the fact that the substances in suspension are of such nature that they absorb any color which might have been dissolved. On the other hand, in New England streams the color due to the decay of vegetable matter, such as peat or muck, remains in solution, and while the water is generally very clear the color itself is apparent in varying degrees.

The importance of the color determination arises from the fact that in public supplies consumers demand a clear, colorless liquid, and are reluctant to accept any other. In manufacturing processes a colored water often works harm. In certain classes of waters the dissolved color is a fair index of the amount of organic matter contained. These facts pertain primarily to unpolluted water, for it is apparent that a water contaminated by wastes may have colors arising from sources such as dyes, sediments, etc. On the whole, the color of a natural water which can be applied to domestic and manufacturing purposes affects its value along economic lines. The periodical determination of dissolved color is necessary, as its intensity varies with the seasons and is influenced by sunlight, precipitation, and various other natural phenomena.

COLOR STANDARDS.

GEOLOGICAL SURVEY STANDARD.

The standard of color determinations adopted by the United States Geological Survey is known as the platinum-cobalt method, devised by Mr. Allen Hazen, from whom so many practical and extremely valuable ideas with reference to the determination of quality of water have come.

The method is as follows:

A standard solution which has a color of 500 is made by dissolving 1.246 grams potassium-platinic chloride[a] ($PtCl_4,2KCl$), containing 0.5 gram platinum, and 1 gram of crystallized cobalt chloride ($CoCl_2,6H_2O$), containing 0.25 gram of cobalt in water, with

[a] Potassium-platinous chloride is a salt that is often substituted by dealers in place of the potassium-platinic chloride. It is sometimes incorrectly labeled. The platinous salt has a reddish color, while the platinic salt has a yellow color.

100 cubic centimeters concentrated hydrochloric acid, and making up to 1 liter with distilled water. By diluting this solution, standards are prepared having values of 0, 5, 10, 15, 20, 25, 30, 35, 40, 50, 60, and 70. The numbers correspond to the metallic platinum in the solutions in parts per million. These are kept in 100 c. c. Nessler jars of such diameter that the liquid shall have a depth between 20 and 25 centimeters and shall be protected from dust. The color of a sample is observed by filling a similar tube with water and comparing it with the standards. The observation is made by looking vertically downward through the tubes upon a white surface placed at such an angle that light is reflected upward through the column of liquid. The reading is recorded to the nearest unit. Waters that have a color darker than 70 are diluted before making the comparison, in order that no difficulties may be encountered in matching the hues. Water containing matter in suspension is filtered until no visible turbidity remains. If the suspended matter is coarse, filter paper may be used for this purpose; if the suspended matter is fine, the use of the Berkfeld filter is recommended. The use of a Pasteur filter is to be avoided, as it exerts a decolorizing action.

It is impracticable to carry the standard tubes above described into the field for observations, and yet field observations are of great convenience and value to the sanitary engineer, and in general to the investigations of the United States Geological Survey.

FIELD STANDARDS.

DESCRIPTION.

Disks of colored glass have been prepared by Mr. Allen Hazen, in cooperation with the Survey, as standards for measuring color of water in the field.[a] These disks have been rated by Mr. George C. Whipple to correspond with the platinum-cobalt standard. The color is measured by balancing the color of the water in a metallic tube with glass ends against the colors of glass disks of known value. The number on each disk represents the corresponding color of a water. This is not a new standard, but a new application of an old standard. The glass disks are rated to correspond with the platinum-cobalt color standard. The process bears the same relation to the usual laboratory process that an aneroid barometer bears to a mercurial barometer. The metallic tubes and glass standards are more portable and better adapted to field use than the Nessler tubes and color solutions heretofore used. The standards are disks of amber-colored glass, mounted with aluminum. Each disk carries two numbers. One number is over 100, and is a serial number for the purpose of identification.

[a] Pressey, H. A., Observations on flow of rivers in vicinity of New York City: Water-Sup. and Irr. Paper No. 76, U. S. Geol. Survey, 1903, Pl. X.

The other number is less than 100, and shows the color value of the disk; that is to say, the color of each disk is equal to the color of a solution of the designated number of parts per million of platinum with the required amount of cobalt to match the hue when seen in a depth of 200 millimeters. When a water comes between two disks its value can be estimated between them by judgment. Two or more disks can be used, one behind the other, in which case their combined value is the sum of the individual values. By combining the disks of a series in different ways a considerable number of values can be produced, allowing the closer matching of many waters.

USE OF FIELD STANDARDS.

Filling the tubes.—The tube, having an aluminum stopper, is to be filled with water, the color of which is to be determined. Rinse the tube once or twice by filling and emptying it. The second tube, having the clips to hold the glass disks, is made much like the one holding the water, to facilitate comparison. Theoretically this tube should be filled with distilled water. Practically it makes very little difference whether it is filled with distilled water or empty. Use distilled water when it is convenient to do so, and when distilled water of unquestionable quality is at hand; otherwise wipe the inside of the tube dry to prevent fogging of the glass ends, and proceed with the tube empty.

Holding the tubes.—Hold the tubes at such a distance from the eye that the sides of the tubes just can not be seen. This occurs when the near end of the tube is 8 or 9 inches from the eye. Hold the tubes at such an angle that both can be seen at once with one eye. Good results can not be obtained in any other way. Interchange the tubes once or twice, as sometimes the light on the right and left is not quite equal.

Background.—There should be a clear white background with a strong illumination. The best results can not be obtained with either too little or too much light. In a gray day look at the sky near the horizon away from the sun. In a bright day look at a piece of white paper or tile upon which a strong light falls. The white surface may be vertical and the tubes held horizontally, or the tubes may be held at an angle directed downward toward a horizontal surface, as may be most convenient. Good results can not be obtained by artificial light.

Turbid water.—The colors of very turbid waters can not be measured in this way. Slight turbidities do not interfere seriously with the results. Waters too turbid for direct observations should be filtered through thick filter paper before being tested; and in case the suspended matter causing the turbidity is fine in grain and large in amount, even this method may fail. The turbidity of water should be taken as far as possible in connection with color observations, except in cases where it is obvious from inspection that there is practically no turbidity.

Highly colored waters.—Some waters will be found having a higher color than can be matched by the standards. In general, waters with colors above 100 should not be matched in 200-millimeter tubes, and the results with waters having colors below 80 will be considerably more accurate than with more highly colored ones. Two procedures are possible with waters having higher colors; namely, to dilute with distilled water before measuring the color, or to use shorter tubes. The latter procedure is the more convenient, but both are equally accurate. To measure the color with short tubes, put the highly colored water in a tube of one-half the usual length and match as usual. It is not necessary to have a short standard holder. The 200-millimeter tube can be used. After the water is matched the result is multiplied by 2. In case the color is too high to be read in a 100-millimeter tube it can be put in a 50-millimeter tube, and the result multiplied by 4. When dilution is used the highly colored water is mixed with one or more volumes of distilled water, the color matched, and the result multiplied by a corresponding factor. The tube itself can be used for measuring the colored water and the distilled water, and the mixing can be done in a tumbler or any convenient clean vessel.

Cleaning the tubes.—Always keep the tubes clean. Take particular care of the glass ends. All the ends are removable for the purpose of cleaning, and should not be screwed on too tightly. They should be water-tight when screwed up only loosely, for if screwed on hard they may stick so as to come off with difficulty.

IRON.

One of the important determinations which it is necessary to include in many special investigations is that of iron. Water containing an appreciable amount of this metal can not be used in many manufacturing processes. It is objectionable in domestic uses by reason of its taste and the discoloration of linen. Certain solutions of iron in boiler feed waters are particularly destructive. Iron in ground waters stimulates the growth of *Crenothrix*, which frequently clogs water pipes. On the other hand, iron has a certain medicinal value, and when it is in the form of sulphate has valuable coagulating properties. The last-named effect is well demonstrated in streams draining coal regions.[a]

[a] Leighton, M. O., Quality of water in Susquehanna River drainage basin: Water-Sup. and Irr. Paper No. 108, U. S. Geol. Survey, 1904, p. 36.

Colorimetric methods are believed to be the simplest and best for the determination of iron in natural waters, and they readily lend themselves to modification for field purposes. That which involves the use of potassium ferrocyanide, described on page 226 of Sutton's Volumetric Analysis, ninth edition, was selected as best adapted for the purposes in view. The process involves the addition of acid and KCNS to the water under investigation or to the residual solution of that water, thereby producing a characteristic blood-red color. The depth of this color is absolutely fixed by the amount of iron in the water. It is then necessary to add to a similar mixture, made up with distilled water, such a quantity of standard iron solution as will produce in this solution exactly the same shade of red as is shown in the water under investigation. Then from the amount of standard iron solution used to produce that shade of red the amount of iron in the water under investigation may easily be determined.

The modification of this method for field purposes consists of the use of fixed color standards, each having been previously rated to correspond with some known equivalent of iron. The apparatus used in the work is that already described for the determination of natural color. (Pl. II.) The color standards are red glass disks, rated and used in precisely the same way as the natural color standards. The colored light is transmitted directly through the disk tube, and the disks may be changed or combined until the color of the sample under examination is matched. Then from the rating of the disks the amount of iron may be stated.

A sample of the clear water to be tested is poured into a 50- or 100-c. c. graduate to the 45 c. c. mark, 2 cubic centimeters of concentrated nitric acid added, and the contents thoroughly mixed in order to convert all ferrous iron present into ferric iron. The fluid should then be allowed to stand about five minutes.

The mixing and oxidation is preferably accomplished by pouring the solution from the graduate into another vessel, such as the glass turbidimeter tube, and vice versa, at least eight or ten times. To the acidified solution in the graduate is then added 3 cubic centimeters of a solution of potassium sulphocyanide containing 20 grams KCNS per liter, and the liquids are thoroughly mixed and allowed to stand 10 minutes. The solution is now transferred to the aluminum colorimeter tube, which has a capacity of about 45 cubic centimeters and is about 8 inches long.

Nitric acid is used in the above method instead of hydrochloric acid, commonly employed, in the first place to avoid any corrosion of the aluminum tube, the desirability of the use of which will be explained later. With the employment of nitric acid instead of hydrochloric acid, moreover, it was discovered that the color produced in

TUBES AND DISKS FOR DETERMINING COLOR OF WATER.

iron solutions by the addition of sulphocyanide is somewhat deeper and does not fade nearly so rapidly. Finally, the addition of nitric acid not only effects the required acidity of the solution essential to the test, but obviates the need of employing potassium permanganate in order to convert any ferrous iron present to the ferric state.

The aluminum tube is employed in the assay for iron in natural waters, because (1) it is light, (2) it can not be easily broken in transportation, like the glass tubes used in the water laboratories, (3) it is provided with a suitable spring for supporting the colored-glass disks, (4) it provides a suitable depth of column of the water for the iron determination, and (5) it is generally used by the hydrographic division in determining the natural colors of waters. The aluminum colorimeter tube thus serves a double purpose, and obviates the necessity of carrying special tubes in the field for the iron assay.

The results reached by this method of determination should be as accurate for practical purposes as those attained by the laboratory method.

CHLORIDES.

The determination of chlorides in water is significant in two general lines of investigation. In connection with sanitary analyses, it is in certain parts of the country a valuable index of sewage pollution. In analyses of water for boiler and industrial purposes it is also important, as the chlorides of calcium and magnesium corrode boiler plates.

LABORATORY DETERMINATION.

With reference to the determination of chlorides from a sanitary standpoint, the following article by Mr. Daniel D. Jackson, chemist in charge of the Mount Prospect laboratory, department of water supply, gas, and electricity, of New York City, is presented:

Chlorine, a constituent of common salt, is present in nearly all natural waters. Its original sources are mineral salt deposits and finely divided salt spray from the sea. This latter is carried with dust particles by the wind and precipitated with the rain. All salt found in waters not coming from these original sources comes from domestic drainage, and indicates that the water is at the present time polluted, or was polluted and has since been purified. By a comparison of the salt contents of any water under examination with the normal chlorine figure for that region, the extent of past or present pollution may be determined.

PHYSIOLOGICAL FUNCTIONS OF COMMON SALT.

Salt always occurs in drainage from animal sources because in all animal economy a certain fairly definite amount of common salt is eaten with the food daily and later expelled from the body in practically the same condition in which it was absorbed. That it plays an important rôle in the blood is indicated by the fact than on an average it constitutes about one-half of the total blood ash. It is also found that normal gastric juice can not be formed without the presence of salt, and that in many other secretions of the body its presence is probably a necessity.

SALT AS AN INDICATION OF POLLUTION.

The amount of salt in a water is a valuable indication of pollution because of the following facts: The animal body expels the same amount of salt that it absorbs; this salt is unchangeable in the soil and is very soluble in water: it must eventually form a part of the drainage and become mixed with the general run-off of the region in which it is expelled. The average amount of salt entering the drainage of any particular district is so constant for each inhabitant that it has been claimed that the number of people living on a drainage area may be determined with a fair degree of accuracy from the average run-off and the excess of chlorine over the normal.[a] Stearns estimates the chlorine in the run-off of any drainage area not receiving factory waste to be increased about one-tenth of a part per million by every 20 inhabitants per square mile.

SALT IN THE WATERS OF INLAND STATES.

All salt in natural unpolluted waters farther inland than Ohio comes from mineral deposits. The salt winds from the sea have no effect beyond this State, but, unfortunately, west of this State a large proportion of the natural waters are more or less affected by the salt deposits. The underground salt seems to spread over a broad area, and exerts not only a wide but a variable influence over most of the waters. In these inland States, while the "normal chlorine" would be practically zero, the value of the determination of chlorine is in most cases vitiated by the variable quantity of salt from mineral sources. Determinations of chlorine in samples of water taken above and below a city which runs its drainage into the stream examined may give the extent of pollution due to the city sewage, but the waters so far analyzed in the inland States give indications that the question of normal chlorine does not to any great extent enter into sanitary problems.

SALT IN COAST STATE WATERS.

On the other hand, the coast State waters are practically unaffected by this mineral salt, and while very extensive deposits exist, especially in the State of New York, they are in narrow pockets and exert an influence over a very limited area. Except in these pockets the mineral salt has apparently been washed into the sea.

It is found that in the coast States the salt in the natural waters which comes from original sources is practically all brought in by the sea winds, and that a certain normal amount is present in the waters of each locality.

The difference in the normal amount in different localities is due to variations in distance from the seacoast, in the amount of rainfall, in the rate of evaporation, in the amount of protection from ocean winds, and in the direction of the prevailing winds. In spite of the great variety of causes which affect the normal chlorine in natural waters, the normal for any particular region is surprisingly constant.

The chlorine decreases as waters farther and farther inland are tested, so that by connecting with lines on the map localities having the same normal we find that these lines of equal chlorine (isochlors) follow in a general way the

[a] Rept. Massachusetts State Board of Health, 1890, pt. 1, p. 680.

coast lines, and as they extend inland are still more or less parallel to the coast. The distance of these lines from the coast depends chiefly upon the general direction of the wind and the protecting influences of mountains on the coast or of islands near the mainland.

COLLECTION OF SAMPLES.

In order to obtain the normal chlorine lines for any State it is first necessary to collect a large number of analyses for chlorine in waters taken at different seasons over the entire area to be covered. It is evident that near the seacoast, where the variations in chlorine within a limited area are greatest, the largest amount of data must be collected. A large number of samples of water taken from surface and ground sources must be obtained. The pond waters usually give the best results, and careful inspection of the drainage area of such sources gives a good idea of whether or not the water is subject to pollution. Samples for analysis should be chosen as far from human habitation as possible.

SOLUTIONS REQUIRED IN THE ANALYSIS OF WATER FOR CHLORINE.

The following solutions are employed in the analysis of water for chlorine:

Salt solution.—A solution of chemically pure fused salt, containing 1 milligram of chlorine in each cubic centimeter, is made by dissolving 1.648 grams of the fused sodium chloride in 1 liter of distilled water free from chlorine.

Silver-nitrate solution.—Two and one-half grams of crystallized silver nitrate are dissolved in 1 liter of distilled water free from chlorine. To this solution water or strong silver nitrate is added until by actual titration 10 cubic centimeters of it are equal to 5 cubic centimeters of the standard salt solution. One cubic centimeter of this solution is then equal to 0.5 milligram of chlorine.

Potassium-chromate solution.—An indicator solution is made by adding 50 grams of potassium chromate to 1 liter of distilled water and then adding sufficient silver-nitrate solution to precipitate all the chlorine present and turn the precipitate slightly reddish. This is allowed to stand, and by filtering or decanting the clear solution is then obtained.

Emulsion of alumina.—This is made by dissolving 125 grams of potassium or ammonium alum in 1 liter of water and precipitating the alumina from boiling solution by ammonia. After precipitation the alumina must be washed free from chlorine, sulphate, and ammonia by successive treatments, settlings, and decantations with cold distilled water.

METHOD OF PROCEDURE IN THE ANALYSIS OF WATER FOR CHLORINE.

Pour 25 cubic centimeters of the water to be tested into a white porcelain dish. Add about one-half a cubic centimeter of chromate solution and run in standard silver-nitrate solution from a burette until the first faint reddish tint appears. This is more easily noted if for comparison a dish containing the same amount of water and chromate is kept beside the dish in which the test is made.

If 1 or more cubic centimeters of silver nitrate are necessary to reach an end point, the test may be made without evaporation, but if less is required then evaporate 250 cubic centimeters to 25 cubic centimeters volume before making the test. It may at times be necessary to evaporate more than this if the chlorine present is very close to zero in amount.

It is best to always titrate with 25 cubic centimeters of the water. In this case 0.1 cubic centimeter is subtracted from the results as an indicator error.

If more than this amount is used in titration, subtract 0.1 cubic centimeter for each 25 cubic centimeters of the volume of water titrated.

If 250 cubic centimeters of water are taken, the number of cubic centimeters of silver-nitrate solution used to obtain an end point minus 0.1 cubic centimeter multiplied by 2, gives the chlorine in parts per million.

Example: 250 cubic centimeters are evaporated to a volume of 25 cubic centimeters and chromate solution added. In the titration 3.5 cubic centimeters of silver nitrate are used. Then $(3.5 - 0.1) \times 2 = 6.8$. The water, then, contains 6.8 parts per million of chlorine.

If the sample is highly colored and very turbid it may be necessary to clarify it by treating it with an emulsion of alumina. This is best accomplished by bringing the water just to the boiling point, and then adding alumina and shaking the emulsion. In a few minutes the clarified water may be decanted. This is allowed to cool and the required amount is measured out for titration.

OBSERVATIONS ON THE USE OF THE NORMAL CHLORINE MAP.

Having drawn a map of this character for any coast State, we are then able to estimate the pollution in any natural water by the amount of chlorine present over the normal. In some instances it is first necessary to ascertain that the chlorine is not from mineral sources.

It will be seen that the normal chlorine lines are of great practical value, both to the chemist and to the engineer, as they give an index from which may be estimated the sanitary quality of most waters analyzed within the coast States. The chlorine also furnishes information as to the source of deep-seated springs or artesian wells.

While this chlorine in the general run-off is in direct proportion to the population on a drainage area, provided none of the sewage is carried outside of that area, yet waters in this region may have been purified before reaching the source from which they are collected. The chlorine would still be present and it is necessary to find from other tests whether the pollution is present or past.

It will be noted in Mr. Jackson's discussion that the determination of chlorine for the location of isochlors should be made by very precise laboratory methods. In fact, such precision should be used in all cases in which it is necessary to decide whether or not a water in a country where normal chlorine is significant contains chlorides in an amount which corresponds to or approximates a normal for the country from which the water comes. In general water surveys, however, it is necessary to make such nice distinctions only in rare cases. In the determination of chlorides in a water which is to be used for boiler or industrial purposes this refinement is not necessary, and field methods will generally suffice.

FIELD DETERMINATION.

STANDARD SILVER-NITRATE TABLETS.

The Geological Survey proposes to use a method for the rapid determination of chlorides which, from numerous experiments, seems to meet the conditions in a satisfactory manner. In place of the

UNITED STATES GEOLOGICAL SURVEY TABLET CASE.

standard solution of silver nitrate, which must be measured in a burette when applied to a water under examination, there are used tablets of silver nitrate containing a known equivalent of this reagent. These tablets are packed in tubes and carried in a leather case, the details of which are shown in Pl. III. This method of packing is well designed to avoid mechanical agitation of the tablets, which would result in their loss of active equivalent. The tablets are held securely in place by the stoppers, which are sufficiently small to be pushed through the lumen of the tube as fast as the tablets are used. This maintains a constant pressure against the tablets and prevents their agitation.

The manufacture of stable silver-nitrate tablets proved to be somewhat difficult. A number of pharmaceutical experts who were engaged at various times to prepare them failed to produce a tablet which was reasonably stable. Those which are now supplied to the Survey are made by the Kremers-Urban Company, of Milwaukee, Wis., and are of superior quality.

In connection with the determination of chlorides it is necessary to carry into the field only a small bottle of potassium-chromate crystals or solution and a heavy glazed porcelain mortar, together with a pestle of approved design. The tablets are dissolved in a measured quantity of water, a small amount of potassium chromate having first been placed in the solution. The end point is indicated by the change in color in the usual way and the number of tablets of known equivalent indicates the amount of chloride in the water.

This method may be objected to by some chemists because it is generally believed that the results of a chlorine determination made on a water without first evaporating the same are too high. This is apparently true with waters like those of New England, which contain only minute quantities of chlorine, but in the waters of the greater part of the country the chlorides are so high in amount that the error which arises from direct titration is not large enough to be of significance in industrial work.

While the tablets may be manufactured to contain almost any reasonable amount of silver nitrate, it has been found that the most convenient equivalents for general field use are those of approximately 1 and 10 milligrams of chlorine. In all special cases the strength of tablets should be adjusted to suit conditions. As it is practically impossible to manufacture tablets of the exact equivalent desired, it is necessary to determine the strength of each new supply and to make calculations of all field results accordingly.

The following statement includes various tests of a supply of tablets. Each tablet was made up to contain an equivalent of as near 1 milligram of chlorine as possible. The purposes of the tests were to determine the variation in equivalent of single tablets and the combined equivalent of tablets in sets of 5 and 10, the actual values in milligrams of chlorine being determined by volumetric methods.

Tests to determine variation in silver-nitrate tablets.

[Milligrams.]

Single tablets.		5 tablets.			10 tablets.		
Equivalent of 1 tablet.	Deviation from mean.	Equivalent of 5 tablets.	Mean equivalent of 1 tablet.	Deviation of 1 tablet from mean.	Equivalent of 10 tablets.	Mean equivalent of 1 tablet.	Deviation of 1 tablet from mean.
0.96	−0.045	5.060	1.012	−0.015	10.24	1.024	+0.0055
.95	− .055	5.230	1.046	+ .019	10.15	1.015	− .0035
1.049	+ .044	5.210	1.042	+ .015	10.17	1.017	− .0015
1.062	+ .057	5.110	1.022	− .005	10.18	1.018	− .0005
		5.090	1.018	− .009			
		5.200	1.040	+ .018			
		5.080	1.016	− .011			
		5.110	1.022	− .005			
Mean.	Maximum deviation.	Mean.		Maximum deviation.	Mean.		Maximum deviation.
1.005	+0.057	5.211	1.027	+0.019	10.185	1.018	+0.0055

It will be seen from the above results that the maximum variation in the equivalent of single tablets is 0.057 milligram of chlorine for each tablet. Therefore, considering the maximum variation shown in the single tablets and allowing, for purposes of illustration, that the maximum error may always be present in a determination, it would be necessary to use 18 tablets in a determination in order to reach an error equivalent to 1 milligram of chlorine. The mean value, however, of the single tablet is thoroughly representative of all the tablets tested, the variations lying above and below the mean value equally. It will be seen, further, that when the tablets are used in larger quantities and the combined equivalents of such quantities are compared, the deviations from the mean are considerably less and the maximum deviation is practically negligible. This is shown especially well in the statement of the comparison of the ten tablets. It is therefore apparent that the tablets do not vary by an appreciable amount, and that having established their equivalent by taking the mean of several determinations, such mean can be used in connection with the field determinations of chlorine in natural waters.

PRACTICAL TESTS WITH TABLET METHOD.

Three chloride solutions were made up at random, which, when tested by precise methods, were found to contain 4,280, 12,940, and 1,372 parts of chlorine per million, respectively. These solutions were titrated with tablets, and the end points reached in the same manner as that used in the field. The results are set forth in the following table:

Tests of silver-nitrate tablets, with solutions of known equivalent.

SOLUTION NO. 1.—CHLORINE, 4,280 PARTS PER MILLION.

Volume of solution.	Number of tablets.	Value of tablets.	Parts chlorine, tablet method.	Deviation from actual value.	Per cent deviation.
c. c.					
22	183.0	1.052	6,350	+2,070	48.4
5	21.0	1.052	4,410	+ 130	3.04
5	21.0	1.052	4,410	+ 130	3.04
25	10.4	10.1	4,200	− 80	1.87
25	10.0 3.0	10.1 1.052	4,160	− 120	2.80
50	20.8	10.1	4,200	− 80	1.87
50	20.0 5.0 1.0	10.1 1.052 .481	4,154	− 126	2.44
					15.06

Mean deviation (six results)=15.06÷6=2.51 per cent.

SOLUTION NO. 2.—CHLORINE 12,940 PARTS PER MILLION.

26	32.25	10.1	12,510	−430	3.32
26	32.0	10.1	12,430	−510	3.94
15	19.5 1.0	10.1 1.052	13,200	+260	2.01
15	18.5 8.0	10.1 1.052	13,020	+ 80	.62
5	64.0	1.052	13,470	+530	4.09
					13.98

Mean deviation=13.98÷5=2.80 per cent.

Tests of silver-nitrate tablets, with solutions of known equivalent—Continued.

SOLUTION NO. 3.—CHLORINE 1,372 PARTS PER MILLION.

Volume of solution.	Number of tablets.	Value of tablets.	Parts chlorine, tablet method.	Deviation from actual value.	Per cent deviation.
c. c.					
50	6.7	10.1	1,353	−19	1.38
50	6.0 7.0	10.1 1.052	1,358	−14	1.02
50	6.0 6.0 2.0	10.1 1.052 .481	1,358	−14	1.02
20	26.0 1.0	1.052 .481	1,392	+20	1.46
5	6.0 1.0 2.4	1.052 .481 .253	1,480	+108	7.80
					12.27

Mean deviation=12.27÷5=2.45 per cent.

The first result in the table above set forth is so radically wrong that it is inserted to illustrate a condition which must always be avoided when this method is employed, viz, the use of a large number of tablets. It will be noted that a considerable amount of strong chloride solution was used with tablets of low equivalent, i. e., 1.052 milligrams of chlorine. This made it necessary to use 133 tablets to reach the end point, with a result that is absurd. But 25 and 50 cubic centimeters of the same solution are titrated with the tablets with only a small error when the tablets of larger equivalent, 10.1 milligrams of chlorine, are used. On the whole, the results shown in the above table are very satisfactory, the error involved in the determinations averaging about 2.5 to 3 per cent, which is well within the limits of field work. Indeed, when the method was designed it was believed that an error of 5 per cent would be as small as could be expected.

A very simple way of testing for small amounts of chlorine is afforded by cutting tablets into quarters with a jackknife. Only ordinary care need be used and the quarters may then be taken for analysis without extreme regard to the selection of large or small pieces. The following account of some experiments performed is submitted:

No. 1. Twenty tablets were cut into quarters. Each quarter should precipitate 0.25 milligram of Cl. To 100 cubic centimeters of water (blank = 0.13 cubic centimeter) 1 cubic centimeter of NaCl was added, making the actual value of the water = 1.13 milligrams Cl. Four quarters gave no end reaction, but five quarters did. This experiment was done fifteen times with the same results. One and one-fourth tablets are equivalent to 1.25 milligrams Cl.

No. 2. Twenty more tablets were cut up and added as above, except that 1.1 cubic centimeters of NaCl were used, making the value of water=1.23 milligrams Cl (blank+0.13 cubic centimeter). Five quarters used in eight out of twelve titrations. This shows a variation in quartering of possibly an equivalent of 0.02 milligram Cl, or 0.2 part per million, an insignificant amount.

No. 3. Next 1 cubic centimeter of NaCl was used (=1.13 milligrams Cl). One whole tablet gave no end reaction, but one whole tablet+one quarter tablet gave a reaction in every case.

No. 4. The same was done with 0.9 cubic centimeter NaCl (=1.03 milligrams Cl). One tablet, no end reaction; 1¼ tablets, end reaction.

No. 5. The same, using 1.2 cubic centimeters NaCl (=1.33 milligrams Cl). One tablet, no end reaction,=1 milligram Cl; 1¼ tablets, no end reaction,=1.25 milligrams Cl; 1½ tablets gave end reaction,=1.50 milligrams Cl.

Nos. 3, 4, and 5 were each done ten times.

The value of these results and their accuracy are shown below:

No.	Milligrams chlorine.	
	Actual content.	Found by titration.
1	1.13	1.25
2	1.23	8 show 1.25 4 show 1.50
3	1.13	1.25
4	1.03	1.25
5	1.33	1.50

This method comes within 0.25 milligram of the amount of chlorine present, or within 2.5 parts per million of chlorine. This procedure is recommended. If only ordinary care be used in cutting tablets, their value will be within 1 part in a million.

ESTIMATION OF CHLORINE.

A known amount (about 50 c. c.) of the water to be tested is measured into a glazed porcelain mortar (4 inches diameter) and 5 drops of potassium chromate (5 per cent solution) added.

One silver-nitrate tablet is then cut into quarters, using ordinary care to get the quarters equal. Whole tablets are added to the water till near the end point, when quarter tablets are used. The end point is the appearance of the red color of silver chromate.

$\frac{1,000 \, nA}{W}$=milligrams per liter of chlorine, when W=cubic centimeters of water used, n = number of tablets used, and A = value of one tablet in milligrams of chlorine. Proper allowance should be finally made for the amount of silver nitrate consumed in the end reaction.

Each tube of $AgNO_3$ tablets is marked with its equivalent of chlorine.

NOTE.—Silver-nitrate tablets should not be handled with the fingers nor exposed to sunlight. Keep all tubes well stoppered.

HARDNESS.

GENERAL STATEMENT.

A hard water is popularly recognized as a water with which it is difficult to obtain a soap lather. Strictly defined, it is that property imparted to water by the carbonates, sulphates, chlorides, and nitrates of calcium and magnesium. Chemical methods for the determination of hardness are not yet well defined; in fact, the whole subject is somewhat chaotic. A method which may be satisfactory for the waters of one part of the United States may be of little value for those of another. Consequently there has developed a rude geographic distribution of methods, each being adapted to the peculiarities of the waters in the regions in which they are used. The result is that comparisons between hardness determinations made in different regions are somewhat uncertain and nearly always unsatisfactory.

The effect of hard water upon the lathering properties of sodium soap—that is, the soap used for laundry and toilet purposes—has been made use of in determining hardness. Indeed, the soap test was the earliest and is still the commonest of all those employed for this purpose. A standard solution of a pure soap, usually a high-grade castile, is standardized against a solution of calcium chloride, the equivalent of which has been determined in terms of calcium carbonate. The details of this process are too familiar to warrant further description. Practically, the principal weakness of the test is the determination of the end point, which is not sharply defined. The sodium salts of oleic, palmitic, and stearic acids which compose a pure soap are definite chemical compounds, and their transformation into calcium and magnesium soaps should follow the usual course of chemical change, and therefore the soap test is not merely a test of the soap-consuming power of water, as maintained by some chemists.

In practice the soap test is limited in its usefulness, and its results are modified by many conditions. A few authorities who have made minute studies of the soap method are impressed with its possibilities, but the modifications which they suggest are somewhat cumbersome, involve superrefinements, and in the end require so much time that the importance of the results is not commensurate. In those parts of the country where soft waters abound and where it is known that magnesium salts are not abundant the test has great value. Its usefulness is limited in the waters of the Mississippi basin, and it fails entirely when applied to western waters.

FIELD METHOD.

USE OF SODIUM-OLEATE TABLETS.

The soap test has been modified for field use by the substitution of tablets of pure sodium oleate for the soap solution. Sodium oleate can be obtained in pure form and readily divided in tablets, each containing a known amount of the reagent. The tablets used by the Geological Survey in field work are made by the Kremers-Urban Company, of Milwaukee, Wis. They are of three grades, "full," "half," and "quarter," or, as they are usually denoted, "F," "H," and "Q," according to their content of 10, 5, or 2.5 milligrams of sodium oleate, respectively.

In using these tablets, 100 cubic centimeters of the water to be tested are placed in a specially designed bottle (Emil Greiner) having a heavy semispherical bottom. Tablets are then added one at a time and dissolved in the water. This process is greatly hastened by trituration of the tablets in the bottle with a blunt glass rod. After each tablet is dissolved the bottle should be shaken and laid upon its side, and the determination conducted in precisely the same manner as that prescribed in the case of the soap solution. From the number of tablets used and their equivalent the hardness may be determined.

The facility with which this determination may be carried on is largely determined by practice. When first attempted it seems awkward, but after a few trials the operator finds that it can be readily performed. The proper way is to start with the F tablets until near the end point, then apply the H, and finally the Q tablets. This of course may become a method of "trial and error," but the skilled field man will seldom add tablets beyond the end point. The appearance of the lather at various stages is characteristic and affords a guide for the operator. In order to show certain characteristic features in the tablet method for the hardness determination the following results of a test made of a supply of tablets are set forth:

TEST OF SODIUM-OLEATE TABLETS.

For a standard of hardness, 0.2 gram of Iceland spar was dissolved in HCl, evaporated to dryness three times in HCl and twice with water. Finally the residue was dissolved and diluted to one liter with redistilled water. Five cubic centimeters of this solution of $CaCl_2$ is equivalent to 1 milligram $CaCO_3$.

The tablets used in the work are of three equivalents, and are designated as follows:

One F tablet contains an approximate equivalent of 0.0014 gram calcium carbonate.

One H tablet contains an approximate equivalent of 0.0007 gram calcium carbonate.

One Q tablet contains an approximate equivalent of 0.0003 gram calcium carbonate.

The first test was to determine the amount of sodium-oleate tablet reacted with 100 cubic centimeters distilled water. In the following tabulated statement the sign + denotes a permanent foam or satisfactory end point, while — denotes no end point.

Standardization of sodium-oleate tablets against 100 cubic centimeters distilled water.

F.	H.	Q.	2Q.	$\frac{3Q}{2}$	$H+\frac{Q}{2}$
+	+	—	+	—	+
+	+	—	+	—	+
+	+	—	+	—	+
+	+	—	+	—	+
	+	—	+	—	+
	+			—	
	+			—	

Results in the above series of experiments indicate that 100 cubic centimeters distilled water requires about 0.005 grams of sodium oleate.

To reduce this to terms of calcium carbonate experiments were made with 100 cubic centimeters distilled water containing varying amounts of calcium chloride. It was found by repeated trial that where two H tablets were used it required 3.2 cubic centimeters of the $CaCl_2$ solution in 100 cubic centimeters of distilled water to react exactly. Consequently, as the 100 cubic centimeters distilled water required one H tablet, the remaining H tablet was equivalent to the $CaCl_2$. Therefore the distilled water is equivalent in the soap reaction to 3.2 cubic centimeters of $CaCl_2$, or 0.64 milligram $CaCO_3$.

The first experiments in standardizing the tablets were made with the standard $CaCl_2$ solution diluted with 100 cubic centimeters distilled water against one F tablet, with results as follows:

Standardization of sodium-oleate tablet F.

$CaCl_2$ against 1 F tablet.	Result.
c.c.	
3.0	Foam.
3.5	Foam.
3.6	Foam.
3.7	Foam.
3.7	Foam.
3.7	Foam.
3.7	Foam.
3.8	No foam.
3.8	No foam.
4.0	No foam.

From the above it appears that one F tablet is equivalent to 3.7 cubic centimeters $CaCl_2$ solution with 100 cubic centimeters distilled water, or actually equivalent to 6.9 cubic centimeters (3.7+3.2 cubic centimeters), which expressed in terms of $CaCO_3$ is equal to 1.38 milligrams.

Using seven F tablets in 100 cubic centimeters distilled water against different amounts of the $CaCl_2$ solution the following results were reported:

$CaCl_2$ against 7 F tablets.	Result.
c. c.	
41.7	Foam.
41.5	Foam.
42.5	Foam.
44.9	Foam.
45.0	Foam.
45.9	No foam.
46.0	No foam.

From the above results it appears that the end point is practically reached with 45 cubic centimeters $CaCl_2$ solution. With the amount for 100 cubic centimeters distilled water added, seven tablets are therefore equivalent to 48.2 cubic centimeters $CaCl_2$, or 9.64 milligrams $CaCO_3$. This allows for one tablet an equivalent of 1.37 milligrams $CaCO_3$, which agrees with the previous determinations within the limit of experimental error.

Experiments were made with various combinations of tablets as follows:

$CaCl_2$.	Tablets (number and value).	Result.
c. c.		
3.2	4 Q	No foam.
3.2	5 Q	Foam.
3.0	5 Q	Foam.
3.0	5 Q	Foam.
2.8	4 Q	Foam.
2.9	4 Q	Foam.
45.0	7 F	Foam.
45.9	7 F	No foam.
45.9	7 F + 1 Q	Foam.
45.0	14 H	No foam.
45.0	15 H	Foam.
45.0	6 F + 2 H	Foam.
45.0	6 F + 2 H	Foam.

Analyzing the results above given we have:

1.

$CaCl_2$+100 c. c. distilled water.	Tablets (number and value).	Result.
c. c.		
3.2	4 Q	No foam.
3.2	5 Q	Foam.
3.0	5 Q	Foam.
2.8	4 Q	Foam.
2.9	4 Q	Foam.

Therefore four Q tablets are equivalent to an amount of $CaCO_3$ lying between 1.28 milligrams and 1.22 milligrams, or one Q tablet to an amount lying between 0.32 and 0.30. Using either as the value, the error will not be significant. 0.31 milligram is probably the most accurate factor.

2.

$CaCl_2$+100 c. c. distilled water.	Tablets (number and value).	Result.
c. c.		
45.0	7 F	Foam.
45.9	7 F	No foam.
45.9	7 F+1 Q	Foam.

Therefore seven F tablets are equivalent to an amount of $CaCO_3$ lying between 9.82 and 9.64 milligrams, or one tablet to an amount lying between 1.40 and 1.38 milligrams. The error introduced if either 1.40 or 1.38 is used as a factor will be less than $\frac{Q}{7}$ and therefore insignificant.

3.

$CaCl_2$+100 c. c. distilled water.	Tablets (number and value).	Result.
c. c.		
45	14 H	No foam.
45	15 H	Foam.
45	6 F+2 H	Foam.

Therefore the equivalent of 9.64 milligrams $CaCO_3$ lies between the value of 14 and 15 H tablets, or one tablet is equivalent to an amount of $CaCO_3$ between 0.69 and 0.64 milligram. But from the third experiment it is seen that two H tablets are equivalent to 1.36 milligrams (9.64—[6×1.38]), or one tablet to 0.68 milligram.

ESTIMATION OF HARDNESS.

The directions for using these tablets should be followed absolutely. The end-point foam should be permanent for at least five minutes, the bottle lying upon its side. The smallest number of tablets possible should be used; for example, if the hardness of a water is 120 parts per million on direct titration without correction, the best procedure assuming the given value of the tablets used would be as follows:

	Milligrams $CaCO_3$.
8 F tablets equivalent to	11.04
1 H tablet equivalent to	.68
1 Q tablet equivalent to	.31
Total	12.03
Subtracting 0.64 milligram $CaCO_3$ for distilled water	.64
Hardness expressed by tablets	11.39
Or in parts per million	113.9

In this case the error would be equivalent to 0.03 milligrams $CaCO_3$, an amount unimportant in practical work.

The end point can be approached in the manner above described after the operator has had a short experience with the method. The comparative permanence of the preliminary foam pellicles which do not remain unbroken for the entire five minutes is a guide.

HARDENING CONSTITUENTS.

CLASSES.

It is customary to distinguish between temporary and permanent hardness. Temporary hardness is due to the carbonates (and bicarbonates) of calcium and magnesium. Calcium and magnesium carbonates are not readily soluble in water unless accompanied by carbon dioxide. Under such circumstances it is supposed that the carbonates become bicarbonates, although the bicarbonates of these two elements have never been isolated. When waters containing calcium and magnesium bicarbonates are boiled the carbon dioxide is driven off and the normal carbonates of calcium and magnesium are precipitated. Therefore, the properties which they impart to the water are designated temporary hardness. Permanent hardness, on the other hand, is that property which is imparted to waters by the sulphates, chlorides, and nitrates of the alkali earths. They are not precipitated by ordinary boiling, and therefore their effects are regarded as permanent. The usual method of determining temporary and permanent hardness by the soap test consists in making the test on a sample of water before boiling, and another on a similar sample after boiling; the difference in the two results representing the temporary hardness.

Temporary and permanent hardness are often expressed as alkalinity and incrusting constituents, respectively, and it is common to see, even in analytical reports of well-informed chemists, the expression "alkalinity or temporary hardness." This expression is misleading. It is approximately correct when the waters of New England and certain other portions of the country are referred to, but as a general statement concerning the majority of waters nothing could be more inaccurate. There are abundant instances in which waters are alkaline to an extraordinary degree, and yet are widely known as soft waters, giving little or no reaction with the soap test. The alkalinity in such cases is due to the carbonates of sodium and potassium, which, while they impart a truly alkaline reaction, have no hardening effect. The majority of the waters of the United States contain alkali carbonates, and therefore any interpretation of alkalinity as being equivalent to temporary hardness with such waters is erroneous.

If the alkalinity found in a water is the result of the carbonates of the alkali-earth elements, its industrial significance is considerably different from that of the carbonates of the alkalies. For example, calcium carbonate forms soft scale when used in boilers, while sodium carbonate forms no scale, but presents a much less important difficulty, that of foaming. If, however, a water containing calcium carbonate were used for irrigation purposes, it would not damage crops unless it were present in extremely high proportions—higher, in fact, than it is almost ever found in nature. On the other hand, a small amount of sodium carbonate is destructive to crops. It is as desirable to know whether the sulphates and chlorides which are found in the water are of the alkali earths or the alkalies, for they present variations in usefulness with reference to industries similar to those above described in the case of the carbonates.

It has been the endeavor of the Geological Survey to so modify the methods by which the various determinations of the hardening constituents of water may be made that they can be used in the field. These methods are set forth in subsequent pages. They do not include all of the determinations desirable for some classes of work, but sufficient to allow of a very comprehensive interpretation concerning the quality of any water under investigation.

CARBONATES.

The determination of alkalinity or carbonates is a simple volumetric process. It requires only a standard solution of an acid, preferably a mineral acid, with accurate means for measuring the same, and a proper indicator solution. On account of the carbon dioxide set free by the determination, methyl orange is the indicator in commonest use. The objections already cited to carrying standard solutions and burettes in the field led to an attempt to adopt an acid which could be preserved in tablet form. Many organic acids were tried, but is was found that they were either too weak to afford a definite end point or were of so deliquescent a character that they could not be made to form stable tablets. It was finally decided to adopt the use of sodium acid sulphate. Tablets made from this reagent are easily regulated in equivalent and are of an extremely stable nature. The results which can be procured through their use are very satisfactory.

TESTS OF SODIUM ACID-SULPHATE TABLETS.

For use in titrating against sodium acid-sulphate tablets a fiftieth normal solution of sodium carbonate was made, in which each cubic centimeter equals 1.06 milligram $NaCO_3$. Six sets of five sodium acid-sulphate tablets each were then dissolved in 50 cubic centimeters of distilled water and each solution was titrated with the standard sodium carbonate. The results of these titrations are shown in the following table:

Standardization of sodium acid-sulphate tablets.

$NaHSO_4$.	Na_2CO_3.	Value of 1 tablet.
c. c.	*c. c.*	*MgCaCO₃.*
10.60	2.15	2.03
10.15	2.05	2.02
10.85	2.20	2.08
10.30	2.10	2.04
10.10	2.05	2.03
10.10	2.05	2.03

Assuming these final reactions,

$$Na_2CO_3 + 2NaHSO_4 = 2Na_2SO_4 + H_2O + CO_2,$$

$$\text{and } CaCO_3 + 2NaHSO_4 = CaSO_4 + H_2O + CO_2 + Na_2SO_4,$$

then for the expression of the value of our tablet in milligrams of $CaCO_3$ we have the following proportion:

$$\frac{Na_2CO_3}{CaCO_3} = \frac{106}{100}.$$

The experiments above described were made with solutions of the acid-sulphate tablets, and the results show the constancy of the reaction between the normal carbonate and the sulphate. They do not show, however, the variations which would occur in the practical use of the tablets applied directly to the alkaline solution, as would be done in the field. Therefore the following tests are submitted to show the deviations which may arise in successive tablets or successive sets of tablets. The tablets used were marked "Lot 715, sodium acid-sulphate equivalent to 1.995 milligrams calcium carbonate." These tablets had been in stock for several months, had received some rough handling, and were in poor condition. In fact, they represented the most unfavorable conditions that might be supposed to occur in connection with the field use of tablets, and the variations which are shown may be accepted as the extreme variations which are likely to occur in common use.

In testing and standardizing the sodium acid-sulphate tablets normal solutions of sulphuric acid and sodium carbonate were used. Tests were made as follows:

Solutions of unknown strength of sodium carbonate were made up and the alkalinity was determined volumetrically with standard sulphuric acid solution. Following this, determinations of the same unknown solutions were made with the tablets. Varying amounts of the solution were used, with a corresponding variation in the number of tablets.

Comparative determinations of alkalinity in carbonate solutions with standard sulphuric-acid and sodium acid-sulphate tablets.

CARBONATE SOLUTION NO. 1.—ALKALINITY, 5,764 PARTS PER MILLION IN TERMS OF CALCIUM CARBONATE.

Amount of solution.	Number of tablets.	Parts per million.	Deviation in parts per million.	Per cent deviation.
c. c.				
1.9	5	5,225	−539	9.3
3.5	10	5,674	− 90	1.56
7.0	20	5,674	− 90	1.56
11.0	32	5,726	− 38	.66
10.5	30			
17.5	50	5,674	− 90	1.56
46.5	135	5,767	+ 3	.05

CARBONATE SOLUTION NO. 2.—ALKALINITY, 2,312 PARTS PER MILLION IN TERMS OF CALCIUM CARBONATE.

Amount of solution.	Number of tablets.	Parts per million.	Deviation in parts per million.	Per cent deviation.
4.5	5	2,207	−105	4.55
10.0	12	2,383	+ 71	3.1
15.0	18	2,384	+ 72	3.1
21.0	25			
19.0	23			
25.5	30	2,359	+ 47	2.0
25.0	30			
51.0	60	2,328	+ 14	.61
50.5	59			

The figures of the above tables show that when a large number of tablets are used to determine alkalinity the results are more nearly correct than when a few are used. This is especially noticeable in the first entries in the two tables, where the small amount of the alkaline solution used requires only five tablets. The error in each of these cases is larger than is permissible even in field work. In the remainder of the tests, however, the variation is not sufficiently great to be appreciable, especially in the weaker carbonate solutions, where it is shown that the use of a larger number of tablets involves a minimum error. This suggests that in connection with the field determination of carbonates it is advisable, wherever waters of a low alkalinity are tested, to use a large amount of the water in order that a large number of tablets can be used to neutralize the alkalinity, and thereby avoid the error arising from the variation which occurs in the single tablets.

ESTIMATION OF ALKALINITY.

Measure 100 cubic centimeters of water to be tested into a glazed porcelain mortar (4 inches diameter). Add two drops methyl-orange indicator. Add $NaHSO_4$ tablets till an acid reaction is reached. Then add some of the original water that is being tested, drop by drop, till an alkaline reaction is exactly reached. Measure the liquid in the mortar and to the amount of the reading add 1 cubic centimeter for the wetted interior of the dish. The following formula is convenient for use in making calculations of alkalinity:

$$\frac{1{,}000\ n\ A}{W} \text{ equals milligram per liter of } CaCO_3.$$

When W equals cubic centimeter of water used;
n equals number tablets used;
A equals value of 1 tablet in milligrams of $CaCO_3$.

Each consignment of tablets is marked with its value in equivalent of $CaCO_3$.

NORMAL AND ACID CARBONATES.

It is nearly always of value to determine the proportion of normal and acid carbonates in a water, for it affords a fairly good index to the character of the base with which the carbon dioxide is united. It is a generally accepted idea that the carbonates of the alkaline-earth metals are, when in solution in water, in the form of bicarbonates. For the general purposes of field work it may be considered that all bicarbonates occurring in natural waters are alkaline-earth carbonates and may conveniently be calculated as $CaCO_3$. All normal carbonates, on the other hand, must be alkali carbonates, conveniently calculated as Na_2CO_3. This generalization is not uniformly true, especially in certain classes of western waters. It has been plainly shown, by the work of Messrs. Frank K. Cameron and Lyman J. Briggs, of the Bureau of Soils, United States Department of Agriculture, that there is considerable complexity in the occurrence and equilibrium of carbonates and bicarbonates in waters. There are, however, few practical water problems occurring outside of the alkali-desert regions in which the interpretation of bicarbonates as alkaline-earth carbonates and normal carbonates as alkali carbonates would lead to erroneous results. The field men of the United States Geological Survey are therefore instructed to report bicarbonates as $CaCO_3$ and normal carbonates as Na_2CO_3, in the absence of data which will allow of other interpretations.

The method of determining carbonates and bicarbonates in aqueous solution is discussed by Mr. Frank K. Cameron, chemist of the Bureau of Soils, in Bulletin No. 18 of the United States Department of Agriculture, Bureau of Soils, pages 77–89. The method depends upon the fact that while phenolphthalein reacts with the normal carbonates of the alkali and alkaline-earth metals and not with the bicarbonates, methyl orange reacts with either. The water under investigation is titrated with a standard solution of potassium acid sulphate, using phenolphthalein as an indicator, the first end point being the complete disappearance of the red color. Methyl orange is then added to the solution, and the titration is continued with the same standard until a pink acid reaction is obtained. The amount of standard solution used to reach the first end point is a measure of the amount of normal carbonates, while the total amount used in securing both end points, less twice that for the first end point, is a measure of the bicarbonates.

The reaction taking place before and up to the total neutralization of the phenolphthalein is a conversion of carbonates into bicarbonates and can probably be expressed as follows:

$$2KHSO_4+2Na_2CO_3=Na_2SO_4+K_2SO_4+2NaHCO_3$$

or

$$2KHSO_4+2MgCO_3=MgSO_4+K_2SO_4+Mg(HCO_3)_2.$$

The neutralization of bicarbonates probably takes place in this manner:

$$2KHSO_4+2NaHCO_3=Na_2SO_4+K_2SO_4+2H_2O+2CO_2$$

or

$$2KHSO_4+Mg(HCO_3)_2=MgSO_4+K_2SO_4+2H_2O+2CO_2.$$

It is evident that when the end point with phenolphthalein has been reached there remains as a product of the first reaction, in addition to the bicarbonates originally present, an amount of bicarbonates equal in reacting power to the reaction shown by the phenolphthalein. In other words, double the amount of potassium acid sulphate required to convert the carbonates to bicarbonates, and so destroy the color of the phenolphthalein, is necessary to completely neutralize the normal carbonates as indicated by methyl orange. This must be taken into consideration in computing the results from the titration.

Inasmuch as the reactions taking place when sodium acid sulphate is used must be similar to those with the use of potassium acid sulphate, there is no reason to believe that the tablets now in use in this division may not be substituted for the standard solution suggested by Mr. Cameron. Experiments have been made to determine the accuracy of the results obtainable and are discussed in the following paragraphs.

An unknown amount of thoroughly fused Kahlbaum's sodium bicarbonate was dissolved in distilled water that had previously been boiled to drive out carbonic acid. Twenty-five cubic centimeters of this solution, which should contain only sodium carbonate, was tested by adding phenolphthalein, triturating and dissolving standard tablets of sodium acid sulphate till decolorized, adding methyl orange, and continuing the trituration until the methyl-orange end point was reached. Another equal portion of the solution was tested by adding methyl orange alone and dissolving tablets until the end point was reached. The results in the two cases were as follows:

1. Necessary for phenolphthalein end point	9 tablets.
Excess necessary for methyl-orange end point	9 tablets.
2. Total necessary for methyl-orange end point	18 tablets.

Two solutions were then made, one similar to the first, of sodium carbonate in boiled distilled water, the other of supposedly pure Kahlbaum's sodium bicarbonate in distilled water, also boiled. These were tested by solution of tablets, using first phenolphthalein and then methyl orange as an indicator, as in the first case.

Sodium carbonate solution	25 cubic centimeters.
Necessary for phenolphthalein end point	27 tablets.
Necessary excess for methyl-orange end point	27 tablets.
Sodium bicarbonate solution	25 cubic centimeters.
Necessary for phenolphthalein end point	15 tablets.
Necessary excess for methyl-orange end point	57 tablets.

It is evident that the solution of bicarbonate was impure, 30 tablets out of a total of 72 being required for the neutralization of the normal carbonate. A mixture of these two solutions was then made, as follows:

	Cubic centimeters.
Sodium carbonate solution	100
Solution containing bicarbonate	100
Distilled water (boiled)	200

It is evident that the number of tablets required by this solution, if the method is reliable, will be equal to one-fourth of the sum of all the tablets used to neutralize the two original solutions, 25 cubic centimeters being taken in each case. This should be true, not only of the whole determination, but of each part. Tests made of the mixture resulted as follows:

Necessary for phenolphthalein end point	10½ tablets.
Necessary excess for methyl-orange end point	21 tablets.

Inspection of these figures and comparison with those preceding indicate that, in so far as it is possible to judge under the conditions, the method is accurate and reliable.

To make the determination, measure a convenient quantity of the water to be tested into a porcelain mortar and add 4 drops of phenolphthalein (1 per cent). Triturate the standard $NaHSO_4$ tablets in the mortar, one at a time, until the color disappears. Note the number of tablets and then add 4 drops of methyl orange (1 per cent). Continue the titration with the tablets until the orange color of the solution changes to a faint pink. Then note the total number of tablets used in both titrations. The equivalent of the sodium acid-sulphate tablets is usually given in terms of calcium carbonate. Therefore the amount of bicarbonates in the water may be calculated directly from this valuation. In order to calculate the normal carbonates as Na_2CO_3, it will be necessary to multiply the valuation of the sodium-sulphate tablets given by 1.06, the conversion factor of $CaCO_3$ to Na_2CO_3.

For computation of the normal carbonates in parts per million, double the number of tablets used for the decolorization of phenolphthalein, multiply by the equivalent of each tablet in milligrams Na_2CO_3. Then multiply this product by 1,000 and divide the whole by the number of cubic centimeters of the sample tested. To find bicarbonates in parts per million, subtract from the total number of tablets used in the two titrations twice the number required for the phenolphthalein end point and multiply this difference by the equivalent of each tablet in terms of calcium carbonate. Then multiply this product by 1,000 and divide by the number of cubic centimeters of water tested.

For the convenient expression of the above in formulas, assume the following symbols:

A = equivalent of $NaHSO_4$ tablets in terms of milligrams of $CaCO_3$.
B = equivalent of $NaHSO_4$ tablets in terms of milligrams of Na_2CO_3.

The conversion factor being 1.06, we have

$$A = \frac{B}{1.06} \text{ or } B = 1.06A.$$

n = number of tablets used to reach first or phenolphthalein end point.
N = number of tablets used to reach second or methyl-orange end point.
W = amount in cubic centimeters of water tested.

Then for the determination of normal carbonates we have the formula

$$\frac{2{,}000nB}{W}$$

and for the determination of bicarbonates

$$\frac{1{,}000(N-2n)A}{W}$$

The results of the two above equations will be the expression of parts per million.

SULPHATES.

Water generally contains either one or more of the sulphates of sodium, potassium, calcium, magnesium, and iron. If present in minute amounts the effect of any or all of them is negligible, but if they appear in large proportions they do damage in every branch of science or industry in which it is necessary to use water. Calcium, magnesium, and iron sulphates damage boilers, textiles, soaps, malt liquors, paper, and many other manufactured products, while they render water undesirable for domestic purposes. The sulphates of sodium and potassium are troublesome in boilers, and damage crops when water containing large amounts is used for irrigation. A knowledge of the amount of sulphates in a water is of great importance.

The determination of sulphates, as it is usually performed in the laboratory, is a slow, laborious, and expensive process. A field method has, however, been devised by which the sulphates can be determined in a few minutes and with a degree of accuracy sufficient for all practical purposes. The determination involves the use of the Jackson turbidimeter, described on previous pages. In the following paragraphs the determination of sulphates is described by the originator of the method, Mr. Daniel D. Jackson:

DETERMINATION BY TURBIDIMETER.

Knowledge of the amount of sulphates in a water to be used for industrial purposes is especially important. The scale which is most troublesome to remove from boilers is produced by the precipitation of sulphate of lime. If the amount of sulphate is considerable the determination of lime may be made by the turbidimeter with a fair degree of accuracy. The method is as follows:

To 100 cubic centimeters of water to be tested add 1 cubic centimeter of hydrochloric acid (1–1) and 1 gram of solid barium-chloride crystals. If the amount of sulphate is low, 200 or 300 cubic centimeters of water must be treated in order to fill the longer tube employed. In this case add 1 cubic centimeter of acid and 1 gram of barium chloride for each 100 cubic centimeters of water taken.

The mixture should be allowed to stand for ten minutes, and in this time it should be frequently shaken. It is best to employ a bottle for this purpose. Treating the water in the cold with solid barium chloride causes the barium sulphate to be precipitated in a finely divided state, and the turbidity produced may then be read by either the candle or the electric turbidimeter.

In the lower part of the tube the end point is taken when the hazy cross of light disappears. This is a higher reading than the point where the sharp cross disappears. Higher up in the tube there is no hazy cross of light, and the end point is the disappearance of the sharp cross of light. When this point is obtained, remove the glass tube and find the depth of the liquid (using the bottom of the meniscus in reading). Refer this reading to the accompanying table to obtain the parts per million or grains per gallon of sulphate present.

The readings of these instruments are only to a very slight extent affected by the amount of light used, so that a fairly wide variation in this respect gives little or no error in the result. This is also true with variations in the color of different natural waters. The reason for this lies in the fact that the end point is not to any great extent dependent upon the amount of light cut out but to the complete covering up of the image of light by the particles in suspension. It is surprising to find that the interposition of disks, even of highly colored glass, produces little or no effect upon the end point.

In using the electric turbidimeter, if the image becomes perceptibly dim the battery is replaced by a fresh one, but if the analyst is careful to keep the light turned off except when actually making readings the batteries will last for a considerable period of time. Fresh electric bulbs and batteries may be obtained from the Howard Electric Novelty Company, 221–227 Canal street, New York City, or 183 Lake street, Chicago. If any parts of the instrument are lost or broken they may be replaced by Baker & Fox, 83 Schermerhorn street, Brooklyn, N. Y.

Table for converting readings in depths by the turbidimeter into parts per million or grains per gallon of sulphate.

Reading in centimeters.	Parts per million, SO_3.	Grains per United States gallon, SO_3.	Reading in centimeters.	Parts per million, SO_3.	Grains per United States gallon, SO_3.
1.0	522	30.5	3.9	144	8.4
1.1	478	28.0	4.0	140	8.2
1.2	442	25.8	4.1	137	8.0
1.3	410	24.0	4.2	133	7.8
1.4	383	22.4	4.3	131	7.7
1.5	359	21.0	4.4	128	7.5
1.6	338	19.8	4.5	125	7.3
1.7	319	18.6	4.6	122	7.1
1.8	302	17.7	4.7	119	7.0
1.9	287	16.8	4.8	117	6.8
2.0	273	16.0	4.9	115	6.7
2.1	261	15.3	5.0	113	6.6
2.2	250	14.6	5.1	110	6.4
2.3	239	14.0	5.2	108	6.3
2.4	230	13.5	5.3	106	6.2
2.5	221	12.9	5.4	104	6.0
2.6	213	12.4	5.5	103	6.0
2.7	205	12.0	5.6	101	5.9
2.8	198	11.6	5.7	99	5.8
2.9	191	11.2	5.8	97	5.7
3.0	185	10.8	5.9	96	5.6
3.1	179	10.5	6.0	94	5.5
3.2	173	10.1	6.1	93	5.4
3.3	168	9.8	6.2	91	5.3
3.4	164	9.6	6.3	90	5.2
3.5	159	9.3	6.4	88	5.1
3.6	155	9.1	6.5	87	5.1
3.7	151	8.8	6.6	86	5.0
3.8	147	8.6	6.7	84	4.9

Table for converting readings in depths by the turbidimeter into parts per million or grains per gallon of sulphate—Continued.

Reading in centimeters.	Parts per million, SO_3.	Grains per United States gallon, SO_3.	Reading in centimeters.	Parts per million, SO_3.	Grains per United States gallon, SO_3.
6.8	83	4.9	12.4	46	2.7
6.9	82	4.8	12.6	45	2.6
7.0	81	4.8	12.8	44	2.6
7.1	80	4.7	13.0	43	2.5
7.2	79	4.7	13.5	42	2.5
7.3	78	4.6	14.0	41	2.4
7.4	77	4.5	14.5	39	2.3
7.5	76	4.4	15.0	38	2.3
7.6	75	4.4	15.5	37	2.2
7.7	74	4.3	16.0	36	2.1
7.8	73	4.3	16.5	35	2.0
7.9	72	4.2	17.0	34	2.0
8.0	71	4.2	17.5	33	1.9
8.1	70	4.1	18.0	32	1.9
8.2	69	4.0	18.5	31	1.8
8.3	68	4.0	19.0	30	1.8
8.5	67	3.9	20.0	29	1.7
8.6	66	3.9	21.0	28	1.7
8.7	65	3.8	22.0	27	1.6
8.8	64	3.8	22.5	26	1.6
9.0	63	3.7	23.0	25	1.5
9.1	62	3.7	24.0	24	1.4
9.3	61	3.6	25.0	23	1.3
9.5	60	3.6	26.5	22	1.3
9.7	59	3.5	28.0	21	1.2
9.8	58	3.4	29.0	20	1.2
10.0	57	3.3	31.0	19	1.1
10.2	56	3.3	33.0	18	1.1
10.4	55	3.2	35.0	17	1.0
10.6	54	3.2	37.5	16	1.0
10.8	53	3.1	40.0	15	.9
11.0	52	3.1	43.0	14	.9
11.2	51	3.0	46.5	13	.8
11.4	50	3.0	50.0	12	.7
11.6	49	2.9	55.5	11	.6
11.8	48	2.8	62.0	10	.6
12.0	47	2.7	68.0	9	.5

PRECAUTIONS IN USE OF INSTRUMENT.

1. The same care should be taken as in measuring turbidity to have the turbidimeter in good running order. 2. Always shake the solution until all the barium chloride is dissolved. Otherwise a flaky precipitate may be obtained. 3. Since the barium-sulphate precipitate is very heavy, the solution should be mixed frequently by pouring and shaking while readings are being made. 4. Only sufficient hydrochloric acid should be added to make the water acid.

CALCIUM.

The determination of calcium is made by means of the turbidimeter, the method being similar to that described in the chapter on sulphates. It is the latest, and therefore the least known, of all the determinations here described. While the results which have been reached by this method appear to be satisfactory, no particular plan has yet been offered to determine certain necessary facts with reference to the behavior of precipitated calcium oxalate. The method depends upon the turbidity produced by the precipitation of calcium oxalate upon the addition of ammonium oxalate to the water under investigation. Whether or not the variations which occur in the character of this precipitate under different conditions are sufficient to affect appreciably the degree of turbidity produced is a matter which is yet to receive attention.

The test is made in the following manner: To 100 cubic centimeters of the water to be tested add a few drops of ammonium hydroxide, NH_4OH. The amount added should be barely sufficient to impart to the water a perceptible ammoniacal odor. Then add crystals of ammonium oxalate. The amount of crystals to be added depends, of course, upon the amount of lime in the water. As this is yet undetermined, care should be taken to add an excess of ammonium oxalate. Mix thoroughly and allow the solution to stand for ten or fifteen minutes. Then determine the turbidity with the Jackson turbidimeter precisely as described in previous pages in the case of sulphates and state the amount of calcium according to the table given below.

The treatment above described will precipitate materials other than calcium, but they are usually in so small a proportion in natural waters that they do not often give trouble. The most frequent complication arises from the precipitation of magnesium on the addition of ammonia. If the precipitate is sufficient in amount to materially affect the degree of turbidity it should be filtered before the addition of ammonium oxalate.

The table given below for the determination of calcium is less satisfactory than that for sulphates, and it will probably be found that corrections must be made as future experience dictates.

Table for determining calcium with Jackson's turbidimeter.

Reading in centimeters.	Parts per million.	Reading in centimeters.	Parts per million.	Reading in centimeters.	Parts per million.	Reading in centimeters.	Parts per million.
1.0	1,150	4.0	167	7.0	80	10.0	53
1.1	1,000	4.1	162	7.1	78	10.2	52
1.2	890	4.2	156	7.2	77	10.4	51
1.3	795	4.3	151	7.3	76	10.6	50
1.4	715	4.4	146	7.4	74	10.8	49
1.5	650	4.5	142	7.5	73	11.0	48
1.6	595	4.6	137	7.6	72	11.2	47
1.7	550	4.7	133	7.7	71	11.4	46
1.8	505	4.8	130	7.8	70	11.7	45
1.9	470	4.9	126	7.9	69	11.9	44
2.0	435	5.0	123	8.0	68	12.2	43
2.1	410	5.1	119	8.1	67	12.4	42
2.2	380	5.2	116	8.2	66	12.7	41
2.3	360	5.3	113	8.3	65	13.0	40
2.4	340	5.4	110	8.4	64	13.3	39
2.5	320	5.5	107	8.5	64	13.7	38
2.6	305	5.6	105	8.6	63	14.0	37
2.7	288	5.7	102	8.7	62	14.4	36
2.8	274	5.8	100	8.8	61	14.8	35
2.9	261	5.9	98	8.9	60	15.3	34
3.0	248	6.0	96	9.0	60	15.7	33
3.1	238	6.1	94	9.1	59	16.2	32
3.2	228	6.2	92	9.2	58	16.7	31
3.3	218	6.3	90	9.3	57	17.3	30
3.4	209	6.4	88	9.4	57	17.9	29
3.5	200	6.5	87	9.5	56	18.5	28
3.6	194	6.6	85	9.6	55	19.2	27
3.7	186	6.7	84	9.7	55	20.0	26
3.8	179	6.8	82	9.8	54	21.7	24
3.9	173	6.9	81	9.9	54	22.7	23

INSTRUMENTS AND REAGENTS.

The field case (see Pl. IV) contains the instruments and reagents described below:

1. A Berkfeld army filter for removing suspended matter from water under investigation. The porous stone in this filter should

UNITED STATES GEOLOGICAL SURVEY FIELD CASE.

be removed from the tube frequently and thoroughly cleansed with the small stiff brush provided for this purpose. If it is desired to secure sterile water, or if the only water available is known to be polluted and a supply for drinking purposes is desired, the filter stone should be boiled or baked frequently. Watch the filter stone closely for cracks and imperfections. When water is pumped through the filter, care should be taken that the suction end does not rest on sand or mud; such materials, if drawn into the buckets of the pump, are troublesome and materially shorten the term of usefulness of the filter.

2. One or more leather cases containing tubes of reagent tablets. The equivalent of each tablet of the various reagents should be noted on a slip pasted upon the inside of the case.

The tablets are packed in tubes to prevent mechanical agitation. This is highly important, because if the tablets are loosely packed a loss of active chemical reagent is inevitable. Tablets which show signs of extraordinary wear should be rejected. In using a tube one of the cork stoppers should be removed and the tablets poured out as needed. When the end point is reached the cork should be replaced and the stoppers in the opposite end of the tube should be pushed through the lumen until the tablets remaining in the tube are projected against the opposite stopper, thus holding them securely.

Sodium-oleate tablets are packed in unmarked transparent glass tubes. Two grades of silver-nitrate tablets are usually issued. The tubes containing tablets of the higher equivalent have a cross etched on the glass, while those with the lower equivalent are etched with a single transverse line. The sodium acid-sulphate tablets are packed in transparent glass tubes, upon each of which is etched the symbol $NaHSO_4$.

3. One case containing four aluminum tubes for natural color and for iron determinations. There will also be provided brown-glass disks for the color determination or red-glass ones for the iron, or both. The equivalent of each disk in terms of parts per million is engraved on the aluminum rim.

4. One Jackson candle or electric turbidimeter with two graduated cylinders for same. An extra electric bulb, a ground-glass disk, a brass cross disk, a standard English candle, and a dry battery will be provided with each turbidimeter. The field observer should not use any dry battery which has been in his possession over sixty days, irrespective of the intensity of the light produced by it. Candles other than the standard English sperm should not be used.

The candle turbidimeter should be used in preference to the electric whenever possible, as the former is the more steady instrument and insures uniformity of results. If, however, it is necessary to make determinations in exposed places when the wind is blowing, the electric turbidimeter must be used, as the slightest flickering of the candle flame will introduce errors in the determinations. Whenever possible, water samples should be carried to a convenient shelter and assayed.

5. Seven special dropping bottles containing the following reagents: Concentrated nitric acid (HNO_3); concentrated hydrochloric acid (HCl); concentrated ammonium hydroxide (NH_4OH); two per cent solution of potassium sulphocyanide ($KCNS$); five per cent solution of potassium chromate (K_2CrO_4); one per cent solution of phenolphthalein; one-tenth per cent solution of methyl orange.

Care should be taken to close the stoppers in these dropping bottles where they are packed in the cases.

6. Two salt-mouth bottles containing pure crystals of barium chloride and ammonium oxalate.

All bottles containing chemicals have etched labels, except the indicators, the colors and odors of which are sufficient for identification.

7. One heavily glazed porcelain mortar and pestle.

8. One round-bottom glass bottle, with glass pestle, for hardness determination.

9. One small horn spoon for handling crystals noted in section 6.

10. One 5 c. c. pipette in case for general use in measuring small amounts of liquid.

11. One centigrade thermometer in brass case.

12. One loose-leaf notebook. This notebook is made up of printed cards, with every alternate leaf a blank.

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PUBLICATIONS OF UNITED STATES GEOLOGICAL SURVEY.

[Water-Supply Paper No. 151.]

The serial publications of the United States Geological Survey consist of (1) Annual Reports, (2) Monographs, (3) Professional Papers, (4) Bulletins, (5) Mineral Resources, (6) Water-Supply and Irrigation Papers, (7) Topographic Atlas of United States—folios and separate sheets thereof, (8) Geologic Atlas of United States—folios thereof. The classes numbered 2, 7, and 8 are sold at cost of publication; the others are distributed free. A circular giving complete lists may be had on application.

Most of the above publications may be obtained or consulted in the following ways:

1. A limited number are delivered to the Director of the Survey, from whom they may be obtained, free of charge (except classes 2, 7, and 8), on application.

2. A certain number are allotted to every member of Congress, from whom they may be obtained, free of charge, on application.

3. Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be had at practically cost.

4. Copies of all Government publications are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted by those interested.

The Professional Papers, Bulletins, and Water-Supply Papers treat of a variety of subjects, and the total number issued is large. They have therefore been classified into the following series: A, Economic geology; B, Descriptive geology; C, Systematic geology and paleontology; D, Petrography and mineralogy; E, Chemistry and physics; F, Geography; G, Miscellaneous; H, Forestry; I, Irrigation; J, Water storage; K, Pumping water; L, Quality of water; M, General hydrographic investigations; N, Water power; O, Underground waters; P, Hydrographic progress reports. This paper is the eleventh in Series L, the complete list of which follows. (PP=Professional Paper; B=Bulletin; WS=Water-Supply Paper.)

SERIES L—QUALITY OF WATER.

WS 3. Sewage irrigation, by G. W. Rafter. 1897. 100 pp., 4 pls. (Out of stock.)

WS 22. Sewage irrigation, Pt. II, by G. W. Rafter. 1899. 100 pp., 7 pls. (Out of stock.)

WS 72. Sewage pollution near New York City, by M. O. Leighton. 1902. 75 pp., 8 pls.

WS 76. Flow of rivers near New York City, by H. A. Pressey. 1903. 108 pp, 13 pls.

WS 79. Normal and polluted waters in northeastern United States, by M. O. Leighton. 1903. 192 pp., 15 pls.

WS 103. Review of the laws forbidding pollution of inland waters in the United States, by E. B. Goodell. 1904. 120 pp.

WS 108. Quality of water in the Susquehanna River drainage basin, by M. O. Leighton, with an introductory chapter on physiographic features, by G. B. Hollister. 1904. 76 pp., 4 pls.

WS 113. Strawboard and oil wastes, by R. L. Sackett and Isaiah Bowman. 1905. 52 pp., 4 pls.

WS 121. Preliminary report on the pollution of Lake Champlain, by M. O. Leighton. 1905. 119 pp., 13 pls.

WS 144. The normal distribution of chlorine in the natural waters of New York and New England, by D. D. Jackson. 1905. 31 pp., 5 pls.

WS 151. Field assay of water, by M. O. Leighton. 1905. 77 pp., 4 pls.

Correspondence should be addressed to

THE DIRECTOR,
UNITED STATES GEOLOGICAL SURVEY,
WASHINGTON, D. C.

OCTOBER, 1905.

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Author.

Leighton, Marshall O[ra] 1874–

. . . Field assay of water, by Marshall O. Leighton. Washington, Gov't print. off., 1905.

77, iii p. illus., IV pl., diagrs. 23^{cm}. (U. S. Geological survey. Water-supply and irrigation paper no. 151)

Subject series: L, Quality of water, 11.

1. Water—analysis.

Subject.

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U. S. Geological survey.

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no. 151. Leighton, M. O. Field assay of water. 1905.

Reference.

U. S. Dept. of the Interior.

see also

U. S. Geological survey.

Water-Supply and Irrigation Paper No. 152 Series L, Quality of Water, 12

DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

A REVIEW

OF THE

LAWS FORBIDDING POLLUTION OF INLAND WATERS IN THE UNITED STATES

SECOND EDITION

By EDWIN B. GOODELL

WASHINGTON
GOVERNMENT PRINTING OFFICE
1905

CONTENTS.

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., August 17, 1905.

SIR: I transmit herewith a manscript entitled "A Review of the Laws Forbidding Pollution of Inland Waters of the United States," prepared by Edwin B. Goodell, and request that it be published as a water-supply and irrigation paper. This paper is a second edition of Water-Supply Paper No. 103, published last year. The subject-matter has been brought to date by the incorporation of the statutes passed since the first edition was prepared, and the section on pollution under the common law has been amplified to include the arid States and Territories.

One of the important features involved in the determination of the water supplies of the United States and the preparation of reports upon the best methods of utilizing water resources is the character of those supplies. In the more populous sections of the country the quality of water is dependent to a large degree upon the amount and character of the pollution which is allowed to discharge into the streams. Therefore it has been found desirable to study different State laws regulating and controlling this matter and to determine the scope of the work to a large degree according to them.

Mr. Goodell has presented the subject of antipollution laws in a manner which will be of assistance to public officials, water companies, manufacturers, farmers, and legislators, rather than to members of the bench and bar. The broad legal principles under which antipollution statutes become operative are explained, and important court decisions are quoted to show authority for various deductions. The statutes enacted in the different States are classified according to the general scope, and an opportunity is thereby afforded to compare their effectiveness and desirability. In short, the paper provides specific information necessary to a popular knowledge of the conditions in each State with respect to one feature of the conservation of natural water resources. Its distribution should be of material assistance in bringing about a general apprehension of correct principles upon the subject.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

A REVIEW OF THE LAWS FORBIDDING POLLUTION OF INLAND WATERS IN THE UNITED STATES.

By Edwin B. Goodell.

This subject naturally divides into two parts: (1) A summary of the common law upon the subject of water pollution—i. e., the law as pronounced and determined by the courts independently of legislative action, and (2) a summary or abstract of the statutes enacted by the various legislatures for the correction of the evil.

WATER POLLUTION UNDER THE COMMON LAW.

The full treatment of this branch of the subject involves the examination of the very numerous decisions which have been rendered by the courts in England and the United States in the determination of litigation arising from alleged violations of the right to have inland waters preserved in their natural state. It necessarily follows that a full treatment of this branch of the subject would be beyond the scope of this paper. It is not the purpose of the present publication to furnish a complete work upon water pollution for the use of members of the bench and bar, but rather to put into the hands of public officials and others who may be interested in the subject a guide for their action and references to the sources from which a more exhaustive knowledge of the subject may be obtained if required.

No attempt, accordingly, will be here made to present a detailed statement of the entire law against water pollution as it exists independently of statutes, but this branch of the subject will be confined to a statement of the general principles which are to be deduced from the decisions, with references to some of the leading cases.

PRINCIPLES AND DECISIONS.

CLASSIFICATION.

These principles and decisions have been classified and are presented in the following groups:

A. The rights of riparian owners to pure water as against one another.

B. The rights of the public (as distinguished from individual owners) to have inland waters kept free from pollution by riparian owners or others.

C. The conditions under which, and the extent to which, public municipalities may use inland waters in disposing of sewage matter from public sewers.

A. RIGHTS OF RIPARIAN OWNERS TO PURE WATER AS AGAINST ONE ANOTHER.

In contemplation of law the water flowing over the land is part of the realty and belongs to the owner of the soil. But the latter's ownership thereof is a qualified one. He may use it in certain ways as it passes, may take from it for his own use to a certain extent, and may thus, incidentally, somewhat diminish its volume and slightly alter its character. But its nature is to pass on to the owners of the adjoining soil, and the next owner has precisely the same rights therein as every other owner. It follows, therefore, that as no riparian owner of a stream may appropriate all the water which comes to him, neither may he so corrupt or pollute it as to injure the other owners by diminishing the value of their property in the natural stream. This prohibition is independent of any statute; it is a part of the law of the land, except in certain of the arid and mining States of the West; its application in these is discussed on pages 21–23.

The conflict of rights between the several owners has given rise to litigation in many hundreds of instances, and it is impossible to give a rule, limiting the owner's right to use the water of a stream as it passes, more exact than this: Every owner may make such use of the water for farming and domestic purposes as is reasonable, and in the States in which the doctrine of prior appropriation obtains may use the water which he has acquired by appropriation, and the lower owners must accept the diminution and perturbation of the water which necessarily follows from this reasonable use.

If the use for farming or domestic purposes is challenged by another owner, the question of its reasonableness, in that case, is to be determined by court or jury as a question of fact.

If the water is used for any other than farming or domestic purposes, it must be such a use as will not change the character of the water from its natural state or make it less useful to other owners.

If the riparian owner cast sewage, filth, or waste material therein, he does it at his peril.

Independent of statutory provisions there is a remedy for these wrongs in the following ways:

By private suit against the wrongdoer for damages.

By injunction when the wrong is a continuing one.

By indictment when the injury affects the rights of the public.

Where the acts causing the pollution are done in one jurisdiction and the injuries suffered are in another, the injured party has his remedy in a civil action to the same extent as if the injurious act and the resulting injury were in the same jurisdiction.

These general principles will be found to be fully sustained by the cases. The following are given, not as an exhaustive list, but to enable the reader to find authorities if his needs require:

Alabama:

- Drake v. Iron Co., 14 So. Rep., 749; 102 Ala., 501; 24 L. R. A., 64; 48 Am. St. Rep., 77.
- Tenn. Coal Co. v. Hamilton, 14 So. Rep., 167.
- Lewis v. Stein, 16 Ala., 214.

Arkansas:

- State v. Chapin, 17 Ark., 561.

California:

- Potter v. Froment et al., 47 Cal., 165.
- People v. Elk River Mill and Lumber Co., 107 Cal., 214; S. C. 40 Pac. Rep., 486.
- Mining Co. v. Mining Co., 48 Pac. Rep., 828.
- People v. Gold Run Ditch Co., 66 Cal., 138; 56 Am. Rep., 80; 4 Pac. Rep., 1152.[a]

Colorado:

- City of Durango v. Chapman, 60 Pac. Rep., 635.

Connecticut:

- Morgan v. Danbury, 67 Conn., 484.
- Nolan v. New Britain, 69 Conn., 668.

Georgia:

- Satterfield v. Rowan, 83 Ga., 187; S. C. 9 S. E. Rep., 677.

Indiana:

- Muncie Pulp Co. v. Martin, 55 N. E. Rep., 796.
- State v. Herring (Ind., 1897), 48 N. E. Rep., 598.
- State v. Wabash Paper Co., 48 N. E. Rep., 653.
- Weston Paper Co. v. Pope, 155 Ind., 394.
- Indianapolis Water Co. v. Am. Straw Board Co., 57 Fed. Rep., 1000.

Iowa:

- Ferguson v. Mfg. Co., 77 Ia., 576; S. C. 42 N. W. Rep., 448.
- Kinnaird v. Oil Co. (Ky.), 12 S. W. Rep., 937.

Maine:

- Gerrish v. Brown, 51 Me., 256, 81 Am. Dec., 569.

Maryland:

- Baltimore v. Warren Mfg. Co., 59 Md., 96.
- Price v. Lawson, 74 Md., 499.

Massachusetts:

- Ball v. Nye, 99 Mass., 582.
- Martin v. Gleason, 139 Mass., 183.
- Merrifield v. Lombard, 13 Allen, 16.
- Woodward v. Worcester, 121 Mass., 245.
- Dwight Printing Co. v. Boston, 122 Mass., 583.
- McGenness v. Adriatic Mills, 116 Mass., 177.

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[a] This case was one brought in behalf of the people to restrain a public nuisance, caused by discharging the refuse from mining operations into an unnavigable stream. The injunction was granted and it was held that the right to pollute the stream in this manner could not be gained by prescription.

Minnesota:
    Roller Mills *v.* Wright, 30 Minn., 254.
Mississippi:
    Mississippi Mills *v.* Smith, 69 Miss., 299; S. C. 11 So. Rep., 26.
Missouri:
    Smith *v.* Conathy, 11 Mo., 517.
New Hampshire:
    Hayes *v.* Waldron, 44 N. H., 580.
New Jersey:
    Holsman *v.* Boiling Springs Co., 1 McCart., 335.
    Acquackanonk Water Co. *v.* Watson, 2 Stew. Eq., 366.
    Beach *v.* Sterling Iron and Zinc Co., 9 Dick., 65.
    Same case affirmed on appeal, 10 Dick, 824.
    (See the opinion of Pitney, V. C., in the last-cited case, given in full on pp. 11–20.)
    O'Riley *v.* McChesney, 3 Lans., 278.
    Covert *v.* Cranford, 141 N. Y., 521.
    Townsend *v.* Bell, 24 N. Y. S. (70 Hun, 557), 193.
    Smith *v.* Cranford, 32 N. Y. S., 375.
Ohio:
    The Columbus, etc., Co. *v.* Tucker, 48 Ohio St., 41; S. C. 26 N. E. Rep., 630.
    Thayer *v.* Brooks, 17 Ohio, 489.
Pennsylvania:
    Elder *v.* Lykens Valley Coal Co., 157 Pa. St., 490.
    Hindson *v.* Markle, 171 Pa. St., 138.
    Stevenson *v.* Ebervale Coal Co., 201 Pa. St., 112.
Rhode Island:
    Stillman *v.* Mfg. Co., 3 W. & M. (R. I.), 546.
    Richmond Mfg. Co. *v.* Atlantic De Laine Co., 10 R. I., 106.
South Carolina:
    Threatt *v.* Mining Co. (S. C., 1897), 26 S. E. Rep., 970.
Vermont:
    Snow *v.* Parsons, 28 Vt., 459.
    Canfield *v.* Andrew, 54 Vt., 1.
Wisconsin:
    Middlestadt *v.* Starch Co. (Wis.), 66 N. W. Rep., 713.
    Hazeltine *v.* Case, 46 Wis., 391.
    Greene *v.* Nunnemacher, 36 Wis., 50.
Wyoming:
    Howell *v.* Johnson, 89 Fed. Rep., 556.[a]
English:
    Mason *v.* Hill, 5 B. & Ad., 1.
    Embry *v.* Owen, 6 Exch., 353.
    Wood *v.* Waud, 3 Exch., 748.
    Bealey *v.* Shaw, 6 East, 208.

CASE IN EXCEPTION.

A single case in Pennsylvania seems to create an exception to the operation of the principles above stated, viz, Sanderson *v.* Pennsylvania Coal Company, 113 Pa. St., 126.

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[a] In this case the injury arose from an act done in Montana, but the injurious result occurred in the State of Wyoming.

This was a case brought by a riparian owner who had established a home on the banks of a stream, after ascertaining, by a careful investigation, that its waters were uncontaminated by any influx of deleterious matter, and who used the waters of the stream for domestic purposes. Subsequently a coal mine was opened higher up the stream, and the mining company, in the course of its mining operations, pumped the water from the mine to the surface, where it ran into this stream and rendered the water unfit for domestic use. The case was bitterly contested, and came before the court several times. (See 86 Pa. St., 401; 94 Pa. St., 303, and 102 Pa. St., 370.)

In the final decision the court refused damages to the riparian owner. The reasoning of the court indicates that this result was due to its unwillingness to impose upon the immense coal-mining interests of the State the burden of paying for the damage to property in the water of streams caused by their operations; but the reason given for the decision, in the court's attempt to harmonize it with the principles firmly established by precedent in Pennsylvania, was that the water which the defendant conducted into the stream was contaminated only by the coal, which was a natural product, and hence was said to be conducted into the stream in its "natural state." This reasoning is specious, since the presence of coal in the brook was due wholly to the operations of the defendant company, the stream in its natural state showing no trace of coal, and the doctrine thus established for Pennsylvania has not found favor in any other jurisdiction.

But in subsequent decisions the courts of Pennsylvania have been careful not to extend the force of Sanderson *v.* Pennsylvania Coal Company beyond the single act of turning the natural drainage from a mine into a stream. (See Elder *v.* Lykens Valley Coal Co., 157 Pa. St., 490; Hindson *v.* Markle, 171 Pa. St., 138, and Stevenson *v.* Ebervale Coal Co., 201 Pa. St., 112.)

OPINION OF VICE-CHANCELLOR PITNEY, OF NEW JERSEY.

The whole subject was thoroughly treated in Beach *v.* Sterling Iron and Zinc Company (9 Dick. (N. J.), 65).

This was an action for an injunction, brought by the manufacturers of a white tissue paper against a mining company, the water from whose mines was pumped into the stream above the paper works and befouled the water, making it unfit for the purposes of the complainant. The opinion gives a careful and most lucid and interesting review of the course of decisions sustaining and enforcing the rights of riparian owners upon streams above tide water to have the water in the stream maintained in its natural condition. The decision of the court in this case was affirmed by the court of errors and appeals

(10 Dick., 824) upon the opinion of the court below, which is given here in full:

The material facts of the case are undisputed. The only dispute is as to the degree of discoloration caused by the defendant's operations and the length of time over which such discoloration extended.

The facts clearly established are as follows:

The Wallkill River rises in the southern part of Sussex County and flows upon a course nearly north, passing through the villages of Franklin and Hamburg. At the latter place is situated an artificial pond, called the Furnace Pond, caused by an old dam, upon which, for several years, has been a paper mill driven by the waters of the river from that pond. The complainant, Beach, purchased this water power and lands connected with it in the summer of 1891, for the purpose of erecting a plant for the manufacture of what is known as white tissue paper. Associated with him were two gentlemen by the name of Sparks, who had previously been engaged in the business of waxing white tissue paper according to a process which they controlled, and the project was to both manufacture and wax, for market, white tissue paper. For that purpose the corporation was formed, of which Mr. Beach and the Messrs. Sparks were stockholders, and the latter were the active managers. A large amount of money was spent in erecting a plant between the date of the purchase and the 1st of February, 1892, when they commenced the manufacture of white tissue paper and carried it on with success for about a year.

The manufacture of such paper requires a perfectly clear, pure water, and before purchasing the Hamburg water power the complainants inspected the stream and inquired as to its character for clearness, and satisfied themselves that they would be able to use it for making white tissue paper without incurring the expense of filtration, and their experience for a year proved that their expectations were just.

In the month of February, 1893, complaints began to come in from the purchasers of their paper that it was deteriorating in the matter of whiteness, and they investigated the cause. The pond was frozen over, but they knew by reputation that mining operations were being carried on at Greenspot by the defendant, and they went there March 1 and found a stream of highly colored water flowing from the defendant's mine shaft into the river, traced its effect in discoloration to their pond, and by subsequent observations by themselves and others in the neighborhood traced its effect not only in and through the Furnace Pond, but for miles down the river to the north of Hamburg. In fact, several respectable and credible witnesses, called by the complainants, testified to the discoloration of the water in the Furnace Pond and beyond, and the complainants were stopped by the court from producing further evidence on that subject in the opening of their case. Several witnesses called by the defendant, among them its superintendent, corroborated this evidence, and there is no attempt to meet it.

The color was a peculiar reddish-yellow tint, which was in marked contrast with the discoloration due to the ordinary road and field wash after a heavy storm or spring thaw.

This peculiar discoloration continued throughout the month of March and, with some intermissions and variations in degree of discoloration, through the month of April. Complainants early in March were obliged to stop the making of white tissue paper. Negotiations between the parties for some arrangement of the matter failing, the bill was filed on the 21st of April, 1893.

The immediate origin of the discoloration was as follows: The defendant corporation was organized by two gentlemen by the name of Heckscher and two by the name of Wetherill for the purpose of reaching and working a bed of franklinite ore which had been located by boring exploration at a depth of about a thousand feet below the surface near this point called Greenspot. It was the continuation of a seam of ore for many years worked to the southwest of Greenspot by two companies, one of which—viz, the Lehigh Zinc and Iron Company—was owned and controlled by the Heckschers and Wetherills. In the spring or early summer of 1891 the defendant commenced to sink a perpendicular shaft, known as the "Parker shaft," 10 by 20 feet in diameter, and after passing through a small amount of superincumbent earth struck solid limestone rock. It continued the working without cessation until August 11, 1892, when, having attained a depth of 560 feet (many feet lower than the bed of the Wallkill), the workmen struck a water-bearing fissure or rent in the rock, which instantly flooded the mine and drove them out. Previous to that time they had encountered small seams or fissures from time to time, producing a little water and sometimes a little mud, which they pumped up, of course, carried through a trough or trunk several hundred feet westerly toward the Wallkill till it reached a small spring run, where it was discharged, and from thence it ran into the Wallkill. The amount of water up to August was small, and its discoloration was slight, so that it was not felt or observed by complainants. The influx in August, 1892, was discolored by a fine clay, amounting almost to a pigment, having a high reddish-yellow tint and intermixed with a small quantity of very fine sand. This water rose to within 40 feet of the surface, and resisted all attempts to lower it by the pumps then in use and until very large and heavy pumps were introduced. This was done in September. After the shaft filled with water there was no further movement; it became perfectly quiet, and the clay and sand began to settle, so that the water in the upper reach of the shaft became comparatively clear. The first water that was discharged after heavy pumping commenced came from near the top and was but slightly discolored, such discoloration being due to the disturbance of the clay and sand which had settled on the timbering of the shaft. The quantity of water struck in the fissure was so great that with these powerful pumps very slow advance was made, the pumps being lowered from time to time, and the greater the depth attained the less rapid the advance and the greater the discoloration.

On about the 1st of January, 1893, the water was reduced to a depth of 420 feet from the surface, and a delay there occurred of about three weeks, caused by the necessity of establishing a pumping station at that point. When the rapid pumping commenced again, at or near the 1st of February, the discharge was much discolored, and continued growing worse and worse until the bottom was reached, and there, without detailing the circumstances, the greatest discoloration was reached, and continued during the month of March. The discoloring clay is so very fine in its texture that a very slight movement of particles of water with which it comes in contact will thoroughly mix it, and it will only subside in perfectly still water. This accounts for the fact that it did not fully subside in passing through complainants' pond, which is quite narrow, so that it is probable that the volume of the water of the Wallkill causes continued motion throughout its length.

After the shaft had been entirely pumped out and the volume of water stored in the fissure had been entirely exhausted and the flow reduced to the natural supply of the fissure, and the various water channels which had been created throughout it by the sudden drawing off of the water had arrived at what the experts call an "angle of repose," so that no further scouring resulted from the flow of the ordinary quantity of water, there was no discoloration and the water ran clear. This condition was, as claimed by the defendant, reached some time in the summer of 1893, and the case shows that from about the middle of April or the 1st of May till about the middle of July the discolorations were temporary and increasingly infrequent, and usually the result of clearing out the different settling basins, called "sinks," which had been established in the rock at different points in the shaft. Since that time the shaft has been sunk over 200 feet without finding any more water or fissures.

The proof is clear that the result of the contribution of this discolored water to the waters of the river was to render the mixture when it reached complainants' mill unfit, without filtration, for use in making white paper.

An ingenious experiment was made by an expert, as follows: He ascertained, by a rough measurement, that the flow of the river was about forty times that of the output from the mine, and he took a jar of perfectly clear water and mixed with it one-fortieth of its quantity of the dirty water that came from the mine, and exhibited the sample to show to what a slight extent it was discolored.

The dirty water which he used had been confined in a jar for several months, with the result that the fine particles of clay had partially coagulated and gathered into little flakes, and when shaken up did not produce the same degree of discoloration as exhibited when freshly taken from the running stream. But even that experiment showed that the result of so slight a mixture made the whole mass palpably roily. In point of fact, as shown by the evidence of the expert paper makers, a very small admixture of mud or clay will render the water improper, without filtration, for making white tissue paper; and the effect of that evidence is that the river in its ordinary clear state is no clearer than is necessary for that purpose. A very small admixture of coloring or dirty matter renders it unfit for use.

Several matters are urged in defense to this case. First, but faintly, that the doctrine finally established by a bare majority of a divided court in Pennsylvania, in Sanderson v. The Coal Company (86 Pa. St., 401; 94 Pa. St., 303; 102 Pa. St., 370, and 113 Pa. St., 126), should be adopted here. The history of that case, in its various phases, is given by a writer in the American Law Register (n. s.), vol. 1, p. 1 (1894). It was an action, as here, by a riparian proprietor against a mining company for polluting a natural stream with water pumped from its mine. After three decisions by the supreme court of Pennsylvania in favor of the plaintiff's right, that court finally held the contrary and affirmed the right of the coal company to discharge its acid mine water into the creek, without regard to its effect upon lands below, upon the broad ground that the necessities of the mining interests of the Commonwealth required it. This result was attributed by the author of the article in the American Law Register (pp. 5, 18), in part, to a lack of care on the part of the learned judge who prepared the first prevailing opinion (86 Pa. St., 406). The doctrine of that case is shown by that writer to be inharmonious with a long line of previous decisions in Pennsylvania, and has not been, so far as I can learn, followed in any other State—certainly not in this State. It was repudiated in Ohio, whose mining interests are quite large, in the recent and well-considered case of The Columbus, etc., Co. v. Tucker (48 Ohio St., 41). I refer particularly to the lucid expressions of the learned judge found on pages 58 and 62.

It was not suggested on the argument that the doctrine ever had the least foothold in this State. No case of a stream fouled by mining operations has indeed ever, so far as I know, been presented to our courts, but the right of a riparian proprietor to have the waters of the stream come to him unchanged in quality, as well as undiminished in quantity, has been determined in the clearest and most positive manner. In fact, the doctrine stated so tersely by Chancellor Kent in Gardner v. Newburgh (2 Johns. Ch. 162, at p. 166)—"A right to a stream of water is as sacred as a right to the soil over which it flows. It is a part of the freehold"—has always been adhered to by our courts. I need refer only to Holsman *v.* Boiling Spring Co. (1 McCart., 335), and Acquackanonk Water Co. *v.* Watson (2 Stew. Eq., 366). In the last case the right was stated by the learned master in an extremely clear and comprehensive manner, and the decree advised by him was unanimously affirmed on appeal, for the reasons by him given.

The facts of that case are, in a manner, analogous to those here under consideration. Watson owned and operated a bleachery which required for use clear and pure water, which he obtained from a small stream running through his land. The water company, desiring to supply the city of Passaic with potable water, proposed to take this small stream above the bleachery and substitute for it an equal or greater quantity of Passaic River water, drawn from the Dundee Canal and used to drive its pumps. This the court restrained, on the ground that the substituted water was not of equal purity with that abstracted.

There is a line of cases of pollution by mine water in England which sustains the general doctrine. Hodgkinson *v.* Ennor (4 Best and S., 229) was the case, as here, of a paper maker against a miner who had permitted dirty washings of lead ores to run through rents, called "swallets," in limestone rock into a subterraneous stream, rendering the water, which in its course came to plaintiff's paper mill, unfit for use in the manufacture of paper, and the action was sustained by Chief Justice Cockburn and Justices Blackburn and Mellor.

Magor *v.* Chadwick (11 Ad. and E., 571) was a suit by a brewer against a miner.

Pennington *v.* The Brinsop Coal Co. (L. R., 5 Ch. Div. 769) (1877) was a suit by a manufacturer against a coal miner, where the only allegation of injury was that the acid contributed to the water from the mine rendered it less fit for use in the engine boilers driving the machinery of the plaintiff's mill. An injunction was allowed. Defendant relied, without success, upon the ground taken in Sanderson *v.* The Coal Co., supra, that the acid could not be removed from the water; that there was no means of remedying the evil, and an injunction would absolutely stop its work. The learned judge (Fry) refused even to exercise the right given by the English statute to give damages instead of an injunction, relying on Clowes *v.* Staffordshire Waterworks (L. R., 8 Ch. App., 125) (1873), and he declared that he would have granted the injunction, although the present damage was only nominal, because of the injury to the riparian rights of the plaintiff, and such is the doctrine of the case relied on, which was a suit by a silk dyeing and washing establishment against a waterworks company for rendering the water coming to their works less clear and pure.

The English cases dealing with pollution by mine water culminated in the case of Young *v.* Bankier (L. R., App. Cas., 691) (1893), in the House of Lords, on appeal from Scotland. The case was argued, elaborately, of course, before six law lords, whose unanimous judgments were delivered after consideration. The riparian proprietor (Bankier), the plaintiff there, was a distiller, and used the water of the stream in his distilling process, presumably for making mash, for which it was peculiarly fit by reason of its softness. The added mine water did not render it unfit for ordinary purposes—there called primary purposes—but by reason of its hardness rendered it less fit for distilling purposes. Sanderson *v.* The Coal Co. was cited, but the court repudiated its doctrine and was unanimous in judgment in favor of the respondent, who was the plaintiff and had judgment below. Lord Macnaghten, at page 699, says: "Then the appellant urged (precisely as does the defendant here) that working coal was the natural and proper use of their mineral property. They said they could not continue to work unless they were permitted to discharge the water which accumulates in their mine, and they added that this water course is the natural and proper channel to carry off the surplus water of the district. All that may be very true, but in this country, at any rate, it is not permissible in such a case for a man to use his own property so as to injure the property of his neighbor."

There are numerous English cases upon the general right of a riparian proprietor to have the waters of his stream come to him in its natural condition, of which I cite Crossley *v.* Lightowler (L. R., 3 Eq. Cas., 279; 2 Chan. App., 478) (1867); Attorney-General *v.* Lunatic Asylum (L. R., 4 Ch. App., 145) (1868). Numerous other cases will be found cited in Gould, Waters, section 219, and in Higg. Pol. Waterc., 132 et seq.

The argument was advanced by the defendant that the use of the defendant's property for mining purposes is what was termed, unfortunately, I think, by Lord Cairns, in Fletcher *v.* Rylands (L. R., 3 H. L., 330, at pp. 338, 339) (1868), a natural user, and similar in that respect to plowing a field, and that if it be unlawful for defendant here to cast into the stream the muddy waters from its mine it is also unlawful for the farmer to plow his land and allow the muddy water which runs from it after a heavy rain to reach the river. But the very statement of the two cases shows the absence of analogy between them. In the first place, the water from the plowed field comes thereon by natural causes beyond the farmer's control and runs by gravity to the stream, while in the case of the mine the water is, as here, found and raised by artificial means from a level far below that of the river and would never reach it but for the act of the miner, and in the second place, by the common law of the land every owner may cultivate his land without regard to its effects upon his neighbor, while such is not the law as to mining. The supreme court of Ohio, in Columbus Company *v.* Taylor (48 Ohio, 41, at p. 58), repudiates the notion that mining was a natural use of the land in the sense that farming is.

The ground of a reasonable natural user seems to be at the bottom of what was said in Merrifield *v.* Worcester (110 Mass., 216) upon this topic. So far as the expressions there used favor the notion that a city or town may collect and discharge sewage matter into a fresh-water stream to the injury of a riparian owner without liability to action they are contrary to the law as held in England for centuries. See Higg. Pol. Waterc., 127 et seq., where several cases besides these above cited are collected.

Equally untenable is another position advanced by the defendant, viz. that the river was always more or less polluted by contributions from other mines and from the washing of plowed fields, public roads, and railroad embankments. Such insistments have been frequently made and always overruled. The question in such cases seems to be whether the stream has already become so far polluted by contributors who have acquired a right so to do by adverse use or otherwise as that the pollution presently opposed will not sensibly alter its condition. And even in such a case the courts have held that the party has the right to deal with each contributor in detail and to buy off such contributors as have acquired a right, and is not obliged to submit to fresh contributors. I cite the following authorities: Ross *v.* Butler (4 C. E. Gr., 294, at p. 306); Attorney-General *v.* Steward (5 C. E. Gr., 415, at p. 419), where the learned chancellor says: "The defendants have no right to pollute or corrupt the waters of the creek, or if they are already partially polluted to render them more so;" to Cleveland *v.* The Gas Co. (5 C. E. Gr., 201, at p. 208); and to Meigs *v.* Lister (8 C. E. Gr., 199, at p. 205), where the learned chancellor says: "The position taken by counsel that the complainants were entitled to no relief from this nuisance because the locality was surrounded by other nuisances and dedicated to such purposes has no foundation in law or in fact. If there were severa. nuisances of the like nature surrounding them, they must seek relief from each separately. They can not be joined in one suit nor need the suits proceed pari passu."

In Crossley v. Lightowler (L. R., 2 Ch. App., 478, p. 481, 1867) Lord Chelmsford says: "But the defendants contend that the plaintiffs have no right to complain of any pollution of the Hebble occasioned by them, because there are many other manufacturers who pour polluting matter into the stream above the plaintiffs' works, so that they never could have the water in a fit state for use even if the defendants altogether ceased to foul it. The case of St. Helen's Smelting Co. v. Tipping (11 H. L. Ch., 642; 11 Jur. N. S.. 785), is, however, an answer to this defense. Where there are many existing nuisances, either to the air or to water, it may be very difficult to trace to its source the injury occasioned by any one of them; but if the defendants add to the former foul state of the water and yet are not to be responsible on account of its previous condition, this consequence would follow that if the plaintiffs were to make terms with the other polluters of the stream so as to have water free from impurities produced by their works, the defendants might say, 'We began to foul the stream at a time when, as against you, it was lawful for us to do so, inasmuch as it was unfit for your use, and you can not now, by getting rid of the existing pollutions from other sources, prevent our continuing to do what, at the time when we began, you had no right to object to.'" (Attorney-General v. Lunatic Asylum, 4 Ch. App., 145, p. 150, report of the expert, and p. 155.)

In Attorney-General v. Leeds (L. R., 5 Ch. App., 583, p. 595, 1870) the lord chancellor says: "I think the argument deduced from the foul state of the water before it gets to Leeds is not deserving of any weight for two reasons: First—and it is hardly disputed—the evil did become seriously aggravated when the new sewer was opened—that is to say, sixteen or seventeen years ago; and, secondly, the nuisance might terminate; and no one can say it was right that when one nuisance terminates there should be another brought into existence."

The sensible and material increase in the discoloration of the water, in this case resulting from the contribution of the defendant's mine, is clearly proved. The complainant was able to make white paper successfully and satisfactorily from February 1, 1892, for nearly a year, and until the serious discharge of discolored water from the defendant's shaft, in January, 1893; and they were also able to make such paper after the discolored water ceased to run, in June or July, 1893. During the intermediate period, while the discoloration of the water being discharged from the defendant's mine was the greatest, complainant could not make white paper satisfactorily.

In whatever point of view the complainant's case is considered it seems entirely clear and free from doubt. I can not think the least doubt is cast upon the law by the last decision in the Sanderson case, in Pennsylvania, and the facts of the case are substantially undisputed. The complainants' title and possession of the ripa, though put in issue by the answer, is established by the proofs and was finally admitted at the hearing. Their right to have the water come to them in its natural condition follows inevitably. (Holsman v. Boiling Spring Co., 1 McCart., 335, at p. 343, bottom, and cases there cited.) The learned chancellor there says: "Where the complainant seeks protection in the enjoyment of a natural water course upon his land, the right will ordinarily be regarded as clear. And the mere fact that the defendant denies the right by his answer or sets up title in himself by adverse user will not entitle him to an issue before the allowance of an injunction."

There can be no doubt that, upon the facts presented, it would be the duty of a judge to direct a verdict, and the rule adopted by the court of errors and appeals in Higgins *v.* The Water Co. (9 Stew. Eq., 538) applies. I refer to the language of the chief justice on page 544 et seq.

The jurisdiction of this court to adopt, on final hearing, the extreme remedy of an injunction in this class of cases, when the right is clear, is well established, not only by the case just cited, but by Acquackanonk Water Co. *v.* Watson, supra, which was decided by the court of errors and appeals, and by Holsman *v.* Boiling Spring Co., supra, decided by Chancellor Green, and by Shields *v.* Arndt (3 Gr. Ch., 234), and by Carlisle *v.* Cooper (6 C. E. Gr., 576).

It was suggested that in this case no injunction should be ordered, but that the complainants should be left to their action at law for damages. I am unable to adopt that view. It must now be considered as settled law in this State that the maintenance of a nuisance of the kind here in question is, in effect, a taking of property. Pennsylvania Railroad Co. *v.* Angel (14 Stew. Eq., 316, p. 329), where Judge Dixon, speaking for the court of errors and appeals, says: "This principle rests upon the express terms of the Constitution. In declaring that private property shall not be taken without recompense, that instrument secures to owners not only the possession of property, but also those rights which render possession valuable. Whether you flood the farmer's fields so that they can not be cultivated, or pollute the bleacher's stream so that his fabrics are stained, or fill one's dwelling with smells and noise so that it can not be occupied in comfort, you equally take away the owner's property. In neither instance has the owner any less of material things than he had before, but in each case the utility of his property has been impaired by a direct invasion of the bounds of his private dominion. This is the taking of his property in a constitutional sense. Of course, mere statutory authority will not avail for such an interference with private property. This doctrine has been frequently enforced in our courts," and he proceeds to cite previous authorities in the same court. If this be so, then the legislature has no power to authorize the maintenance of a nuisance for the promotion of private objects, even upon terms of making compensation; for no authority is necessary for the position that the legislature is powerless to enact a law declaring that defendant may have complainants' mill and water power upon terms of paying them what a court may ascertain it is worth. And I am unable to distinguish such action and that of leaving complainants to the remedy of repeated actions at law to recover damages as often as they are suffered. In this respect our system of laws varies from that of England, where Parliament is omnipotent and is not confined to the mere making of laws—the true function of a legislature—but may take private property for private purposes, with or without making compensation, the only restraint upon its power being its own innate sense of justice. Hence the English courts are authorized, in cases of certain nuisances, to give damages once for all instead of an injunction.

The result of my consideration of the subject is that there is no principle which will sustain a court of equity in refusing an injunction against the maintenance of an established continuing nuisance and leaving the injured party to his remedy at law. To do so is, in effect, to permit a party to take his neighbor's land for his own use upon terms of making such compensation as a jury shall assess. This is inadmissible.

The object and office of a verdict and judgment at law is to establish the right and give compensation for past injuries. The right being once made clear, whether by judgment at law or upon incontrovertible rules of law and well-established facts, the remedy in equity by injunction to prevent future injury is a matter of right, and the relief can not be refused.

The ground, however, mainly relied upon by defendant is that the proofs show that the nuisance has entirely abated and that there is no danger of its recurrence, and hence an injunction is unnecessary and improper.

At about the time the injunction was issued—July 17, 1893—defendant purchased a small tract of land skirting the railroad, between the shaft and the river, and established on it a settling basin, into which the mine water was turned and given opportunity for subsidence before reaching the river. The result was that it was substantially clear, and no further injury has been since felt at the paper mill. It is also in proof that from that time up to July, 1894, the water was usually clear when it came from the mine. At the sessions of December 27 and December 28, 1893, Professor Nason, a competent geologist and mining expert, testified that, in his opinion, no further clay and water-bearing seams or rents would be met in the course of defendant's mining operations, and that the rent which had given so much trouble had, by natural causes, become harmless. It was not suggested that all or any large proportion of the discolored clay deposit had been removed, but the theory was that the descending water had worn channels in the clay, resulting in little rivulets centering at the section by the shaft, and that the scouring power of the water—that is, its power to bring down clay—had ceased by reason of the clay banks and beds of the little rivulets having arrived at an "angle of repose." The stability of this state of affairs depends, of course, upon the uniformity of the flow of water, both as to quantity and source of inflow, and Professor Nason, on cross-examination, admitted some uncertainty in this respect. After his examination and the close of the evidence on both sides, and before the argument, viz, about July 16, 1894, an unexpected influx of muddy water occurred, due to an overflow from a flume carrying water from the neighboring mine of the Lehigh Zinc and Iron Company, which found its way into the seam or rent at a point where it came to the surface, about 1,800 feet from the Parker (defendant's) shaft. This opening was a surface fissure or swallet in the rock—quite common where limestone rocks come to the surface. In this case, as I understand Professor Nason, he did not suppose or infer, from the trend of the fissure, that it reached the surface in that neighborhood, but such was the fact. It was promptly stopped by defendant and filled up, so as to prevent any more water getting in at that point.

Now, it seems to me that this occurrence shows the impossibility of affirming that there will be no further incursions of muddy water. It is true that with the continued use of the settling ground no injury will probably result to complainants from such an irruption. I say "probably," because, in case of a sudden irruption of discolored water, the quantity might be so great as to overwork the present settling basin. But without a decree and injunction the defendant will be at liberty to discontinue its use and permit any muddy water that may appear to flow into the Furnace Pond as of old.

At the time the complainants filed their bill the injury was serious and continuous. The defendant positively declined to stop it, but claimed the right to continue it. To complainants' bill was interposed a general denial, and setting up a right to persist in the injury as long as its necessities required. On all these issues the defendant is beaten. The complainants have established their case, and it would seem to be a most lame and impotent conclusion to refuse to give them the very relief prayed for, viz, a perpetual injunction. I am unable to imagine any other decree in their favor which would adequately meet the case and give them the just fruits of their suit; and, surely, if there is no danger of further discoloration the injunction will do the defendant no harm, but will be of value as a muniment of title to the complainants' property. The language of Lord Justice Turner, in Goldsmid *v.* Tunbridge Wells Commissioners (L. R., 1 Ch., App., 349, p. 355), applies: "In this particular case I think that regard must be had not merely to the comfort or convenience of the occupier of the estate, which may only be interfered with temporarily and in a partial degree, but that regard must also be had to the effect of the nuisance upon the value of the estate and upon the prospect of dealing with it to advantage; and I can not but think that the value of this estate, and the prospect of advantageously dealing with it, is and will be affected by the continuance of this nuisance."

But the defendant further urges that the complainants have manifested a disposition to make an unreasonably harsh and oppressive use of their rights in the premises, and have thereby weakened their standing in equity and disentitled them to the extreme decree asked for.

In the month of March, 1893, while the outflow from the mine was at its worst, negotiations took place between the parties for some sort of settlement, and a filter was mentioned. The complainants offered to be satisfied if defendant would furnish them with a filter of proper size, which they said, and about which there is no dispute, would cost $5,000. The defendant offered to pay one-half of the expense of the filter, the same to be in full compensation for all damages up to the time it was furnished, which offer the complainants refused to accept. I can see nothing harsh or oppressive in that refusal.

Next, and after bill filed, as I now recollect, defendant made an arrangement with the tenant of a gristmill, located upon a little stream which empties into the Furnace Pond, for a right to divert water from the mill and carry it by a flume several hundred feet down to the complainants' works and furnish them with clear water from that stream. Complainants employed an expert to examine the stream and see whether it would supply sufficient water for their paper engines, with the result that they were informed and believed that it was not sufficient, and declined to accept it as a substitute for the river water. The defendant, nevertheless, in the face of complainants' refusal, built the flume—a mere wooden trough, set upon benches and trestles—along the surface of the ground down to the mill yard of the complainants. The complainants refused to allow it to be put across their mill yard, because it would prevent them from having access to their works and from free passage with carts and wagons from one part to the other, and said that anything of that kind must be put underground in iron pipes. But the radical difficulty with that movement on the part of the defendant was that the right to the use of the water was merely obtained temporarily from a mere tenant of the mill property, and did not give the complainants any permanent right to the flow of the stream, even if it had been large enough for their purposes. I can see nothing harsh or oppressive in complainants' action in refusing this offer of substitution. They not only had the strict right in law to refuse to accept them, but their conduct in so doing, in my judgment, was not inequitable.

I shall advise a decree establishing the complainants' right to the flow of the stream in its natural condition and an injunction with costs.

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Where an injurious act in one State is so far-reaching in its injurious consequences as to threaten the rights of property and the health of a large number of citizens in another State, the latter State may become a party complainant in the Supreme Court of the United States to enforce the legal remedies of its citizens for such injuries. (Missouri *v.* Illinois et al. (U. S. Supreme Court, October term, 1900), 180 U. S., 208.)

This was a case in which the State of Missouri sued to restrain the State of Illinois and the Sanitary District of Chicago from carrying the sewage of Chicago through an artificial channel to the Mississippi River. The right of the State of Missouri to protect its citizens by this action and to implead the State of Illinois as a party defendant and to have an injunction against the defendants in case the facts alleged in its bill should be established was upheld by a divided court in overruling a demurrer to the bill. The defendants have answered, but at the time of the present writing the final hearing has not been reached.

RIGHTS OF RIPARIAN OWNERS IN ARID AND MINING STATES.

In certain of the arid and mining States of the West the doctrine of riparian rights has been in whole or in part abrogated by what is known as the doctrine of prior appropriation. Where the latter doctrine prevails the rights of riparian owners as given above do not exist, and where the doctrine of prior appropriation has been adopted in part the rights of appropriators to some extent supersede the rights of riparian owners.

"Appropriation" is an actual use of the water for a beneficial purpose by a person having the right to make such use, i. e., by any person having lawful access to the water. The appropriator, by the fact of appropriation, acquires the right, as against riparian owners, to use the water in the state and condition and to the extent necessary for the purpose for which he has appropriated it. Subsequent appropriators also acquire rights, but such are subordinate to the rights of the prior appropriator.

The doctrine of prior appropriation has been adopted to the extent indicated in the States mentioned below:

Arizona:

Clough *v.* Wing, 17 Pac. Rep., 453.

Colorado:

"The right to divert unappropriated waters of any natural stream shall never be denied." Const., Art. XVI, sec. 6.

Wheeler *v.* Northern Colorado I. Co., 10 Col., 582.

3 Am. St. Rep., 603; 17 Pac. Rep., 487.

Idaho:

Constitution of 1889, Art. XV, sec. 3.

Wilterding *v.* Green, 45 Pac. Rep., 134; 4 Idaho, 773.

Montana:

Constitution, Art. III, sec. 15.

Smith *v.* Denniff, 24 Mont., 20.

81 Am. St. Rep., 408; 60 Pac. Rep., 398; 50 L. R. A., 741.

Nevada:

Reno Smelting, etc., Works *v.* Stevenson, 20 Nev., 269.

New Mexico:

Compiled Laws of New Mexico, sec. 23.

Albuquerque Land and Irr. Co. *v.* Gutierrez, 61 Pac. Rep., 357.

North Dakota:

Springville *v.* Fullmer, 7 Utah, 450.

Stowell *v.* Johnson, 7 Utah, 215.

Wyoming:

Farm Investment Co. *v.* Carpenter, 9 Wyo., 110; 50 L. R. A., 747; 61 Pac. Rep., 288.

In California the common law as to riparian rights seems to prevail, except as to rights acquired by appropriation upon public lands made before any riparian owner has acquired title to lands below. (Lux *v.* Haggin, 69 Cal., 254.)

In Oregon the right of appropriation is confined to such rights as were acquired before Washington became a State, under an act of Congress passed in 1866. (Simmons *v.* Winters, 21 Oreg., 35; 25 Am. St. Rep., 727; 27 Pac. Rep., 7.)

In Washington the right of appropriation seems to be recognized, at least as to the portion east of the Cascade Mountains in that State, but not as against settlers who have obtained riparian rights before the appropriation.

Isaacs *v.* Barber, 30 L. R. A., 665.

10 Wash., 124; 45 Am. St. Rep., 772.

38 Pac. Rep., 871.

Benton *v.* Johncox, 39 L. R. A., 107; 17 Wash., 277.

61 Am. St. Rep., 912; 49 Pac., 495.

In several of the arid or partly arid States not included in the above list the riparian owner holds subject to the right of those owning above him to a *reasonable* use of the water for irrigation purposes.

Rhodes *v.* Whitehead, 27 Tex., 309; 84 Am. Dec., 631.

Tolle *v.* Carreth, 31 Tex., 362; 98 Am. Dec., 540.

Fleming *v.* Davis, 37 Tex., 173.

Baker *v.* Brown, 55 Tex., 377.

Mud Creek Irr., etc., Co. *v.* Vivian, 74 Tex., 170; 11 S. W. Rep., 1078.

Barrett *v.* Metcalf, 12 Tex. Civ. App., 247; 33 S. W. Rep., 758.

So far as the doctrine of prior appropriation is recognized, the rights of riparian owners are pro tanto extinguished. In such States, therefore, the general statements already given require modification.

In States where the doctrine of prior appropriation is established it may be safely asserted:

1. That the riparian owners can not complain of pollution so far as such pollution necessarily results from the use for which the appropriator has appropriated the water.

2. That no person, except a prior appropriator, may pollute the stream so as to render the water less fit for use by one who has law-

fully appropriated it, and such prior appropriator can not so pollute the water by a subsequent appropriation to a new use.

Fairplay Hydraulic Mining Co. *v.* Weston, 29 Colo., 125.

3. No appropriator or other person may pollute waters to the extent of creating a public nuisance.

Woodruff *v.* North Bloomfield Gravel Mining Co., 8 Lawy., 628; 16 Fed. Rep., 25; 9 Lawy., 441; 18 Fed. Rep., 753.
People *v.* Gold Run Ditch, etc., Co., 66 Cal., 138; 56 Am. Rep., 80; 4 Pac. Rep., 1152.
Carson *v.* Hayes, 39 Oreg., 97; 65 Pac. Rep., 814.
Suffolk Gold Min. and Milling Co. *v.* San Miguel Consd. Mining and Milling Co., 9 Colo. App., 407; 48 Pac. Rep., 828.
Nixon *v.* Bear River and A. Water Co., 24 Cal., 367; 85 Am. Dec., 69.
Levaroni *v.* Miller, 34 Cal., 231; 91 Am. Dec., 692.
Yuba Lake, etc., Co. *v.* Yuba Co., Super. Ct., 66 Cal., 311; 5 Pac. Rep., 490.
McLaughlin *v.* Del Re, 71 Cal., 230; 16 Pac. Rep., 881.

B. RIGHTS OF THE PUBLIC (AS DISTINGUISHED FROM INDIVIDUAL OWNERS) TO HAVE INLAND WATERS KEPT FREE FROM POLLUTION BY RIPARIAN OWNERS OR OTHERS.

Whenever the pollution of a stream or other body of water injuriously affects the health or materially interferes with the peace and comfort of a large and indefinite number of people in the neighborhood, such pollution becomes what is known as a public nuisance. But, except under such circumstances, the public, as such, has no standing to prevent the pollution of waters. When, however, there is a public or quasi-public ownership of the banks of a stream, as in the case of a source of water supply owned by a municipality or owned by a company which supplies the inhabitants of a municipality with water, the public is interested in the enforcement of the rights of riparian proprietors, as stated under heading "A."

Where there is a public nuisance caused by the pollution of water, it is the duty of public authorities to cause its abatement, and their right to do so has been sustained in numerous cases. Where the public is injured in its capacity of riparian owner the remedy is either by injunction or by criminal proceedings, according to the nature of the wrong and the laws and practice of the jurisdiction in which the offense occurs.

The following are cases in which the pollution of water has been held to be a public nuisance:

Board of Health *v.* Casey, 3 N. Y. S., 399.
People *v.* Elk River Mill and Lumber Company, 107 Cal., 214.
State *v.* Taylor, 29 Ind., 517.
Greene *v.* Nunnemacher, 36 Wis., 50.

C. CONDITIONS UNDER WHICH, AND EXTENT TO WHICH, PUBLIC MUNICIPALITIES MAY USE INLAND WATERS IN DISPOSING OF SEWAGE FROM PUBLIC SEWERS.

This subject has but recently been receiving attention from the courts. It seems to have been the custom of municipalities to discharge their sewers freely into the larger streams, and until within the last few years but little, if any, objection to the practice has found its way into the courts. Latterly the increase of population, with the consequent increase of the amount of sewage matter so discharged, has brought about a condition of affairs that has produced opposition and in many cases litigation. The principles established by the decisions thus made necessary are briefly summarized as follows:

Municipalities, if riparian owners, have the same rights and are subject to the same restrictions in the use and treatment of the water flowing over their lands as private owners are—i. e., they may deposit sewage and other filth in such waters, provided always that by so doing they cause no injury to property below them. They may drain the surface water from their streets into water courses, with the impurities which it naturally carries, provided they do not thereby increase the flow of water into the stream so as to exceed the capacity of the channel to the injury of property below.

Brainerd v. Newton, 154 Mass., 255; 27 N. E., 995.
Cone v. Hartford, 28 Conn., 363.

Where municipalities are expressly authorized by statute to construct a system of sewerage, and to cause the sewage matter to be discharged into any particular waters, the statutory authority is to be exercised subject to the implied condition that such discharge will not constitute a nuisance. Legislative authority can go no further than to authorize municipalities to acquire the rights of lower owners by purchase or condemnation, because of the constitutional restriction against taking private property for public use without just compensation.

It will thus be seen that the increase of population under the present conditions and with the now prevalent methods of sewage disposal in cities is rapidly leading to a condition of affairs which will call for radical changes. Many cities will find themselves unable to dispose of their sewage matter by means of rivers without enormous expense, and probably not without additional legislation. As will be seen hereafter, the subject is already receiving serious attention from legislators.

CITATION OF CASES.

The following cases will be found to sustain the general principles above stated:

English:

Goldsmid *v.* Tunbridge Wells Imp. Com., L. R., 1 Chan. App., 349.
Holt *v.* Rochdale, L. R., 10 Eq. Cases, 354.
Attorney-General *v.* Leeds, L. R., 5 Chan. App., 583.
Attorney-General *v.* Richmond, L. R., 2 Eq. Cases, 306.
Attorney-General *v.* Hackney Local Board, L. R., 20 Eq. Cases, 626.
Attorney-General *v.* Cockermouth Local Board, L. R., 18 Eq. Cases, 172.
Attorney-General *v.* Luton Local Board, 2 Jurist, 180.
Attorney-General *v.* Halifax, 39 L. J. (N. S.), 129.
North Staffordshire R. R. Co. *v.* Tunstall Local Board, 39 L. J., Chan., 131.
Attorney-General *v.* Kingston on Thames, 34 L. J., 481.
Attorney-General *v.* Basingstoke, 45 L. J. (N. S.), 726.
Attorney-General *v.* Colney Hatch Lunatic Asylum, L. R., 4th Ch. Div., 146.
Attorney-General *v.* Birmingham, 4 Kay & Johns., 528.
Attorney-General *v.* Metropolitan Board of Works, 1 H. & M., 298.
Bidder *v.* Croyden Local Board, 6 L. T., 778.
Manchester, etc., Railway Co. *v.* Worksop Board of Health, 23 Beav., 198.
Oldaker *v.* Hunt, 6 De Gex, McN. & G., 376.

Alabama:

Birmingham *v.* Land, 374 So. Rep., 613.

California:

People *v.* City of San Luis Obispo, 116 Cal., 617.
Peterson *v.* City of Santa Rosa, 51 Pac. (Cal.), 557.

Connecticut:

Morgan *v.* Danbury, 67 Conn., 484.
Nolan *v.* New Britain, 69 Conn., 668.
(See extracts from opinions in the Conn. cases given below.)

Georgia:

Columbia Av. Savings Fund, etc., Co. *v.* Prison Commission of Georgia, 92. Fed. Rep., 801 (Cir. Ct. West Div. Ga., 1899).

Illinois:

Village of Dwight *v.* Hayes, 150 Ill., 273.
Robb *v.* Village of La Grange (1895), 158 Ill., 21.
Barrett *v.* Cemetery Assn., 159 Ill., 385.

Indiana:

Valparaiso *v.* Hagen, 153 Ind., 337; 48 L. R. A., 707; 74 Am. St. Rep., 305; 54 N. E., 1062.[a]

Iowa:

Randolf *v.* Town of Bloomfield, 77 Ia., 50.
Loughran *v.* City of Des Moines, 72 Ia., 382; S. C. 34 N. W. Rep., 172.

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[a] In this case it was held that where a municipality acts in conformity to the statute, skillfully and without negligence, it may discharge its sewage into a stream and the lower proprietors may not have an injunction, and are entitled to no compensation for the damages suffered by them.

This seems to settle the law in that State; but the reasoning is not convincing, and it is believed no other State has, so far, adopted that rule, which might, perhaps, be held violative of that clause of the Constitution of the United States which forbids the taking of private property for public use without compensation.

Kansas:

Topeka Water Supply Co. *v.* City of Potwin, 43 Kan., 404.

Massachusetts:

Brainard *v.* Newton (Mass. Sup.), 27 N. E. Rep., 995, and 154 Mass., 255.
Morse *v.* Worcester, 139 Mass., 389.
Boston Rolling Mills *v.* Cambridge, 117 Mass., 396.
Haskell *v.* New Bedford, 108 Mass., 208.
Woodward *v.* Worcester, 121 Mass., 245.
Middlesex Co. *v.* Lowell, 149 Mass., 509.
Merrifield *v.* Worcester, 110 Mass., 216.[a]

Missouri:

The Joplin Consolidated Mining Co. *v.* City of Joplin, 124 Mo., 129.

New Hampshire:

Vale Mills *v.* Nashua, 63 N. H., 136.

New Jersey:

Doremus *v.* Paterson, 65 N. J. Eq., 711.
State *v.* Freeholders of Bergen, 1 Dick., 173
Atty. Gen. *v.* City of Paterson, 45 Atl. (N. J., 1900), 995; 60 N. J. Eq., 385

New York:

Butler *v.* Village of Edgewater, 6 N. Y. S., 174.
Chapman *v.* City of Rochester, 110 N. Y., 273.

Pennsylvania:

Good *v.* Altoona City, 162 Pa. St., 493.

EXCERPTS FROM IMPORTANT DECISIONS.

In Owens *v.* Lancaster City (182 Pa. St., 257, and 193 Pa. St., 436) the right of a city to use a stream passing through it as an open sewer, subject only to liability for any injury done to adjoining property through its negligence, seems to be conceded.

As to the limits of this right, and the consequences for which the municipality would be liable in the State of Pennsylvania, see the following cases:

The city was held liable for injury done to plaintiff's wharf by deposits from a sewer, in Butcher's Ice and Coal Company *v.* Philadelphia. (156 Pa. St., 54.)

It was held liable to a lot owner for maintaining a sewer mouth upon his lot, in Harris *v.* City. (155 Pa. St., 76.)

It was held liable for destroying the value of wells, caused by the flowing of polluted river water into them by underground passages, in Good *v.* Altoona. (162 Pa. St., 493.)

It was held liable for damages caused by accumulations of filth, ashes, or other material, that obstruct the flow of the water and throw

[a] In Merrifield *v.* Worcester damages were refused to a riparian owner who sued in tort for the pollution of his stream. The decision turned upon the nonliability of municipal corporations for the consequences of the judicial acts of their governing bodies. It holds that the plaintiff might recover for injury caused by pollution due to the improper construction or unreasonable use of the sewers, or to the negligence or other fault of the defendant in the care and management of them. It is no authority for the principle established in Indiana in Valparaiso *v.* Hagen.

it out upon the lands of adjoining owners, in Blizzard *v.* The Borough of Danville. (175 Pa., 479.)

In Owens *v.* Lancaster City (182 Pa. St., 257), at page 262, Mr. Justice Green remarks, obiter: "We apprehend the same principle would apply to the injury inflicted by allowing offensive and injurious odors and smells to issue from the polluting substances discharged into the stream from the city sewers."

Nolan *v.* New Britain (69 Conn., 668) was an action for damages caused by the defendant's discharge of its public sewers into a stream called Pipers Brook, which ran through plaintiff's land.

The city had, in 1872, under alleged legislative authority, condemned and taken, and condemned the right to take, occupy, and appropriate Pipers Brook for sewer purposes, but plaintiff did not appear in the proceedings, nor was any award made to him.

Significant excerpts from the supreme court's opinion, by Andrews, C. J., are here given:

> The use of Pipers Brook which the complainant charges that the defendant has made, unless there is a lawful warrant therefor, causes a public nuisance. * * * That it would be a public nuisance to render the water of a stream so impure that it could not be used for domestic purposes or for watering cattle, and so that it gave off noxious and unhealthy odors is hardly open to question (Chapman *v.* Rochester, 110 N. Y., 273), for the reason that these causes would injuriously affect every riparian owner along the whole length of the stream and every person who lived near it. If a municipal corporation, in the absence of a legal right to do so, causes sewage to pollute a water course, to the use of which a lower owner through whose premises the water course flows is entitled, it is guilty of a nuisance for which damages may be recovered. [Many authorities cited.]

On page 681, after an examination of the alleged statutory authority, the opinion continues:

> If it had been the intent of the legislature by the act of 1872 to authorize the common council of the city of New Britain to take or to affect any lands outside of the city limits, it is certain there would have been in the act some provision for the ascertainment of damages to be paid to the landowner. The right of the plaintiff to have the water of Pipers Brook flow through his land as it had been accustomed to flow (i. e., pure and uncontaminated) is not an easement, but is inseparably annexed to the soil. (Wadsworth *v.* Tillotson, 15 Conn., 366, 373.) To deprive the plaintiff of that part of his soil for the purposes named in that act would be the taking of private property for public use, and the plaintiff would be entitled to have just compensation.

As the complainant lived outside the city limits, it was held that he was in no way affected by the assessment proceedings.

The other defenses amounted to a claim of right to such use of the stream by prescription. As to this defense the court says, at page 683:

> The sixth defense presents the question of prescription. We have already indicated our opinion that the use of Pipers Brook of which the plaintiff com-

plains is a public nuisance. We suppose the law to be so that a public nuisance can not be prescribed for. No length of time can legitimate, or enable a party to prescribe for, a public nuisance. (People v. Cunningham, 1 Denio., 524; Mills v. Hall, 9 Wend., 315; Veazie v. Dwinel, 50 Me., 479, 490; Commonwealth v. Upton, 6 Gray, 471, 476; Wood on Nuisances, 722; 19 Am. and Eng. Encyc. of Law, 30.) When an action is brought by a party who has suffered a special injury in consequence of a public nuisance, a prescriptive right to do the acts complained of can not be maintained against him. (Bowen v. Wendt, 103 Cal., 236; People v. Gold Run, etc., Mining Co., 66 Cal., 138; Boston Rolling Mills v. Cambridge, 117 Mass., 396; O'Brien v. St. Paul, 18 Minn., 176; Cooley on Torts, 614.) There is no occasion to discuss this defense further, because the defendant's counsel in their brief expressly disclaim that any right can be obtained by prescription to commit such a nuisance.

In Morgan *v.* City of Danbury (67 Conn., 484) the question of restraining a city from polluting the water of a stream by sewage, at the suit of a mill owner below the city, was thoroughly discussed, and the injunction sustained. The opinion is written by Baldwin, J., and the important portions of it are as follows (p. 493):

The nuisance thus complained of consisted, then, of discharging into a river, above the plaintiff's premises, certain substances of a kind and in such a manner that the water came to him polluted, and a deposit was made upon his land and in his mill pond whereby noxious odors were created, dangerous to his health and that of others, his dam partly filled up by filth, and the use and value of his property largely taken away—injuries which the defendant intended to increase by enlarging its sewer system, and adding to the amount of the deposits made from the sewers in the river, the result of which would be to fill up his mill pond with filth and sewage, and make his property valueless.

These allegations were denied, but they have been found true, and there is nothing inconsistent with their truth in the special finding of facts. They stated that the deposits from the sewers both filled up the plaintiff's mill pond, and polluted the air he breathed and the waters that flowed over his property. These, though proceeding from the same act, produced separate injuries. A nuisance was created with a double aspect. That to the waters of the stream and the air above it it was found constituted a public nuisance, though it was one which also wrought a special and peculiar injury to the plaintiff. That from filling up the mill pond constituted simply a private nuisance. (Haskell v. New Bedford, 108 Mass., 208, 216; Brayton v. Fall River, 113 Mass., 218, 229.) It was proper that the injunction should be so framed as to protect the plaintiff against every serious and irreparable injury which he might suffer by the continuance of the nuisance, and its terms are fully conformable to the claims stated in his complaint.

The defendant contends that the decree is too broad, in that it restrains the discharge into the river of any sewage, even if not of a noxious or polluting character, or though entirely and permanently disinfected and purified.

The primary meaning of "sewage" is that which passes through a sewer (Century Dictionary; Webster's International Dictionary). A secondary meaning is derived from the usual character of the contents of a sewer, and as used in that sense the word signifies the refuse and foul matter, solid or liquid, which it so carries off.

In the plaintiff's complaint the connection in which the term is employed is such as to indicate that it was intended to carry the secondary meaning.

And further, at page 496:

The defendant urges that it should not be made responsible for the acts of others, and that if its sewage is thoroughly disinfected, sterilized, and purified before its discharge into the river nothing further should be required, even though as it flows down the stream it may be brought into contact with other substances in such a way as to work a nuisance. But the right to deposit a thing in any place must always be dependent not only on its own nature but on the nature of the place in question and the uses to which that has already been put. A lighted match may be safely thrown into a brook under ordinary circumstances, but not should it happen to be covered with oil from a leaky tank.

If different parties by several acts foul the same stream, each may be enjoined against the commission of the wrong with which he is individually chargeable.

And see, also, Watson *v.* Town of New Milford (72 Conn., 561); Platt Bros. & Co. *v.* Waterbury (72 Conn., 531); and note on "Rights of municipal corporations to drain sewage into waters," appended to a report of the last-named case in 48 Lawyers' Rep. Annotated, page 691.

In Mayor, etc., of Birmingham, *v.* Land (34 So. Rep., 613), decided by the supreme court of Alabama in June, 1903, the Connecticut cases above cited were followed. Among other things, the court, per McClellan, C. J., say:

The fact that the city of Birmingham had statutory authorization to construct a sewer emptying into Valley Creek, upon the condemnation of lands taken or injured in its construction and use, is not of importance, since the lands here injured have not been condemned. The nuisance is none the less a nuisance because of the statutory power referred to, the right to exercise the power in respect of this land not having been acquired. City of Mansfield *v.* Balliett (65 Ohio St., 451; 58 L. R. A., 628, and note).

See, to the same effect, Sammons *v.* City of Gloversville (67 N. E. Rep., 622), decided by the court of appeals of New York, June 9, 1903. In this case an injunction was granted, its operation being suspended to enable the defendant to obtain legislative relief, or to abate the nuisance.

In Middlesex Company *v.* Lowell (149 Mass., 509), decided in 1889, it was held that an injunction should be granted to restrain defendant from discharging sewage into plaintiff's mill pond, and that no right to do so could be acquired by prescription.

This places Massachusetts in line with the other States, notwithstanding the decision in Merrifield *v.* Worcester that a city is not liable for damages caused by lawfully laying out and constructing and reasonably using a system of sewers in accordance with plans adopted by the proper corporate body, upon the principle that such body acts quasi judicially in so adopting plans.

In Butler *v.* Village of White Plains (69 N. Y. Supp., 193; N. Y. Sup. Court App. Div., 2d Dept., March, 1901), an injunction was granted against a nuisance caused by the deposit of the effluent of defendant's sewage in the Bronx River. The fact that others were polluting the stream was no defense.

Grey, Attorney-General, *v.* Paterson (13 Dick., 1; on appeal, 15 Dick., 385), was an action brought by riparian owners below Paterson for an injunction restraining the city of Paterson from depositing or discharging its sewage through its drains or sewers into the Passaic River, and from constructing new sewers to discharge into said river, and from enlarging or increasing its present sewerage system with outlets into said river.

By an act passed in 1867 (P. L. of 1867, p. 653, sec. 17) Paterson had been authorized by the legislature as follows:

That the mayor and aldermen of the city of Paterson are hereby authorized to cause such surveys, maps, and returns to be made as may be necessary to enable them to prescribe and adopt, either for the whole or any part of said city, the location of streets and sewers, or either, and the width thereof, hereafter to be opened or constructed therein, and when such location, width, and grade shall be adopted, the surveys, maps, and returns prescribing and defining the same shall be recorded in the clerk's office of the county of Passaic, and thereupon no street or sewer shall thereafter within the district comprised in any such survey, map, or return be opened or constructed, except in conformity therewith as to location, width, and grade, and fully to accomplish the purposes contemplated by this section the said mayor and aldermen may employ such engineers, surveyors, and other persons, and provide for their compensation and pass such ordinances as they may deem to be proper, and may enter upon any land for making surveys and examinations.

On the 26th of February, 1868, Paterson was further authorized to construct sewers and drains (P. L., 1868, p. 126). The second section provides:

That all such sewers and drains shall be constructed in conformity with the plans thereof adopted or which shall be adopted by said mayor and aldermen pursuant to the seventeenth section of the act approved April 4, 1867, entitled "A further supplement to the act entitled 'An act amending and revising the act to incorporate the city of Paterson.'"

It was found by the court that, so far as the authority of the State can avail for that purpose, the legislative consent, in this case, furnishes ample protection to the city for the appropriate exercise of the power granted.

It was further said that riparian owners below the point where the tide ebbs and flows were not entitled to an injunction, because the title to their lands did not extend below high-water mark.

The title of owners above the ebb and flow of the tide extends to the middle of the stream, subject only to the rights of the public for purposes of navigation; and it is held that, notwithstanding the legislative grant of authority, such owners can not be deprived of their right of property in the river without just compensation. Following the case of Beach *v.* Sterling Iron and Zinc Company (9 Dick., 65), as affirmed in 10 Dick., 824, it was decided that the owners above tide water were entitled to compensation, but in view of the great detriment to the city if an injunction should be granted and the comparatively small injury done to the owners the injunction was refused, except in the alternative that the city should refuse to make such compensation for the diminished value of their lands as shall be ascertained to be just.

In this case there is no recognition of the damage done to the lands adjoining or near the stream. The complainant's right to redress arises wholly from the injury done to the water, in which they have a proprietary right.

In Winchell *v.* Waukesha (110 Wis., 101), Dodge, J., gave the opinion, which in part is as follows:

The findings and evidence disclose a very obvious nuisance, which, if created and maintained by an individual, would entitle the plaintiff to the aid of a court of equity to effect its abatement, and to damages if pecuniary injury be established, with the decisions of this court. * * * It has been declared by this court in Harper *v.* Milwaukee (30 Wis., 365, 372), that "the general rule of law is that a municipal corporation has no more right to erect and maintain a nuisance than a private individual possesses, and an action may be maintained against such corporation for injuries occasioned by a nuisance for which it is responsible in any case in which, under like circumstances, an action could be maintained by an individual." Again, in Hughes *v.* Fond du Lac (73 Wis., 380, 383) it is said: "A municipal corporation is no more exempt from liability in case it creates a nuisance, either public or private, than an individual." These statements are very broad, and, appellant insists, must yield to various exceptions and limitations (pp. 105 and 106).

When, if ever, the legislature shall enact that streams generally or any stream shall be used as sewers without liability to the owners of the soil through which they run, the question of constitutional protection to private rights may be forced upon the courts for decision. Until such enactment is made, however, in clear and unambiguous terms, we shall be slow to hold by inference or implication that it has been made at all. The right of the riparian owner to the natural flow of waters, substantially unimpaired in volume and purity, is one of great value, which the law nowhere has more persistently recognized than in Wisconsin. Not alone the strictly private right, but important public interests, would be seriously jeopardized by promiscuous pollution of our streams and lakes. Considerations of aesthetic attractiveness, industrial utility, and public health and comfort are involved. Amid this conflict of important rights, we can not believe that the legislature concealed, in words merely authorizing municipalities to raise and expend money for the construction of sewers, a declaration of policy that each municipality might, in its discretion, without liability to individuals, take practical possession of the nearest stream as a vehicle for the transportation of its sewage in a crude and deleterious condition. At that stage in its logic we can not agree with the Indiana court in Valparaiso *v.* Hagen (153 Ind., 337).

STATUTORY RESTRICTIONS OF WATER POLLUTION.

CLASSIFICATION.

Speaking generally, jurisdiction over the pollution of waters in the United States is confined to the several States. There is no pro sion in the Constitution which gives to Congress authority in the p emises, partly, no doubt, because at the time of its adoption the great importance of the subject from an interstate point of view was not thought of. Hence, by the familiar principle that the several States retain full sovereign powers except so far as such powers are restricted by the National Constitution or expressly delegated thereby to the National Government, the States have full control of this subject. In reviewing these laws, accordingly, we must examine the statutes of all the States and Territories.

Uniformity of legislation is not to be expected. The natural conditions existing in different portions of the vast territory are so various, the density of population differs so widely in the different sections involved, and public enlightenment as to the deleterious effects of water pollution and the necessity to restrain it is, in sparsely settled districts, so far behind that which has been developed in congested areas by the terrible consequences, that statutory regulations must necessarily differ. In some States there is found nothing more than a simple provision making it a crime to poison wells and springs, while others have made elaborate provisions designed to check and, so far as possible, absolutely to prevent all pollution of waters by mingling with them the refuse products of animal life or the wastes of human industry. If, therefore, we are to avoid making this review a mere catalogue of statutes, it will be necessary to adopt some system of classification and grouping. Doubtless a mere citation of the statutes of all the States, taken in their alphabetical order, would serve a useful purpose in enabling the reader to turn to the particular section in which his interest lies and to find the legislation whic affects this section. But if, by a logical grouping of States accordi to their progress in this particular, we can give a clearer idea o status of such legislation as a whole, without seriously interfe. with the usefulness of the book as a compendium of State ws u this subject, much will be gained.

Accordingly, I have arranged the States and Territories in th groups or classes, placing those in each group in alphabetical for convenience of reference.

CLASS I: STATES WITH PARTIAL RESTRICTIONS.

This group comprises those States and Territories in which the legislature has confined itself to forbidding the poisoning or pollution of drinking water in certain ways or in certain localities. They belo. ~ in the same category because they are all at the same stage of growth .. sanitary education—i. e., there is manifest in their legislation no sense o. the general desirability of pure natural waters, but only a desire to prevent certain acts recognized as criminal in intent or as likely to injure special groups of persons (public or private corporations) whom the legislature desires to protect.

An alphabetical list of the States and Territories in Class I, with the statutes in force in each at the close of 1905, either given in full or abstracted so as to show their nature and force, is here presented.

ALABAMA.

[Acts of Alabama, 1896–97, p. 1281.]

AN ACT to punish any person who pollutes or contaminates water supplied to cities and towns of the State.

SECTION 1. *Be it enacted by the general assembly of Alabama*, That it shall be unlawful for any person to knowingly deposit any dead animal or nauseous substance in any source, standpipe, or reservoir from which water is supplied to any city or town of said State. Any person violating the provisions of this act shall be guilty of a misdemeanor, and upon conviction shall be punished by a fine not exceeding $500 and may be sentenced to hard labor for the county not exceeding one year.

Approved, February 17, 1897.

[General Acts, Alabama, 1903, Act No. 542, p. 499.]

AN ACT to amend, reconstruct, and provide for the enforcement of the laws relating to the public health.

SEC. 15 (p. 508). Whenever complaint shall be made in writing to a ..h officer of a county, city, or town that there is in any pond, lake, ..ream owned or maintained by a private individual or corporation ... source of infection, or unsanitary condition, which is prejudicial to the public health, or likely to become so, or any material or thing that is grossly offensive or indecent, it shall be the duty of such ..h officer to thoroughly investigate such complaint. If upon ..stigation said health officer shall be of the opinion that said com...t is well founded, he shall at once notify the person responsible for that he must remove or abate, at his own expense, said source ... ion, unsanitary condition, or grossly offensive or indecent material or thing. Should such person responsible for said nuisance refuse or neglect to obey such order, said officer shall refer the matter to the county board of health for investigation, and either party to the contest may request the State health officer to be present and participate in the investigation. Should said county board of health agree with the opinion of said health officer, and should the person responsible for said nuisance or for said indecent material or thing still refuse or neglect to comply with the decision reached by said county board of health, the health officer to whom said complaint was first made shall proceed with as little delay as possible to cause said source of infection, unsanitary condition, or grossly offensive materail or thing to be removed or abated at the expense of the person responsible therefor.

ARKANSAS.

[Sandel and Hill's Digest, 1894.]

SEC. 1903. The throwing or dragging of dead animals, or animals in a dying condition, into any running stream or other body of water in this State is a misdemeanor.

Anyone violating the provisions of this chapter, on conviction thereof, shall be fined in any sum not less than ten nor more than fifty dollars. (Act March 27, 1891.)

[Laws of 1895, Act CXXVI, p. 183.]

AN ACT authorizing municipal corporations and other corporations to exercise certain privileges, and for other purposes.

SEC. 7. If any person shall * * * commit such a nuisance in or near the impounding dams or reservoirs of any water plant, or shall pollute the water or effect [affect] its wholesome qualities, he shall be deemed guilty of a misdemeanor and be fined for each and every offense in any sum not exceeding $200.

[Sandel and Hill's Digest, sec. 5134.]

They [municipal corporations] shall have the power to provide a supply of water by constructing or acquiring, by purchase or otherwise, wells, pumps, cisterns, reservoirs, or waterworks; to regulate the same; to prevent the unnecessary waste of water; to prevent the pollution of the water and injury to the waterworks; and for the purpose of establishing or supplying waterworks any municipal corporation may go beyond its territorial limits; and its jurisdiction to prevent or punish any pollution or injury to the stream or source of water, or to the waterworks, shall extend five miles beyond its corporate limits. [As amended by laws of 1903, act 88, p. 152.]

DELAWARE.

[Laws of 1893, p. 1024.]

AN ACT to amend chapter 242, volume 19, of the Laws of Delaware, entitled "An act to provide for the lighting of Middletown."

SEC. 10 (p. 1029). That if any person or persons shall designedly or maliciously injure the said light and water works, or obstruct the water to and from the same, or in any manner pollute the water supply * * * they shall forfeit and pay to the commissioner of the town of Middletown a fine not exceeding one hundred (100) dollars, to be recovered, etc.

FLORIDA.

[Revised Statutes of Florida, approved January 8, 1891.]

SEC. 2658. *Poisoning food or water.*—Whoever mingles any poison with food, drink, or medicine, with intent to kill or injure another person, or wilfully poisons any spring, well, or reservoir of water with such intent, shall be punished by imprisonment in the State prison for life or any term of years.

SEC. 2665. *Corrupting or interfering with water supply.*—Whoever wilfully or maliciously defiles, corrupts, or makes impure any spring or other source of water or reservoir, or destroys or injures any pipe, conductor of water, or other property pertaining to an aqueduct, or aids or abets in any such trespass, shall be punished by imprisonment not exceeding one year or by fine not exceeding one thousand dollars.

GEORGIA.

[Laws of 1896, p. 84.]

No. 57. AN ACT to prohibit the poisoning of any spring, well, or reservoir of water, to provide a penalty for the violation of the same, and for other purposes.

SEC. 1. *Be it enacted by the general assembly of the State of Georgia, and it is hereby enacted by authority of the same,* That from and after the passage of this act any person who wilfully and wantonly poisons or procures another to poison any spring, fountain, well, or reservoir of water shall be deemed guilty of a felony, and on conviction therefor shall be imprisoned in the penitentiary for a term of not less than two nor more than twenty years.

SEC. 2. Repeals inconsistent laws.

Approved, December 19, 1896.

IDAHO.

[Penal Code, passed 1901.]

SEC. 4916. Every person * * * who wilfully poisons any spring, well, or reservoir of water is punishable by imprisonment in the State prison for a term not less than one nor more than ten years.

IOWA.

[Code of Iowa, annotated, 1897.]

Sec. 4979. *Throwing dead animals in stream, spring, etc.*—If any person throw, or cause to be thrown, any dead animal into any river, well, spring, cistern, reservoir, stream, or pond, he shall be imprisoned in the county jail not less than ten nor more than thirty days or be fined not less than five nor more than one hundred dollars.

KANSAS.

[Laws of 1905, chap. 267, fish and game law.]

Sec. 6. It shall be unlawful for any person to empty or throw into or place in any lake, pond, river, creek, or stream, or other water within or bordering on this State, any acid, drug, lime, or other deleterious substance, or fishberries, or dynamite, giant powder, or other explosive matter of whatever kind, or any material or liquid which may kill, stun, poison, or craze fish; provided, that nothing in this section shall be construed to prevent the proper use of explosives for the exclusive purpose of improving navigation, or for blasting rock on [in] preparing foundations, or other improvements on or along the streams or waters of the State.

KENTUCKY.

[Compilation by John D. Carroll, 2d ed., 1899.]

Sec. 1278. If any person shall cast or place the carcass of any cattle or that of any other dead beast in any water course or within twenty-five yards thereof, or shall cast the same into any spring, or into any pond, such person, for every such offence, shall be fined for the first offense not less than five nor more than twenty dollars, and every subsequent offense not less than twenty nor more than one hundred dollars. (Under head of "Offences against public health.")

LOUISIANA.

[Revised Laws (Wolff).]

Sec. 924. Amending law of 1882, page 109.

Makes it an offense to "throw or cause to be thrown or conveyed into any navigable stream, bay, or lake within this State, bagasse from sugar mills, ballast from vessels, sinking timber of any kind, or any other matter of a nature to form an obstruction to its free navigation."

MICHIGAN.

[Compiled Laws of the State of Michigan (Lewis M. Miller).]

Sec. 11496. Willfully poisoning spring, well, or reservoir made a crime.

SEC. 2806. The council (of any village located upon or adjacent to any of the navigable waters of this State) shall have authority to "provide by ordinance for the preservation of the purity of the waters of any harbor, river, or other waters within the village," and other powers.

SEC. 3146. The council (of any city located upon or adjacent to any of the navigable waters of the State) "shall have authority to provide by ordinance for the preservation of the purity of the waters of any harbor, river, or other waters within the city, and within one-half of a mile from the corporate boundaries thereof; to prohibit and punish the casting or depositing therein of any filth, logs, floating matter, or any injurious thing," and other powers.

[Public Acts, 1899, No. 80, p. 115.]

AN ACT to prevent and punish the pollution and contamination of the waters of the stream known as Wolf Creek, in Lenawee County, Michigan, and the tributaries thereof.

The people of the State of Michigan enact:

SECTION 1. It shall be unlawful for any person or persons to wilfully or in any other manner knowingly to befoul, pollute, contaminate in any manner, so as to render said water offensive for drinking purposes, the waters of that stream situated in the townships of Adrian, Rome, and Cambridge, Lenawee County, Michigan, and known commonly as Wolf Creek, or any tributary thereof situated in said county, at any place in said stream above the dam from which the water supply of the city of Adrian is taken.

SEC. 2. Whoever mischievously, maliciously, or wilfully puts any dead animal, carcass or part thereof, or any other putrid, nauseous, noisome, or offensive substance in said stream or its tributaries, or in any other manner befouls the waters of said stream or its tributaries in an unwholesome or offensive manner, or shall drain the contents of any barnyard, waste factory products, or other unwholesome substance, into the water of said stream or its tributaries, shall be deemed guilty of a violation of this act.

SEC. 3. Any person convicted of a violation of this act shall be punished by a fine not exceeding one hundred dollars and not less than five dollars and costs of prosecution, and in default of the payment of said fine and costs he shall be imprisoned in the jail of Lenawee County not less than ten nor more than ninety days, or both such fine and imprisonment, in the discretion of the court.

This act is ordered to take immediate effect.

Approved, May 17, 1899.

[House Enrolled Act No. 404.]

AN ACT in relation to the pollution of the waters of Pine River in the counties of Midland and Gratiot, and Cass River in the county of Tuscola.

SEC. 1. It shall be unlawful for any person, firm, or corporation, except municipal corporations, or any agent or employe of such firm or corporation to pollute the waters of Pine River in the counties of Midland and Gratiot, and Cass River in the county of Tuscola, by depositing or attempting to deposit therein any beet pulp or other waste matter of any kind or character liable to decomposition.

SEC. 2. Any person, firm, or corporation, or any agent or employe of such firm or corporation, found guilty of a violation of this act shall be punished by a fine of not less than one hundred fifty dollars, or more than three hundred dollars, or by imprisonment in the county jail for not less than three months nor more than six months, or by both such fine and imprisonment in the discretion of the court.

MISSISSIPPI.

[Annotated Code of the General Statute Laws (Thompson, Dillard & Campbell).]

SEC. 1326 (under "Crimes and misdemeanors"). If any person shall in any manner permanently obstruct any of the navigable waters, or shall place any obstruction therein and not remove the same within a reasonable time, or if any person shall pollute any such waters by putting therein the carcass of any dead animal, or any refuse or foul matter, or any matter or thing calculated to render the water thereof less fit for drink or the sustenance of fish, the person so offending, in either case, shall be guilty of a misdemeanor, and, on conviction, shall be punished by a fine of not more than fifty dollars, or by imprisonment in the county jail not more than thirty days, or both; but this shall not apply to the Mississippi or Yazoo rivers.

[Amended; Laws of 1898, chap. 89, p. 101.]

Exception of Mississippi and Yazoo rivers dropped out, and the following clause added: "But this act shall not be so construed as to prevent any city or town in this State from constructing sewers so as to empty into any navigable streams of water in this State." (Approved February 10, 1898.)

NEBRASKA.

[Compiled Statutes of Nebraska, 1897.]

SEC. 6892 (Criminal Code, sec. 229). *Putting offensive matter into well or spring.*—If any person or persons shall put any dead animal, carcass, or part thereof, or other filthy substance, into any well, or into any spring, brook, or branch of running water, of which use is made for domestic purposes, every person so offending shall be fined in any sum not less than two nor more than forty dollars.

SEC. 6893 (230). If any person or persons shall put the carcass of any dead animal, or the offals from any slaughterhouse or butcher's establishment, packing house, or fish house, or any spoiled meats or spoiled fish, or any putrid animal substance, or the contents of any privy vault, upon or into any river, bay, creek, pond, canal, road, street, alley, lot, field, meadow, public ground, market space, or common * * * he shall be fined in any sum not less than one nor more than fifty dollars.

NORTH DAKOTA.

[Revised Codes of North Dakota, 1899.]

SEC. 7291 (Penal Code, sec. 435). *Fouling water with gas tar.*—Every person who throws or deposits any gas tar or refuse of any gas house or factory into any public waters, river, or stream, or into any sewer or stream emptying into any such public waters, river, or stream, is guilty of a misdemeanor.

[Chap. 69. Fouling the public waters of this State.]

SEC. 7653. *Fouling public waters.*—Every person who deposits or places or causes to be deposited or placed any dead animal, offal, or other refuse matter offensive to the sight or smell or deleterious to health upon the banks or in the waters of any lake or stream, so far as the same is within the jurisdiction of the State is guilty of a misdemeanor, and upon conviction thereof is punishable by a fine of not less than twenty and not exceeding one hundred dollars.

SEC. 7654. *Extent of last section.*—The provisions of the last section shall be construed to include privies and privy vaults and any stable, shed, pen, yard, or corral wherein is kept any horse, cattle, sheep, or swine and located nearer than sixty feet from the top of the bank of such lake or stream, and also any slaughter house, grave, graveyard, or cemetery located nearer than eighty feet therefrom. But the provisions of said section shall not be construed to prevent any incorporated city within this State from running its sewers into any river: *Provided*, That where there is a dam across said river within the corporate limits of any such city, any such sewer shall connect with such river below such dam.

OKLAHOMA.

[Wilson's Revised and Annotated Statutes of Oklahoma, vol. 1, p. 894.]

SEC. 3732. From "An act to prevent public nuisances and fixing penalties for maintaining the same."

Sec. 16. It shall be unlawful for any person or persons or corporations to put any dead animal, carcass, or part thereof into any well, spring, brook, or branch of running water of which use is made for domestic purposes. Every person or persons so offending shall, on conviction thereof, be fined in any sum not less than five nor more than one hundred dollars.

Sec. 3733. Any person or persons or corporations who shall put any dead animal or any part of the carcass of a dead animal into any river, creek, or pond shall, upon conviction thereof, be fined in any sum not less than two nor more than twenty-five dollars.

Sec. 2344. Every person who throws or deposits any gas tar or refuse of any gas house or factory into any public waters, river, or stream, or into any sewer or stream emptying into any such public waters, river, or stream is guilty of a misdemeanor.

RHODE ISLAND.

[Revision of 1896, sec. 16, p. 977.]

OFFENCES AGAINST THE PERSON.

Sec. 16. Every person who shall mingle any poison with any food, drink, or medicine, with intent to kill or injure any person, and every person who shall wilfully poison any spring, well, or reservoir of water with such intent shall be imprisoned for life or for any term of years.

[Laws of Rhode Island, 1904, chap. 1222, p. 33.]

AN ACT for the better protection of the shell fisheries in the public waters of this State.

Sec. 1. No person shall deposit in, or allow to escape into, or shall cause or permit to be deposited in, or allowed to escape into any of the public waters of this State any substance which shall in any manner injuriously affect the growth of the shellfish in or under said waters, or which shall in any manner affect the flavor or odor of such shellfish so as to injuriously affect the sale thereof, or which shall cause any injury to the public and private fisheries of this State.

Sec. 2. Any person violating any of the provisions of this act shall, upon conviction thereof, be fined not less than five hundred dollars or more than two thousand dollars, one-half thereof to the use of the complainant and one-half thereof to the use of the State: *Provided*, That in case of conviction upon prosecution by the commissioners of shell fisheries the whole of any fine imposed shall go to the use of the State.

Sec. 3. Every person violating any of the provisions of this act shall be liable to pay to the party injured by such violation double the amount of damages caused thereby, to be recovered in an action of the case in any court of competent jurisdiction. It shall not be necessary, before bringing suit for the recovery of such damages, for a criminal prosecution to have been first instituted for the violation of the provisions of this act, nor shall the recovery of damages under this section be a bar to such criminal prosecution.

SEC. 4. It shall be the duty of the commissioners of shell fisheries to investigate all complaints made to them of the violation of any of the provisions of this act. For the purpose of such investigation said commissioners may make examination of the premises, hold public hearings, summon witnesses, and take testimony under oath, and they shall have power to punish, by fine or imprisonment or both, all contempt of their authority in any hearing before them. They may employ professional or expert services as they may deem desirable.

SEC. 5. It shall be the duty of the shell fish commissioners to prosecute any person in their opinion guilty of the violation of any of the provisions of this act, and in all such prosecutions said commissioners shall not be required to enter into any recognizance or to give surety for costs. It shall be the duty of the attorney-general to conduct the prosecution of all cases brought by said commissioners under the provisions of this act. Complaints may also be brought and prosecuted by any citizen for any violation of its provisions.

SEC. 6. The expenses incurred by the commissioners of shell fisheries in the performance of the duties imposed upon them by this act shall be paid by the general treasurer out of any funds in the treasury not otherwise appropriated, upon the presentation of vouchers therefor duly certified by their chairman.

SEC. 7. All provisions of the General Laws, of the Public Laws, and of any special law inconsistent herewith are hereby repealed, and this act shall take effect upon its passage.

[Laws of Rhode Island, 1904, chap. 1178, p. 58.]

AN ACT to prevent pollution of the sources of the water supply of the cities of Pawtucket and Woonsocket and the towns of Bristol and East Providence.

SEC. 1. Section 1 of chapter 491 of the Public Laws is hereby amended so as to read as follows:

"SEC. 1. No person shall throw or discharge, or suffer to be discharged from land owned, occupied, or controlled by him, into any stream, pond, or reservoir used as a source of water supply by the city of Woonsocket, the city of Pawtucket, the city of Newport, the town of Bristol, the town of Warren, the town of East Providence, the town of Narragansett, the town of Jamestown, the East Greenwich fire district, or by any water company supplying water for domestic use in any of said cities or towns, or into any tributary or feeder of any such stream, pond, or reservoir, any sewerage, drainage, refuse, or noxious or polluting matter of such nature as will corrupt or impair the quality of the waters of said stream, pond, or reservoir, or render the same injurious to health, which water shall be of the recognized standard of purity to be determined by the State board of health or other recognized authority. But the provisions of this section shall not interfere with or prevent the enriching of land for agricultural purposes by the owner or occupant thereof if no human excrement is used thereon. Any person violating the provisions of this section shall be punished for each offence by a fine of fifty dollars or by imprisonment for not to exceed thirty days or by both such fine and imprisonment.

SEC. 2. Section 2 of chapter 491 is hereby amended so as to read as follows:

"SEC. 2. The State board of health or the secretary of said board, when satisfied that any sewerage, drainage, or refuse or polluting matter exists in a locality such that there is danger that said sewerage, drainage, or refuse or polluting matter may corrupt or impair the quality of said waters or render them injurious to health, may order the owner or occupant of the premises where said sewerage, drainage, or refuse or polluting matter exists to remove the same from said premises within such time after the serving of the notice prescribed in the next succeeding section as said board or secretary may designate; and if the owner or occupant neglects or refuses so to do he shall be fined twenty dollars for each day during which he permits said sewerage, drainage, or refuse or polluting matter to remain upon said premises after the time prescribed for the removal thereof."

SEC. 3. Section 3 of chapter 491 is hereby amended so as to read as follows:

"SEC. 3. Such notice shall be in writing, signed by the secretary of the State board of health or the person performing the duties of that official, and shall be served by any sheriff, deputy sheriff, or constable by reading the same in the presence or hearing of the owner, occupant, or his authorized agent, or by leaving a copy of the same in the hands or possession of, or at the last and usual place of abode of, said owner, occupant, or agent if within this State: *Provided, however*, That if said owner, occupant, or agent be a corporation incorporated in this State, said notice shall be served by leaving a copy thereof at the last and usual place of abode of the president or person performing the duties of president of said corporation. But if said premises are unoccupied, or the residence of the owner is unknown or without this State, or if the said owner is a corporation incorporated without this State, the notice may be served by posting a copy of the same on the premises and by advertising the same in some newspaper published in Providence County in such manner and for such length of time as the State board of health or the Secretary thereof may determine."

SEC. 4. Section 4 of chapter 491 is hereby amended so as to read as follows:

"SEC. 4. The secretary of the State board of health, when so directed by said board, shall prosecute for all violations of this chapter and shall not be required to give surety for costs upon complaints made by him; but the cities of Woonsocket and Pawtucket and the towns of Bristol and East Providence shall be directly liable to the State for the costs incurred in the prosecution for violation of this chapter in their respective cases."

SEC. 5. Section 5 of chapter 491 is hereby amended so as to read as follows:

"SEC. 5. The appellate division of the supreme court, upon the application of the mayors of said cities or the presidents of the town councils of said towns, or upon the application of the secretary of the State board of health, may issue an injunction to enforce the orders of the State board of health, or the secretary thereof, provided for in this chapter."

SEC. 6. All acts and parts of acts inconsistent herewith are hereby repealed, and this act shall take effect upon its passage.

Passed April 12, 1904.

WISCONSIN.

[Wisconsin Statutes, 1898, p. 651.]

POWERS OF COUNCIL IN CITIES UNDER GENERAL LAW.

57. To provide for the preservation of any harbor within or of the city; prevent any use of the same or of such part of any lake, river, stream, spring, or pond as is within the city, or any action in relation thereto inconsistent with or detrimental to the public health or calculated to render the water of the same or any part thereof impure or offensive; or tending in any degree to fill up and obstruct the same; prohibit and punish the casting or depositing therein of any earth, dead animals, ashes, or other substance, or filth, logs, or floating matter. * * *

PRESERVATION OF PUBLIC HEALTH.

[Idem, p. 1065.]

SLAUGHTERHOUSES. SEC. 1418. No person shall erect, maintain, or keep any slaughterhouse upon the bank of any river, running stream, or creek, or throw or deposit therein any dead animal or any part thereof or any of the carcass or offal therefrom, nor throw or deposit the same into or upon the banks of any river, stream, or creek which shall flow through any city, village, or organized town containing two hundred or more inhabitants, or erect, maintain, or use any building for a slaughterhouse within the limits of any village, incorporated or unincorporated, or at any place within one-eighth of a mile of any dwelling house or a building occupied as a place of business; and every person who shall violate any of the provisions of this section shall forfeit for each such violation not less than ten dollars nor more than one hundred dollars; and the mayor of the city, president of the village, and the chairman of the town in which any such slaughterhouse is located shall have the power to and shall cause the same to be immediately removed; and every such officer who shall knowingly permit any such slaughterhouse to be used or maintained contrary to the provisions of this section shall forfeit not less than fifteen dollars nor more than fifty dollars. In any county containing a population of one hundred thousand or over all the provisions of this section relating to slaughterhouses shall apply to all establishments and manufactories in which dead animals or any part thereof or any of the carcasses or offal therefrom are collected and converted into marketable products.

OFFENSES AGAINST LIVES AND PERSONS.

[Idem, p. 2669.]

SEC. 4384. *Poisoning food, drink, etc.*—Any person who shall mingle any poison with any food, drink, or medicine, with intent to kill or injure any other person, or who shall wilfully poison any spring, well, or reservoir of water with such intent, shall be punished by imprisonment in the State prison not more than ten years nor less than one year.

[Laws of Wisconsin, 1905, chap. 402.]

AN ACT to amend section 4567 of the statutes of 1898, as amended by chapter 325, laws of 1903, prohibiting depositing of deleterious substances in waters and providing a penalty.

SEC. 1. Section 4567 of the statutes of 1898, as amended by chapter 325, laws of 1903, is hereby amended by adding after the words "decayed wood," where they occur in line 14 of chapter 325, laws of 1903, the words: "Sawdust, sawmill offal, and planing mill shavings;" also by adding after the word "paper" where it occurs in line 16 of chapter 325, laws of 1903, the words "beet sugar;" further amend by striking out the word "or" in line 20, all of lines 21, 22, 23, 24, and 25, and the words "mill shavings" in line 26 of chapter 325, laws of 1903; also further amend by adding after the word "mouth," where it occurs in line 30 of chapter 325, laws of 1903, the words "nothing in this section shall apply to the following streams: The Kickapoo River, the Pine River in Richland County, Balsam branch in Polk County, the Chippewa River from mouth of Thornapple River to its mouth, Flambeau River from dam at Ladysmith to its mouth, Black River from Falls Dam down, in Jackson County, and the Wisconsin River from the north boundary line of the city of Rhinelander down to its mouth," so that said section 4567 when so amended shall read as follows:

"SECTION 4567. Any person who shall cast, deposit, or throw overboard from any row, sail, or steam boat or other craft into any of the inland waters of this State or into Green Bay, Sturgeon Bay, and Chequamegon Bay, or deposit or leave upon the ice thereof until it melts, any fish offal, which shall be construed to mean and include the head, intestines, blood, and cleanings of fish and dead fish, or throw or deposit or permit to be thrown or deposited any lime, tanbark, ship ballast, stone, sand, slabs, decayed wood, sawdust, sawmill offal, and planing-mill shavings, or any acids or chemicals or waste or refuse arising from the manufacture of pulp, paper, or beet sugar, or other substances deleterious to fish life (authorized drainage and sewage from municipalities excepted), into any of the rivers, lakes, or streams of this State, including Green Bay, Chequamegon Bay, Sturgeon Bay, or into any streams wherein there have been planted trout fry, or in which trout naturally abound, shall be punished by a fine of not less than twenty-five dollars nor more than one hundred dollars, or by imprisonment in the county jail not less than thirty days nor more than four months. (Nothing in this section shall apply to the following streams: The Kickapoo River, the Pine River in Richland County, Balsam branch in Polk County, the Chippewa River from the mouth of Thornapple River to its mouth, Flambeau River from dam at Ladysmith to its mouth, the Black River from the Falls Dam down in Jackson County, and the Wisconsin River from the north boundary line of the city of Rhinelander down to its mouth.) The fact of any fisherman coming to the shore with dressed fish in his boat and without the offal produced by such dressing shall be prima facie evidence of the violation of the first clause of this section."

SEC. 2. All acts or parts of acts inconsistent with or in conflict with the provisions of this act are hereby repealed.

SEC. 3. This act shall take effect and be in force from and after its passage and publication.

Approved, June 17, 1905.

CLASS II. STATES WITH GENERAL RESTRICTIONS.

This group consists of those States and Territories in which the importance of pure water for every inhabitant of the State or Territory for drinking and domestic purposes has received legislative recognition. It will be noted that the laws are general in their application, varying much in the elaborateness of the wording and in the emphasis laid upon the remedies and penalties provided for infractions of the law.

This class logically includes all States not included in Class I, but inasmuch as certain States have recently adopted stringent and elaborate methods, novel and extraordinary in their character, to restore and protect the purity of their navigable and potable waters, these States have been omitted from Class II and are treated in a class by themselves, forming Class III (see p. 57).

CALIFORNIA.

[Penal code as in force at the close of the session of 1901.]

SEC. 374. *Putting dead animals in streets, rivers, etc.*—Every person who puts the carcass of any dead animal, or the offal from any slaughter pen, corral, or butcher shop into any river, creek, pond, reservoir, stream, street, alley, public highway, or road in common use, or who attempts to destroy the same by fire within one-fourth of a mile of any city, town, or village, except it be in a crematory, the construction and operation of which is satisfactory to the board of health of such city, town, or village; and every person who puts any water-closet or privy, or the carcass of any dead animal, or any offal of any kind in or upon the borders of any stream, pond, lake, or reservoir from which water is drawn for the supply of the inhabitants of any city, city and county, or any town in this State, so that the drainage for such water-closet, privy, or carcass, or offal may be taken up by or in such stream, pond, lake, or reservoir; or who allows any water-closet or privy, or carcass of any dead animal, or any offal of any kind to remain in or upon the borders of any such stream, pond, lake, or reservoir within the boundaries of any land owned or occupied by him, so that the drainage from such water-closet, privy, carcass, or offal may be taken up by or in such stream, pond, lake, or reservoir, or who keeps any horses, mules, cattle, swine, sheep, or live stock of any kind penned, corralled, or housed on, over, or on the borders of any such stream, pond, lake, or reservoir, so that the waters thereof become polluted by reason thereof, or who bathes in any such stream, pond, lake, or reservoir, or who by any other means fouls or pollutes the waters of any such stream, pond, lake, or reservoir is guilty of a misdemeanor, and upon conviction thereof shall be punished as described in section 377. (Commissioners' amendments, approved March 16, 1901; took effect July 1, 1901.)

SEC. 374½. *Discharging coal tar, etc., into waters.*—Every person, firm, association, or corporation which shall discharge or deposit, or shall cause or suffer to be discharged or deposited, or to pass in or into the waters of any navigable bay or river in this State any coal tar or refuse or residuary product of coal, petroleum, asphalt, bitumen, or other carbonaceous material or substance is guilty of a misdemeanor, and for each offense is punishable by imprisonment in the county jail for not exceeding one year or by fine not exceeding $1,000 or by both such fine and imprisonment. (New section, approved March 25, 1901; took effect immediately. Statutes, 1901, p. 813.)

[Statutes of California, 1905, chap. CXXXV, p. 138.]

AN ACT to amend the penal code of the State of California by adding a new section thereto, to be numbered section 377b, making it a misdemeanor to refuse or neglect to conform to the rules, orders, and regulations of the State board of health, concerning the pollution of water, used or intended to be used for human or animal consumption:

SEC. 1. A new section to be numbered section 377b is hereby added to the penal code of the State of California, to read as follows:

377b. Any person who shall violate or refuse or neglect to conform to any sanitary rule, order, or regulation prescribed by the State board of health for the prevention of the pollution of springs, streams, rivers, lakes, wells, or other waters used or intended to be used for human or animal consumption shall be guilty of a misdemeanor.

SEC. 2. All acts and parts of acts inconsistent or in conflict with this act are hereby repealed.

SEC. 3. This act shall take effect immediately. (Act of March 18, 1905.)

[Statutes of California, 1905, Chap. CXXXVI, p. 138.]

AN ACT To amend the penal code of the State of California by adding a new section thereto, to be numbered section 377c, making it a misdemeanor to refuse or neglect to conform to the rules, orders, and regulations of the State board of health, concerning the pollution of ice used or intended for public consumption.

SEC. 1. A new section, to be numbered 377c, is hereby added to the penal code of the State of California, to read as follows:

377c. Any person who shall violate, or refuse or neglect to conform to any sanitary rule, order, or regulation prescribed by the State board of health for the prevention of the pollution of ice or the sale or disposition of polluted ice offered, kept, or intended for public use or consumption, shall be guilty of a misdemeanor.

SEC. 2. All acts and parts of acts inconsistent or in conflict with this act are hereby repealed.

SEC. 3. This act shall take effect immediately. (Act of March 18, 1905.)

COLORADO.

[Mills' Annotated Statutes, 1891, p. 949.]

SEC. 1376. *Polluting streams—penalty.*—If any person or persons shall hereafter throw or discharge into any stream of running water or into any ditch or flume in this State any obnoxious substance, such as refuse matter from slaughterhouse or privy, or slops from eating houses or saloons, or any other fleshy or vegetable matter which is subject to decay in the water, such person or persons shall, upon conviction thereof, be punished by a fine not less than one hundred dollars nor more than five hundred dollars for each and every offense so committed.

SEC. 1357 provides a penalty not exceeding five hundred dollars for anyone "who shall in anywise pollute or obstruct any water course, lake, pond, marsh, or common sewer, or continue such obstruction or pollution so as to render the same offensive or unwholesome," &c.

SEC. 3330 (p. 1861). *Emptying oil into the waters of the State a misdemeanor—penalty.*

AN ACT To prohibit the emptying or running of oil or petroleum, or other oleaginous substance into any waters of this State, and to impose a penalty for the violation of this act.

[Laws, 1889, p. 287, approved March 7, 1889, in force June 7, 1889.]

If any person or persons, corporation or corporations shall hereafter empty or cause to be emptied, or allow the emptying or flowing of oil, petroleum, or other oleaginous substance into any of the waters of this State, or deposit or cause the same to be deposited at such distance that the same may be carried into such waters by natural causes, such person or persons, corporation or corporations so offending shall be deemed guilty of a misdemeanor, and upon conviction thereof shall be punished by a fine not exceeding one thousand dollars, or imprisonment in the county jail not exceeding six months, or both such fine and imprisonment, for each such offense.

ILLINOIS.

[Hurd's Revised Statutes, 1901, sec. 202, p. 627.]

Whoever willfully and maliciously defiles, corrupts, or makes impure any spring or other source of water or reservoir * * * shall be fined not exceeding one thousand dollars or confined in a county jail not exceeding one year.

Page 631, section 221, makes it a public nuisance—

1. To cause or suffer the carcass of any animal or any offal, filth, or noisome substance to be collected or deposited or to remain in any place to the prejudice of others.

2. To throw or deposit any offal or other offensive matter, or any carcass of any dead animal, in any water course, lake, pond, spring, well, or common sewer, street, or public highway.

3. To corrupt or render unwholesome or impure the water of any spring, river, stream, pond, or lake to the injury or prejudice of others.

INDIANA.

[Burns's Annotated Statutes, 1904.]

SEC. 2156. *Nuisance by dead animals.*—Whoever puts the carcass of any dead animal or the offal from any slaughterhouse or butcher's establishment, packing house, or fish house, or any spoiled meats or spoiled fish, or any putrid animal substance, or the contents of any privy vault upon or into any river, pond, canal, lake, public ground, market place, common, field, meadow, lot, road, street, or alley, and whoever, being the owner or occupant of any such place, knowingly permits any such thing to remain therein to the annoyance and injury of any of the citizens of the State, or neglects or refuses to remove or abate the nuisance occasioned thereby within twenty-four hours after knowledge of the existence of such nuisance upon any of the above described premises owned or occupied by him, or after notice thereof, in writing, from any health officer of the city or the trustee of the township in which such nuisance exists, shall be fined not more than one hundred dollars nor less than one dollar.

SEC. 2169. Whoever maliciously or mischievously puts any dead animal carcass or part thereof on, or any other putrid, nauseous, noisome, or offensive substance into, * * * or in any manner befouls any well, cistern, spring, brook, canal, or stream of running water, or any reservoir of waterworks of which any use is made or may be made for domestic purposes shall be fined not more than one hundred dollars nor less than five dollars, to which may be added imprisonment in the county jail not more than sixty days nor less than ten days.

(The foregoing section is repealed by the act of 1905 hereafter quoted.)

SEC. 3538. *Streams and ferries.*—The common council shall have exclusive power to keep open streams, and preserve, and, if necessary and expedient, change the course of rivers passing through or bordering upon the corporate limits of such city; to prevent encroachment or injury to the banks thereof, or the casting into the same of offal, dead animals, logs, or rubbish. * * *

[Acts of 1901, Chap. LXI, p. 96.]

AN ACT prohibiting the discharge of waste water and refuse of manufacturing establishments into streams of water, conferring certain powers upon the State board of health in such cases, providing penalties for the violation thereof, and declaring an emergency.

SECTION 1. *Be it enacted by the general assembly of the State of Indiana*, That it shall be unlawful for any person, firm, or corporation owning or operating any manufacturing establishment to discharge or permit to be discharged into any stream of water any waste water or refuse from said factory of such character as to pollute said stream, except by and in pursuance to a written permission so to do, first obtained from the State board of health as hereinafter provided.

SEC. 2. Whenever any person, firm, or corporation owning or operating a manufacturing establishment shall file with the secretary of the State board of health a verified application in writing, asking permission to be allowed to discharge into any stream any waste water or refuse from such establishment, and showing therein that the water of said stream is at such stage as that such refuse or waste water may be safely discharged into such stream without injury to the public, it shall be the duty of such board to inspect the said stream at and below the point of such proposed discharge, and if it is found that such refuse and waste water may be safely discharged therein without injury as aforesaid, the said board may, in its discretion, grant and issue a written permit allowing such discharge into said stream for a time to be limited therein, which permit shall be void and of no effect after the time so fixed, and may be revoked by said board at any time. The holder of any such permit regularly issued by such board shall be authorized to discharge any such refuse or waste water into such stream during the time fixed and limited in such permit, and shall not be liable therefor in any suit at law or in equity: *Provided*, That nothing herein contained shall prevent any person specially damaged by any such discharge from recovering the amount of such special damages so sustained in an action at law brought for such purpose.

SEC. 3. Any person, firm, or corporation violating any of the provisions of this act shall be fined in any sum not less than twenty-five dollars nor more than five hundred dollars.

SEC. 4. Whereas an emergency exists for the immediate taking effect of this act, the same shall be in force on and after its passage.

[Laws of 1905, chap. 169, p. 584.]

SEC. 553. *Befouling water.*—Whoever maliciously or mischievously puts any dead animal, carcass, or part thereof, or any other putrid, nauseous, noisome, or offensive substance into, or in any manner befouls any well, cistern, spring, brook, canal, or stream of running water, or any reservoir of waterworks, of which any use is or may be made for domestic purposes, shall, on conviction, be fined not less than five dollars nor more than one hundred dollars, to which may be added imprisonment in the county jail not less than ten days nor more than sixty days.

SEC. 689. *Repeal.*—All laws within the purview of this act are hereby repealed; but this repeal shall not affect any prosecutions pending or offenses heretofore committed under existing laws, and such prosecutions and offenses shall be continued and prosecuted to a final determination, as if this act had not passed; nor shall this repeal affect the enforcement of any fine or penalty or other punishment provided as a punishment for the violation of any civil statute; nor shall this act be construed to repeal any act passed at this session of the general assembly.

(Approved March 9, 1905.)

MAINE.

[Laws of 1891, chap. 82, p. 67.]

AN ACT to protect waters used for domestic purposes.

SEC. 1.[a] Whoever knowingly and willfully poisons, defiles, or in any way corrupts the waters of any well, spring, brook, lake, pond, river, or reservoir used for domestic purposes for man or beast, or knowingly corrupts the sources of the water supply of any water company or of any city, town, or municipal corporation supplying its inhabitants with water, or the tributaries of said sources of supply, in such manner as to affect the purity of the water so supplied, or knowingly defiles such water in any manner, whether the same be frozen or not, or puts the carcass of any dead animal or other offensive material into said waters or upon the ice thereof, shall be punished by a fine not exceeding one thousand dollars or by imprisonment not exceeding one year.

SEC. 2. Whoever shall wilfully injure any of the property of any water company or of any city, town, or municipal corporation used by it in supplying water to its inhabitants shall be punished by a fine not exceeding one thousand dollars or by imprisonment not exceeding one year, and such person shall also forfeit and pay to such water company, city, or town three times the amount of actual damages sustained, to be recovered in an action of the case. (As amended by the laws of 1905, Chap. 93, p. 97.)

SEC. 3. Inconsistent acts repealed.

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[a] As amended by laws of 1905, chap. 97, p. 100.

[Laws of 1903, Special Laws, chap. 94, p. 156.]

AN ACT to prevent the pollution of the waters of Sebago Lake.

SEC. 1. No person or corporation shall use or occupy any structure hereafter built upon or near the shores of Sebago Lake, in the county of Cumberland, or upon any of the islands of said lake for such purposes or in such manner that the sewage or drainage therefrom shall enter the waters of said lake or pollute the same.

SEC. 2. No sewage, drainage, refuse, or polluting matter of such kind and amount as either by itself or in connection with other matter will corrupt or impair the quality of the water of said Sebago Lake or render it injurious to health shall be discharged into said lake, but nothing herein shall prohibit the cultivation and use of the soil in the ordinary methods of agriculture if no human excrement is used thereon within three hundred feet of the shores of said lake.

SEC. 3. The supreme judicial court shall have jurisdiction in equity to enjoin, prevent, or restrain any violation of the provisions of this act.

SEC. 4. This act shall take effect when approved.

Approved February 26, 1903.

MARYLAND.

[Poe's Maryland Code, adopted March 14, 1888.]

RIVERS.

SEC. 240. If any ballast, ashes, filth, earth, soil, oysters, or oyster shells be taken, unladen, or cast out of any ship, steamboat, scow, pungy, or other vessel, on any pretense whatever, in the Chesapeake Bay above "Sandy Point," or in the waters of Herring Bay, or in any river, creek, or harbor within this State, below high-water mark, the master or other person having charge of such vessel shall, upon conviction thereof, be fined.

Waters of Potomac River above the canal dam near the mouth of Wills Creek are protected by section 242 against pollution calculated to render the waters of said river "impure or unfit for use."

WATER SUPPLY—POLLUTION OF SOURCES OF.

[Passed in 1886, chap. 6.]

SEC. 277. If any person shall put, or cause to be placed, any dead animal or part of the carcass of any dead animal, or any decayed or filthy animal or vegetable matter, into any stream, or the tributary of any stream, well, spring, reservoir, pond, or other source from which water or ice is drawn, taken, or used for drinking or domestic purposes, or shall knowingly suffer any sewage, washings, or other offensive matters from any privy, cesspool, factory, trades establishment, slaughterhouse, tannery, or other place over which he shall have control, to flow therein, or into any drain or pipe communicating therewith, whereby the water supply of any city, town, village, community, or household is fouled or rendered unfit for drinking and domestic purposes, he shall be guilty of a misdemeanor and shall, upon conviction thereof in a court of competent jurisdiction, be fined not more than two hundred dollars for every such offence; and after reasonable notice, not exceeding fifteen days, from the State board of health, or any local sanitary authority, to discontinue the act whereby such water supply is fouled, a further sum of not more than fifty dollars for every day during which the offence is continued.

MISSOURI.

[Revised Statutes, 1899.]

CRIMES AND PUNISHMENTS.

SEC. 2234. *Putting dead animals in well, &c.*—If any person or persons shall put any dead animal, carcass, or part thereof, the offal, or any other filth into any well, spring, brook, branch, creek, pond, or lake, every person so offending shall, on conviction thereof, be fined in any sum not less than ten nor more than one hundred dollars. If any person shall remove, or cause to be removed and placed * * * in any of the streams and water courses other than the Missouri or Mississippi River, any dead animal, carcass, or part thereof, or other nuisance, to the annoyance of the citizens of this State, or any of them, every person so offending shall, upon conviction thereof, be fined for every such offence any sum not less than ten dollars nor more than fifty dollars, and if such nuisance be not removed within three days thereafter, it shall be deemed a second offence against the provisions of this section.

SEC. 2235. *Corrupting or diverting water supply.*—Whoever willfully or maliciously poisons, defiles, or in any way corrupts the water of a well, spring, brook, or reservoir used for domestic or municipal purposes, or whoever willfully or maliciously diverts, dams up, and holds back from its natural course and flow any spring, brook, or other water supply for domestic or municipal purposes, after said water supply shall have once been taken for use by any person or persons, corporations, town, or city for their use, shall be adjudged guilty of a misdemeanor and punished by a fine not less than fifty nor more than five hundred dollars, or by imprisonment in the county jail not exceeding one year, or by both such fine and imprisonment, and shall be liable to the party injured for three times the actual damage sustained, to be recovered by suit at law.

SEC. 1974. *Injury to schoolhouses and church buildings.*—Every person * * * who shall in any manner pollute the water contained in any well, cistern, or reservoir (in which water is gathered or kept for the supply of a schoolhouse or those attending the same) shall be guilty of a misdemeanor.

[Section 28 of House bill No. 15, laws of 1905, p. 163.]

AN ACT relating to the preservation, propagation, and protection of game animals, birds, and fish; creating the office of game and fish warden; creating a game protection fund, and appropriating money therefrom.

SEC. 28. It shall be unlawful for any person or persons, firm, or corporation to suffer or permit any dyestuff, coal tar, oil, sawdust, poison or deleterious substances to be thrown, run, or drained into any of the waters of this State in quantities sufficient to injure, stupefy, or kill fish which may inhabit the same at or below the point where any such substances are discharged or permitted to flow or thrown into such waters. Any person or persons, firm, or corporation offending against any of the provisions of this section shall be deemed guilty of a misdemeanor, and upon conviction shall be fined not less than $200.00 nor more than $500.00 for each offense.

Approved March 10, 1905.

NEVADA.

[General Statutes of Nevada.]

SEC. 4617. (Crimes and punishments, sec. 54.) * * * Every person who shall willfully poison any spring, well, or reservoir of water shall, upon conviction thereof, be punished by imprisonment in the State prison for a term not less than one nor more than ten years.

Sawdust in rivers.—It is made a misdemeanor to deposit sawdust in or on the waters of any lake, river, or running stream by laws of 1889, page 24, Chapter XV.

[Laws of Nevada, 1903, Chap. CXXII, p. 214.]

AN ACT to prevent the pollution or contamination of the waters of the lakes, rivers, streams, and ditches in the State of Nevada, prescribing penalties, and making an appropriation to carry out the provisions of this act. (Approved March 20, 1903.)

The people of the State of Nevada, represented in senate and assembly, do enact as follows:

SECTION 1. *Unlawful to pollute any body of water.*—Any person or persons, firm, company, corporation, or association in this State, or the managing agent of any person or persons, firm, company, corporation, or association in this State, or any duly elected, appointed, or lawfully created State officer of this State, or any duly elected, appointed, or lawfully created officer of any county, city, town, municipality, or municipal government in this State, who shall deposit, or who shall permit or allow any person or persons in their employ or under their control, management, or direction to deposit in any of the waters of the lakes, rivers, streams, and ditches in this State any sawdust, rubbish, filth, or poisonous or deleterious substance or substances liable to affect the health of persons, fish, or live stock, or place or deposit any such deleterious substance or substances in any place where the same may be washed or infiltered into any of the waters herein named, shall be deemed guilty of a misdemeanor, and upon conviction thereof in any court of competent jurisdiction shall be fined in any sum not less than fifty dollars nor more than five hundred dollars, exclusive of court costs: *Provided*, That in cases of State institutions, municipalities, towns, incorporated towns or cities, when, owing to the magnitude of the work, immediate correction of the evil is impracticable, then in such cases the authorities shall adopt all new work, and as rapidly as possible reconstruct the old systems of drainage, sewerage, and so as to conform with the provisions of this act: *And provided further*, That all such new and reconstructed systems shall be completed within four years from the date of passage hereof: *Provided*, That nothing in this act shall be so construed as to permit mining or milling companies to dump tailings directly into any stream in this State so as to prevent or impede the natural flow of such stream. Nothing in this act shall be so construed as to apply to any quartz mill or ore reduction works in this State.

SEC. 2. For the purposes of this act the word "ditch" shall be construed to mean any ditch, canal, channel, or artificial waterway used for carrying or conducting water into any reservoir from which it may be used or distributed for domestic purposes to any person in this State, or to any person in any county, city, town, or municipality in this State.

SEC. 3. The sum of three thousand dollars is hereby appropriated out of any money in the State treasury, not otherwise appropriated, subject to the disposal of the governor of this State, for the purpose of enforcing the provisions of this act, either in the courts of this State or in the courts of the United States, such expenditure to be allowed and paid as other claims against the State are allowed and paid.

SEC. 4. This act shall take effect and be in force from and after the first day of July, A. D. nineteen hundred and four.

NEW MEXICO.

[Compiled laws, act of March 16, 1897.]

STREAMS AND LAKES.

SEC. 54. It shall not be lawful for any person or persons to throw or cast the dead body or carcass of any animal or fowl, or to run or empty any sewers or other polluted or befouled substances into any river, stream, lake, pond, reservoir, ditch, or any water course, or to in any manner or by any means pollute or befoul the waters thereof, within this Territory, so as to render the same unwholesome or offensive or dangerous to the health of the inhabitants of any community or of any person having the right to use and who uses the same, for drinking or domestic purposes, or that may render such waters unfit or dangerous for watering stock, or for agricultural or horticultural purposes.

SEC. 55. That the polluting of waters in any of the manners above specified is hereby declared to be a public nuisance, which shall be immediately removed by the person or persons creating the same, upon the demand of any public officer or of any person or persons, who may have a right to the use of said waters.

SEC. 56. That any person or persons violating any of the provisions of sec. 54 may be tried therefor before any justice of the peace of the county where the offence is committed, and upon conviction thereof shall be punished by a fine in any sum not less than ten dollars nor more than one hundred dollars, or by imprisonment in the county jail for any period of time not less than ten days nor more than sixty days, or by both fine and imprisonment. And in addition thereto the justice of the peace shall direct the sheriff of the county or the constable of the precinct to relieve such nuisance, at the expense of the person or persons creating the same, which said expenses shall be taxed as other costs against the person or persons so offending, and shall be collected in the manner provided by law for the collection of costs in criminal cases.

[Laws of 1899, chap. 79, p. 175.]

AN ACT to amend section 54 of the compiled laws of 1897. (Approved March 16th, 1899.)

Be it enacted by the legislative assembly of the Territory of New Mexico:

SECTION 1. That section 54 of the compiled laws of 1897 be, and the same is hereby, amended to read as follows:

SEC. 54. It is hereby made unlawful for any person to cast the dead body of any animal or fowl, or any refuse matter, such as tin cans, paper, ashes, bones, or other garbage into any running stream, spring, lake, pond, reservoir, ditch, or water course, or to run or empty any sewer or other foul substance into the same, or in any other manner or means to pollute or foul the said water so as to render the same offensive or dangerous to the health of the inhabitants of any community or of any person having the right to use the same for drinking or domestic purposes, or that may render said waters unfit or unhealthy for watering stock. But it shall be the duty of every person, outside of incorporated towns, cities, or villages, to destroy all domestic refuse and garbage by burning the same; any violation of this section shall be considered a misdemeanor and punished as provided by law.

SEC. 2. All acts and parts of acts in conflict herewith are hereby repealed; and this act shall take effect from and after its passage.

[Laws of 1903, chap. 21, p. 32.]

AN ACT to prevent injury to ditches, pipe lines, reservoirs, and the taking of and befouling of water therefrom. (Approved March 10th, 1903.)

Be it enacted by the legislative assembly of the Territory of New Mexico:

SECTION 1. Any person who shall wilfully and maliciously cut, break, or injure, or who shall by shooting or by damming or obstructing the same cause to break any ditch, flume, pipe line, or reservoir, or any of the attachments or fixtures used in connection therewith, shall be guilty of a misdemeanor and shall be punished by a fine of not less than ten dollars nor more than fifty dollars, or by confinement in the county jail for not more than sixty days, or by both such fine and imprisonment, in the discretion of the court trying the case, except in cases where such pipe line or reservoir is used for the purpose of supplying water to any community, village, town, or city for domestic purposes, in which event the person committing such offence shall be punished by a fine of not less than fifty nor more than one hundred dollars, or by imprisonment in the county jail not less than thirty nor more than sixty days, or by both such fine and imprisonment in the discretion of the court trying the case.

SEC. 2. Any person who shall bathe in, or wilfully cast any filth in, any reservoir or ditch used for supplying water for domestic use shall be guilty of a misdemeanor, and upon conviction shall be fined not less than ten dollars or not more than twenty-five dollars.

SEC. 3. All acts and parts of acts in conflict herewith are hereby repealed, and this act shall take effect from and after its passage.

NORTH CAROLINA.

[North Carolina Criminal Code and Digest (2d ed.), p. 436.]

SEC. 500. Putting poisonous substance in water for the purpose of killing fish is forbidden.

Laws of 1903, chapter 245, page 321, forbids throwing sawdust into the water courses of Yancey County.

[Laws of North Carolina, 1903, chap. 159, p. 182.]

AN ACT to protect water supplies.

Sections 1 to 10, inclusive, provide a thorough system of inspection and forbid any person or corporation to supply water for the public without taking the precautions therein prescribed.

Sections 11 to 17 are as follows:

SEC. 11. Whoever defiles, corrupts, pollutes any well, spring, drain, branch, brook, or creek, or other source of public water supply used for drinking purposes, in any manner, or deposits the body of any dead animal on the watershed of any such water supply, or allows the same to remain thereon unless the same is buried with at least two feet cover, shall be guilty of a misdemeanor, and fined and imprisoned, in the discretion of the court.

SEC. 12. Whoever shall collect and deposit human excreta on the watershed of any public water supply shall be guilty of a misdemeanor, and punished by fine and imprisonment, in the discretion of the court.

SEC. 13. No person, firm, corporation, or municipality shall flow or discharge sewage into any drain, brook, creek, or river from which a public drinking-water supply is taken, unless the same shall have been passed through some well-known system of sewage purification approved by the State board of health. Any person, firm, corporation, or the officer of any municipality having this work in charge, who shall violate this section shall be guilty of a misdemeanor, and the continued flow and discharge of such sewage may be enjoined by any person.

SEC. 14. That all schools, hamlets, villages, towns, or industrial settlements which are now located or may be hereafter located on the shed of any public water supply not provided with a sewerage system, shall provide and maintain a tub system for collecting human excrement, and provide for removal of the same from the watershed at least twice each week. Every person, firm, corporation, or municipality violating this section shall be guilty of a misdemeanor, and fined or imprisoned, in the discretion of the court.

SEC. 15. No burying ground or cemetery shall be established on the watershed of any public water supply nearer than five hundred yards of the source of supply.

SEC. 16. All water companies now operating under charters from the State or municipalities, which may maintain public water supplies, may acquire by condemnation such lands and rights in land and water as are necessary for the successful operation and protection of their plants, said proceedings to be the same as prescribed by chapter 49, volume 1, of the Code of North Carolina.

SEC. 17. For carrying out the provisions of this act the State board of health is authorized and empowered to have the bacteriological examination made as hereinbefore provided for, and to charge for the same the sum of five dollars ($5.00) for each examination.

[Laws of 1905, chap. 415.]

AN ACT to establish a State laboratory of hygiene.

SECTION 1. That for the better protection of the public health and to prevent the spread of communicable diseases there shall be established a State laboratory of hygiene, the same to be under the control and management of the State board of health.

SEC. 2. That it shall be the duty of the State board of health to have made in such laboratory monthly examinations of samples from all the public water supplies of the State. The board shall also cause to be made examinations of well and spring waters when in the opinion of any county superintendent of health or any registered physician there is reason to suspect such waters of being contaminated and dangerous to health. The board shall likewise have made in this laboratory examinations of sputum in cases of suspected tuberculosis, of throat exudates in cases of suspected diphtheria, of blood in cases of suspected typhoid and malarial fever, of *fæces* in cases of suspected hook-worm diseases, and such other examinations as the public health may require.

SEC. 3. For the support of the said laboratory the sum of twelve hundred dollars is hereby appropriated and an annual tax of sixty dollars, payable quarterly, by each and every water company, municipal, corporate, and private, selling water to the people, said tax to be collected by the sheriff as other taxes and paid by said sheriff directly to the treasurer of the State board of health, and the printing and stationery necessary for the laboratory to be furnished upon requisition upon the State printer.

SEC. 4. Section seventeen of chapter one hundred and fifty-nine of the laws of one thousand nine hundred and three is hereby repealed.

SEC. 5. This act shall be in force from and after its ratification.

In the general assembly read three times, and ratified this 4th day of March, 1905.

OHIO.

[Bates's Annotated Revised Statutes of Ohio, p. 3343.]

SEC. 6921. *Nuisance.*—Whoever * * * corrupts or renders unwholesome or impure any water course, stream, or water * * * shall be fined not more than five hundred dollars.

SEC. 6923. (*Unlawful deposit of dead animals, offal, &c., into or upon land or water.*)—Whoever puts the carcass of any dead animal, or the offal from any slaughterhouse or butcher's establishment, packing house, or fish house, or any spoiled meat or spoiled fish, or any putrid substance, or the contents of any privy vaults, upon or into any lake, river, bay, creek, pond, canal, road, street, alley, lot, field, meadow, public ground, market place or common, and whoever, being the owner or occupant of any such place, knowingly permits any such thing to remain therein, to the annoyance of any of the citizens of this State, neglects or refuses to remove or abate the nuisance occasioned thereby, within twenty-four hours after knowledge of the existence of such nuisance upon any of the above-described premises, owned or occupied by him, or after notice thereof in writing from any supervisor, constable, trustee, or health officer of any municipal corporation or township in which such nuisance exists, or from a county commissioner of such county, shall be fined not more than fifty dollars nor less than ten dollars and pay the costs of prosecution, and in default of the payment of said fine and costs be imprisoned not more than thirty days; but the provisions hereinbefore made shall not prohibit the depositing of the contents of privy vaults and catch-basins into trenches or pits not less than three feet deep, excavated in any lot, field, or meadow, the owner thereof consenting, outside the limits of any municipal corporations, and not less than thirty rods distant from any dwelling, well, or spring of water, lake, bay, or pond, canal, run, creek, brook, or stream of water, public road or highway: *Provided*, That said contents deposited in said trenches or pits are immediately thereafter covered with dry earth to the depth of at least twelve inches; nor shall said provisions prohibit the depositing of said contents into furrows situate and distinct, as specified for said trenches or pits, provided the same are immediately thereafter wholly covered with dry earth by plowing or otherwise: *And provided also*, That the owner or occupant of the land in which said furrows are plowed consents and is a party thereto: *Provided also*, That the board of health of any municipal corporation may allow said contents to be deposited within corporate limits into trenches or pits or furrows, situate distant and to be covered as aforesaid.

SEC. 6925. *Emptying of coal dirt, petroleum, &c., into lakes, rivers, &c., or permitting same; penalty.*—Whoever intentionally throws or deposits, or permits to be thrown or deposited, any coal dirt, coal slack, coal screenings, or coal refuse from coal mines, or any refuse or filth from any coal-oil refinery or gas works, or any whey or filthy drainage from a cheese factory, upon or into any of the rivers, lakes, ponds, or streams of this State, or upon or into any place from which the same will wash into any such river, lake, pond, or stream; or whoever shall, by himself, agent, or employe, cause, suffer, or permit any petroleum, or crude oil, or refined oil, or any compound or mixture or other product of such well, except fresh or salt water, or residuum of oil or filth from oil well, or oil tank, or oil vat, or place of deposit, of crude or refined oil, to run into, or be poured, or emptied, or thrown into any river, or ditch, or drain, or water course, or into any place from which said petroleum, or crude oil, or residuum, or refined oil, or filth may run or wash, or does run or wash, into any such river, or ditch, or drain, or water course, upon indictment and conviction in the county in which such coal mines, coal-oil refinery, gas works, cheese factory, oil well, oil tank, oil vat, or place of deposit of crude or refined oil are situated, shall be fined in any sum not more than one thousand dollars nor less than fifty dollars.

(*Fine and costs a lien; execution.*)—And such fine and costs of prosecution shall be and remain a lien on said oil well, oil tank, oil refinery, oil vat, and place of deposit, and the contents of said oil well, oil tank, oil refinery, oil vat, or place of deposit until said fine and costs are paid; and said oil well, oil tank, oil refinery, oil vat, or place of deposit, and the contents thereof, may be sold for the payment of such fine and costs upon execution duly issued for that purpose.

Sec. 6927. (*Befouling well, spring, &c.*)—Whoever maliciously puts any dead animal carcass, or part thereof, or any other putrid, nauseous, noisome, or offensive substance into, or in any manner befouls, any well, spring, brook, or branch of running water, or any reservoir of waterworks, of which use is or may be made for domestic purposes, shall be fined not more than fifty nor less than five dollars, or imprisoned not more than sixty days, or both.

[Laws of 1904, house bill 277, p. 135.]

AN ACT to amend section 2433, Revised Statutes of Ohio, for the purpose of preventing the pollution of water and providing penalty therefor.

Sec. 1. That section 2433, Revised Statutes of Ohio, be, and the same is hereby, amended to read as follows:

"Sec. 2433. The jurisdiction of any municipal corporation to prevent the pollution of its water supply and to provide penalty therefor shall extend twenty miles beyond the corporation limits. Whoever pollutes any running stream, the water of which is used for domestic purposes by any municipality by putting therein any putrid or offen-

sive substance (other than fresh or salt water) injurious to health shall be guilty of a misdemeanor, which shall be punishable by a fine of not less than five or more than five hundred dollars. It shall be the duty of the board of public service or board of trustees of public affairs of any municipal corporation to enforce the provisions of this section."

SEC. 2. Original section 2433 is hereby repealed.

OREGON.

[Bellinger and Colton's Annotated Codes and Statutes of Oregon, vol. 1, p. 735.]

OF CRIMES AGAINST THE PUBLIC HEALTH.

SEC. 2128. *Polluting with sewage, &c., water for domestic use unlawful.*—Any person who shall put any sewage, drainage, or refuse, or polluting matter, as either by itself or in connection with other matter will corrupt or impair the quality of any well, spring, brook, creek, branch, or pond of water which is used or may be used for domestic purposes, shall be deemed guilty of misdemeanor. (Laws 1885, p. 110, sec. 1.)

SEC. 2129. *Animal carcass, &c., unlawful to place in water for domestic use or near dwelling.*—If any person shall put any dead animal carcass, or part thereof, excrement, putrid, nauseous, noisome, decaying, deleterious, or offensive substance into, or in any other manner not herein named befouls, pollutes, or impairs the quality of any spring, brook, creek, branch, well, or pond of water, which is or may be used for domestic purposes, or shall put any such dead animal carcass, or part thereof, excrement, putrid, nauseous, noisome, decaying, deleterious, or offensive substance within one-half mile of any dwelling house or public highway, and leave the same without proper burial, or, being in the possession or control of any land, shall knowingly permit or suffer any such dead animal carcass, or part thereof, excrement, putrid, nauseous, noisome, decaying, deleterious, or offensive substance to remain without proper burial upon such premises, within one-half mile of any dwelling house or public highway, whereby the same becomes offensive to the occupants of such dwelling or the traveling public, he shall be deemed guilty of a misdemeanor. (1885, p. 110, sec. 2.)

SEC. 2130. *Penalty for violating preceding provisions and jurisdiction to enforce.*—Any person violating the provisions of this act shall, upon conviction, be fined not less than ten nor more than fifty dollars, or be imprisoned not less than five days nor more than twenty-five days, or by both fine and imprisonment. Justices of the peace shall have jurisdiction of offences committed against the provisions of this act.

SEC. 2131. *Polluting water used for domestic purposes, or to which live stock have access, unlawful.*—If any person or persons shall put any dead animal's carcass, or part thereof, or any excrement, putrid, nauseous, decaying, deleterious, or offensive substance in any well, or into any spring, brook, or branch of running water, of which use is made for domestic purposes, or to which any cattle, horses, or other kind of stock have access, every person so offending shall, on conviction thereof, be fined in any sum not less than three nor more than fifty dollars.

SEC. 2133. *Animal carcass, unlawful to put in river or elsewhere to injury of health.*—If any person or persons shall put any part of the carcass of any dead animal into any river, creek, pond, road, street, alley, lane, lot, field, meadow, or common, or if the owner or owners thereof shall knowingly permit the same to remain in any of the aforesaid places to the injury of the health or to the annoyance of the citizens of this State, or any of them, every person so offending shall, on conviction thereof, be fined in a sum not less than two nor more than twenty-five dollars, and every twenty-four hours during which said owner may permit the same to remain thereafter shall be deemed an additional offence against the provisions of this act.

SOUTH DAKOTA.

[Revised Codes of 1903, Penal Code, p. 1146.]

SEC. 445. Every person who throws or deposits any gas tar, or refuse of any gas house or factory into any public waters, river, or stream, or into any sewer or stream emptying into such public waters, river, or stream, is guilty of a misdemeanor.

SEC. 446. It shall be unlawful for any person, persons, company, or corporation to place or cause to be placed any manure, butcher's offal, carcasses of animals, or other deleterious substances into any river, stream, or lake, in the State of South Dakota, or upon the banks thereof in such proximity that such substances may be washed into said water or water courses.

SEC. 447. Any violation of the provisions of this chapter is a misdemeanor, and the person, persons, company, or corporation so violating are deemed guilty thereof, and upon conviction shall be liable to a fine not less than ten dollars nor more than one hundred dollars, and in addition thereto such offending person or persons shall be subjected to imprisonment in the county jail for the period of thirty days unless he or they cause such deleterious substances to be removed.

SEC. 448. This act shall not be construed as to interfere with or prevent any necessary or legitimate mining operation or sewerage system.

sive substance (other than fresh or salt water) injurious to health shall be guilty of a misdemeanor, which shall be punishable by a fine of not less than five or more than five hundred dollars. It shall be the duty of the board of public service or board of trustees of public affairs of any municipal corporation to enforce the provisions of this section."

SEC. 2. Original section 2433 is hereby repealed.

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SEC. 447. Any violation of the provisions of this chapter is a misdemeanor, and the person, persons, company, or corporation so violating are deemed guilty thereof, and upon conviction shall be liable to a fine not less than ten dollars nor more than one hundred dollars, and in addition thereto such offending person or persons shall be subjected to imprisonment in the county jail for the period of thirty days unless he or they cause such deleterious substances to be removed.

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SEC. 2133. *Animal carcass, unlawful to put in river or elsewhere to injury of health.*—If any person or persons shall put any part of the carcass of any dead animal into any river, creek, pond, road, street, alley, lane, lot, field, meadow, or common, or if the owner or owners thereof shall knowingly permit the same to remain in any of the aforesaid places to the injury of the health or to the annoyance of the citizens of this State, or any of them, every person so offending shall, on conviction thereof, be fined in a sum not less than two nor more than twenty-five dollars, and every twenty-four hours during which said owner may permit the same to remain thereafter shall be deemed an additional offence against the provisions of this act.

SOUTH DAKOTA.

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SEC. 447. Any violation of the provisions of this chapter is a misdemeanor, and the person, persons, company, or corporation so violating are deemed guilty thereof, and upon conviction shall be liable to a fine not less than ten dollars nor more than one hundred dollars, and in addition thereto such offending person or persons shall be subjected to imprisonment in the county jail for the period of thirty days unless he or they cause such deleterious substances to be removed.

SEC. 448. This act shall not be construed as to interfere with or prevent any necessary or legitimate mining operation or sewerage system.

TENNESSEE.

[Code of Tennessee, 1896.]

SEC. 6869. It is a public nuisance— * * *

3. To corrupt or render unwholesome or impure the water of any river, stream, or pond to the injury or prejudice of others.

SEC. 6520. If any person place or throw the dead body of any animal in any spring, well, cistern, or running stream of water he is guilty of a misdemeanor.

[1903, chap. 310, p. 905.]

Section 1 makes it a misdemeanor for "any person to in any way wilfully * * * disturb, pollute, contaminate, or injure the water in the tanks, standpipes, or reservoirs of any such waterworks by bathing therein or by any other act or acts tending to injure the water or to make it unpalatable, unwholesome, or unfit for domestic or manufacturing purposes of any plant supplying water for domestic or manufacturing purposes, however owned."

SEC. 2. That it shall be a misdemeanor for any person to wilfully corrupt or to permit anything to run or fall into any stream from which water shall be taken for the purpose of supplying water to any water plant such as is referred to in section 1 of this act, and any person violating this section shall be punished as provided in section 1 hereof.

Act takes effect April 7, 1903, on its passage.

TEXAS.

[White's Annotated Penal Code of Texas, p. 256.]

OFFENCES AFFECTING PUBLIC HEALTH.

ART. 424. If any person shall in any wise pollute of [a] [or?] obstruct any water course, lake, pond, marsh, or common sewer, or continue such obstruction or pollution so as to render the same unwholesome or offensive to the inhabitants of the county, city, town, or neighborhood thereabout, he shall be fined in a sum not exceeding five hundred dollars.

OFFENCES AGAINST THE PERSON.

ART. 647. If any person shall mingle or cause to be mingled any other noxious potion or substance with any drink, food, or medicine, with intent to kill or injure any other person, or shall wilfully poison or cause to be poisoned any spring, well, cistern, or reservoir of water with such intent, he shall be punished by imprisonment in the penitentiary not less than two nor more than ten years.

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[a] So in original.

UTAH.

[Revised Statutes, p. 910, Penal Code: Public Health and Safety.]

SEC. 4274. *Befouling waters.*—Any person who shall either:

1. Construct or maintain any corral, sheep pen, stable, pigpen, chicken coop, or other offensive yard or outhouse, where the waste or drainage therefrom shall flow directly into the waters of any stream, well, or spring of water used for domestic purposes; or

2. Deposit, pile, unload, or leave any manure heap, offensive rubbish, or the carcass of any dead animal where the waste or drainage therefrom will flow directly into the waters of any stream, well, or spring of water used for domestic purposes; or

3. Dip or wash sheep in any stream, or construct, maintain, or use any pool or dipping vat for dipping or washing sheep in such close proximity to any stream used by the inhabitants of any city, town, or village for domestic pusposes as to make the waters thereof impure or unwholesome; or

4. Construct or maintain any corral, yard, or vat to be used for the purpose of shearing or dipping sheep within twelve miles of any city, town, or village, where the refuse or filth from said corral or yard would naturally find its way into any stream of water used by the inhabitants of any city, village, or town for domestic purposes; or

5. Establish and maintain any corral, camp, or bedding place for the purpose of herding, holding, or keeping any cattle, horses, sheep, or hogs within seven miles of any city, town, or village, where the refuse or filth from said corral, camp, or bedding place will naturally find its way into any stream of water used by the inhabitants of any city, town, or village for domestic purposes, shall be guilty of a misdemeanor.

[Laws of 1899, chap. 45, p. 66.]

SEC. 2. No house refuse, offal, garbage, dead animals, decaying vegetable matter, or organic waste substance of any kind shall be thrown on or allowed to remain upon any street, road, ditch, gutter, public place, private premises, vacant lot, water course, lake, pond, spring, or well.

VIRGINIA.

[Pollard's General Laws, 1887–1896, chap. 72, p. 44 (Acts 1887–88, p. 83).]

AN ACT to prevent the pollution of drinking water in this State. (Approved February 3, 1888.)

1. *Be it enacted by the general assembly of Virginia*, That any person or persons who shall knowingly and wilfully throw or cause to be thrown into any reservoir or other receptacle of drinking water,

or spring, or stream of running water ordinarily used for the supply of drinking water or domestic purposes of any person or family, town, or city in this Commonwealth the dead body of any animal, or shall drown and leave, or cause to be drowned and left any animal therein shall be guilty of a misdemeanor, and upon conviction thereof shall be fined not exceeding one hundred dollars or imprisoned not exceeding six months, or both, at the discretion of the court in which such conviction is made.

[Idem, p. 115 (Acts 1891–92, p. 759).]

AN ACT to prevent the pollution of potable water used for the supply of cities and towns. (Approved February 29, 1892.)

1. *Be it enacted by the general assembly of Virginia*, That it shall be unlawful, except as hereinafter provided, for any person to defile or render impure, turbid, or offensive the water used for the supply of any city or town of this State, or the sources or streams used for furnishing such supply, or to endanger the purity thereof by the following means, or any of them, to wit, by washing or bathing therein, or by casting into any spring, well, pond, lake, or reservoir from which such supply is drawn, or into any stream so used, or the tributary thereof above the point where such supply is taken out of such stream or is impounded for the purposes of such supply, or into any canal, aqueduct, or other channel or receptacle for water connected with any works for furnishing a public water supply, any offal, dead fish, or carcass of any animal, or any human or animal filth or other foul or waste animal matter, or any waste vegetable or mineral substance, or the refuse of any mine, manufactory, or manufacturing process, or by discharging or permitting to flow into any such source, spring, well, reservoir, pond, stream, or the tributary thereof, canal, aqueduct, or other receptacle for water, the contents of any sewer, privy, stable, or barnyard, or the impure drainage of any mine, any crude or refined petroleum, chemicals, or any foul, noxious, or offensive drainage whatsoever, or by constructing or maintaining any privy vault or cesspool, or by storing manure or other soluble fertilizer of an offensive character, or by disposing of the carcass of any animal, or any foul, noxious, or putrescible substance, whether solid or fluid and whether the same be buried or not, within two hundred feet of any water course, canal, pond, or lake aforesaid, which is liable to contamination by the washing thereof or percolation therefrom: *Provided*, That nothing in this act contained shall be construed to authorize the pollution of any of the waters of this State in any manner now contrary to law: *And provided further*, That this act shall not apply to streams the drainage area of which, above the point where the water thereof is withdrawn for the supply of any city or

town, or is impounded for the purposes of such supply, shall exceed fifty square miles.

2. That any person knowingly or wilfully violating the terms of this act shall be deemed guilty of a misdemeanor, and shall be punished for each offence by a fine not exceeding one hundred dollars or by imprisonment not exceeding thirty days, or by both, at the discretion of the court: *And provided further*, That nothing herein contained shall be so construed as to prevent the washing of ore or minerals in any of the streams or waters of this Commonwealth other than such as may be used for the water supply of any city or town.

3. This act shall take effect fifteen days after its passage.

WASHINGTON.

[Ballinger's Annotated Codes and Statutes, including acts of 1897.]

NUISANCES. Sec. 3085. It is a public nuisance:

* * * * * * *

2. To throw or deposit any offal or other offensive matter, or the carcass of any dead animal, in any water course, stream, lake, pond, spring, well, or common sewer, street or public highway, or in any manner to corrupt or render unwholesome or impure the water of any such spring, stream, pond, lake, or well, to the injury or prejudice of others.

Punishment provided in section 3097.

[Acts of 1899, Chap. LXX, p. 114: Providing for a pure water supply.]

AN ACT to preserve from pollution the water supplied to the inhabitants of cities and towns in the State of Washington; to declare what are nuisances in the vicinity of the source of such water supply; providing for the abatement thereof, and for the punishment of the violations of this act.

Be it enacted by the legislature of the State of Washington:

SECTION 1. That for the purpose of protecting the water furnished to the inhabitants of towns and cities within this State from pollution, the said towns and cities are hereby given jurisdiction over all property occupied by the works, reservoirs, systems, springs, branches, and pipes by means of which, and of all sources of supply from which, such cities or the companies or individuals furnishing water to the inhabitants of such cities or towns obtain their supply of water or store or conduct the same.

SEC. 2. That the establishment or maintenance of any slaughter pen, stock-feeding yards, hogpens, or the deposit or maintenance of any uncleanly or unwholesome substance, or the conduct of any business or occupation, or the allowing of any condition upon or sufficiently near the sources from which the supply of water for the inhabitants of any such city or town is obtained, or where such water is stored, or the property or means through which the same may be conducted or conveyed, so that such water would be polluted or the purity of such water or any part thereof destroyed or endangered, is hereby prohibited and declared to be unlawful, and is hereby declared to be and constitute a nuisance, and as such to be abated as other nuisances are abated under the provisions of the existing laws of the State of Washington, or under the laws which may be hereafter enacted in relation to the abatement thereof; and that any person or persons who shall do, establish, maintain, or create any of the things hereby prohibited for the purpose of or which shall have the effect of polluting any such sources of water supply or water, or shall do any of the things hereby declared to be unlawful, shall be deemed guilty of creating and maintaining a nuisance, and may be prosecuted therefor, and upon conviction thereof may be fined in any sum not exceeding five hundred dollars.

SEC. 3. If upon the trial of any person or persons for the violation of any of the provisions of this act such person or persons shall be found guilty of creating or maintaining a nuisance as hereby defined or of violating any of the provisions of this act, it shall be the duty of such person or persons to forthwith abate such nuisance, and in the event of their failure so to do within one day after such conviction, unless further time be granted by the court, a warrant shall be issued by the court wherein such conviction was obtained directed to the sheriff of the county in which such nuisance exists, and the sheriff shall forthwith proceed to abate the said nuisance, and the cost thereof shall be taxed against the party so convicted as a part of the costs of such case.

SEC. 4. It is hereby made the duty of the city health officer, city physician, board of public health, mayor of the city, or such other officer as may have the sanitary condition of such city or town in charge, to see that the provisions of this act are enforced, and, upon complaint being made to any such officer, to immediately investigate the said complaint and see if the same shall appear to be well founded; and if the same shall appear to be well founded, it shall be, and is hereby, declared to be the duty of such officer to proceed and file a complaint against the person or persons violating any of the provisions of this act and cause the arrest and prosecution of such person or persons.

SEC. 5. That any city supplied with water from any source of supply as hereinbefore mentioned, or any corporation owning waterworks for the purpose of supplying any city or the inhabitants thereof with water, in the event that any of the provisions of this act are being violated by any person, may, by civil action in the superior court of the proper county, have the maintenance of the nuisance which pollutes or tends to pollute the said water, as provided for by section 2 of this act, enjoined, and such injunction may be perpetual.

WEST VIRGINIA.

[Code of West Virginia, 1891, p. 933.]

OFFENCES AGAINST PUBLIC HEALTH—MISDEMEANOR TO PUT DEAD ANIMALS, ETC., INTO WATER USED FOR DOMESTIC PURPOSES.

If any person or persons shall knowingly and willfully throw or cause to be thrown into any well, cistern, spring, brook, or branch of running water which is used for domestic purposes, any dead animal, carcass, or part thereof, or any putrid, nauseous, or offensive substance, he or they shall be guilty of a misdemeanor, and upon conviction thereof shall be fined not less than five dollars nor more than one hundred dollars, and may, at the discretion of the jury, be confined in the jail of the county not exceeding ninety days, and shall moreover be liable to the party injured in a civil action for damages. (Acts 1872–73, ch. 176.)

PREVENTING THE DEPOSIT OF THE CARCASSES OF DEAD ANIMALS AND OTHER NOXIOUS MATTER IN CERTAIN WATERS OF THE STATE, ETC.

It shall be unlawful to put the carcass of any dead animal, or the offal from any slaughterhouse, butcher's establishment, or packing house, or slop or other refuse from any hotel or a tavern, or any spoiled meats or spoiled fish, or any putrid animal substance, or the contents of any privy vault, upon or into any river, creek, or other stream within this State, or upon the surface of any road, street, alley, city lot, public ground, market space, or common, or on the surface within one hundred feet of any public road.

III. A justice of the peace shall have jurisdiction of any offence against the provisions of this act, committed within his county. Any such offence shall be punished by a fine of not less than five or more than fifty dollars, and the proceedings in the case, as well as in all other cases under this act, shall be in conformity with sections 221 to 230, inclusive, or chapter 50 of the Code of West Virginia, which sections are hereby made applicable to such cases. Upon a conviction for any such offence the accused must bury at least three feet under the ground, or destroy by fire, any of the things named in the first section which he has placed in any of the waters or places named in such section, or which he has knowingly permitted to remain upon a city lot, public ground, market space, or common, contrary to the provisions of the second section, within twenty-four hours after such conviction, and if he shall fail to do so, the justice shall further fine him not less than ten nor more than fifty dollars. (Acts 1887, ch. 25.)

WYOMING.

[Revised Statutes, 1899.]

CRIMES AGAINST THE PERSON.

SEC. 4966. *Poisoning springs.*—Whoever poisons any spring, well, cistern, or reservoir of water with intent to injure or kill any human being shall be imprisoned in the penitentiary not more than fourteen years.

CRIMES AGAINST PUBLIC HEALTH AND SAFETY.

SEC. 5114. *Putting offensive substances in creek or highway declared a nuisance.*—If any person or persons, association of persons, company, or corporation shall deposit, place, or put, or cause to be deposited, placed, or put upon or into any river, creek, bay, pond, canal, ditch, lake, stream, railroad, public or private road, highway, street, alley, lot, field, meadow, public place or public ground, common, market place, or in any other and different locality in this State, where the same may become a source of annoyance to any person or detrimental to the public health, the carcass of any dead animal or the offal or refuse matter from any slaughterhouse, butcher's establishment, meat market, packing house, fish house, hogpen, stable, or any spoiled meats, spoiled fish, or any animal or vegetable matter in a putrid or decayed state, or liable to become putrid, decayed, or offensive, or the contents of any privy vault, or any offensive matter or substance whatever, or shall cause to be maintained any privy, slaughterhouse, meat market, or any other or different place, building, or establishment that shall directly or indirectly be the cause of polluting the waters of any spring, reservoir, stream, lake, or water supply used wholly or partly for domestic purposes, or if the owner or owners, tenant or tenants, occupant or occupants of any lands or tenements, dwellings, or places of business, or any other and different places or localities, whether defined in this section or not, shall knowingly permit any of the said offensive matters or substances, or any other and different offensive matter or substances, to remain in any of the aforesaid places or other and different places or localities, or shall permit any of the aforesaid places to be maintained which shall cause the pollution of any stream, spring, reservoir, lake, or water supply, either directly or indirectly, in any locality, place, or situation in this State, to the annoyance of the citizens or residents of this State, or any of them, or to the detriment of the public health, or who shall neglect or refuse to remove or abate the nuisance, offence, or inconvenience occasioned or caused thereby within twenty-four hours after knowledge of the existence of such nuisance, offence, or inconvenience in or upon any of the above-described premises or places, or any other and different place or locality, owned or occupied by him her, it, or they,[a] [them?] or after notice in writing from the sheriff, deputy sheriff, or coroner of any county in this State, or the constable of any precinct, or the marshal or any of the policemen of any city, town, or village in which such nuisance shall exist, or from any peace officer in this State of the locality wherein such nuisance shall exist, every such person so offending shall be guilty of a misdemeanor, and upon conviction thereof shall be punished by a fine of not less than ten dollars nor more than fifty dollars, and if such nuisance is not abated within forty-eight hours after the same is created or exists to the knowledge of such offender, or within forty-eight hours after said written notice is given, such failure to abate such nuisance shall be deemed a second offence against the provisions of this section, and every like failure and neglect to abate such nuisance of each twenty-four hours thereafter shall be considered an additional offence, and shall be subject to a like penalty as is herein provided.

SEC. 5115. *Abatement of nuisance.*—Provides that officer shall remove nuisance, on neglect of owners so to do, expenses collectible in civil action.

SEC. 5116. *Throwing sawdust into streams.*—If any person or persons who may own, run, or have charge of any sawmill in this State shall throw or permit the sawdust therefrom to be thrown or placed in any manner into any river, stream, creek, bay, pond, lake, canal, ditch, or other water course in this State, such person or persons shall be liable to a like penalty as is provided in section 5114.

[Laws of Wyoming, 1905, chap. 31, p. 25.]

FISH—POLLUTING WATERS.

AN ACT To repeal section 1 of chapter 22 of the session laws of Wyoming of the year A. D. 1903, being an act entitled "An act to amend and reenact section 2146, revised statutes of Wyoming, 1899, relating to the unlawful taking or having in possession of certain kinds of fish," and to amend and reenact section 2148, revised statutes of Wyoming, 1899, relating to the unlawful placing of deleterious substances, poisons, or explosives in the waters of the State.

SEC. 1. That section 1, chapter 22, of the session laws of Wyoming, 1903, being "An act to amend and reenact section 2146 of the revised statutes of Wyoming, 1899, relating to the unlawful taking or having in possession of certain kinds of fish," be and the same is hereby repealed.

SEC. 2. That section 2148 of the revised statutes of Wyoming, 1899, be amended and reenacted so as to read as follows:

"SEC. 2148. Any owner or owners of any sawmill, reduction works, smelter, refining or contraction works, or any of the employees

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[a] So in original.

thereof, who shall throw, deposit, or in any way permit to pass into any natural stream or lake wherein are living fish, any sawdust, chemicals, or other matter or substance that will tend to drive away from such waters any fish shall be deemed guilty of a misdemeanor and shall be fined not less than twenty-five dollars nor more than one hundred dollars, or shall be imprisoned in the county jail not less than thirty days nor more than sixty days. Any person who shall kill in any of the waters of this State, by use of any poison or deleterious drug, or by the use of any explosive substance, or explode or cause to be exploded any powder, giant powder, hercules powder, dynamite, nitroglycerine, lime gas, or any other explosive substance for the purpose of catching, killing, or destroying food fish in such waters shall be deemed guilty of a misdemeanor and upon conviction thereof shall be fined not less than fifty dollars nor more than two hundred dollars, and shall be imprisoned in the county jail not less than ninety days nor more than one year: *Provided further*, That nothing in this title contained shall prevent the owner or owners of any quartz mill or reduction works in this State, now located or to be hereafter located upon any natural stream or lake, from operating or working said quartz mill or reduction works, where the said owner or owners thereof shall build or cause to be built a suitable dam, to be used in connection with said quartz mill or reduction works, and which dam shall be so constructed as to prevent any tailings or substance from passing into the stream or lake which will destroy or drive away the fish or any number of them from said stream, lake, or water."

SEC. 3. This act shall take effect and be in force from and after its passage.

Approved February 15, A. D. 1905.

[Chapter 83, House bill No. 87.]

FISH.

AN ACT to repeal section 1 of chapter 22 of the session laws of Wyoming of the year A. D. 1903, being an act entitled "An act to amend and reenact section 2146, revised statutes of Wyoming, 1899, relating to the unlawful taking or having in possession of certain kinds of fish," and to amend and reenact section 2148, revised statutes of Wyoming, 1899, relating to the unlawful placing of deleterious substances, poisons, or explosives in the waters of the State.

SEC. 1. *Repeal of sec. 1, chap. 22, laws 1903.*—That section 1, chapter 22, of the session laws of Wyoming, 1903, being "An act to amend and reenact section 2146 of the revised statutes of Wyoming, 1899, relating to the unlawful taking or having in possession of certain kinds of fish" be, and the same is hereby, repealed.

SEC. 2. *Use of explosives and poison.*—That section 2148 of the revised statutes of Wyoming, 1899, be amended and reenacted so as to read as follows:

"SEC. 2148. Any owner or owners of any sawmill, or any of the employees thereof, who shall throw, deposit, or in any way permit to pass into any natural stream or lake wherein are living fish any sawdust or other matter or substance that will tend to drive away from such waters any fish, shall be deemed guilty of a misdemeanor, and shall be fined not less than twenty-five dollars nor more than one hundred dollars, or shall be imprisoned in the county jail not less than thirty days nor more than sixty days. Any person who shall kill in any of the waters of this State, by use of any poison or deleterious drug, or by use of any explosive substances, or explode or cause to be exploded any powder, giant powder, hercules powder, dynamite, nitroglycerine, lime gas, or any other explosive substance for the purpose of catching, killing, or destroying the food fish in such waters, shall be deemed guilty of a misdemeanor, and upon conviction thereof shall be fined not less than fifty dollars nor more than two hundred dollars, and shall be imprisoned in the county jail not less than ninety days nor more than one year."

SEC. 3. This act shall take effect and be in force from and after its passage.

Approved February 21, A. D. 1905.

CLASS III. STATES WITH SEVERE RESTRICTIONS.

This group consists of those States which have adopted unusual and stringent methods to enforce the right of their citizens to unpolluted natural waters. The adoption of the legislation embodied in the following pages under this group indicates that the inhabitants of the States in which these laws have been adopted have begun to realize the immense harm which the increased pollution of waters, owing to increase of population, is doing to persons and property within their borders. It is noticeable that in several of the States stringent methods are adopted by which pollution by cities can be regulated and controlled; while in at least one State (New Jersey) a system has been instituted which, carried to its logical conclusion, will result in conveying all sewage matter from cities and large towns so far beyond the borders of the land as to render it wholly inoffensive or in some other way preventing its getting into any inland waters in an offensive form.

CONNECTICUT.

[General Statutes, revision of 1902, sec. 1328, as amended by chap. 28, of the laws of 1905.]

Every person who shall wantonly and indecently expose his person, or who shall bathe in any reservoir from which the inhabitants of any town, city or borough are supplied with water, or in any lake, pond or stream tributary to such reservoir, or who shall cast any filthy or impure substance into said reservoir, or any of its tributaries, or commit any nuisance in or about it or them, shall be fined not more than one hundred dollars, or imprisoned not more than six months, or both.

[General Statutes, revision of 1902, p. 668.]

SEC. 2593. *Pollution of water from which ice is taken.*—Every person who shall put any substance into waters from which ice is procured for consumption which shall defile, pollute, or injure the quality of said ice, or who shall throw anything into such waters or upon the ice with intent to injure the quality of the ice or obstruct the cutting or gathering of the same, shall be fined not more than thirty dollars or imprisoned not more than thirty days. This section shall not affect the rights of any manufacturing establishment now existing or hereafter established to use any waters in carrying on its business.

SEC. 2594. *Pollution of waters.*—Every person who shall put or leave a dead animal or carcass in a pond, spring, or reservoir, the water of which is conveyed to any building, or who shall wilfully put and leave in any of the waters of this State a dead animal, shall be fined not more than fifty dollars or imprisoned not more than thirty days.

SEC. 2595. *Penalty for polluting drinking water.*—Every person who shall put anything into a well, spring, fountain, or cistern, or other place from which water is procured for drinking or other purposes, with the intent to injure the quality of said water, shall be fined not more than five hundred dollars or imprisoned not more than six months.

SEC. 2596. *Analysis of water.*—Town, borough, and city health officers shall, when in their judgment health is menaced or impaired through a water supply, send, subject to the approval of the county health officer, samples of such water to the State board of health for examination and analysis, and the expense of such examination and analysis shall be paid out of the funds appropriated to said board to investigate the pollution of streams.

SEC. 2598. *Location of cemeteries.*—No cemetery or place of sepulture shall hereafter be located or established within one-half mile of any reservoir from which the inhabitants of a town, city, or borough are supplied with water; nor shall such reservoir be located or established within one-half mile of a cemetery or place of sepulture unless the superior court of the county wherein such cemetery or place of sepulture or reservoir is located shall, upon application or notice find that such cemetery or place of sepulture or such reservoir so proposed to be located is of public convenience and necessity and will not be detrimental to the public health.

SEC. 2602. *Pollution of reservoirs—Penalty.*—No person, after notice shall have been posted that any reservoir, or any lake, pond, or stream tributary thereto, is used for supplying the inhabitants of a town, city, or borough with water, shall wash any animal, clothing, or other article therein. No person shall throw any noxious or harmful substance into such reservoir, lake, pond, or stream, nor shall any person, after receipt of written notice from any county or town health officer having jurisdiction that the same is detrimental to such water supply, suffer any such substance to be placed upon land owned, occupied, or controlled by him, so that the same may be carried by rains or freshets into the water of such reservoir, lake, pond, stream, or drain, or allow to be drained any sewage from said land into such water. Every person who shall violate any provision of this section shall be fined not more than one hundred dollars or imprisoned not more than thirty days, or both.

SEC. 2603. *Appointment of special police.*—The governor may, upon the application of such town, borough, city, or company, commission during his pleasure one or more persons who, having been sworn, may act as policemen for the purpose of preventing and abating nuisances and protecting such water supply from contamination. Such policemen shall arrest without previous complaint and warrant any person for any offense under the provisions of any law for the protection of water supplies when the offender shall be taken or apprehended in the act or on the speedy information of others, and all persons so arrested shall be immediately presented before proper authority. Every such policeman shall, when on duty, wear in plain view a shield bearing the words "Special police" and the name of the town, city, borough, or company for which he is commissioned.

[Acts of 1903, chap. 192, p. 148.]

AN ACT concerning injunctions.

Be it enacted by the senate and house of representatives in general assembly convened.

SECTION 1. Section 2599 of the General Statutes is hereby amended to read as follows: Whenever any land or building is so used, occupied or suffered to remain, that it is a source of injury to the water stored in a reservoir used for supplying a town, city, or borough with water, or to any source of supply to such reservoir, or when such water is liable to pollution in consequence of the use of the same, either the authorities of such town, city, or borough, or the company having charge of said water, may apply to the superior court, or any judge thereof in vacation, in the county in which said town, city, borough, or company is located, for relief; and said court or judge may order the removal of any building, enjoin any use or occupation of any land or building or of said water which is detrimental to said water, or make any other order, temporary or permanent, which in its or his judgment may be necessary to preserve the purity of said water. Said town, city, borough, or company may, by its officers or agents, duly appointed for such purpose, at all reasonable times enter upon and inspect any premises within the watershed tributary to such water supply, and in case any nuisance shall be found thereon which pollutes or is likely to pollute such water, may abate such nuisance at its own expense after reasonable notice to the owner or occupant of said premises and upon his neglect or refusal to abate the same: but such town, city, borough, or company shall be liable for all unnecessary or unreasonable damage done to said premises.

Sec. 2. Section 2600 of the General Statutes is hereby amended to read as follows: Any city, town, borough, or corporation authorized by law to supply the inhabitants of any city, town, or borough with pure water for public or domestic use may take and use such lands, springs, streams, or ponds, or such rights or interests therein as the superior court or any judge thereof in vacation may, on application, deem necessary for the purposes of such supply. For the purpose of preserving the purity of such water and preventing any contamination thereof, such city, town, borough, or corporation may take such lands or rights as the superior court or any judge thereof in vacation may, on application, deem necessary therefor. Compensation shall be made to all persons entitled thereto in the manner provided by section 2601.

Sec. 3. Section 2601 of the General Statutes is hereby amended to read as follows: In all cases where the law requires compensation to be made to any person whose rights, interests, or property are injuriously affected by said orders, such court or judge shall appoint a committee of three disinterested freeholders of the county who shall determine and award the amount to be paid by such authorities before such order is carried into effect.

Approved June 18, 1903.

MASSACHUSETTS.

[Revised laws of the Commonwealth of Massachusetts, enacted November 21, 1901, taking effect January 1, 1902, chap. 75, p. 677.]

OF THE PRESERVATION OF THE PUBLIC HEALTH.

SEC. 112. *Supervision of inland waters.*—The State board of health shall have the general oversight and care of all inland waters and of all streams and ponds used by any city, town, or public institution, or by any water or ice company, in this Commonwealth as sources of water supply, and of all springs, streams, and water courses tributary thereto. It shall be provided with maps, plans, and documents suitable for such purposes and shall keep records of all its transactions relative thereto.

SEC. 113. *Examination of water supply.*—Said board may cause examinations of such waters to be made to ascertain their purity and fitness for domestic use or their liability to impair the interests of the public or of persons lawfully using them or to impair the public health. It may make rules and regulations to prevent the pollution and to secure the sanitary protection of all such waters as are used as sources of water supply.

SEC. 114. *Effect of publication of notice.*—The publication of an order, rule, or regulation made by the board under the provisions of the preceding section, or section one hundred and eighteen, in a newspaper of the city or town in which such order, rule, or regulation is to take effect, or, if no newspaper is published in such city or town, the posting of a copy of such order, rule, or regulation in a public place in such city or town, shall be legal notice to all persons, and an affidavit of such publication or posting by the person causing such notice to be published or posted, filed and recorded with a copy of the notice in the office of the clerk of such city or town, shall be admitted as evidence of the time at which, and the place and manner in which, the notice was given.

SEC. 115. *Report and recommendations.*—Said board shall annually, on or before the tenth day of January, make a report to the general court of its doings for the preceding year, recommend measures for the prevention of the pollution of such waters and for the removal of polluting substances in order to protect and develop the rights and property of the Commonwealth therein and to protect the public health, and recommend any legislation or plans for systems of main sewers necessary for the preservation of the public health and for the purification and prevention of pollution of the ponds, streams, and inland waters of the Commonwealth. It shall also give notice to the attorney-general of any violation of law relative to the pollution of water supplies and inland waters.

SEC. 116. *Agents and assistants.*—Said board may appoint, employ, and fix the compensation of such agents, clerks, servants, engineers, and expert assistants as it considers necessary. Such agents and servants shall cause the provisions of law relative to the pollution of water supply and of the rules and regulations of said board to be enforced.

SEC. 117. *Advice as to methods.*—Said board shall consult with and advise the authorities of cities and towns and persons having, or about to have, systems of water supply, drainage, or sewerage as to the most appropriate source of water supply, and the best method of assuring its purity or as to the best method of disposing of their drainage or sewage with reference to the existing and future needs of other cities, towns, or persons which may be affected thereby. It shall also consult with and advise persons engaged or intending to engage in any manufacturing or other business whose drainage or sewage may tend to pollute any inland water as to the best method of preventing such pollution, and it may conduct experiments to determine the best methods of the purification or disposal of drainage or sewage. No person shall be required to bear the expense of such consultation, advice, or experiments. Cities, towns, and persons shall submit to said board, for its advice, their proposed system of water supply or of the disposal of drainage or sewage, and all petitions to the general court for authority to introduce a system of water supply, drainage, or sewerage shall be accompanied by a copy of the recommendation and advice of said board thereon. In this section the term "drainage" means rainfall, surface, and subsoil water only, and "sewage" means domestic and manufacturing filth and refuse.

SEC. 118. *Removal of causes of pollution.*—Upon petition to said board by the mayor of a city or the selectmen of a town, the managing board or officer of any public institution, or by a board of water commissioners, or the president of a water or ice company, stating that manure, excrement, garbage, sewage, or any other matter pollutes or tends to pollute the waters of any stream, pond, spring, or water course used by such city, town, institution, or company as a source of water supply, the board shall appoint a time and place within the county where the nuisance or pollution is alleged to exist for a hearing, and after notice thereof to parties interested and a hearing, if in its judgment the public health so requires, shall, by an order served upon the party causing or permitting such pollution, prohibit the deposit, keeping, or discharge of any such cause of pollution, and shall order him to desist therefrom and to remove any such cause of pollution; but the board shall not prohibit the cultivation and use of the soil in the ordinary methods of agriculture if no human excrement is used thereon. Said board shall not prohibit the use of any structure which was in existence on the eleventh day of June, in the year eighteen hundred and ninety-seven, upon a complaint made by the board of water commissioners of any city or town or by any water or ice company, unless such board of water commissioners or company files with the State board a vote of its city council, selectmen, or company, respectively, that such city, town, or company will, at its own expense, make such changes in said structure or its location as said board shall deem expedient. Such vote shall be binding on such city, town, or company. All damages caused by such changes shall be paid by such city, town, or company; and if the parties can not agree thereon, the damages shall, on petition of either party, filed within one year after such changes are made, be assessed by a jury in the superior court for the county where the structure is located.

SEC. 119. *Appeal from order.*—Whoever is aggrieved by an order passed under the provisions of the preceding section may appeal therefrom in the manner provided in sections 95 and 97, but such notice as the court shall order shall also be given to the board of water commissioners and mayor of the city or chairmen of the selectmen of the town or president or other officer of the water or ice company interested in such order. While the appeal is pending the order of the board shall be complied with, unless otherwise authorized by the board.

SEC. 120. *Enforcement of law.*—The supreme judicial court or the superior court shall have jurisdiction in equity, upon the application of the State board of health or of any party interested, to enforce its orders or the orders, rules, and regulations of said board of health, and to restrain the use or occupation of the premises or such portion thereof as said board may specify, on which said material is deposited or kept, or such other cause of pollution exists, until the orders, rules, and regulations of said board have been complied with.

SEC. 121. *Entry on premises.*—The agents and servants of said board may enter any building, structure, or premises for the purpose of ascertaining whether sources of pollution or danger to the water supply there exist, and whether the rules, regulations, and orders aforesaid are obeyed. Their compensation for services rendered in connection with proceedings under the provisions of section 118 shall be fixed by the board and shall in the first instance be paid by the Commonwealth; but the whole amount so paid shall, at the end of each year, be justly and equitably apportioned by the tax commissioner between such cities, towns, or companies as, during said year, have instituted said proceedings, and may be recovered in an action by the treasurer and receiver-general, with interest from date of the demand.

SEC. 122. *Penalties.*—Whoever violates any rule, regulation, or order made under the provisions of section 113 or section 118 shall be punished for each offence by a fine of not more than five hundred dollars, to the use of the Commonwealth, or by imprisonment for not more than one year, or by both such fine and imprisonment.

SEC. 123. *Application of preceding sections.*—The provisions of the eleven preceding sections shall not apply to the Merrimac or Connecticut rivers, nor to so much of the Concord River as lies within the limits of the city of Lowell, nor to springs, streams, ponds, or water courses over which the metropolitan water board has control.

SEC. 124. *Sources of water supply—as to.*—The provisions of the refuse, or polluting matter of such kind and amount as either by itself or in connection with other matter will corrupt or impair the quality of the water of any pond or stream used as a source of ice or water supply by a city, town, public institution, or water company for domestic use, or render it injurious to health, and no human excrement shall be discharged into any such stream or pond, or upon their banks if any filter basin so used is there situated, or into any feeders of such pond or stream within twenty miles above the point where such supply is taken.

SEC. 125. *Prescriptive rights unaffected—application limited.*—The provisions of the preceding section shall not destroy or impair rights acquired by legislative grant prior to the first day of July in the year 1878, or destroy or impair prescriptive rights of drainage or discharge, to the extent to which they lawfully existed on that date; nor shall it be applicable to the Merrimac or Connecticut rivers, or to so much of the Concord River as lies within the limits of the city of Lowell.

SEC. 126. *Injunction against pollution of water supply.*—The supreme judicial court or the superior court, upon application of the mayor of a city, the selectmen of a town, managing board or officer of a public institution, or a water or ice company interested, shall have jurisdiction in equity to enjoin the violation of the provisions of sec. 124.

SEC. 127. *Penalty for corrupting spring, etc.*—Whoever willfully and maliciously defiles or corrupts any spring or other source of water, or reservoir, or destroys or injures any pipe, conductor of water, or other property pertaining to an aqueduct, or aids or abets in any such trespass, shall be punished by a fine of not more than one thousand dollars or by imprisonment for not more than one year.

SEC. 128. *Penalty for corrupting sources of water supply.*—Whoever willfully deposits excrement or foul or decaying matter in water which is used for the purpose of domestic water supply, or upon the shore thereof within five rods of the water, shall be punished by a fine of not more than fifty dollars or by imprisonment for not more than thirty days; and a police officer or constable of a city or town in which such water is wholly or partly situated, acting within the limits of his city or town, and any executive officer or agent of a water board, board of water commissioners, public institution, or water company furnishing water or ice for domestic purposes, acting upon the premises of such board, institution, or company, and not more than five rods from the water, may, without a warrant, arrest any person found in the act of violating the provisions of this section and detain him until a complaint can be made against him therefor. But the provisions of this section shall not interfere with the sewage of a city, town, or public institution, or prevent the enriching of land for agricultural purposes by the owner or occupant thereof.

SEC. 129. *Penalty for bathing in public ponds.*—Whoever bathes in a pond, stream, or reservoir the water of which is used for the purpose of domestic water supply for a city or town, shall be punished by a fine of not more than ten dollars.

SEC. 130. *Penalty for driving on ice of pond used for water supply.*—Whoever, not being engaged in cutting or harvesting ice, or in hauling logs, wood, or lumber, drives any animal on the ice of a pond or stream which is used for the purpose of domestic water supply for a city or town, shall be punished by a fine of not more than fifty dollars or by imprisonment for not more than thirty days.

NOTE.—Sections 95 and 97, referred to in section 119, provide for an appeal to the superior court of the county and a jury trial. The verdict may alter, affirm, or annul the order, and shall be returned to the court for acceptance, and, if accepted, shall have the authority and effect of a valid order of the board.

MINNESOTA.

[General Statutes, p. 120.]

SEC. 430. *Pollution of sources of water supply forbidden.*—No sewage, drainage, or refuse, or polluting matter of such kind as, either by itself or in connection with other matter, will corrupt or impair the quality of the water of any spring, well, pond, lake, stream, or river for domestic use, or render it injurious to health, and no human or animal excrement shall be placed in or discharged into or placed or deposited upon the ice of any pond, lake, stream, or river used as a source of water supply by any town, village, or city; nor shall any such sewage, drainage, refuse, or polluting matter or excrement be placed upon the banks of any such pond, lake, stream, or river within five miles above the point where such supply is taken, or into any feeders or the banks thereof of any such pond, lake, stream, or river:

Provided, nothing in this section contained shall apply to Lake Superior. (1885. Chap. 225, sec. 1.)

SEC. 431. *Supervision of sources of water supply—procedure in cases of pollution.*—The State board of health shall have the general supervision of all springs, wells, ponds, lakes, streams, or rivers used by any town, village, or city as a source of water supply, with reference to their purity, together with the waters feeding the same, and shall examine the same from time to time, and inquire what, if any, pollution exists and their causes. In case of a violation of any of the provisions of section one of this act (sec. 430) said board may appoint a time and place for hearing parties to be affected, and shall give due notice thereof, as hereinafter provided, to such parties; and after such hearing, if in its judgment the public health requires it, may order any person or corporation, or municipal corporation, to desist from the acts causing such pollution, and may direct any such person or corporation to remedy the pollution, or to cleanse or purify the polluting substances in such a manner and to such a degree as shall be directed by said board, before being cast or allowed to flow into the waters thereby polluted, or placed or deposited upon the ice or banks of any of the bodies of water in the first section of this act mentioned. Upon the application of the proper officers of any town, village, or city, or of not less than ten legal voters of any such town, village, or city, to said State board, alleging the pollution of the water supply of any such town, village, or city by the violation of any of the provisions of this act, said State board shall investigate the alleged pollution, and shall appoint a time and place when and where it will hear and examine the matter, and shall give notice of such hearing and examination to the complainant, and also to the person or corporation or municipal corporation alleged to have caused such pollution, and such notice shall be served not less than ten days prior to the time so appointed, and shall be served in the same manner that now is or hereafter may be by law provided for the service of a summons in a civil action in the district court. Said board, if in its judgment any of the provisions of this act have been violated, shall issue the order or orders already mentioned in this section. (1885. Chap. 225, sec. 2.)

NEW HAMPSHIRE.

[Public statutes of New Hampshire and general laws in force Jan. 1, 1901, p. 337, chap. 108. (Wm. M. and Arthur H. Chase.)]

Section 13, entitled, "The prevention and removal of nuisances," is as follows: "If a person shall place, leave, or cause to be placed or left in or near a lake, pond, reservoir, or stream tributary thereto, from which the water supply for domestic purposes of a city, town, or village is taken, in whole or in part, any substance or fluid that may cause the water thereof to become impure or unfit for such purposes, he shall be fined not exceeding twenty dollars or be imprisoned not exceeding thirty days, or both."

SEC. 14. The board of health of the town or the water commissioners having charge of the water supply or the proprietors thereof may remove such substance or fluid, and they may recover the expense of removal from the person who placed the same or caused it to be placed in or near the water as aforesaid in an action on the case.

[Laws of 1895, chap. 76, p. 433.]

AN ACT to protect waters used for domestic purposes.

Be it enacted by the senate and house of representatives in general court convened:

SECTION 1. Whoever knowingly and wilfully poisons, defiles, pollutes, or in any way corrupts the waters or ice of any well, spring, brook, lake, pond, river, or reservoir used as the source of a public water or ice supply for domestic purposes, or knowingly corrupts the sources of the water of any water company or of any city or town supplying its inhabitants with water, or the tributaries of said sources of supply, in such a manner as to affect the purity of the water or ice so supplied at the point where the water or ice is taken for such domestic use, or puts the carcass of any dead animal or other offensive material into said waters or upon the ice thereof, shall be punished by a fine not exceeding one thousand dollars or by imprisonment not exceeding one year. The provisions of this section shall not apply to the deposit of any bark, sawdust, or any other waste of any kind arising from the business of cutting, hauling, driving, or storing logs, or the manufacture of lumber; and the use of any stream for the purposes of manufacturing and for the necessary drainage connected therewith, if more than four miles distant from the point where the water is taken for such domestic purposes, shall not be deemed a violation of this section.

SEC. 2. No person shall cut or take ice from any lake, pond, or reservoir used as the source of a public water or ice supply for domestic purposes for man, unless he first shall comply in all respects with such reasonable rules and regulations in regard to the manner and place of cutting and taking such ice on said lake, pond, or reservoir as may be prescribed by the local board of control or officers of a water company who may have charge of the works of any city or town supplying its inhabitants with water from said lake, pond, or reservoir. The supreme court shall have power to issue injunctions restraining any person from cutting or taking ice from such lakes, ponds, or reservoirs until they have complied with the reasonable regulations made as aforesaid.

SEC. 3. Said local boards and officers may also make all reasonable rules and regulations in regard to fishing and the use of boats in and upon any such lake, pond, or reservoir, and in regard to racing or speeding horses upon the ice thereof, which they may deem expedient. Any person who shall violate any of said rules and regulations after notice thereof shall be fined not exceeding twenty dollars, or imprisoned not exceeding six months.

SEC. 4. If any person shall bathe in such lake, pond, or reservoir within one-fourth mile of the point where said water is taken, he shall be fined not exceeding twenty dollars, or imprisoned not exceeding six months.

SEC. 5. Whoever shall wilfully injure any of the property of any water company or of any city or town, used by it in supplying water to its inhabitants, shall be punished by a fine not exceeding one thousand dollars, or by imprisonment not exceeding one year; and such person shall also forfeit and pay to such water company, city, or town three times the amount of actual damages sustained, to be recovered in an action on the case.

SEC. 6. All acts and parts of acts inconsistent with this act are hereby repealed, but nothing in this act shall be construed to repeal any special act applying to cities and towns.

[Laws of 1897, chap. 85, p. 82.]

SECTION 1. It shall be the duty of boards of health of the cities and towns of the State to examine and inspect the sources from which ice is cut, or is proposed to be cut, for domestic use in such cities and towns, and to employ such means as may be necessary to determine whether the waters of such sources of ice supply have been polluted, or whether ice taken therefrom will be deleterious to the public health.

SEC. 2. In each case where the waters of the sources of ice supplies shall be found so polluted that the ice taken therefrom will be unhealthy or unsafe for domestic use, the board of health of the city or town concerned in the same shall immediately notify such person or persons as may have taken, or who propose to take ice from such polluted source for their own domestic use or for sale for domestic use, of the dangerous character of the waters inspected and that the taking of such ice for domestic use must cease.

SEC. 3. Whoever knowingly or wilfully shall cut or take any ice for domestic purposes from any waters which are polluted with sewage or other substance deleterious or dangerous to life or health, or from waters which a board of health has condemned, shall be fined not exceeding two hundred and fifty dollars or imprisoned not exceeding six months.

[Acts of 1899, chap. 57.]

SECTION 1. Whenever any board of water commissioners, local board of health, or ten or more citizens of any town or city have reason to believe that a public water or ice supply is being contaminated or is in danger of contamination, and that the local regulations are not sufficient or effective to prevent such pollution, they may petition the State board of health to investigate the case and to establish such regulations as the said board may deem necessary for the protection of the said supply against any pollution that in its judgment would endanger the public health.

SEC. 2. The State board of health shall, after due investigation, make such regulations as it may deem best to protect the said supply against any dangerous contamination, and the regulations so made shall be in force when a copy is filed with the town clerk and posted in two or more public places in said town, or published in some newspaper in the county, and it shall be the duty of the local board of health to enforce said regulations.

SEC. 3. Any person violating any regulation established by the State board of health shall be punished by a fine of twenty dollars for each offense, and a certified copy under oath of such regulation, made by the secretary of the State board of health or by the town clerk where the regulations are filed, shall be received as prima facie evidence of such regulations in any court of the State.

[Laws of 1905, chap. 12.]

AN ACT to protect the waters of Alton Bay from pollution by sawdust and other waste.

SEC. 1. That no sawdust, shavings, or other waste product of sawmills, planing mills, or other manufactories shall be deposited, dumped, or placed in that part of Lake Winnipesaukee known as Alton Bay, nor shall any sawdust, shavings, or other waste products be allowed to escape into, or be deposited, dumped, or placed in any stream which runs or empties into said bay.

SEC. 2. Any person, or any officer of any corporation, violating the provisions of this act shall be fined not exceeding twenty-five dollars for each offense, and each day of a violation of the same shall be deemed a separate offense.

SEC. 3. All acts and parts of acts inconsistent with this act are hereby repealed.

SEC. 4. This act shall take effect on April 1, 1905.

Approved February 9, 1905.

[Laws of 1905, chap. 73.]

AN ACT to prohibit the deposit of sawdust and other sawmill refuse and other waste in Swift River and its tributaries in the town of Tamworth.

SEC. 1. Any person who shall deposit, dump, place, or cause to be deposited, dumped, or placed any sawdust or other sawmill refuse, rubbish, or other waste in Swift River and its tributaries, in the town of Tamworth, shall be fined not less than ten dollars nor more than fifty dollars.

SEC. 2. This act shall take effect upon its passage.

Approved March 9, 1905.

[Laws of 1905, chap. 74.]

AN ACT to protect Mink Brook from pollution by sawdust and other waste.

SEC. 1. No person or corporation shall put or place, or cause or allow to be put or placed, any sawdust, shavings, edgings, chips, bark, or other waste from woodwork establishments into Mink Brook in the town of Hanover.

SEC. 2. Any person or corporation violating the provisions of this act shall be punished by a fine not exceeding ten dollars for each offense, and every day that they violate the same shall be deemed a separate offense.

SEC. 3. This act shall take effect June 1, 1905.

Approved March 9, 1905.

[Laws of 1905, chap. 88.]

AN ACT to protect Union River and its tributaries from pollution by sawdust and other waste.

SEC. 1. No person or corporation shall put or place, or cause to be put or placed, any sawdust, shavings, edgings, chips, bark, or other waste from sawmills or other woodwork establishments into Union River, so called, or its tributaries, in the towns of Brookfield and Wakefield in Carroll County, and the town of Milton in Strafford County. Any person or corporation violating the provisions of this act shall be punished by a fine not exceeding one hundred dollars for each offense.

SEC. 2. This act shall take effect on April 15, 1905.

Approved March 10, 1905.

NEW JERSEY.

[General Statutes, p. 1109.]

AN ACT to prevent the pollution of the waters of any of the creeks, ponds, or brooks of this State. (Approved March 29, 1878.)

322. SECTION 1. That if any person or persons shall throw, cause or permit to be thrown, into the waters of any creek, pond, or brook of

this State, the waters of which may be used for the cutting or harvesting of ice, any carcasses of any dead animal or any offal or offensive matter whatsoever, calculated to render said waters impure or create noxious or offensive smells, or shall connect any water-closet with any sewer or other means whereby the contents thereof may be conveyed to and into any such creek, pond, or brook, shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding one hundred dollars, or imprisonment not exceeding thirty days, or both.

[2 General Statutes, p. 2215.]

AN ACT to enable towns and townships in this State to construct waterworks for the extinguishment of fires and supplying the inhabitants thereof with pure and wholesome water. (Approved March 9, 1893.)

419. SEC. 13. That if any person or persons shall willfully pollute or adulterate the waters in any reservoir erected under the provisions of this act, any person so offending shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding five hundred dollars, or by imprisonment at hard labor not exceeding three years, or both, at the discretion of the court before whom such conviction shall be had.

[General Statutes, p. 1107.]

Supplement to an act to prevent the willful pollution of waters of any of the creeks, ponds, or brooks of this State. (Approved February 27, 1880.)

SECTION 1. (As amended by act passed March 14, 1893. General Statutes, p. 1107, sec. 311.) That if any person or persons shall throw, cause or permit to be thrown into any reservoir, or into the waters of any creek, pond, or brook of this State which runs through or along the border of any city, town, or borough of this State, or the waters of which are used to supply any aqueduct or reservoir for distribution for public use, any carcass of any dead animal, or any offal or offensive matter whatsoever calculated to render said waters impure, or to create noxious or offensive smells, or shall connect any water-closet with any sewer, or other means whereby the contents thereof may be conveyed to and into any such creek, pond, or brook, or shall so deposit or cause or permit to be deposited any such carcass, offal, or other offensive matter that the washing or waste therefrom shall or may be conveyed to and into any such creek, pond, brook, or reservoir, such person or persons shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding one thousand dollars, or by imprisonment not exceeding two years, or both.

309. SEC. 2. That it shall be the duty of the owner or owners, occupant or occupants of any land whereon any such carcass, offal, or other offensive matter may be to cause the same to be buried forth-

with, so that all portions thereof shall be covered with solid earth to a depth of at least two feet below the surface of the ground, and not within a distance of two hundred feet from such creek, pond, or brook used as aforesaid; and any such owner or occupant who shall refuse or neglect for the space of two days to remove and bury as aforesaid, or cause to be removed and buried, any such carcass, offal, or offensive matter shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding one thousand dollars, or by imprisonment not exceeding two years, or both.

[Laws of 1898, chap. 136, p. 233.]

AN ACT authorizing the appointment of commissioners to consider the subject of the pollution of rivers and streams within this State, to provide a plan for the prevention thereof, and for the relief of the persons and property affected thereby, and to provide for the expenses necessary for that purpose.

Be it enacted by the senate and general assembly of the State of New Jersey:

1. The governor of this State shall have power and authority to appoint and commission not less than three suitable persons commissioners to consider the subject of the pollution of any stream or river within this State, whose duty it shall be, after having duly investigated the cause, character, and extent of such pollution, if they shall deem it necessary and expedient, to prepare and perfect a plan for the prevention thereof and for the relief of the persons and property affected thereby, and to report their conclusions and present their plan to the legislature of this State, together with a bill providing therefor and for the expenses thereof.

2. Such commissioners, when so appointed, shall organize by the selection of one of their number as chairman and one to act as treasurer, and they are authorized to select a clerk and to employ such other agents and assistants as may be necessary. The salary and compensation of such commissioners shall be fixed by the governor, and shall not exceed one thousand dollars each, and they shall have power and authority to fix the compensation of their agents and assistants.

3. Such commissioners are authorized to raise and expend for the purposes of this act a sum not exceeding twenty-five thousand dollars, which sum, or such part thereof as may be required and be necessary, they are hereby authorized to apportion among the several local municipalities which the said commissioners shall deem to be affected by such pollution, in proportion to the population of such municipalities as shown by the last State or National census, and the sum or sums so apportioned shall be certified by the said commissioners under their hands to the assessors or other taxing officers of the said several municipalities, and it shall be the duty of the proper taxing

officer or officers in each of the said municipalities to whom such apportionment is made to proceed to have the same levied and assessed and collected in the same manner and at the same time as other taxes are levied and collected therein, and it shall be the duty of the collector or other equivalent officer of each of the said municipalities to pay over the said several sums of money, when so levied, assessed, and collected, to the said commissioners or to such person or persons as they may appoint to receive the same, and the said commissioners are authorized to use and disburse the same for the purposes of this act.

4. The commissioners appointed under the authority of this act shall have the power and authority to anticipate the collection and receipt of the sums of money hereby authorized to be raised by taxation, and may issue from time to time certificates of indebtedness, or other obligations, to be paid from the funds to be raised by taxation in the manner herein provided; and they are authorized to use the funds received from the sale or negotiation of such certificates or obligations authorized to be issued by this act.

5. Said commissioners are hereby required, at any time, on the order of the governor, to render to him a report and statement of their receipts and expenditures under the authority of this act.

6. Vacancies caused by the death or resignation of any commissioner appointed under the authority of this act, or from other cause, shall be filled by the governor, and the governor may remove any of the persons so appointed and appoint another commissioner in his place.

7. This act shall take effect immediately.

Approved April 2, 1898.

[Laws of 1899, chap. 41, p. 73.]

AN ACT to secure the purity of the public supplies of potable waters in this State.

Be it enacted by the senate and general assembly of the State of New Jersey:

1. No sewage, drainage, domestic or factory refuse, excremental or other polluting matter of any kind whatsoever which, either by itself or in connection with other matter, will corrupt or impair, or tend to corrupt or impair, the quality of the water of any river, brook, stream, or any tributary or branch thereof, or of any lake, pond, well, spring, or other reservoir from which is taken, or may be taken, any public supply of water for domestic use in any city, town, borough, township, or other municipality of this State, or which will render, or tend to render, such water injurious to health, shall be placed in, or discharged into, the waters, or placed or deposited upon the ice, of

any such river, brook, stream, or any tributary or branch thereof, or of any lake, pond, well, spring, or other reservoir above the point from which any city, town, borough, township, or other municipality shall or may obtain its supply of water for domestic use, nor shall any such sewage, drainage, domestic or factory refuse, excremental or other polluting matter be placed or suffered to remain upon the banks of any such river, brook, stream, or of any tributary or branch thereof, or of any lake, pond, well, spring, or other reservoir above the point from which any city, town, borough, township, or other municipality shall or may obtain its supply of water for domestic use as aforesaid; and any person or persons, or private or public corporation, which shall offend against any of the provisions of this section shall be liable to a penalty of one hundred dollars for each offense: and each week's continuance, after notice by the State or local board of health to abate or remove the same, shall constitute a separate offense: *Provided, however*, That this section shall not be held to apply to any city, town, borough, township, or other municipality of this State which, at the date of the passage of this act, has a public sewer or system of sewers, drain or system of drains, legally constructed under municipal or township authority, discharging its drainage or sewage into any such river, brook, stream, lake, pond, well, spring, or other reservoir: *And provided further*, That nothing in this section contained shall be construed to repeal, modify, or otherwise affect any law or statute now conferring upon any local board of health the power or authority to institute any proceedings in any court of this State for the recovery of any penalty for, or obtaining any injunction against, the pollution of any of the waters of this State.

2. Any penalty incurred under any of the provisions of the first section of this act may be recovered, with costs, in a summary proceeding, either in the name of the board of health of the State of New Jersey or in the name of the local board of health of the township, city, borough, town, or other local municipal government within whose jurisdiction the penalty may have been incurred; it shall be the duty of any health inspector, or member of any local board of health, who shall know or be informed of any violation of any of the provisions of the first section of this act whereby any penalty may have been incurred, to make, and any other person having such knowledge may make, under oath or affirmation, a complaint against the person or persons or private or public corporation incurring such penalty, setting forth the facts of such violation, which complaint shall be filed in the office of the clerk of the district court, or with any justice of the peace of the county within which the offense may have been committed, or with any police justice or recorder of the township, city, or other municipality within which any local board bring-

ing suit shall have jurisdiction; and the district court, justice of the peace, police justice, or recorder, with whom any complaint shall be filed as aforesaid, setting forth facts sufficient to show that the penalty prescribed by the first section of this act has been incurred, is hereby authorized and required to issue process either in the nature of a summons or warrant, which process, when in the nature of a warrant, shall be returnable forthwith, and when in the nature of a summons shall be returnable in not less than five nor more than fifteen days. On the return of such process, or at any time to which the trial shall have been adjourned, the said court, justice of the peace, police justice, or recorder shall proceed to hear the testimony of witnesses and the proofs in the case, and to determine and give judgment in the matter without the filing of any pleadings, and, if judgment shall be given in favor of the plaintiff, execution shall forthwith issue against the goods and chattels of the defendant for the amount of the penalty, with costs; and all judgments so rendered shall have the same force and effect as other judgments in civil actions before civil courts and officers, and may be docketed in like manner in the office of the clerk of the court of common pleas; the officers to serve and execute any process or execution issued as aforesaid shall be the constables of the counties, which service and execution, in the case of any execution issued out of the district court, shall be made in the same manner and under the same liabilities as other executions issued out of said court are served and executed; the officers to serve and execute any process or execution issued by a justice of the peace, police justice, or recorder, shall be the constables of the county, which service and execution shall be made in the same manner and under the same liabilities as prescribed in cases of the service and execution of processes and executions by the act entitled "An act constituting courts for the trial of small causes," and the supplements thereto; all moneys recovered in any such proceeding shall be paid to the plaintiff therein and applied by such plaintiff to any purpose for which it may be legally authorized to expend money.

3. The State board of health shall have the general supervision, with reference to their purity, of all rivers, brooks, streams, lakes, ponds, wells, springs, or other reservoirs in this State the waters of which are or may be used as the source or sources of public water supplies for domestic use, together with the waters feeding the same, and shall have the authority from time to time, as they deem necessary or proper, to examine the same and to inquire what, if any, pollutions exist and their causes; and the said State board of health, in carrying out the provisions of this section, may from time to time, as they deem it necessary or proper, address inquiries in printed or written form to any local board of health, municipal or township authority, corporation, or person or persons, which inquiries it shall

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luties of their office, shall make and subscribe an oath (before some person authorized by the laws of this inister the same) to truly, faithfully, and impartially discharge the duties of their office according to law and with the secretary of state. The terms of office of the said commission (except those appointed by the governor cies as aforesaid) shall commence on the first Monday of acceeding their appointment by the governor and confir- he senate. On the first Monday of May next succeeding l appointment of said commission the members thereof at the statehouse in the city of Trenton and organize by n of one of their number to be chairman of said commis- ne to be treasurer thereof, which officers shall hold office at re of the commission. After having so met and organized t meetings of the commission shall be held at such times s as the commission may direct or as it may be called to the chairman.

l commission shall keep a record of all its proceedings and ons, also full and accurate account of its receipts, disburse- xpenditures, assets, and liabilities, and shall annually report gislature its operations, proceedings, and transactions for the ng year, with a statement or abstract of such receipts, dis- nts, expenditures, assets, and liabilities.

he members of said commission shall each receive an annual of one thousand dollars, to be paid as other salaries of State are paid. Said commission may have a secretary (not a er of the commission), to be appointed by the commission or a ity thereof, who shall hold his office at the pleasure of the com- on or a majority thereof, and receive such salary as the commis- or a majority thereof, with the approval of the governor, may said commission or a majority thereof may also from time to employ or appoint such experts, engineers, officers, agents, loyes, workmen, and servants as it may deem necessary or proper enable it to perform its duties and carry out the objects and pur- ses of this act; and said commission or a majority thereof may and determine the duties and compensation of said experts, engi- ers, officers, agents, employes, workmen, and servants, and remove r discharge the same or any of them at pleasure.

4. It sha[illegible] f the secretary to keep a record of all the proceedin[illegible] s of the commission, to prepare the annual report t[illegible] d perform such other duties as the com- mission [illegible] iall be the duty of the treasurer to take charge [illegible] ved by the commission, to keep accurate accour[illegible] disbursement thereof, and to deposit and

be the duty of the persons or parties addressed to answer within such time as the said State board of health may in such inquiries prescribe.

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5. It shall be the duty of said commission to investigate the various methods of sewage disposal, either in this country or elsewhere, in order that they may be able to make proper recommendations in regard thereto. They shall investigate all complaints of pollution of the waters of this State which shall be brought to their notice, and shall advise as to the best methods of sewage disposal in order to prevent such pollution.

6. It shall be unlawful for any person, corporation, or municipality to build any sewer or drain or sewerage system from which it is designed that any sewage or other harmful and deleterious matter, solid or liquid, shall flow into any of the waters of this State so as to pollute or render impure said waters, except under such conditions as shall be approved by the State sewerage commission: *Provided*, That the provisions of this section shall not be deemed to prohibit the use or extension of existing sewers, drains, or sewerage systems.

7. It shall be unlawful for any person, corporation, or municipality to build or cause to be built any plant for the treatment of sewage or other polluting substance from which the effluent is to flow into any of the waters of this State, except under such conditions as shall be approved by the State sewerage commission, to whom the plans shall be submitted before building.

8. On or before the first day of January, one thousand nine hundred, and thereafter whenever required by said commission, the mayor of every municipality and the chairman of every township committee of every township now having, using, owning, leasing, or controlling a sewerage plant or system shall furnish to said commission, on blanks to be provided by said commission, a statement showing the disposition made of the sewage of their respective municipalities or townships, and, as near as possible, the amount discharged each twenty-four hours, and such other information and data as may be called for by said blanks to be provided as aforesaid by said commission.

9. The words " waters of this State," as used in this act, shall be held and construed to mean and include any and all waters of any pond, lake, creek, inlet, bay, estuary, river, or stream of this State.

10. To enable said commission to carry out and enforce the provisions of this act, the said commission may expend a sum not exceeding five thousand dollars, when duly appropriated.

11. And whereas, in order to prevent the pollution of the waters of this State, it is deemed necessary to establish a proper system or sys-

tems of sewerage and drainage wherein may or may not be included a system or systems of sewage-disposal works for the scientific treatment and proper disposal of sewage and sewage matter and the effluent thereof, and the establishment of any such system or systems may render proper or necessary the formation or creation of sewerage districts embracing portions or the whole of the territory of two or more of the municipalities of this State, within which districts such system may be constructed, maintained, and operated, and such municipalities may be unable, through lack of power and authority or otherwise, to agree upon the establishment of any such system or systems or upon the extent or limits of the territory of their respective municipalities to be included in any such district or districts and devoted to the uses and purposes of any such system or systems as aforesaid; therefore upon presentation to said the State sewerage commission of a petition in writing, setting forth that in order to prevent the pollution of the waters of this State, or any of them, it is proper or necessary that portions or the whole of the territory of two or more of the municipalities of this State should be erected into a sewerage district for the construction, maintaining, and operation within such district of a system of sewerage and drainage or a system of sewage-disposal works, or of both such systems, and naming each municipality, the whole or any portion of the territory whereof it is proposed shall be included in such district, and stating generally the boundaries and outlines of such proposed district with sufficient exactness to show approximately the quantity or extent of territory of each municipality to be embraced in such proposed district, and requesting said commission to create and establish such district for either or both of the purposes aforesaid; and if said petition be signed by the mayors or other chief executive officers of all of the municipalities named in said petition, any of whose territory is proposed to be included in said district, said signatures being respectively affixed to said petition by authority or direction of the respective governing bodies of such municipalities (full power and authority to authorize and direct the signing of any such petition being hereby conferred upon and vested in all such governing bodies), and the signing of said petition by such authority or direction being made to appear by affidavit or other due proof thereof, it shall be lawful for said the State sewerage commission to appoint a time and place when and where it will attend and give public hearing of the matters contained in said petition to all persons and parties interested therein; said commission shall cause at least twenty days' notice to be given of the time and place of any such hearing by publishing the same in the newspaper or newspapers, if any, published within said proposed district, and if none be published therein, then in a newspaper or newspapers published in the neighborhood of said proposed district and circulating therein; said notice may also, at the discretion of said commission, be published in a newspaper or newspapers published outside of said proposed district, whether or not any paper or papers be published within the same; said commission shall also, at least ten days prior to the day fixed for such hearing, cause notice of the time and place thereof to be mailed to or served upon the mayor or other chief executive officer of any and all municipalities named in said petition, any territory whereof is included in said proposed district; and said commission may, if it deem proper so to do, require a copy of said petition to be mailed to or served upon such mayors or other chief executive officers such number of days prior to said hearing as it may direct; said hearing may be adjourned from time to time as said commission may decide; the sessions of said commission on said hearing, or any adjournment thereof, when sitting for the taking of testimony or hearing argument of counsel, shall be open and public, and witnesses may be examined under oath or affirmation, which any member of said commission or the secretary thereof is hereby authorized and empowered to administer; the secretary of said commission shall attend at all such hearings and keep minutes of the proceedings thereat; said commission may, if it deem proper so to do, employ a stenographer to take and transcribe the testimony produced before it at any such hearing; and said commission may require the persons or parties presenting to it any such petition as aforesaid to pay in advance or assume or guarantee to pay all or such part of the costs, charges, and expenses to be made or incurred by reason of the filing of said petition and subsequent proceedings to be had thereupon or thereunder, as said commission may think proper.

12. If, after such hearing, said commission, or a majority thereof, shall deem it advisable to comply with the request of said petition, and that a district for the purpose or purposes, or either of them therein stated, should be created and established, said commission shall adopt a resolution to that effect, defining the limits and boundaries of such district with certainty and declaring the territory included within such limits and boundaries to be a sewerage district, within which a system of sewerage and drainage, or a system of sewage-disposal works, or both, may be constructed, maintained, and operated under the provisions of this act; the said districts shall be called and known as "sewerage districts," and the boards to construct, maintain, and operate the system or systems of sewerage or sewage-disposal works within such districts shall be called and known as "sewerage boards;" in and by said resolution, said commission shall assign to the district therein and thereby established a name and number, thus, "Sewerage district number ——," and shall also specify the name by which the board thereafter to be elected in such

district shall be called and designated, thus, "Sewerage board of district number ——," the number of any such district and that of the sewerage board therein to be always the same. The first sewerage district created and established under this act shall be "Sewerage district number one," the second number two, and so on in regular order as the same may be respectively created. Said commission shall also cause a map to be prepared of said district so created and established, whereon and whereby shall be shown with accuracy the limits and boundaries of such district, of what municipalities the lands included in said district form a part, and what extent or quantity of territory of each municipality (whether the whole or a portion thereof) is included in said district. The original of said map shall be filed with said commission, and within ten days after the adoption of said resolution a copy thereof and of said map shall be filed in the office of the secretary of state and in the clerk's office of each county in which any of the lands included in said district may be situate; and from and after the filing of such resolution and maps as aforesaid the territory included in said district as stated and shown in and by said resolution and map shall be deemed to be and constitute a sewerage district by the name and number and for the purposes stated in said resolution.

13. The members of the several sewerage boards shall consist of two members from each municipality, in whole or part, within the sewerage district, to be appointed by the governing body of each of such municipalities, and one member to be appointed by the State sewerage commission, all of whom shall be residents of the district; provided that in case more than three municipalities shall be included in whole or part in any sewerage district there shall be but one member from each municipality in addition to the number appointed by the State sewerage commission.

14. The members of any district sewerage board first appointed shall meet at such time and place as the State sewerage commission shall designate; each member of said board (and all members thereof afterwards appointed thereto) shall take and subscribe an oath or affirmation, before some person authorized to administer the same, to faithfully and truly perform his duty as member of such board to the best of his ability, and within two days after making thereof forward the same to the secretary of state; said board when met as aforesaid (the members thereof having each made and subscribed said oath or affirmation) shall organize by the election of one of their number as chairman, one as secretary, and one as treasurer; the members of said board shall serve for the term of three years each, and the terms of such members shall commence on the date of their first meeting as

designated by the State sewerage commission; the chairman, secretary, and treasurer of said board shall, respectively, serve for the period of one year and until their successors are elected; a certificate or statement of such meeting and organization of said board shall, on the day of such meeting, be prepared and mailed to the secretary of state, to be filed in his office; meetings of said board subsequent to such first meeting for organization shall be held at such times and places as the board may decide or as it may be called to meet by the chairman.

15. From and after such meeting and organization of said board and the filing of such certificate as aforesaid, said board shall be deemed to be and shall be a body politic and corporate, under the same name and title as that designated and specified in the resolution of the State sewerage commission creating and defining the said sewerage district, to wit, "Sewerage board of district number ——," and by such name and title said sewerage board shall have perpetual succession, with power to sue and be sued, and the right, power, and authority to acquire, hold, use, and dispose of all such property, real or personal, as may be proper or necessary for the objects, uses, and purposes for which said sewerage board was created, and with all other powers necessary or incident to bodies politic and corporate or that may be necessary or proper to carry out and effectuate the objects and purposes of this act and the objects and purposes for which said sewerage board was created.

16. Any such board incorporated as aforesaid shall have full power and authority within its respective district, under the supervision, direction, and control of the State sewerage commission as hereinbefore or hereinafter provided, to construct, maintain, and operate in said district a system of sewerage and drainage, or of sewage-disposal works, or both, with the necessary pipes, drains, conduits, fixtures, pumping works, and other appliances for the purpose of taking up sewage and all other offensive and deleterious matter and convey the same to some proper place or places of deposit or disposal to be selected by the said board, there to be deposited, treated, disinfected, or disposed of as to the said board may seem proper and as may be deemed most advantageous; and it shall be the duty of all persons and all corporate bodies and municipalities owning or controlling sewers or drains or having charge thereof within the limits of the district wherein intercepting or main sewers have been or may be constructed by the said board as herein provided, to cause the same to be connected therewith; and it shall be the duty of said board in constructing such main or intercepting sewers to have them so constructed that such connection can be made therewith at all necessary and proper points and places; all such connections shall be made in

accordance with the rules and regulations from time to time adopted by the said board in relation thereto, and under the direction and supervision of its officers and agents.

17. The said board shall have power and authority to purchase and acquire all lands, rights, or interest in lands which may be deemed necessary for the construction of sewers, drains, disposal pumping, and other works authorized by this act; and if in any case the said board shall be unable to agree with the owner or owners of any lands, rights, or interests in lands deemed necessary by the said board in the construction of the works herein authorized, or when, by reason of the legal incapacity or absence of such owner or owners, no agreement can be made for the purchase thereof, the lands or rights in lands so desired shall be acquired in the manner provided by the general laws of this State relating to the condemnation of lands for public use.

18. Before determining upon the final plan or route for the building or construction of any work authorized by this act, the said board may, by its officers, agents, servants, and employés, enter at all times upon any lands or waters for the purpose of exploring, surveying, leveling, and laying out the route of any drain or sewer, locating any disposal, pumping, or other works, establishing grades and doing all necessary preliminary work, doing, however, no unnecessary damage or injury to private or other property.

19. The said board shall have power and authority to construct any sewer or drain, by it to be made or constructed under or over any water course, under, over, or across, or along any street, turnpike, road, railroad, highway or other way, and in or upon private or public lands under water, in such way and manner, however, as not unnecessarily to obstruct or impede travel or navigation, and may enter upon and dig up any road, street, highway, or private or public land, for the purpose of laying down sewers and drains upon or beneath the surface thereof, and for maintaining and repairing the same, and in general may do all other acts and things necessary, convenient, and proper for the purposes of this act; and whenever the said board shall dig up any road, street, or way, as aforesaid, it shall, as far as practicable, restore the same to as good condition and order as the same was when such digging commenced.

20. The said board shall have power and authority also to alter or change the course or direction of any water course, and, with the consent of the board or body having control of the streets and highways in any city, town, or municipality, to alter or change the location or grade of any highway, public street, or way crossed by any sewer or drain constructed or to be constructed under the provisions of this act, or in which such sewers or drains may be located.

21. The said board shall at all times keep full and accurate accounts of its receipts, expenditures, disbursements, assets, and liabilities, and shall annually make a report of its operations and doings, in which report it shall include an abstract of such receipts, expenditures, disbursements, assets, and liabilities, and publish the same in one or more newspapers, published in each of the counties in said district.

22. To provide for the payment of the costs and expenses incurred or to be incurred by the said board in making the constructions and executing the work and performing the duties imposed upon it by this act, it shall have power and authority from time to time to issue bonds in its corporate name, not to exceed in amount such costs and expenses, and not to exceed that part of such cost and expense incurred in the work of constructing sewers, drains, disposal, and other works, including the cost of lands, rights and interests in lands, of which a separate account is to be kept by said board as hereinafter provided; such bonds shall be of the form and payable at such time, not exceeding thirty years from the date thereof, and at such place, either in currency or coin, as the said board may determine; they shall bear interest at a rate not exceeding five per centum per annum; in issuing such bonds the said board may, in its discretion, make the same or any part thereof, fall due at stated periods less than thirty years, or may reserve therein an option to redeem and pay the same or any part thereof at stated periods at any time between the date thereof and the date at which they would otherwise fall due, and all such bonds may be negotiated, sold, or disposed of at not less than their par value, and the same or the proceeds thereof may be used by the said board for the purpose aforesaid.

23. The said board shall keep the costs and expenses of the construction of sewers, drains, disposal and other works, in which shall be included the cost of lands, rights, and interests in lands, separate from the costs and expenses of maintenance, operation, and repairs, and shall, after having prepared and adopted plans (which, however, the board or the State sewerage commission shall have the power to change or modify, if such change or modification shall be found necessary or desirable), make a careful estimate of the cost and expense of such construction, and shall divide and apportion the same, according to their best judgment, to and between the several municipalities or parts thereof (if any) included within such sewerage districts ratably and proportionally to the benefits received or to be received by such municipalities or parts thereof from such construction, and shall furnish to the governing body of each and every municipality the whole or any part whereof is included in such sewerage district, a statement of such estimated cost and expense and of the division and

apportionment thereof as aforesaid, and service of said statement upon the mayor or other chief executive officer or upon the clerk of any such municipality shall be deemed to be a service upon the municipality; if the governing body of any such municipality (whether a whole or only a part thereof is included in such sewerage district) shall be dissatisfied with such division and apportionment and shall within twenty days after service thereof as aforesaid express such dissatisfaction by a resolution adopted by a majority of such body, then it shall be lawful for such body, in the corporate name of such municipality, to make application to any justice of the supreme court of this State for the appointment of three disinterested persons, residents of this State, commission to review such division and apportionment, and correct, amend, revise, alter, or confirm the same, as they or a majority of them shall deem just and proper, and it shall be the duty of said justice to make such appointment; the commissioners so appointed (having respectively taken and subscribed an oath or affirmation before some person authorized to administer the same faithfully and impartially to perform the duties imposed upon them by [a]), shall forthwith, at such time and place as they or a majority of them may appoint, and upon such notice as the said justice in the order appointing said commissioners shall direct to be given, hear the parties interested in said matter and such proofs and witnesses as may be produced before them; said commissioners may adjourn said hearing from time to time as occasion may require; on any such hearing the parties, if they so choose, may be represented by counsel, and the witnesses may be examined under oath or affirmation, which any of said commissioners are hereby authorized to administer; said commissioners may designate one of their number to act as chairman and one to act as clerk or secretary; at the conclusion of such hearing, and within ten days thereafter, said commissioners, or a majority of them, shall correct, amend, revise, alter, or confirm such division and apportionment as they or a majority of them shall deem just and proper under the evidence and proofs produced before them and shall make and sign a statement or certificate thereof, which statement or certificate shall be final and conclusive and binding upon all parties; the application for the appointment of such commissioners, the order of the justice appointing them, the oath or affirmation of said commissioners, and their said statement or certificate shall, within two days after the making of such statement or certificate, be filed with the secretary of the sewerage board which made the division or apportionment reviewed by said commissioners; and such sewerage board, within five days after the filing of such statement or certificate as aforesaid, shall cause a certified copy thereof to be served in manner

[a] So in original.

aforesaid upon each of the municipalities that the original division and apportionment made by said sewerage board was served upon, which certified copy so served shall be in lieu and stead of that originally served and (as aforesaid) be final and conclusive and binding upon all parties; if, in any case, where only a part of a municipality is included in a sewerage district, the governing body of such municipality shall not within said twenty days after service upon it of any such original statement as aforesaid adopt a resolution expressing its dissatisfaction as aforesaid provided, then, and in every such case, it shall and may be lawful for one or more of the residents and taxpayers, or residents and nonresident taxpayers of said sewerage district, to join in such application as aforesaid to any justice of the supreme court for the appointment of commissioners to review, as aforesaid, the said division and apportionment, and thereupon the said justice may, in his discretion, appoint such commissioners, and if such appointment be made, said commissioners shall proceed in the same manner, and the proceedings before them had shall be similar to those hereinbefore provided, and the statement or certificate of said commissioners made upon any such last-mentioned application shall be final and conclusive and binding upon all parties.

24. The said sewerage board shall also, in the manner hereinbefore directed, serve upon or furnish to each of said municipalities after every issue and sale of bonds a statement of the amount of such bonds and the date of interest thereon and the proportion thereof allotted to each municipality (where such municipality is entirely within the sewerage district) or (where only a part of the municipality is included in the sewerage district) of the proportion of such division and apportionment allotted to the part of the municipality in said sewerage district; and it shall be the duty of each of said municipalities, and of its proper officers, in the next annual tax levy made in such municipality and in each succeeding year thereafter to include and raise by taxation the amount required to pay the interest on the proportion of such bonded indebtedness allotted to such municipality or part thereof, as the case may be, and if such municipality be entirely within such sewerage district then it shall be the duty of such municipalities to cause to be levied and assessed therein a sum equal to the amount of interest so apportioned and allotted to such municipality together with such additional sum, to be divided and apportioned and allotted to and between said municipalities or parts thereof as aforesaid as may be necessary to establish and maintain a sinking fund sufficient to pay the principal of the bonds issued by the said sewerage board under authority of this act when the same fall due. If only a part of the municipality be included in the sewerage district, then it shall be the duty of such municipality and its

proper officers, instead of levying and assessing the same upon the whole municipality, to cause, in manner aforesaid, the sum or sums that may, as aforesaid, be apportioned and allotted to such part of the municipality as is included in the sewerage district to be levied and assessed in and upon such part of the municipality as is included in the sewerage district, in the same manner as other taxes may be levied and assessed therein; and it shall be the duty of all taxing officers and all collecting officers in the said municipalities to levy, assess, and collect the said amount or sums so to be raised in such municipalities or parts thereof, as the case may be; and it shall also be the duty of the collector of taxes in each of the said municipalities, or other proper officer, to pay to the sewerage board thereunto entitled the money so levied, assessed, and collected. After each census, State or national, a new allotment shall be made of the sinking fund or redemption fund in the manner herein provided.

25. As soon as the work of construction by this act authorized (or the cost and expense of which a preliminary estimate shall have been made as herein provided) has been completed the said board shall proceed at once to ascertain the actual cost and expense of such work, and shall furnish to each of the said municipalities or municipal divisions a statement of such cost and expense.

26. The cost of maintenance, operation, and repairs, together with the cost of supervision, and all other expenses of every kind not included in the cost and expense of construction, shall be annually estimated by the said board and divided and apportioned between the said several municipalities or parts thereof upon the same basis as herein provided for the division of the cost and expense of construction; and that the same, when so divided and apportioned, shall be levied, assessed, collected, and paid annually in the same manner provided for the levying, assessment, and collection of the cost and expense of construction: *Provided, however*, That if at the end of any year when such cost and expense shall have been accurately ascertained such estimate shall be found to have been more or less than the proper proportion of any such municipality, then the surplus or deficiency, as the case may be, shall be deducted from or added to the sum to be levied, assessed, and collected for the succeeding year.

27. The said board shall, immediately after receiving from the said municipalities, or either of them, or from the collector or treasurer of any such municipality, any moneys on account of the apportionment made, as hereinbefore provided, or as soon thereafter as practicable, cause the same to be invested in securities, the character of which shall be the same as required by law for savings banks of this State, except so much thereof as may be required to pay interest due or to fall due during the current year; and all such funds, and the

securities in which the same or any part thereof shall be invested. and the interest received therefrom, shall be held, used, and applied by the said board as a sinking fund to meet and pay the interest and principal on the bonds issued by the said board under the authority of this act, and for no other purpose whatever, until all such bonds and all arrears of interest thereon are fully paid. It shall be the duty of said sewerage board to include in its annual report the amount of money received by it for the purposes aforesaid, the sources from which such money was received, and the investment of the same; and the said board shall keep a record and account of all bonds issued by it, when the same fall due, the time and place of payment, and the rate of interest thereon, and of the amount received on the sale or disposition thereof, and shall also keep an account of all moneys invested, held, and used as a sinking fund, and of the securities in which the same may be invested. The books, records, accounts, papers, and documents of the said board shall be open to the inspection of any person appointed by the governing body of any municipality within said district to inspect the same: *Provided, however*, That in case the said board shall issue bonds which shall fall due and become payable at stated periods less than thirty years, or shall retain in any such issue the option to redeem bonds prior to the date at which they would otherwise fall due as hereinbefore provided, then it shall be lawful for the said board to make application of the moneys received by it from the several municipalities and of the funds temporarily invested by the said board so received for the purpose of paying off and discharging the said obligations according to their tenor and effect.

28. During the year preceding the year in which the bonds issued under the authority of this act shall fall due the said board shall cause a careful computation to be made of the moneys that will be available for the payment of the same, and if it shall be found that any deficiency will exist in the fund that will be available therefor, after the application of moneys received and the use of all securities held, such deficiency shall be apportioned and allotted to the said municipalities in the same manner and upon the same basis as the original apportionment, and shall be added to the amount so levied, assessed, collected, and paid by the said municipalities, respectively, in the succeeding year; and if any excess shall be found to exist in such fund the surplus shall be credited to each of the said municipalities in the same proportion and deducted from future estimates of the respective shares or proportions of such municipalities of the cost and expense of maintenance, operation, and repairs.

29. In and about the performance and discharge of the duties imposed upon it by this act any such sewerage board as aforesaid,

or a majority thereof, may employ such experts, engineers, contractors, officers, agents, employes, clerks, workmen, and servants as it may deem necessary or proper to enable it to perform its duties and carry out the objects and purposes of this act; and said board, or a majority thereof, may fix and determine the duties and compensation of such experts, engineers, contractors, officers, agents, employees, clerks, workmen, and servants, and remove or discharge the same, or any of them, at pleasure.

30. The secretary of any such sewerage board shall keep a record of all the proceedings and transactions of said board; under the direction of said board he shall prepare the estimate, division, and apportionment provided for in section twenty-six hereof; he shall prepare the annual report of said board and perform such other duties as the board may from time to time require. The secretary shall receive an annual salary, to be fixed by the board, or a majority thereof, but he shall not receive any per diem allowance.

31. The treasurer of any such sewerage board shall have charge and custody of all moneys and securities received or owned or held by said board; he shall keep accurate record and account of the receipt, disbursement, and disposition of all such moneys and securities, and invest, deposit, dispose of, disburse, and pay out the same at such times and in such manner as the board may direct, and under such rules and regulations as it may from time to time establish. The treasurer shall give bond to such board for the due and faithful performance of his duties as such treasurer in such sum and with such sureties as the board, or a majority thereof, may require. The treasurer shall receive an annual salary, to be fixed and determined by the board, or a majority thereof, but he shall not receive any per diem allowance.

32. The members of any such board, except the secretary and treasurer thereof, when actually engaged in and about the business of said board, shall receive a per diem compensation of five dollars; said per diem compensation, and the salaries to be paid the secretary and treasurer, shall be included in said estimate hereinbefore mentioned.

33. Any such sewerage board is authorized and empowered to rent an office or offices as may be required for the due transaction and carrying out of its work and duties, and to properly equip and furnish such office or offices, the expense thereof to be included in said estimate mentioned in section twenty-six hereof.

34. This act shall take effect immediately.

Approved March 24, 1899.

[Laws of 1902, p. 195, chap. 49.]

AN ACT authorizing the appointment and defining the powers and duties of commissioners in sewage and drainage districts created for the purpose of relieving the streams and rivers therein from pollution, and to provide a plan for the prevention thereof, and providing for the raising, expenditure, and payment of moneys necessary for this purpose.

Be it enacted by the senate and general assembly of the State of New Jersey:

1. Upon the creation and incorporation by the legislature of any sewerage and drainage district for the purpose mentioned in the title of this act it shall be the duty of the governor of this State forthwith to appoint therein and therefor five able and discreet men, residents within such district (having regard in making such appointments to locality, so that each section of the district may be represented, as far as practicable), who, when so appointed, commissioned, and sworn, shall constitute a board of commissioners, to be known as the ——— district sewerage and drainage commissioners (inserting in each case in the blank space the name of the district designated in the act of incorporation), and the persons so appointed shall receive as compensation for their services an annual salary of twenty-five hundred dollars, payable in equal monthly installments. In making the first appointments under this act the members of the said board shall be appointed as follows: One for a term of one year, one for a term of two years, one for a term of three years, one for a term of four years, and one for a term of five years, and thereafter one shall be appointed each year for a term of five years. Any vacancy occurring in the said board by death, resignation, or otherwise, shall be filled in the same manner as the original appointment for the balance of the term. Each of the said commissioners so appointed shall, before they enter upon the duties of their office, take and subscribe an oath that they will faithfully and impartially execute and perform the duties imposed upon them by law, and cause the same to be filed in the office of the secretary of state of this State. The governor of this State shall have power to remove such commissioners from office for cause during their term of office and, upon removal, to fill the vacancy thus occasioned for the unexpired term in the manner herein provided for filling vacancies.

2. The said board shall, as soon as may be after appointment, and annually thereafter on the first Tuesday in May in each year, organize by the choice of one of its members as chairman, and may elect a clerk, who may or may not be a member of the said board, and may from time to time appoint such agents, officers, and servants and employ such engineers and assistants as it may deem necessary to

carry out the purposes of this act, and may determine their duties and compensation and remove the same at its pleasure.

3. The said board of commissioners, when duly organized, shall be deemed to be and shall become a body corporate, with power to sue and be sued and with the right to acquire, hold, use, and dispose of all such property as may be necessary for the uses and purposes for which the said board was created and with all other necessary powers incident to corporate bodies.

4. When duly organized, the said commissioners shall at once, with the aid and assistance of such engineers and other agents as they may deem proper, proceed to investigate methods and plans for relieving the streams and rivers within the said district from pollution and for preventing the pollution of the same, and to determine the apportionment of the capacity of sewer provided for each municipality in any intercepting sewer, sewers, or disposal works: *Provided*, That before a final determination as to the plan or method to be adopted for the purpose, an opportunity shall be given the governing body of each municipality to be heard in relation thereto, and after said hearing, as soon as the said commissioners have adopted a plan or method for this purpose, they shall report the same to the respective municipalities of the district and to the legislature of this State, together with a bill providing therefor and for the expenses thereof.

5. Before determining upon the final plan or route for the building or construction of any work investigated under this act the said board may, by its officers, agents, servants, and employees, enter at all times upon any lands or waters for the purpose of exploring, surveying, leveling, and laying out the route of any drain or sewer, locating any disposal, pumping, or other works, establishing grades, and doing all necessary preliminary work in the way of designating locations, doing, however, no unnecessary damage or injury to private or other property.

6. The said board shall at all times keep full and accurate account of its receipts and expenditures, disbursements, assets, and liabilities, and shall annually cause a detailed statement thereof to be published in one or more newspapers published or circulating in the respective municipalities in said district.

7. To provide for the payment of the cost and expense incurred or to be incurred by the said board in investigating and performing the duties imposed upon it by this act, one-half of said cost of [a] expense shall be paid out of the State treasury on certificate of the governor to the comptroller, who shall draw his warrant on the State treasurer in favor of the said board for the amount thereof, the same to be

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[a] So in original.

ascertained on a duly verified statement of such expenses being filed with the governor and in the office of the secretary of state; as to the balance of the said costs and expense incurred or to be incurred under this act, the said board shall have power and authority:

I. To issue from time to time, for the said one-half of the costs and expenses, temporary certificates, to run for a period not to exceed two years, the aggregate issue of said certificates not to exceed the sum of twenty-five thousand dollars; such certificates, when issued, shall be deemed and considered the indebtedness of the sewerage and drainage district, and shall constitute a charge upon persons and property therein, and shall be retired and paid in the manner hereinafter provided.

II. The said board shall have power and authority to order and cause a tax to be levied, assessed, and collected upon persons and property within the said sewerage and drainage district, the proceeds of which to be used in payment of the said certificates and the interest due and to grow thereon; the amount to be assessed and collected in the respective municipalities composing such districts shall be determined by the said board, and shall be apportioned according to the taxable ratables of the last preceding year as returned by the taxing officers in said district, and a certificate by the said board shall be filed with the taxing officers of such municipalities composing the said sewerage district, and it shall be the duty of the taxing officers within the said municipalities included in the said sewerage and drainage district, to levy, assess, and collect and pay over to the said commissioners any tax ordered by them to be assessed by virtue of the provisions of this act.

8. It shall be the duty of the said board annually to make and file with the secretary of state of this State a report showing the amount of money received by it for the purposes aforesaid, sources from which money was received, and the expenditure of the same; and it shall be the duty of the said board to keep an account of all certificates issued by it, when the same fall due, the time and place of payment, the rate of interest thereon, and of the amount received on the sale or disposition thereof; and the books, records, accounts, papers, and documents of the said board shall be open for the inspection of the governor of this State, or any person or persons whom he may appoint to inspect the same.

9. For the purpose of carrying out the provisions of this act with dispatch the sum of twenty-five thousand dollars is hereby appropriated by the State out of any moneys now in the State treasury not otherwise appropriated, and the governor is hereby authorized and empowered to give an order on the comptroller for advanced payments to the said board on account of the State's share of such expenses to be incurred.

10. When the said board of commissioners are appointed and organized under this act it shall have absolute control of and supervision over the prevention of pollution throughout the said sewerage or drainage district for which the said commissioners were appointed, exclusive of any other body or board in this State now having control of the same: *Provided, however*, That nothing herein contained shall in any way affect or delay or interfere with any action or proceedings which may have heretofore been taken by the State sewerage commission for the purpose of preventing pollution in said sewerage district, or that may hereafter be taken by said State sewerage commission for the enforcement thereof.

11. All acts or parts of acts inconsistent with the provisions of this act be, and the same are hereby, repealed, and this act shall take effect immediately.

Approved March 27, 1902.

[Act passed by the special session of the legislature convened April 21, 1903. Laws of 1903, p. 777.]

AN ACT to relieve from pollution the rivers and streams within the Passaic Valley sewerage district, established and defined by an act of the legislature entitled "An act to create a sewerage district to be called the Passaic Valley sewerage district," approved March twenty-seventh, one thousand nine hundred and two, and for this purpose establishing therefor a district board of commissioners, defining its powers and duties, and providing for the appointment, terms of office, duties, and compensation of such commissioners, and further providing for the raising, collecting, and expenditure of the necessary moneys.

Whereas the legislature of this State has created and defined a sewerage district, embracing a large number of municipalities and parts of municipalities, in the counties of Passaic, Bergen, Hudson, and Essex, under the name of the Passaic Valley sewerage district; and

Whereas the Passaic River and many streams flowing into it within said sewerage district are polluted by sewage and other deleterious matter to the extent that the health of the people residing in said district is seriously endangered; and

Whereas immediate relief therefrom is imperative; and

Whereas the governor of this State, by sanction of the legislature, has appointed five commissioners for said district with power, among other things, to investigate methods and plans for relieving the streams and rivers within said district from pollution, and for preventing the pollution of the same; and

Whereas said commissioners have adopted an effectual plan or method for relieving the streams and rivers within said district from pollution, and for preventing the pollution of the same. and have reported said plan or method to the legislature; and

Whereas in order to carry into effect such plan or method, with such modifications or additions thereto as shall hereafter be approved by said commissioners, it is necessary that further and greater power be given to said commissioners:

Be it enacted by the senate and general assembly of the State of New Jersey:

1. The commissioners heretofore appointed by the governor of this State in and for the Passaic Valley sewerage district shall continue in their respective offices for the terms for which they were severally appointed, and said terms are hereby extended to the first Tuesday of May succeeding the date when their terms under said appointments would respectively expire; and hereafter one commissioner shall be appointed by the governor, by and with the advice and consent of the senate, in each year for a term of five years, beginning on the first Tuesday of May next following the date of his appointment. Any vacancy occurring in the office of commissioner by death, resignation, or otherwise shall be filled by the governor, but for the unexpired term only. Each of the said commissioners hereafter appointed, before he enters upon the duties of his office, shall take and subscribe an oath that he will faithfully and impartially execute and perform the duties imposed upon him by law, and cause the same to be filed in the office of the secretary of state of this State. The commissioners shall each receive for services under this act an annual salary of twenty-five hundred dollars, payable in equal monthly installments, and the said commissioners shall henceforth receive no other compensation than that provided under this act. The governor of this State shall have power to remove any commissioner from office for cause during his term of office, and upon removal to fill the vacancy thus occasioned for the unexpired term. In making appointments, either for full terms or to fill vacancies, regard shall be had by the governor both to ability and fitness, and also to locality, so that each section of the district may be represented as far as practicable. No commissioner shall be directly or indirectly interested in any contract awarded under the provisions of this act, nor in furnishing materials or supplies therefor to any contractor, nor in furnishing security for the performance of any contract. If at any time it shall appear to the satisfaction of the governor of this State that any commissioner is or has been so interested, or is or has been a stockholder in any corporation furnishing material or supplies to any contractor for work done or to be done under the provisions of this act, or that he is the owner of any lands or water or water rights taken or to be taken or used in or for the construction of any work under the provisions of this act, or a stockholder in any corporation owning or leasing any such lands or waters or water rights, it shall be the duty of the governor to remove such commissioner from office forthwith, and all contracts made by such sewerage commissioners wherein any such

commissioner shall have been interested, directly or indirectly, as aforesaid, or otherwise, shall thereupon become and be null and void, and no further payments on account thereof shall be made by said sewerage commissioners.

2. The said commissioners shall, on the first Tuesday in May of each year, at the hour of two o'clock in the afternoon, organize by the choice of one of their members as chairman of the board, and they may elect a treasurer, who may or may not be a member of the board, and a clerk, who may or may not be a member of the board, and may also from time to time appoint such other officers, attorneys, agents, employees, and servants, and such engineers and assistants as they may deem necessary to carry out the purposes of this act, and may prescribe the duties and fix the compensation of all officers, attorneys, agents, employees, servants, engineers, and assistants; and all appointees of said commissioners may be removed at their pleasure. The organization of said board and the appointment of officers, agents, clerks, servants, engineers, and assistants heretofore made by the said board shall have the same effect as if made under this act.

3. The said commissioners heretofore appointed and their successors in office are and shall continue to be a body politic and corporate, with perpetual succession under the name of "Passaic Valley sewerage commissioners," with power to sue and be sued, with power to adopt and use a corporate seal, and the right, power, and authority to acquire, hold, use, and dispose of all such property, real and personal, as may be proper or necessary, and with all other powers proper or necessary to carry out and effectuate the purposes for which said board is created.

4. The board of Passaic Valley sewerage commissioners, incorporated as aforesaid, is hereby given full power and authority to make, construct, maintain, and operate intercepting, main, trunk, and outlet sewers with the necessary pipes, conduits, pumping works, and other appliances for the purpose of taking up, within the said Passaic Valley sewerage district, sewage and other offensive and deleterious matter which would or might otherwise pollute the streams and rivers in said district and convey the same to some proper place or places of deposit, discharge, or outfall in the New York Bay, within the State of New Jersey, to be selected by the said sewerage commissioners, there to be discharged, which place or places of deposit, discharge, or outfall shall be at least one and one-quarter miles, measured at right angles, in an easterly direction, from the exterior line for solid filling in the New York Bay, as now established by the riparian commissioners of this State, and in a tidal channel of not less than forty feet in depth at mean low water; and the said sewerage commissioners shall also have power to establish within said sewerage district, when necessary, sewage disposal works and works

for the treatment, disinfecting, and disposal of sewage: *Provided, however*, That no sewage disposal work and works for the treatment, disinfecting, and disposal of sewage shall be erected, established, or maintained within the distance of five miles from the outfall of said trunk sewer herein provided for: *Provided, however*, That nothing herein contained shall in any way be construed to allow or permit said sewerage commission to establish or build more than one sewage disposal works or more than one plant or works for the treatment, disinfecting or disposal of sewage; no contract of any kind shall be awarded at any one time for more than one million dollars: *Provided, however*, That this provision shall not apply to the sale of bonds. All work done and materials purchased in the prosecution of said work or works, the cost of which shall exceed five thousand dollars, shall be be by contract awarded, after due advertisement, to the lowest responsible bidder, and all contractors shall be required to give bonds satisfactory in security and amount to the said board; and no contract involving an expenditure of more than twenty-five thousand dollars shall be awarded until after the same shall have been submitted to and approved by the governor: *Provided*, That no contract for any of the work herein required to be performed by contract shall be awarded except on the express stipulation that so far as practicable all said work shall be performed by union labor, and preference shall be given to citizens of the State of New Jersey.

5. It shall be the duty of all persons, corporations, and municipalities owning or controlling the sewers or drains within the limits of said sewerage district, which discharge directly or indirectly into the streams or rivers within the said sewerage district any sewage or deleterious matter, to cause the same to be connected with and to be discharged into the sewers constructed by the said sewerage commissioners when the same shall have been constructed, and at the places which shall have been designated for that purpose by the said sewerage commissioners; all sewers and drains hereafter constructed by any person, corporation, or municipality within the said sewerage district conveying or discharging sewage or other deleterious matter, which might otherwise discharge into or be discharged into the streams or rivers within the said sewerage district, directly or indirectly, shall be so constructed that the outfall or discharge therefrom shall be delivered into the drains or sewers provided by the said sewerage commissioners at the points and places designated by the said commissioners; and it shall be the duty of the said sewerage commissioners, in constructing said intercepting or main sewers, to have them so constructed that connection therewith can be made at necessary or proper points; and all such connections shall be made in accordance with the rules and regulations from time to time adopted by the said sewerage

commissioners in relation thereto, and under the direction and supervision of their officers and agents, and all such connections shall be the property of such sewerage commissioners; the main, intercepting or trunk sewer to be constructed by the said sewerage commissioners shall commence at or near the Valley of Rocks, in the city of Paterson, and shall extend to the point of discharge or outfall in the New York Bay, within the limits of the State of New Jersey; before any moneys expended or obligations are incurred for the construction of any trunk or outlet sewer which shall discharge into New York Bay, the said board shall carefully investigate whether said discharge is likely to pollute the waters of said bay within the jurisdiction of the State of New York to such an extent or in such a degree as to cause a nuisance to persons or property within said State, and shall present the result of their investigation to the governor with their opinion thereon and their reasons for their opinion; and thereupon the same shall be considered by the governor and the attorney-general, and no work shall be done or further proceedings taken unless the attorney-general shall, in writing, advise that no cause of action either for damages or an injunction will arise in favor of the State of New York or any of its inhabitants by reason of such discharge of sewage into the waters of New York Bay, and the governor shall, by order, in writing, advise said board that, in his judgment, it is safe and prudent to proceed with its work, due regard being had to all the risks and dangers of injunctive litigation.

6. The said sewerage commissioners shall have power and authority to purchase and acquire lands and rights or interests in lands within and without the said sewerage district which may be deemed necessary for the construction of sewers, drains, disposal, pumping or other works authorized by this act, but no ventilating plant, sewage disposal works, or works for the treatment, disinfecting, or disposal of sewage shall be erected or maintained outside of said sewerage district; and if in any case the said sewerage commissioners shall be unable to agree with the owner or owners of any lands or rights or interests in lands deemed necessary by said sewerage commissioners in the construction and prosecution of the work hereby authorized, or when by reason of legal incapacity or absence of such owner or owners no agreement can be made for the purchase thereof, the lands or rights or interests in lands so deemed necessary for the purposes of this act shall be acquired by condemnation by the said sewerage commissioners in the manner provided by the general laws of this State relating to the condemnation of lands for public uses: *Provided*, That no private property shall be taken for the purposes of this act without compensation therefor shall have first been made or tendered to the owner or owners thereof, or, in lieu thereof, paid to the clerk of the

county in which the lands taken are located for the use of the person or persons entitled to receive the same; and in case such payment or tender to the owner or owners, or payment into court, is made by the said sewerage commissioners upon the award of commissioners, the said sewerage commissioners shall be entitled to take immediate possession of the property so condemned, notwithstanding any appeal. and the acceptance by the owner or owners of the lands or rights so condemned of any award of commissioners shall not interfere with or prevent the taking of any appeal provided by law.

7. The said board of sewerage commissioners shall have power to construct any sewer or drain by it to be made or constructed under or over any water course, under or over or across or along any street, turnpike, railway, canal, highway, or other way, and in or upon private or public lands, and in or upon lands of this State and under waters of this State, in such manner, however, as not unnecessarily to obstruct or impede travel or navigation, and may enter upon and dig up any street, road, highway, or private or public lands either within or without the said sewerage district for the purpose of constructing or laying sewers or drains upon or beneath the surface thereof, and for maintaining and operating the same, and in general may do all other acts or things necessary, convenient, and proper to carry out the purposes of this act; but no part of said sewer where laid under the waters of this State beyond the exterior lines for solid filling, as established by the riparian commissioners of this State, shall in said Newark Bay be above an elevation of thirty feet below mean low water, or shall in said New York Bay be above an elevation of thirty-five feet below mean low water; and the said board of sewerage commissioners shall have power, for the purpose of carrying such sewage or other matter to the place of deposit or discharge in New York Bay, to construct sewers within territory outside of the said sewerage district, and with its sewers, pipes, and drains to pass through or partly through the territory of municipalities outside of said sewerage district; and whenever the said board shall dig up any road, street, or highway as aforesaid, it shall, as far as possible, restore the same to as good condition and order as the same was when such digging commenced: *Provided, however*, That when such streets, roads, or highways lie outside of such sewerage district, the laying down of sewers or drains under or across said streets, roads, or highways shall be subject to such police regulations of the governing bodies of such municipalities as are applicable and enforceable in the construction of sewers or drains for such municipality.

8. The said sewerage commissioners shall have power and authority to alter or change the course or direction of any water course, and, with the consent of the township committee of any township and of the board or body having control of the streets or highways in any

city, town, or other municipality, to alter or change the grade or location of any highway, public street, or way crossed by any sewer or drain to be constructed under the provisions of this act.

9. The said board of sewerage commissioners may, by its officers, agents, servants, and employees, enter at all times upon any lands or waters within or without the said sewerage district for the purpose of exploring, surveying, leveling, and laying out the route of any drain or sewer, locating any disposal, pumping, or other works, establishing grades, and doing all necessary preliminary work; doing, however, no unnecessary damage or injury to private property.

10. The said board of sewerage commissioners shall at all times keep full and accurate accounts of its receipts, expenditures, disbursements, and liabilities, and shall annually cause a detailed statement thereof to be published and a copy thereof mailed to the secretary of state of this State and to the clerk of each of the municipalities in the district. The fiscal year of said sewerage commissioners shall end on the first Tuesday of May in each year, and said report so to be published shall be a report for the previous fiscal year, and shall be made as soon after the end of each fiscal year as conveniently may be; and the mayor or chief officer of any city or other municipality included within said drainage district shall be given full access to all the books, accounts, and vouchers of the said board, at all reasonable times, for the purpose of examination and report in the interest of such municipalities, respectively, and of the taxpayers therein.

11. To provide for the payment of costs and expenses incurred or to be incurred by the said sewerage commissioners for the purchase of lands, rights, or interests in lands or other property or rights, and in the construction of said disposal works, pumping stations, sewers, drains, and all other works by them to be constructed, and for engineering, administrative, and other expenses connected therewith, including interest during construction, said board of sewerage commissioners shall have power from time to time to issue its corporate bonds in an amount not to exceed nine million dollars and not to exceed the total estimated cost and expenses of the whole work; such bonds shall be in the form and payable at a time not exceeding fifty years from the date thereof and at such places, and either in currency or coin, as the said sewerage commissioners may determine; such bonds shall bear interest at a rate not exceeding four per centum per annum, payable semiannually; all such bonds shall be signed by the chairman of the said board of sewerage commissioners and countersigned by the treasurer, and shall be sealed with its corporate seal, attested by the clerk; in issuing such bonds the board of sewerage commissioners may, in its discretion, make the same or any part thereof fall due at stated periods less than fifty years from the date of issue, and may reserve in said bonds an option to redeem or pay

the same or any part thereof at stated periods at any time between the date thereof and the date at which they would otherwise fall due: the said bonds may be either coupon or registered bonds or partly coupon and partly registered bonds, and all such bonds may be negotiated, sold, and disposed of at not less than their par value, and the same or the proceeds thereof may be used by the said sewerage commissioners for the purposes aforesaid; the said board of sewerage commissioners shall keep the cost and expenses of the construction of its plant—in which shall be included the cost of lands, rights, or interests in lands, and the cost of all other property and rights, and the cost of construction of all works, including engineering expenses, administrative expenses, and legal expenses, and including interest during the course of construction—separate from the cost and expenses of maintenance, operation, and repairs; all sales of bonds shall be made after public notice and advertisement calling for bids and shall be made to the highest responsible bidders.

12. The said board of sewerage commissioners may, in anticipation of the issuing of bonds, and from time to time as it may need money, borrow such sum or sums of money, not exceeding at any one time one-fifth of the estimated cost of the whole work, and may issue its certificates of indebtedness, promissory notes, or other obligations therefor, retiring the same from time to time as the bonds hereinbefore authorized to be issued are sold. In order that the said bonds issued for the purchase of land, rights in land, and for the construction of the works, plant and extensions, betterments and improvements thereof may be paid and retired at maturity, the sewerage commissioners shall provide a proper and suitable sinking fund not exceeding in amount to be raised in any one year one per centum of the face value of the bonds issued, which sum shall be raised annually, beginning with the fifth year after the issuing of said bonds, at the time and in the manner herein provided for the raising of the moneys necessary to pay the interest on said bonds. The money so raised for sinking-fund purposes shall be kept in a separate account by the treasurer of the board of sewerage commissioners, and shall, under its direction, be used or invested from time to time in the purchase or retirement of its own bonds, or in the purchase of securities in which savings banks and savings institutions of this State are authorized to invest.

13. All indebtedness of the said board of sewerage commissioners incurred for the purchase of lands, rights, or interests in land or other property, and in the construction of its works or plant, or otherwise lawfully incurred, pursuant to the provisions of this act, whether such indebtedness is represented by bonds, certificates of indebtedness, promissory notes, or other form of indebtedness, with interest accrued or to accrue thereon, shall be a charge upon all per-

sons and property in the municipal or taxing districts lying in whole or in part within said sewerage district as fully as the legislature of this State shall have power to authorize the same; and all bonds, certificates of indebtedness, promissory notes, and other obligations issued by the said board of sewerage commissioners shall be free from all State, county, municipal, and other taxes, and the property, real and personal, of the said board of sewerage commissioners held by it under authority of this act, wherever situated, shall in like manner be free from taxation.

14. The said sewerage commissioners shall, on or before the fifteenth day of June in each year, ascertain and determine the amount of money necessary to be raised for the payment of interest upon bonds and other indebtedness and for sinking-fund charges for the current fiscal year, and shall apportion the same among the respective municipalities and taxing districts lying in whole or in part within said sewerage district, in such proportion as the taxable ratables within so much of said municipality or taxing district as is embraced within said sewerage district bears to the total amount of taxable ratables within the whole of said sewerage district, as returned and certified by the respective taxing boards and taxing officers of the said municipalities or taxing districts for the preceding year: *Provided, however*, That all ratables in said district for this purpose be assessed at their true value; and it shall be the duty of each assessor, taxing board, or taxing officer for the several municipalities and taxing districts lying in whole or in part within said sewerage district for this purpose, to examine, compute, determine, and certify to the said sewerage board annually, and by the first day of April of each year, the amount of taxable property or ratables assessed in the last preceding year to or upon persons and property within so much of the several municipalities and taxing districts as lie within the said sewerage district, and the books of each of the said assessors, taxing boards, and taxing officers shall at all times be open for examination by the board of sewerage commissioners, its officers and agents, for the purpose of examining, checking, and, if necessary, correcting said certificates.

15. The said board of sewerage commissioners shall, on or before the fifteenth day of June in each year, ascertain and determine as near as may be the amount of money necessary to be raised for operating, maintaining, and repairing its works and plant for the current fiscal year, and shall apportion the money so estimated to be necessary among the several municipalities or taxing districts lying in whole or in part within said sewerage district according to the amount of sewage by them respectively delivered to or discharged into any sewers or other receptacles provided or constructed by the said sewerage commissioners for the reception thereof. Before such apportionment is finally made and adopted by the sewerage commis-

sioners for any year and on the fourth Tuesday of May, at two o'clock in the afternoon, the said sewerage commission shall sit at its principal office for the purpose of hearing such municipalities as desire to be heard upon the apportionment of the estimated amount of money required for the operation, maintenance, and repair of said works and plant, but the apportionment when made by the said sewerage commissioners shall be final and conclusive; in case, however, the estimate of moneys necessary to be raised in any year for operating, maintaining, and repairing the works and plant of the sewerage commissioners shall, at the end of the year, be found to have been too low, the deficiency shall be made good by adding the same to the estimated amount required for operating, maintaining, and repairing the said works for the next succeeding year; and if said estimate shall be found to have been excessive, then such excess shall be deducted from the estimate for the next succeeding year.

16. The said board of sewerage commissioners shall, on or before the twentieth day of June in each year, order and cause a tax to be levied and assessed upon all persons and property within each of the municipal and taxing districts lying in whole or in part within said sewerage district, for the purpose of raising the money necessary to pay interest upon its bonds and other indebtedness and necessary sinking-fund charges and for the sum or sums of money estimated as necessary to provide for the proper maintenance and operation of its works and plant, and for all the other expenses of the said sewerage commissioners, and to this end it shall, on or before the twentieth day of June in each year, certify to the tax assessor, taxing board, or taxing officer of each of said municipalities or taxing districts lying in whole or in part within said sewerage district, the amount of tax required to be levied, assessed, and raised in each of their respective municipalities and taxing districts for said purposes; and the said assessors, taxing boards, and taxing officers shall assess said sums so directed to be assessed (and certified to them) upon all the persons and property within their respective municipalities or taxing districts liable to be assessed for State or county taxes, and the said tax shall be levied, assessed, and collected by the same officers at the same time and in the same manner and with the same effect as State or county taxes are required to be levied, assessed, and collected within said municipalities or taxing districts; and the taxes so levied upon real estate in said municipalities and taxing districts shall be and remain a first and paramount lien thereon until paid.

17. Out of the first moneys collected in any year in any municipality or taxing district, and not required by law to be paid to the county collector for State or county purposes, it shall be the duty of the disbursing officer or officers of such municipality or taxing dis-

trict to pay to the treasurer of the sewerage commissioners the sum or sums of money directed by said sewerage commissioners to be assessed, levied, and collected in such municipality or taxing district.

18. The said board of sewerage commissioners may, from time to time, in anticipation of the collection of moneys directed by it to be assessed, levied, and collected within the municipalities or taxing districts lying in whole or in part within its sewerage district, borrow such sum or sums of money as may be necessary for the payment of interest upon bonds or other indebtedness, and for the payment of sinking-fund charges, and for the payment of its officers, agents, employees, and for all other necessary or proper expenses in maintaining and operating its works and plant, and the payment of the moneys so borrowed shall be secured by a lien upon said taxes as levied and assessed, or so directed to be levied and assessed, and said taxes when collected shall be applied to the payment of the moneys so borrowed; all loans made in pursuance of this section shall be after public notice and advertisement, and shall be made or taken from the person or persons offering the most favorable terms.

19. If in any case the streams and rivers within the said sewerage district are or may be polluted by sewage or other deleterious matter discharged therein, directly or indirectly, from any municipality or any part of a municipality lying without the said sewerage district, it shall and may be lawful for the said board of commissioners to enter into contract with such municipality for the disposal of all such sewage and deleterious matter, and every such municipality is hereby authorized to enter into such contract with the said board, and the said board may, in the constructions made by it under the authority of this act, make provisions for such disposal; such contracts may be made upon such terms and for such lengths of time and for such annual or semiannual payments as shall be mutually agreed upon, and the municipalities and taxing districts so contracting shall have the power to raise annually, by taxation, the moneys necessary to make the payments required to be made under such contracts, or to use for this purpose any moneys not otherwise appropriated; and the moneys received by the said commissioners under such contracts shall be applied by them as follows: Two-thirds thereof to the payment of interest upon bonds issued by the said board, and one-third thereof to the payment of the expense of operation, maintenance, and repair of work.

20. The said sewerage commissioners shall have within said sewerage district powers exclusive of all other boards to protect the rivers and streams thereof from pollution and to prevent the pollution of the same, and to this end the said sewerage commissioners may prohibit the deposit or discharge into the rivers or streams within said sewerage district of any sewage or other matter or thing which may

pollute the same; they may also in like manner prohibit or prevent the emptying into any tributary of said rivers or streams, by any municipality or part of a municipality lying within the said sewerage district, of any sewage or other matter or thing which will directly or indirectly cause the rivers or streams within said sewerage district to be polluted; and the said board of sewerage commissioners may at any time, when it has reason to believe that any river or stream within its district is being polluted by any such municipality or part of a municipality by deposit or discharge into said rivers, streams, or their tributaries of any sewage or other matter or thing which will pollute the same, or when such deposit or discharge is threatened, to apply by bill or petition to the court of chancery of this State for injunction to prevent the said pollution or threatened pollution of said rivers or streams or their tributaries, and the court of chancery shall have power to hear and dispose of said petition or bills in a summary manner, and to grant any and all relief necessary to prevent said pollution or threatened pollution or the continuation of any pollution of said rivers, streams, or their tributaries.

21. The said board of sewerage commissioners shall have power from time to time to adopt all such reasonable rules and regulations for its own government and the government of its officers and agents, and also for the use, protection, and management of its works, property, and plant, and for the protection of the rivers and streams within its district from pollution, not inconsistent with the provisions of this act and the laws of this State.

22. The chairman shall preside at all meetings of the sewerage commissioners, and shall, with the treasurer, sign all bonds, promissory notes, certificates of indebtedness, and other obligations of the board; he shall also countersign all checks; in the absence of the chairman, or in case he is incapacitated by illness or other cause, the sewerage commissioners shall have power to elect an acting chairman, who for the time being shall have all the powers and perform all the duties of the chairman; the treasurer shall give bond in such sum as the sewerage commissioners may determine, and shall be the receiving and disbursing officer of the said sewerage commissioners, and all moneys required by law to be paid to said sewerage commissioners shall be paid to the treasurer thereof, and shall be by him deposited in such bank or banks of deposit or trust company or trust companies in this State as shall be determined upon by the said sewerage commissioners; all disbursements shall be by check, signed by the treasurer and countersigned by the chairman; the clerk shall have charge of the seal of the corporation and shall affix it to such instruments as he shall be directed by the said board, and he shall attest the same; he shall keep full minutes of all the meetings of the board and of its committees and shall perform all such other duties

as he may be directed by the said board of commissioners to perform; no deposit of moneys in the charge of the said board shall be made in any bank or trust company except upon the condition that the said board shall receive interest at the rate of not less than two per centum per annum upon the said deposits.

23. In case for any reason any section or any provision of this act shall be questioned in any court and shall be held to be unconstitutional or invalid, the same shall not be held to affect any other section or provision of this act.

24. All acts and parts of acts inconsistent with this act are hereby repealed; and this act shall take effect immediately.[a]

Approved April 22, 1903.

NEW YORK.

[Revised Statutes, 3d ed. (C. F. Birdseye), vol. 2, pp. 2822 ff., Article V: Public health law.]

POTABLE WATERS.

SEC. 70. *Rules and regulations of State board.*—The State board of health may make rules and regulations for the protection from contamination of any or all public supplies of potable waters and

[a] This act was declared unconstitutional by the court of errors and appeals of New Jersey in March, 1905. Van Cleve *v.* Passaic Valley Sewerage Commissioners, 60 Atlantic Rep., 214. It is retained here, however, because the ground upon which it was held unconstitutional affects only the mode of raising the necessary funds for carrying out the work.

It was subjected to an attack by the city of Paterson and by a property owner who had been assessed for public sewers in the city of Paterson. Argument was conducted by several of the ablest counsel in the State on each side and the act was sustained upon all grounds in the supreme court, but by a divided court. In the court of errors and appeals the action of the supreme court was reversed, and the act declared to be unconstitutional upon the ground that it contained an unlawful delegation to the sewerage commissioners of the power of taxation. The court says, per Garrison, J., "To relieve a river from pollution and to construct and maintain for this purpose sewers to the seaboard or to other point of output and to carry away through such sewers all that would otherwise pollute such river is clearly within the power of the central legislative body."

The act under examination authorized the commissioners to raise by taxation any amount in their discretion, subject only to the limit of nine million dollars ($9,000,000) in the matter of construction, but without any limit in the matter of maintenance. The taxation was laid upon a taxation area that was not coterminous with the sewerage district established by the legislature; and neither the taxation area nor the sewerage district is a political division of the State nor invested with any governmental function. The court held that the fundamental law of New Jersey required, "that the district to be taxed shall be coterminous with a district to which some right of local self-government is given." The act is, therefore, held invalid. The court then proceeds as follows: "Having stated the considerations that lead me to the conclusion that the act before us is invalid, because of its fiscal provision, I shall, to avoid misapprehension, add that nothing in this opinion is intended to imply a lack of power in the legislature to effectuate the object expressed in this act by means that are in harmony with the fundamental principles of taxation illustrated by the decisions I have cited. If, for instance, as was suggested by the arguments before us, powers adequate to the execution of the legislative scheme of drainage were conferred upon the entire area to be taxed and duties respecting the exercise of such powers constitutionally imposed in such manner as indicated and that their exercise was compulsory, a question not touched upon in this opinion would be presented."

their sources within the State. If any such rule or regulation relates to a temporary source or act of contamination, any person violating such rule or regulation shall be liable to prosecution for misdemeanor for every such violation, and on conviction shall be punished by a fine not exceeding two hundred dollars, or imprisonment not exceeding one year, or both. If any such rule or regulation relates to a permanent source or act of contamination, said board may impose penalties for the violation thereof or the noncompliance therewith not exceeding two hundred dollars for every such violation or noncompliance. Every such rule or regulation shall be published at least once in each week for six consecutive weeks in at least one newspaper of the county where the waters to which it relates are located. The cost of such publication shall be paid by the corporation or municipality benefited by the protection of the water supply to which the rule or regulation published relates. The affidavit of the printer, publisher, or proprietor of the newspaper in which such rule or regulation is published may be filed with the rule or regulation published in the county clerk's office of such county, and such affidavit and rule and regulation shall be conclusive evidence of such publication and of all the facts therein stated in all courts and places.

SEC. 71. *Inspection of water supply.*—The officer or board having by law the management and control of the potable water supply of any municipality, or the corporation furnishing such supply, may make such inspection of the sources of such water supply as such officer, board, or corporation deems it advisable, and to ascertain whether the rules or regulations of the State board are complied with. If any such inspection discloses a violation of any such rule or regulation relating to a permanent source or act of contamination, such officer, board, or corporation shall cause a copy of the rule or regulation violated to be served upon the person violating the same with a notice of such violation. If the person served does not immediately comply with the rule or regulation violated, such officer, board, or corporation shall notify the State board of the violation, which shall immediately examine into such violation, and if such person is found by the State board to have actually violated such rule or regulation, the secretary of the State board shall order the local board of health of such municipality to convene and enforce obedience to such rule or regulation. If the local board fails to enforce such order within ten days after its receipt, the corporation furnishing such water supply, or the municipality deriving its water supply from the waters to which such rule or regulation relates, may maintain an action in a court of record, which shall be tried in the county where the cause of action arose against such person, for the recovery of the penalties incurred by such violation, and for an injunction restraining him from the continued violation of such rule or regulation.

SEC. 71a. *Rules and regulations legalized.*—All rules and regulations heretofore duly made and published for the sanitary protection of public water supplies, pursuant to chap. 543 of the laws of 1885 and chap. 661 of the laws of 1893, as amended, are hereby legalized, ratified, confirmed, and continued in force until new rules and regulations become operative.

SEC. 71b. *Construction of act.*—This act shall not be construed to repeal or affect any of the provisions of chap. 378 of laws of 1897, or its amendments.

SEC. 72. *Sewerage.*—When the State board of health shall, for the protection of a water supply from contamination, make orders or regulations the execution of which will require or make necessary the construction and maintenance of any system of sewerage, or a change thereof, in or for any village or hamlet, whether incorporated or unincorporated, or the execution of which will require the providing of some public means of removal or purification of sewage, the municipality or corporation owning the waterworks benefited thereby shall, at its own expense, construct and maintain such system of sewerage, or change thereof, and provide such means of removal and purification of sewage and such works or means of sewage disposal as shall be approved by the State board of health. When the execution of any such regulations of the State board of health will occasion or require the removal of any building or buildings the municipality or corporation owning the waterworks benefited thereby shall, at its own expense, remove such buildings and pay to the owner thereof all the damages occasioned by such removal.

When the execution of any such regulation will injuriously affect any manufacturing or industrial enterprise which is not a public nuisance, such municipality or corporation shall pay all damages occasioned by the enforcement thereof. Until such construction or change of such system or systems of sewerage, and the providing of such means of removal or purification of sewage, and such works or means or sewage disposal and the removal of any building, are so made by the municipality or corporation owning the waterworks to be benefited thereby at its own expense there shall be no action or proceeding taken by such municipality or corporation against any person or corporation for the violation of any regulation of the State board of health under this article, and no person or corporation shall be considered to have violated or refused to obey any such rule or regulation. The owner of any building the removal of which is occasioned or required, or which has been removed by any rule or regulation of the State board of health made under the provisions of this article, and all persons whose rights of property are injuriously affected by the enforcement of any such rule or regulation, shall have a cause of action against the municipality or corporation owning the [illegible] the enforcement of such rule or regulation for all damages [illegible] sustained by such removal or enforcement [illegible] against such municipality [illegible] in the county in which the [illegible] therein: or [illegible] a special proceeding in the [illegible] county in which the property is [illegible] shall be commenced by petition [illegible] upon the municipality or corporation [illegible] the commencement of condemnation [illegible] may make and [illegible] condemnation proceedings. [illegible] the claims of the respective [illegible] condemnation law shall be applicable [illegible] upon the petition and answer. if [illegible] of the petition or answer, [illegible] a certain sum, and no costs shall be [illegible] the judgment is more unfavorable [illegible]

[illegible]—Any incorporated city or village [illegible] made such provision for the [illegible] contaminate therewith any [illegible] of water may have and maintain an [illegible] the discharge of any sewage or [illegible] which shall injure the potable [illegible] stream, lake, or other body of [illegible] village shall take or [illegible] provided, that such river, stream, lake, or [illegible] boundaries of the [illegible]

[illegible]—Whenever such action shall be [illegible] provisions of this act, it shall be the duty of the [illegible] of facts justifying the [illegible] under the provisions of this [illegible] incorporated a mandatory [illegible] corporation, municipality [illegible] a defendant in said action which [illegible] agents, or officers, shall discharge [illegible] sewage or any other substance deleterious to [illegible] shall injure the potable qualities of the water in such [illegible] same shall enter into any river, stream, lake, or other [illegible] which such plaintiff shall take or receive its water [illegible] such reasonable time as may be prescribed by the [illegible] action as shall prevent such discharge, or the disposal of such sewage or other substance into such waters, or the pollution thereof, with such further directions in the premises as may be proper and desirable to effect such purpose; provided, that such river, stream, lake, or other body of water is wholly or in part within the boundaries of the county in which such plaintiff is located.

SEC. 72c. *Examination by State board of health.*—But no such action shall be brought as provided for in section 2 (i. e., 72b) of this act until the State board of health has examined and determined whether the sewage does pollute or contaminate the river, stream, lake, or other body of water into which said sewage is discharged. The expense of such examination by said board shall be a charge upon and paid by the municipality in whose interest and on whose behalf such examination is made.

SEC. 72d. *Approval of plans.*—In case the State board of health shall find upon examination that the discharge of said sewage does pollute or contaminate said waters, or any of them, in such manner as to be of menace or danger to the health of those using said waters, the plans for the removal or disposal of the sewage ordered to be prepared by the court as provided in section 2 (i. e., 72b) shall be submitted to the State board of health for its approval.

[Laws of 1901, vol. 3, p. 214, charter of New York City.]

SEC. 481. It shall not be lawful for any person to throw or deposit, or cause to be thrown or deposited, in any lake, pond, or stream, or in any aqueduct from or through which any part of the water supply of the city of New York shall be drawn, or either of the reservoirs, any dead animal or other offensive matter or anything whatever. Any person offending against the provisions of this section shall be deemed guilty of a misdemeanor, and upon conviction thereof shall be punished by fine or imprisonment, or both, in the discretion of the court, such fine not to exceed the sum of one hundred dollars and such imprisonment not to exceed a period of three months, such imprisonment to be in the jail of the county in which the offense shall have been committed.

SEC. 482. If any person shall willfully do or cause to be done any act whereby any work, materials, or property whatever, erected or used, or hereafter to be erected or used, within the city or elsewhere by the said city, or by any person acting under their authority, for the purpose of procuring or keeping a supply of water, shall in any manner be injured, or shall erect or place any nuisance on the banks of any river, lake, or stream from which the water supply of said city shall be drawn, or shall throw anything into the aqueduct or into any reservoir, or pipe, such person on conviction thereof shall be deemed guilty of a misdemeanor.

waterworks benefited by the enforcement of such rule or regulation for all damages occasioned or sustained by such removal or enforcement, and an action therefor may be brought against such municipality or corporation in any court of record in the county in which the premises or property affected is situated and shall be tried therein; or such damages may be determined by a special proceeding in the supreme court or the county court of the county in which the property is situated. Such special proceedings shall be commenced by petition and notice to be served by such owner upon the municipality or corporation in the same manner as for the commencement of condemnation proceedings. Such municipality or corporation may make and serve an answer to such petition as in condemnation proceedings. The petition and answer shall set forth the claims of the respective parties, and the provisions of the condemnation law shall be applicable to the subsequent proceedings upon the petition and answer, if any. Either party may, before the service of the petition or answer, respectively, offer to take or pay a certain sum, and no costs shall be awarded against either party unless the judgment is more unfavorable to him than his offer.

SEC. 72a. *Actions by municipalities.*—Any incorporated city or village in the State of New York which has made such provision for the disposal of its sewage as not to pollute or contaminate therewith any river, stream, lake, or other body of water may have and maintain an action in the supreme court to prevent the discharge of any sewage or substance deleterious to health, or which shall injure the potable qualities of the water in any river, stream, lake, or other body of water from which such incorporated city or village shall take or receive its water supply; provided, that such river, stream, lake, or other body of water is wholly or in part within the boundaries of the county in which such plaintiff is located.

SEC. 72b. *Duty of supreme court.*—Whenever such action shall be brought under the provisions of this act, it shall be the duty of the supreme court, upon proof of the existence of facts justifying the bringing and maintenance of such action under the provisions of this act, to render a judgment in which shall be incorporated a mandatory injunction requiring the person, body, board, corporation, municipality, village, county, or town being a defendant to said action which directly or indirectly, or by its servants, agents, or officers, shall discharge or dispose of its sewage or any other substance deleterious to health, which shall injure the potable qualities of the water in such wise as that the same shall enter into any river, stream, lake, or other body of water from which such plaintiff shall take or receive its water supply, within such reasonable time as may be prescribed by the court, to take such action as shall prevent such discharge, or the disposal of such sewage or other substance into such waters, or the pollution thereof, with such further directions in the premises as may be proper and desirable to effect such purpose; provided, that such river, stream, lake, or other body of water is wholly or in part within the boundaries of the county in which such plaintiff is located.

SEC. 72c. *Examination by State board of health.*—But no such action shall be brought as provided for in section 2 (i. e., 72b) of this act until the State board of health has examined and determined whether the sewage does pollute or contaminate the river, stream, lake, or other body of water into which said sewage is discharged. The expense of such examination by said board shall be a charge upon and paid by the municipality in whose interest and on whose behalf such examination is made.

SEC. 72d. *Approval of plans.*—In case the State board of health shall find upon examination that the discharge of said sewage does pollute or contaminate said waters, or any of them, in such manner as to be of menace or danger to the health of those using said waters, the plans for the removal or disposal of the sewage ordered to be prepared by the court as provided in section 2 (i. e., 72b) shall be submitted to the State board of health for its approval.

[Laws of 1901, vol. 3, p. 214, charter of New York City.]

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SEC. 482. If any person shall willfully do or cause to be done any act whereby any work, materials, or property whatever, erected or used, or hereafter to be erected or used, within the city or elsewhere by the said city, or by any person acting under their authority, for the purpose of procuring or keeping a supply of water, shall in any manner be injured, or shall erect or place any nuisance on the banks of any river, lake, or stream from which the water supply of said city shall be drawn, or shall throw anything into the aqueduct or into any reservoir, or pipe, such person on conviction thereof shall be deemed guilty of a misdemeanor.

matter from any shop, factory, mill, or industrial establishment not constructed or in process of construction when this act takes effect shall be put in or constructed for the purpose of discharging any refuse or waste matter therefrom into any waters in this State, the plan or plans therefor, together with a statement of the purpose for which the same is to be used, shall be submitted to the commissioner. If the same is not detrimental to the public health he shall issue a permit therefor to the applicant. No such conduit, discharge pipe, or other means of discharging or casting any refuse or waste matter from any such shop, factory, mill, or establishment into any of the waters of this State shall be put in or constructed before such permit is granted, and if put in or constructed the person putting in or constructing or maintaining the same shall forfeit to the people of the State five dollars a day for each day the same is used or maintained for such purpose, to be collected in an action brought by the commissioner. He may also maintain an action in the name of the people to restrain a violation of this section.

SEC. 78. *Revocation of permit.*—Every such permit for the discharge of sewage from a sewer system or for the discharge of refuse or waste matter from a shop, factory, mill, or industrial establishment shall, when necessary to conserve the public health, be revocable or subject to modification or change by the State commissioner of health on due notice after an investigation and hearing and an opportunity for all interested therein to be heard thereon being served on the public authorities of the municipality owning and maintaining the sewage system, or on the proprietor, lessee, or tenant of the shop, factory, mill, or industrial establishment. The length of the time after receipt of the notice within which the discharge of sewage or of refuse or waste matter shall be discontinued may be stated in the permit, but in no case shall it exceed two years in the case of a sewer system nor one year in the case of a shop, factory, mill, or industrial establishment, and if the length of time is not specified in the permit it shall be one year in the case of a sewer system and six months in the case of a shop, factory, mill, or industrial establishment. On the expiration of the period of time prescribed after the service of a notice of revocation, modification, or change from the State commissioner of health, the right to discharge sewage or refuse or waste matter into any of the waters of the State shall cease and terminate, and the prohibition of this act against such discharge shall be in full force as though no permit had been granted, but a new permit may thereafter again be granted as hereinbefore provided.

SEC. 79. *Reports of municipal authorities to local boards of health.*—It shall be the duty of the public authorities having by law charge of the sewer system of every municipality in the State, from which sewer system sewage was being discharged into any of the waters of the State at the time of the passage of this act, to file with the board of health of the municipality within which any sewer outlet of the said sewer system is located and within sixty days after the passage of this act a report of each sewer system having an outlet within the municipality, which report shall comprise such facts and information as the State commissioner of health may require and on blanks or forms to be furnished by him on application. The board of health of each municipality being satisfied as to the correctness and completeness of each report submitted to it shall within thirty days after its receipt certify the same and transmit it to the State commissioner of health. Such report when satisfactory to the State commissioner of health shall be filed by him in his office and shall constitute the evidence of exemption from the prohibition of this act. No sewer system shall be exempt from the prohibition of this act against the discharge of sewage into the waters of the State for which a satisfactory report shall not be filed in the office of the State commissioner of health in accordance with this section.

SEC. 79a. *Reports of proprietors of industrial establishments.*—It shall be the duty of the proprietor of every shop, factory, mill, and industrial establishment in the State from which refuse or waste matter was being discharged into any of the waters of the State at the time of the passage of this act to file with the State commissioner of health within sixty days after the passage of this act a report of each shop, factory, mill, and industrial establishment from which refuse or waste matter was being discharged through an outlet within the municipality at the time of the passage of this act, which report shall comprise such facts and information in regard to the size, location, and character of shop, factory, mill, or industrial establishment, the machinery in use therein, and the character and quantity of goods produced as the State commissioner of health may require and on blanks or forms to be furnished by him on application. Such report shall be filed by him in his office, and shall constitute the evidence of exemption of the shop, factory, mill, or industrial establishment from the prohibition of this act. No shop, factory, mill, or industrial establishment shall be exempt from the prohibition of this act against the discharge of refuse or waste matter into the waters of the State, for which a report shall not be made as required by the State commissioner of health in accordance with this section.

SEC. 79b. *Record of permits; inspection of local boards of health.*—Each board of health shall preserve in its office, and in a form to be prescribed by the State commissioner of health, a permanent record of each permit issued by the State commissioner of health granting

the right to discharge sewage or refuse or waste matter into any of the waters of the State within that municipality and of each revocation of a permit; and also a permanent record of each report received by the board of health concerning each sewer system and each shop, factory, mill, or industrial establishment which at the time of the passage of this act was discharging sewage or refuse or waste matter into any of the waters of the State within that municipality. Each local board of health shall make and maintain such inspection as will at all times enable it to determine whether this act is being complied with in respect to the discharge of sewage, refuse, or waste matter or other materials prohibited by this act into any of the waters of the State within that municipality. For the purpose of such inspection every member of such board of health, or its health officers, or any person duly authorized by it, shall have the right to make all necessary examinations of any premises, building, shop, factory, mill, industrial establishment, process, or sewage system.

SEC. 79c. *Violations; service of notice; actions by local boards.*—The local board of health of each municipality shall promptly ascertain every violation of or noncompliance with any of the provisions of this act or of the permits for the discharge of sewage or refuse or waste material into any of the waters of the State herein provided which may occur within that municipality. The board of health shall, on the discovery of every violation of or noncompliance with any of the provisions of this act or of any permit duly issued, serve a written notice on the person or corporation responsible for the violation or noncompliance together with a copy of this act and of the permit, if any, [illegible] complied with, specifying the particular provision [illegible] noncomplied with, and stipulating the length [illegible] the violation or noncompliance must cease [illegible] the stipulated length of time the violation [illegible] continue, the board of health shall [illegible] noncompliance to the State commissioner [illegible] give a hearing to and take the [illegible] such violation or noncompliance [illegible] and if he finds a violation or noncompliance [illegible] certify the fact to the board of [illegible] shall immediately bring an action [illegible] shall be tried in the county wherein [illegible] the person or corporation responsible [illegible] noncompliance for the recovery of the [illegible] against the continuation of [illegible] noncompliance.

[illegible] The penalty for the discharge of sewage [illegible] into any of the waters of the State ithout a duly issued permit for which a permit is required by this ct shall be five hundred dollars, and a further penalty of fifty ollars per day for each day the offence is maintained. The penlty for the discharge of sewage from any public sewer system ito any of the waters of the State without filing a report for which report is required to be filed with the board of health of the muicipality shall be fifty dollars. The penalty for the discharge of efuse or waste matter from any shop, factory, mill, or industrial stablishment for which a permit is required by this act without such ermit shall be one hundred dollars and ten dollars a day for each lay the offence is maintained. The penalty for the discharge of efuse or waste matter from any shop, mill, factory, or industrial stablishment without filing a report where a report is required by his act to be filed shall be twenty-five dollars and five dollars per day for each day the offence is maintained. The penalty for discharging nto any of the waters of the State any other matter prohibited by this act besides that specified above shall be twenty-five dollars and five dollars per day for each day the offence is maintained.

SEC. 2. *Common-law rights not affected.*—Nothing in this act shall be construed to diminish or otherwise to modify the common-law rights of riparian owners in the quality of waters of streams covered by such rights, nor in the case of actions brought against the pollution of waters to limit their remedy to indemnities.

SEC. 3. This act shall take effect immediately.

[Laws of 1905, chap. 454.]

AN ACT regulating the sanitary condition of bathing establishments, and amending section two hundred and twelve of chapter twenty-five of the general public health laws, as amended by the laws of eighteen hundred and ninety-three; being renumbered by the laws of nineteen hundred, chapter six hundred and sixty-seven; number of section being originally two hundred and two.

SECTION 1. Section two hundred and twelve of chapter twenty-five of the general public health laws, as amended by the laws of eighteen hundred and ninety-three, is hereby amended so as to read as follows:

§ 212. *Regulating the sanitary condition of bathing establishments and the preservation of life at bathing places.*—It shall be unlawful for any person to maintain, either as owner or lessee, any bathing establishment of any kind, in this state, for the accommodation of persons, for pay, or any consideration, at a point less than five hundred feet from any sewer connection emptying therein, or thereat, so as to pollute in any way, the waters used by those using or hiring bathing houses at such bathing establishments; it shall be the duty of such owner or lessee to provide separate toilet rooms, with water-closets properly provided with sanitary plumbing, constructed in a manner approved by the local board of health and in such a way as not to contaminate the waters used by the bathers; it shall also be

the right to discharge sewage or refuse or waste matter into any of the waters of the State within that municipality and of each revocation of a permit; and also a permanent record of each report received by the board of health concerning each sewer system and each shop, factory, mill, or industrial establishment which at the time of the passage of this act was discharging sewage or refuse or waste matter into any of the waters of the State within that municipality. Each local board of health shall make and maintain such inspection as will at all times enable it to determine whether this act is being complied with in respect to the discharge of sewage, refuse, or waste matter or other materials prohibited by this act into any of the waters of the State within that municipality. For the purpose of such inspection every member of such board of health, or its health officers, or any person duly authorized by it, shall have the right to make all necessary examinations of any premises, building, shop, factory, mill, industrial establishment, process, or sewage system.

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SEC. 79d. *Penalties.*—The penalty for the discharge of sewage from any public sewer system into any of the waters of the State

without a duly issued permit for which a permit is required by this act shall be five hundred dollars, and a further penalty of fifty dollars per day for each day the offence is maintained. The penalty for the discharge of sewage from any public sewer system into any of the waters of the State without filing a report for which a report is required to be filed with the board of health of the municipality shall be fifty dollars. The penalty for the discharge of refuse or waste matter from any shop, factory, mill, or industrial establishment for which a permit is required by this act without such permit shall be one hundred dollars and ten dollars a day for each day the offence is maintained. The penalty for the discharge of refuse or waste matter from any shop, mill, factory, or industrial establishment without filing a report where a report is required by this act to be filed shall be twenty-five dollars and five dollars per day for each day the offence is maintained. The penalty for discharging into any of the waters of the State any other matter prohibited by this act besides that specified above shall be twenty-five dollars and five dollars per day for each day the offence is maintained.

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SEC. 79c. *Violations; service of notice; actions by local boards.*—The local board of health of each municipality shall promptly ascertain every violation of or noncompliance with any of the provisions of this act or of the permits for the discharge of sewage or refuse or waste material into any of the waters of the State herein provided which may occur within that municipality. The board of health shall, on the discovery of every violation of or noncompliance with any of the provisions of this act or of any permit duly issued, serve a written notice on the person or corporation responsible for the violation or noncompliance, together with a copy of this act and of the permit, if any, violated or noncomplied with, specifying the particular provision being violated or noncomplied with, and stipulating the length of time within which the violation or noncompliance must cease. If at the expiration of the stipulated length of time the violation or noncompliance shall still continue, the board of health shall at once report the violation and noncompliance to the State commissioner of health, who shall at once give a hearing to and take the proof of the persons charged with such violation or noncompliance and investigate the matter, and if he finds a violation or noncompliance to exist he shall at once certify the fact to the board of health of the municipality, which shall immediately bring an action in a court of record, which action shall be tried in the county wherein the cause of action arose against the person or corporation responsible for the violation or the noncompliance for the recovery of the penalties incurred and for an injunction against the continuation of the violation or the noncompliance.

SEC. 79d. *Penalties.*—The penalty for the discharge of sewage from any public sewer system into any of the waters of the State

without a duly issued permit for which a permit is required by this act shall be five hundred dollars, and a further penalty of fifty dollars per day for each day the offence is maintained. The penalty for the discharge of sewage from any public sewer system into any of the waters of the State without filing a report for which a report is required to be filed with the board of health of the municipality shall be fifty dollars. The penalty for the discharge of refuse or waste matter from any shop, factory, mill, or industrial establishment for which a permit is required by this act without such permit shall be one hundred dollars and ten dollars a day for each day the offence is maintained. The penalty for the discharge of refuse or waste matter from any shop, mill, factory, or industrial establishment without filing a report where a report is required by this act to be filed shall be twenty-five dollars and five dollars per day for each day the offence is maintained. The penalty for discharging into any of the waters of the State any other matter prohibited by this act besides that specified above shall be twenty-five dollars and five dollars per day for each day the offence is maintained.

SEC. 2. *Common-law rights not affected.*—Nothing in this act shall be construed to diminish or otherwise to modify the common-law rights of riparian owners in the quality of waters of streams covered by such rights, nor in the case of actions brought against the pollution of waters to limit their remedy to indemnities.

SEC. 3. This act shall take effect immediately.

[Laws of 1905, chap. 454.]

AN ACT regulating the sanitary condition of bathing establishments, and amending section two hundred and twelve of chapter twenty-five of the general public health laws, as amended by the laws of eighteen hundred and ninety-three; being renumbered by the laws of nineteen hundred, chapter six hundred and sixty-seven; number of section being originally two hundred and two.

SECTION 1. Section two hundred and twelve of chapter twenty-five of the general public health laws, as amended by the laws of eighteen hundred and ninety-three, is hereby amended so as to read as follows:

§ 212. *Regulating the sanitary condition of bathing establishments and the preservation of life at bathing places.*—It shall be unlawful for any person to maintain, either as owner or lessee, any bathing establishment of any kind, in this state, for the accommodation of persons, for pay, or any consideration, at a point less than five hundred feet from any sewer connection emptying therein, or thereat, so as to pollute in any way, the waters used by those using or hiring bathing houses at such bathing establishments; it shall be the duty of such owner or lessee to provide separate toilet rooms, with water-closets properly provided with sanitary plumbing, constructed in a manner approved by the local board of health and in such a way as not to contaminate the waters used by the bathers; it shall also be

the right to discharge sewage or refuse or waste matter into any of the waters of the State within that municipality and of each revocation of a permit; and also a permanent record of each report received by the board of health concerning each sewer system and each shop, factory, mill, or industrial establishment which at the time of the passage of this act was discharging sewage or refuse or waste matter into any of the waters of the State within that municipality. Each local board of health shall make and maintain such inspection as will at all times enable it to determine whether this act is being complied with in respect to the discharge of sewage, refuse, or waste matter or other materials prohibited by this act into any of the waters of the State within that municipality. For the purpose of such inspection every member of such board of health, or its health officers, or any person duly authorized by it, shall have the right to make all necessary examinations of any premises, building, shop, factory, mill, industrial establishment, process, or sewage system.

SEC. 79c. *Violations; service of notice; actions by local boards.*—The local board of health of each municipality shall promptly ascertain every violation of or noncompliance with any of the provisions of this act or of the permits for the discharge of sewage or refuse or waste material into any of the waters of the State herein provided which may occur within that municipality. The board of health shall, on the discovery of every violation of or noncompliance with any of the provisions of this act or of any permit duly issued, serve a written notice on the person or corporation responsible for the violation or noncompliance, together with a copy of this act and of the permit, if any, violated or noncomplied with, specifying the particular provision being violated or noncomplied with, and stipulating the length of time within which the violation or noncompliance must cease. If at the expiration of the stipulated length of time the violation or noncompliance shall still continue, the board of health shall at once report the violation and noncompliance to the State commissioner of health, who shall at once give a hearing to and take the proof of the persons charged with such violation or noncompliance and investigate the matter, and if he finds a violation or noncompliance to exist he shall at once certify the fact to the board of health of the municipality, which shall immediately bring an action in a court of record, which action shall be tried in the county wherein the cause of action arose against the person or corporation responsible for the violation or the noncompliance for the recovery of the penalties incurred and for an injunction against the continuation of the violation or the noncompliance.

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the duty of such owner or lessee to thoroughly wash and disinfect, or cause to be thoroughly washed and disinfected, in a manner approved by the local board of health, all bathing suits that have been hired or used, before rehiring or permitting the use of the same again. Any person or persons violating any of the provisions of this section shall forfeit and pay a penalty of not less than fifty dollars nor more than two hundred dollars to be recovered by the sheriff of the county in which such violation is committed, except in the city of New York, when the penalty shall be sued for in the name of the department of health of the city of New York and collected by it. It shall be the duty of the sheriffs and constables of the several counties of this State abutting upon the seashore, to see that in their respective counties the provisions of this section are enforced, and to bring suit for the recovery of the penalty therein provided, unless some other person had already brought suit for the same. A separate penalty may be recovered for each day that any person subject to the provisions of this section may violate any of the provisions of the same; but no penalty shall be recovered for any other violation thereof than shall have occurred during the days when the owner or lessee, or other person or persons, maintaining the said bathing establishments, shall have kept the same open for the use of the public, or for such persons as may be the guests of any hotel that such bathing establishments may be connected with. The owner of a bathing house shall not be subject to the provisions of this section when it is used, occupied or maintained by a lessee for hire, but such lessee shall be deemed the keeper or proprietor or person or persons maintaining such bathing establishment thereof. Nothing in this section shall be construed, in any to affect any bathing establishments, in any city or municipality, at which there is maintained at public expense a life-saving guard.

§ 2. This act shall take effect the first day of June, nineteen hundred and five.

Approved, May 16, 1905.

PENNSYLVANIA.

[Pepper and Lewis Digest, Supplement, p. 98.]

BURIAL. SEC. 7.—*Pollution of water by use of land for burial purposes prohibited.*—That it shall be unlawful to use for the burial of the dead any land the drainage from which passes into any stream furnishing the whole or any portion of the water supply of any city, except beyond the distance of one mile from such city: *Provided, however*, That the prohibitions of this act shall not be enforceable against any land now devoted to burial purposes in which there shall have heretofore been burials and sales of burial lots.

NUISANCES. Same, col. 253.

SEC. 27.—*Penalty for pollution of water used for drinking purposes.*—Any person who shall wilfully enter upon the enclosed land of any company incorporated under the laws of this Commonwealth for the purpose of supplying water to the public for drinking purposes, on which land is erected any dam, reservoir, pond, or other artificial means for storing water, and pollute or attempt to pollute the water on such land, shall be deemed, and the same is hereby declared to be, a misdemeanor, and may be prosecuted and convicted as such under the laws of this Commonwealth, and on conviction thereof in the court of quarter sessions of the proper county shall be fined not exceeding fifty dollars, and imprisoned not exceeding sixty days.

SEC. 28.—*Offender to be arrested on view.*—That any duly constituted watchman of any such water company, or any constable or policeman, is hereby authorized and empowered, upon his own view of any such trespass, to make arrests and bring before any alderman or magistrate of the proper county offenders found violating the provisions of this act.

[Laws of 1905, No. 182.]

AN ACT to preserve the purity of the waters of the State, for the protection of the public health.

SEC. 1. *Be it enacted, &c.*, That the term "waters of the State," wherever used in this act, shall include all streams and springs, and all bodies of surface and of ground water, whether natural or artificial, within the boundaries of the State.

SEC. 2. Every municipal corporation, private corporation, company, and individual supplying or authorized to supply water to the public, within the State, shall, within sixty days after the passage of this act, file with the commissioner of health a certified copy of the plans and surveys of the waterworks, with a description of the source from which the supply of water is derived; and no additional source of supply shall thereafter be used without a written permit from the commissioner of health, as hereinafter provided.

SEC. 3. No municipal corporation, private corporation, company, or individual shall construct waterworks for the supply of water to the public within the State, or extend the same, without a written permit, to be obtained from the commissioner of health, if, in his judgment, the proposed source of supply appears to be not prejudicial to the public health. The application for such permit must be accompanied by a certified copy of the plans and surveys for such waterworks, or extension thereof, with a description of the source from which it is proposed to derive the supply; and no additional source of supply shall subsequently be used for any such waterworks without a similar permit from the commissioner of health. When application shall be made for a permit under either of the above provisions of this section, it shall be the duty of the commissioner to proceed to examine the application without delay, and, as soon as possible, he shall make a decision, in writing; and, within thirty days after such decision, the corporation, company, or individual making such application may appeal to any court of common pleas of the county, and said court shall, without delay, hear the appeal, and shall make an order approving, setting aside, or modifying such decision, or fixing the terms upon which said permit shall be granted. The penalty for failure to file copies of plans, surveys, and descriptions of existing waterworks within the time hereinbefore fixed, and for the construction or extension of waterworks, or the use of an additional source of supply without a permit from the commissioner of health, shall be five hundred dollars, and further penalty of fifty dollars per day for each day that the works are in operation contrary to the provisions of this act, recoverable by the Commonwealth, at the suit of the commissioner of health, as debts of like amount are recoverable by law.

SEC. 4. No person, corporation, or municipality shall place, or permit to be placed, or discharge, or permit to flow into any of the waters of the State, any sewage, except as hereinafter provided. But this act shall not apply to waters pumped or flowing from coal mines or tanneries, nor prevent the discharge of sewage from any public sewer system, owned and maintained by a municipality, provided such sewer system was in operation and was discharging sewage into any waters of the State at the time of the passage of this act. But this exception shall not permit the discharge of sewage from the sewer system which shall be extended subsequent to the passage of this act.

For the purpose of this act, sewage shall be defined as any substance that contains any of the waste products, or excrementitious or other discharges from the bodies of human beings or animals.

SEC. 5. Upon application duly made to the commissioner of health, by the public authorities having by law the charge of the sewer system of any municipality, the governor of the State, the attorney-general, and the commissioner of health shall consider the case of such a sewer system, otherwise prohibited by this act from discharging sewage into any of the waters of the State, and, whenever it is their unanimous opinion that the general interests of the public health would be subserved thereby, the commissioner of health may issue a permit for the discharge of sewage from any such sewer system into any of the waters of the State, and may stipulate in the permit the conditions on which such discharge may be permitted. Such permit, before being operative, shall be recorded in the office of the recorder of deeds for the county wherein the outlet of the said sewer system is located. Every such permit for the discharge of sewage from a sewer system shall be revokable, or subject to modification and change, by the commissioner of health, on due notice, after an investigation and hearing, and an opportunity for all interested therein to be heard thereon being served on the public authorities of the municipality owning, maintaining, or using the sewage system. The length of time after receipt of the notice within which the discharge of sewage shall be discontinued may be stated in the permit, but in no case shall it be less than one year or exceed two years, and if the length of time is not specified in the permit it shall be one year. On the expiration of the period of time prescribed, after the service of a notice of revocation, modification, or change, from the commissioner of health, the right to discharge sewage into any of the waters of the State shall cease and terminate; and the prohibition of this act against such discharge shall be in full force, as though no permit had been granted, but a new permit may thereafter again be granted, as hereinbefore provided.

SEC. 6. It shall be the duty of the public authorities having by law charge of the sewer system of every municipality in the State from which sewage was being discharged into any of the waters of the State at the time of the passage of this act, to file with the commissioner of health, within four months after the passage of this act, a report of such sewer system, which shall comprise such facts and information as the commissioner of health may require. No sewer system shall be exempt from the provisions of this act against the discharge of sewage into the waters of the State for which a satisfactory report shall not be filed with the commissioner of health in accordance with this section.

SEC. 7. The penalty for the discharge of sewage from any public sewer system into any of the waters of the State without a duly issued permit in any case in which a permit is required by this act shall be five hundred dollars, and a further penalty of fifty dollars per day for each day the offense is maintained, recoverable by the Commonwealth at the suit of the commissioner of health as debts of like amount are recoverable by law. The penalty for the discharge of sewage from any public sewer system into any of the waters of the State without filing a report, in any case in which a report is required to be filed, shall be fifty dollars, recoverable by a like suit.

SEC. 8. All individuals, private corporations, and companies that, at the time of the passage of this act, are discharging sewage into any of the waters of the State may continue to discharge such sewage unless, in the opinion of the commissioner of health, the discharge of such sewage may become injurious to the public health. If at any time the commissioner of health considers that the discharge of such sewage into any of the waters of the State may become injurious to the public health he may order the discharge of such sewage discontinued.

SEC. 9. Every individual, private corporation, or company shall discontinue the discharge of sewage into any of the waters of the State within ten days after having been so ordered by the commissioner of health.

SEC. 10. Any individual, private corporation, or company that shall discharge sewage, or permit the same to flow, into the waters of the State contrary to the provisions of this act shall be deemed guilty of a misdemeanor, and shall upon conviction be punished by a fine of twenty-five dollars for each offense and a further fine of five dollars per day for each day the offense is maintained, or by imprisonment not exceeding one month, or both, at the discretion of the court.

SEC. 11. Any order or decision, under this act, of the commissioner of health, or that of the governor, attorney-general, and commissioner of health, shall be subject to an appeal to any court of common pleas of the county wherein the outlet of such sewer or sewer system, otherwise prohibited by this act, is situated; and said court shall have power to hear said appeal, and may affirm or set aside said order or decision, or modify the same, or otherwise fix the terms upon which permission shall be granted. But the order or decision appealed from shall not be superseded by the appeal, but shall stand until the order of the court, as above.

Approved the 22d day of April, A. D. 1905.

[Laws of 1905, No. 223.]

AN ACT authorizing and empowering cities, owning and operating waterworks systems, to enter, by any of its employes, upon private lands through which may pass any stream or streams of water supplying such cities, for the purpose of patrolling the drainage area, and making investigations or inquiries pertaining to the condition of the stream or streams, sanitary or otherwise.

SEC. 1. *Be it enacted, &c.*, That any city owning and operating a waterworks system is hereby authorized and empowered to enter, by any of its employes, upon private lands through which may pass any stream or streams of water supplying such city, for the purpose of patrolling the drainage area of such stream or streams, and making investigations or inquiries pertaining to the condition of the stream or streams, sanitary or otherwise; Provided, however, That any injury or damage done to the property so entered upon shall be paid by such city.

Approved the 2d day of May, A. D. 1905.

VERMONT.

[Statutes, 1894, p. 842, Preservation of public health.]

SEC. 4695. If any person puts or causes to be put a dead animal or animal substance into or upon the bank of a lake, pond, running stream, or spring of water so that it is drawn or washed into the same, and suffers it to remain therein, he shall be fined not more than twenty dollars and not less than five dollars.

[Laws of 1898, No. 150, p. 115.]

AN ACT in amendment of act No. 137 of the acts of 1894, relating to pollution of the waters of Missisquoi River.

It is hereby enacted by the general assembly of the State of Vermont:

SECTION 1, number 137, of the public acts of 1894, is hereby amended so as to read as follows: "A person owning or operating a mill who shall by himself or his agent deposit or suffer to be deposited any sawdust, shavings, or any mill refuse in the waters of the Missisquoi River above Enosburgh Falls, or in any of the tributaries of said Missisquoi River above Enosburgh Falls, shall be fined not less than twenty dollars nor more than one hundred dollars, in the discretion of the court, for each offence."

SEC. 2. This act shall take effect from March 1st, 1899. Approved November 16th, 1898.

[Laws of 1902, No. 115, p. 144.]

AN ACT to prevent the pollution of the sources of water supply, as amended by No. 141, Laws of 1904.

It is hereby enacted by the general assembly of the State of Vermont:

SECTION 1. The State board of health shall have the general oversight and care of all waters, streams, and ponds used by any cities, towns, villages, or public institutions, or by any water or ice companies in this State as sources of water supply, and of all springs, streams, and water courses tributary thereto. It shall have power to call for, and when it calls for it shall be provided with maps, plans, and documents suitable for such purposes, at the expense of such city, town, village, public institution, water or ice company, and shall keep records of all its transactions relative thereto.

Said board shall have authority to prohibit any town, city, village, public institution, individual or water or ice company from using water or ice from any given source whenever in its opinion the same is so contaminated, unwholesome and impure that the use thereof endangers the public health. And the court of chancery shall have jurisdiction and power, upon application therefor by the State board of health, to enforce by proper order and decree any order, rule or regulation which said board may make under and by virtue of this section.

SEC. 2. Said board may cause examinations of such waters to be made to ascertain the purity and fitness for domestic use, or their liability to impair the interests of the public or of persons lawfully using them or to imperil the public health. It may make rules and regulations to prevent the pollution and to secure the sanitary protection of all such waters as are used as sources of water supply.

SEC. 3. The publication of an order, rule, or regulation made by the board under the provisions of sec. 2 or sec. 6 hereof, in the newspaper of any town or village in which such order, rule, or regulation is to take effect, or if no newspaper is published in such city, town, or village, the posting of a copy of such order, rule, or regulation in three public places in such city, town, or village, shall be legal notice to all persons, and an affidavit of such publication or posting by the persons causing such to be published or posted, filed, and recorded with a copy of the notice in the office of the clerk of such city, town, or village shall be admitted as evidence of the time at which and the place and manner in which the notice was given.

SEC. 4. Said board shall include in its biennial report to the general assembly its doings for the preceding biennial term, and shall recommend measures for the prevention of the pollution of such waters and for the removal of polluting substances in order to protect and develop the rights and property of the State therein, and to protect the public health, and shall recommend any legislation or plans for systems of main sewers necessary for the preservation of the public health and for the purification and prevention of pollution of the ponds, streams, and waters of the State. It shall also give notice to the State's attorney for the county wherein any violation of the law relative to the pollution of the water supplies occurs. It shall have the power to employ such expert assistants as it considers necessary.

SEC. 5. Cities, towns, villages, and persons shall submit to said board for its advice their proposed systems of public water supply or for the disposal of drainage or sewage. Said board shall consult with and advise the authorities of the cities, towns, villages, and persons having or about to have systems of public water supply, drainage, or sewage, as to the most appropriate sources of water supply, and the best methods of assuring its purity or as to the best methods of disposing of their drainage or sewage, with reference to the existing and future needs of other cities, towns, villages, or persons which may be affected thereby. It shall also consult with and advise persons engaged or intending to engage in any manufacturing or other business whose drainage or sewage may tend to pollute any water or source of water supply as to the best method of preventing such pollution, and it may conduct experiments to determine the best methods of purification or disposal of drainage or sewage. No person shall be required to bear the expense of such consultation, advice, or experiments. In this section the term "drainage" means the rainfall, surface, and subsoil water only, and "sewage" means domestic and manufacturing filth and refuse.

SEC. 6. Upon petition to said board by the mayor of a city, the selectmen of a town, the trustee or bailiff of a village, the managing board or officer of any public institution, or by a board of water commissioners, or the president of a water or ice company, stating that manure, excrement, garbage, or any other matter is polluting or tending to pollute the water of any stream, pond, spring, or water course used by such city, town, village, institution, or company as a source of water supply, the board shall appoint a time and place within the county where the nuisance or pollution is alleged to exist, for hearing, and after notice thereof to parties interested and a hearing, if in its judgment the public health so requires, shall, by an order served upon the party, company, or premises so polluted, prohibit the deposit, keeping, or discharge of any such cause of pollution, and shall order him to desist therefrom and to remove any such cause of pollution; but the board shall not prohibit the cultivation or use of soil in the ordinary methods of agriculture if no human excrement is used therefor.

Said board shall not prohibit the use of any structure which was in existence at the time of the passage of this act upon a complaint made by the board of water commissioners of any city, town, or village, or by any water or ice company, unless such board of water commissioners or company files with the State board a vote of its city council, selectmen, trustees, or bailiffs, or company, respectively, that such city, town, village, or company will, at its own expense, make such change in said structure or its location as said board shall deem expedient. Such vote shall be binding on such city, town, village, or company. All damages caused by such change shall be paid by such city, town, village, or company, and if the parties can not agree thereon such city, town, village, or company shall tender to the parties sustaining damages such a sum of money as in their judgment is a reasonable compensation for the damages sustained. Whoever is aggrieved by an order under the provisions of the preceding section, or with the sum so tendered as damages, may appeal therefrom in the manner provided in Vermont statutes, sec. 3314 to 3317, inclusive, relating to highways. But the notice therein provided for shall be served on the party or parties who are petitioners in fact under section 6 of this act, and also upon the State board of health. If the appeal be only from the compensation for damages, the order of the board shall be complied with during the pendency of such appeal unless otherwise authorized by said board.

SEC. 7. The court of chancery shall have jurisdiction and power, upon application thereto by the State board of health or any party interested, to enforce its orders, or the orders, rules, and regulations of said board of health, and to restrain the use or occupation of the premises or such portion thereof as said board may specify, on which said material is deposited or kept or such other cause of pollution exists, until the orders, rules, and regulations of said board have been complied with.

SEC. 8. Said board of health may by itself, its servants and agents, enter any building, structure, or premises for the purpose of ascertaining whether sources of pollution or danger to the water supply there exist and whether the rules, regulations, and orders aforesaid are obeyed.

SEC. 9. Whoever violates any rule, regulation, or order made under the provisions of section 2 or section 6 of this act shall be punished for each offense by a fine of not more than five hundred dollars to the use of the State, or by imprisonment for not more than one year, or by both such fine and imprisonment.

SEC. 10. No sewage, drainage, refuse, or polluting matter of such kind and amount as either by itself or in connection with other matter will corrupt or impair the quality of the water of any pond or stream used as a source of ice or water supply by a city, town, village, public institution, or water company for domestic use, or render it injurious to health, shall be discharged into any such streams, ponds, or upon their banks.

SEC. 12.[a] The court of chancery, upon the application of a mayor of a city, the selectmen of a town, the trustees or bailiffs of an incorporated village, the managing board or officer of a public institution, or a water or ice company interested, shall have jurisdiction in equity to enjoin the violation of the provisions of section 10.

SEC. 13. Whoever wilfully deposits excrement or foul or decaying matter in water which is used for the purpose of domestic water supply or on the shore thereof within five rods of the water shall be punished by a fine of not more than fifty dollars or by imprisonment for not more than thirty days; and a constable of a town or police officer of a city or village in which such water is wholly or partially situated may act within the limits of his city or town, and any executive officer or agent of a water board, board of water commissioners, public institution, or water company furnishing water or ice for domestic purposes, acting upon the premises of such board, institution, or company,

[a] Section 11 repealed.

and not more than five rods from the water, may without a warrant arrest any person found in the act of violating the provisions of this section and detain him until complaint may be made against him therefor. But the provisions of this section shall not interfere with the sewerage of a city, town, village, or public institution, or prevent the enriching of land for agriculture by the owner or occupant thereof.

SEC. 14. Each member of the State board of health shall receive four dollars per day and actual expenses while in the discharge of the duties imposed by this act. The State auditor is directed to draw his order on the State treasurer every six months for such sums as are necessary to meet the expenses of said board under the provisions of this act.

Approved December 12, 1902.

GENERAL RULES.

The foregoing compendium of common and statute law may be summarized and stated in a few general rules, which will perhaps be useful to property owners and also to officers charged with the duty of protecting health and property rights in waters.

In the nature of the case these rules can be only general, and many exigencies will appear in which more particular instructions must be obtained from the consultation of text-books and decisions or from the advice of counsel.

I. RIGHTS AND DUTIES OF RIPARIAN OWNERS.

Every riparian owner has the right—

1. To use the waters of streams, navigable or otherwise, which flow across or along his property for the ordinary purposes incidental to domestic life and agriculture, including grazing.

2. To use such waters for water power and for all kinds of manufacturing purposes which do not sensibly diminish the quantity which flows on for the use of lower proprietors nor change the quality of the waters to any appreciable extent, nor interfere with the use of the stream, if navigable by the public.

3. To have such waters flow to him from the premises of higher proprietors not unreasonably diminished nor diverted nor rendered impure by the farming or domestic uses to which the waters are subjected by higher proprietors.

4. To have such waters flow to him not sensibly changed in quality by any manufacturing or other uses to which they may have been put by higher proprietors.

5. To have such waters flow to him in their natural bed, unpolluted by any deposits of filth or any other substance in the bed or channel

previously traversed by them. But 3, 4, and 5 do not apply to riparian owners in those States in which the doctrine of prior appropriation is the law. (See pp. 21–23.)

Conversely, it is the duty of every riparian owner—

1. To so guard his use of the waters of streams which flow across or along his property for domestic and agricultural purposes as not unreasonably to divert nor diminish nor render impure such waters.

2. To refrain from every use in manufacturing which will divert or sensibly diminish the quantity of the waters which flow onward to the lower proprietors or render them appreciably different in quality.

3. To refrain from depositing any filth or other substance in the bed of such streams in such a manner or to such an extent as will cause the waters to flow to the lower proprietors out of their natural bed or will in anywise pollute them or render them impure.

Where the doctrine of prior appropriation is in force the appropriator must confine his use of the appropriated water to the use for which he has appropriated it and take only so much as is reasonably necessary to accomplish that purpose. He may not pollute the stream wantonly, nor by using it for purposes not included in his appropriation. Subject to these restrictions, the prior appropriator has the right to divert from the stream and use as much of the water as is necessary to accomplish the purpose for which it was appropriated.

II. RIGHTS AND DUTIES OF MUNICIPAL CORPORATIONS.

Considered as corporate entities, municipal corporations have such rights and powers only as are conferred upon them by statute, either expressly or by necessary implication.

When, under due authority, they become the owners of lakes, reservoirs, and natural streams, they have the same rights to pure water, and are charged with the same duties as are other riparian proprietors.

If authorized to construct a system of sewers draining into a stream, such authority does not exempt them (except in the State of Indiana) from the duty not to pollute the stream to the damage of lower proprietors.

The rights of property owners, specified in 3, 4, and 5 above are property rights and can not be taken away from owners for public use except upon payment therefor of an amount determined by constitutional condemnation proceedings authorized by statute.

Therefore, until municipal corporations have, by such proceedings, acquired the rights of all lower proprietors and paid for them, they are required in all cases to refrain from the pollution of streams to the same extent as private owners.

III. RIGHTS AND DUTIES OF THE PUBLIC.

By "the public" is meant that indefinite number of individuals, whether larger or smaller, who occupy as a common habitation a neighborhood, village, town, State, or country. Rights and duties which affect inhabitants of the neighborhood, village, town, State, or country as a whole, or a considerable but indefinite number of them, are called "public" rights and duties.

The public, in this sense, aside from the right to use navigable waters for commerce, has the right to enjoy the natural waters and the air which passes over them, so far as life and health are affected by these elements, in a condition so near that in which nature left them that their use will not destroy nor threaten life nor injure health.

And, reciprocally, the public, and each member of it, is charged with the duty not to pollute the natural waters upon which the community depends for life and health in any manner that will render the continued use of the waters, or of the air which passes over them, destructive of or injurious to the life or health of the community.

PUBLIC RIGHTS AND DUTIES ENFORCED BY STATUTE.

The rights and duties attempted to be expressed under III have received some recognition by the courts apart from statutory enactments. They have been enforced chiefly, however, through legislation. These rights and duties have received full recognition, and an active effort has been made to provide an efficient sanction for their enforcement by the legislatures of all the States included in Class II and Class III, as hereinbefore stated. These classes include thirty-eight of the States and Territories.

These statutes, not being in derogation of common-law rights, have been construed as remedial statutes and not unconstitutional, although in some cases they may seem to interfere with prescriptive rights. No one can acquire by prescription a right to do an act which menaces public health or destroys public comfort.

PROGRESS OF LEGISLATION.

It will have been noticed that public opinion, as expressed in public laws, is steadily progressing in the direction of a full, complete, and comprehensive enforcement of all the rights and duties of riparian owners, of municipal corporations, and of the public, as summarized above. Each advance in statutory regulation is an advance in that direction, and more especially in the direction of regulating and enforcing public rights and municipal rights and duties.

Private owners, from time immemorial, have been active in protecting their riparian rights as against other private owners. But the effect of pollution upon public health has not, until a comparatively recent period, been brought prominently into notice. The pollution of streams by cities and private persons has, accordingly, not received the attention which it deserved. This state of affairs is now rapidly passing away. Courts have shown themselves fully alive to the existence and validity of public rights in that respect, and the legislatures in Class III, comprising the States of Connecticut, Massachusetts, New Hampshire, New York, New Jersey, Minnesota, Vermont, and Pennsylvania, which has come into this class by legislation enacted in 1905, have made enactments calculated so to control such pollution as eventually to prevent all danger to public health.

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O

PUBLICATIONS OF UNITED STATES GEOLOGICAL SURVEY.

[Water-Supply Paper No. 152.]

The serial publications of the United States Geological Survey consist of (1) Annual Reports, (2) Monographs, (3) Professional Papers, (4) Bulletins, (5) Mineral Resources, (6) Water-Supply and Irrigation Papers, (7) Topographic Atlas of United States—folios and separate sheets thereof, (8) Geologic Atlas of United States—folios thereof. The classes numbered 2, 7, and 8 are sold at cost of publication; the others are distributed free. A circular giving complete lists may be had on application.

Most of the above publications may be obtained or consulted in the following ways:

1. A limited number are delivered to the Director of the Survey, from whom they may be obtained, free of charge (except classes 2, 7, and 8), on application.

2. A certain number are allotted to every member of Congress, from whom they may be obtained, free of charge, on application.

3. Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be had at practically cost.

4. Copies of all Government publications are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted by those interested.

The Professional Papers, Bulletins, and Water-Supply Papers treat of a variety of subjects, and the total number issued is large. They have therefore been classified into the following series: A, Economic geology; B, Descriptive geology; C, Systematic geology and paleontology; D, Petrography and mineralogy; E, Chemistry and physics; F, Geography; G, Miscellaneous; H, Forestry; I, Irrigation; J, Water storage; K, Pumping water; L, Quality of water; M, General hydrographic investigations; N, Water power; O, Underground waters; P, Hydrographic progress reports. This paper is the twelfth in Series L, the complete lists of which follow. (PP=Professional Paper, B=Bulletin, WS=Water-Supply Paper.)

SERIES L—QUALITY OF WATER.

WS 3. Sewage irrigation, by G. W. Rafter. 1897. 100 pp., 4 pls. (Out of stock.)

WS 22. Sewage irrigation, Pt. II, by G. W. Rafter. 1899. 100 pp., 7 pls. (Out of stock.)

WS 72. Sewage pollution near New York City, by M. O. Leighton. 1902. 75 pp., 8 pls.

WS 76. Flow of rivers near New York City, by H. A. Pressey. 1903. 108 pp., 13 pls.

WS 79. Normal and polluted waters in northeastern United States, by M. O. Leighton. 1903. 192 pp., 15 pls.

WS 103. Review of the laws forbidding pollution of inland waters in the United States, by E. B. Goodell. 1904. 120 pp.

WS 108. Quality of water in the Susquehanna River drainage basin, by M. O. Leighton, with an introductory chapter on physiographic features, by G. B. Hollister. 1904. 76 pp., 4 pls.

WS 113. Strawboard and oil wastes, by R. L. Sackett and Isaiah Bowman. 1905. 52 pp., 4 pls.

WS 121. Preliminary report on the pollution of Lake Champlain, by M. O. Leighton. 1905. 119 pp., 13 pls.

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WS 151. Field assay of water, by M. O. Leighton. 1905. 77 pp., 4 pls.

WS 152. A review of the laws forbidding pollution of inland waters in the United States, second edition, by E. B. Goodell. 1905. 149 pp.

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OCTOBER, 1905.

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Reference.

U. S. Dept. of the Interior.

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Water-Supply and Irrigation Paper No. 158

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DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE UNDERFLOW IN ARKANSAS VALLEY IN WESTERN KANSAS

BY

CHARLES S. SLICHTER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1906

CONTENTS.

ILLUSTRATIONS.

THE UNDERFLOW IN ARKANSAS VALLEY IN WESTERN KANSAS.

By Charles S. Slichter.

INTRODUCTION.

The investigation of the underflow of Arkansas River, described in this paper, was made during the summer of 1904. The field party was under the general supervision of the writer. Mr. Henry C. Wolff had charge of the measurements of the rate of movement of the ground waters. He also made careful determinations of the fluctuation of the position of the water plane, and the success of the field work was largely due to his skill and hard work. Mr. Ray Owen had charge of level and plane-table work, and made a contour map of the water plane.

A few of the principal conclusions may be summarized as follows:

1. The underflow of Arkansas River moves at an average rate of 8 feet per twenty-four hours, in the general direction of the valley.
2. The water plane slopes to the east at the rate of 7.5 feet per mile, and toward the river at the rate of 2 to 3 feet per mile.
3. The moving ground water extends several miles north from the river valley. No north or south limit was found.
4. The rate of movement is very uniform.
5. The underflow has its origin in the rainfall on the sand hills south of the river and on the bottom lands and plains north of the river.
6. The sand hills constitute an essential part of the catchment area.
7. The influence of the floods in the river upon the ground-water level does not extend one-half mile north or south of the channel.
8. A heavy rain contributes more water to the underflow than a flood.
9. On the sandy bottom lands 60 per cent of an ordinary rain reaches the water plane as a permanent contribution.
10. The amount of dissolved solids in the underflow grows less with the depth and with the distance from the river channel.

11. There is no appreciable run-off in the vicinity of Garden, Kans. Practically all of the drainage is underground through the thick deposits of gravels.

12. Carefully constructed wells in Arkansas Valley are capable of yielding very large amounts of water. Each square foot of percolating surface of the well strainers can be relied upon to yield more than 0.25 gallon of water per minute under 1 foot head.

13. There is no indication of a decrease in the underflow at Garden in the last five years. The city well showed the same specific capacity in 1904 that it had in 1899.

14. Private pumping plants in the bottom lands will be profitable for irrigation if proper kind of power be used. There should be a large field of usefulness for suction gas-producer power plants of from 20 to 100 horsepower, with Colorado hard coal or coke as fuel. Kansas crude oil in gas generators should prove profitable for use in the smaller plants. The present cost of pumping with gasoline for fuel is not encouraging.

CHAPTER I.

MEASUREMENTS OF THE UNDERFLOW OF ARKANSAS RIVER.

GENERAL STATEMENT.

Investigations of the underflow of Arkansas River were begun June 11, 1904. The work consisted of the mapping of the water plane or ground-water level within a distance of 6 to 12 miles from the river channel, and of observations by the electrical method of the rate of movement of the underflow. The ground-water levels were obtained by observing the water levels in private wells in the neighborhood of the river and in a few wells which were sunk especially for this purpose. The slope of the water plane was found to be between 7 and 8 feet to a mile in a general easterly direction, and from 2 to 3 feet to a mile toward the river channel from the country immediately to the north and south. The southern margin of the river valley is bordered for 5 to 10 miles to the south by sand hills, which are only partially covered with natural vegetation. These sand hills extend from east of Dodge, Kans., to beyond the Colorado line. The river valley proper varies in width from 1 to 5 miles. Near the river channel there is a strip known as "first bottoms," which is only a few feet above the river level. The principal cultivated portion of the valley lies from 3 to 8 feet higher than first bottoms, and is locally known as "second bottoms." North of the river valley the ground rises rather abruptly to the high plains with their well-known level topography and compact sod of native grasses. The slope of the water plane toward the channel of the river from the north is, as has been stated, about 2½ feet to a mile, but 10 to 14 miles to the north of the valley the slope of the water plane changes from southerly to northerly, and the land at the same time gently dips to the north toward the valley of White Woman Creek. The easterly slope of 7½ to 8 feet to the mile is maintained, however, quite constantly throughout all of this region. Fig. 1 shows the results of the determination of the water plane.

MEASUREMENTS 2 MILES WEST OF GARDEN, KANS. (CAMP 1).

The measurements showed a rate of movement much greater than had been anticipated. The first set of underflow stations were established at a point about 2 miles west of Garden (camp 1), as shown on the map (fig. 1). The stations were in a north-south line, which was about 1½ miles in length. At this point the river flows in an east by south direction, and borders closely on the north margin of the sand

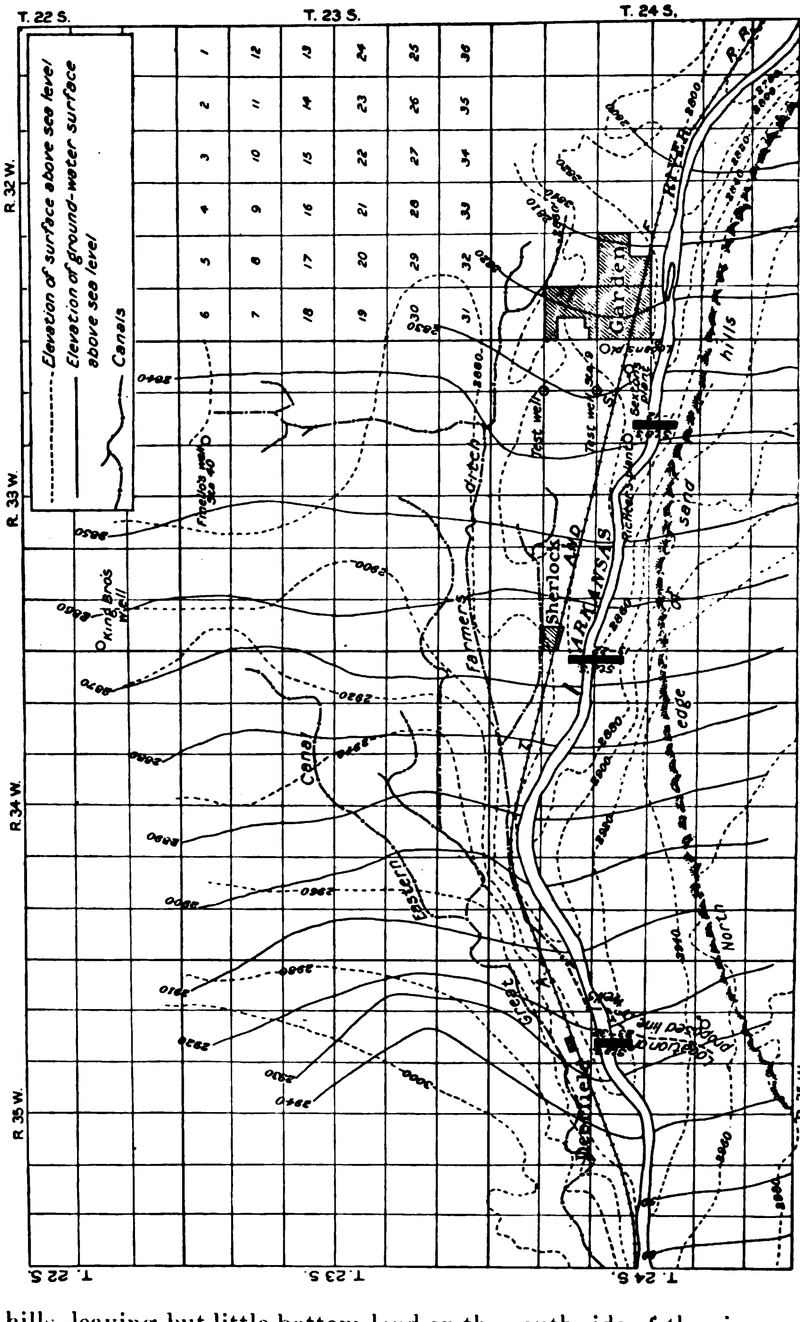


FIG. 1.—Map showing the water plane or ground-water surface between Garden and Deerfield, Kans., as determined during the summer of 1901. Underflow stations shown west of Garden, and near Sherlock and Deerfield, Kans.

hills, leaving but little bottom land on the south side of the river. The channel of the river where the observations were made is about 1,000 feet wide. On the north side is a strip of low land, or first bottoms, about 1,100 feet wide, which is only a few inches above the general bottom of the river bed. This low bottom has several sloughs running through it approximately parallel to the river. North of this low strip of bottom the land abruptly rises several feet and continues

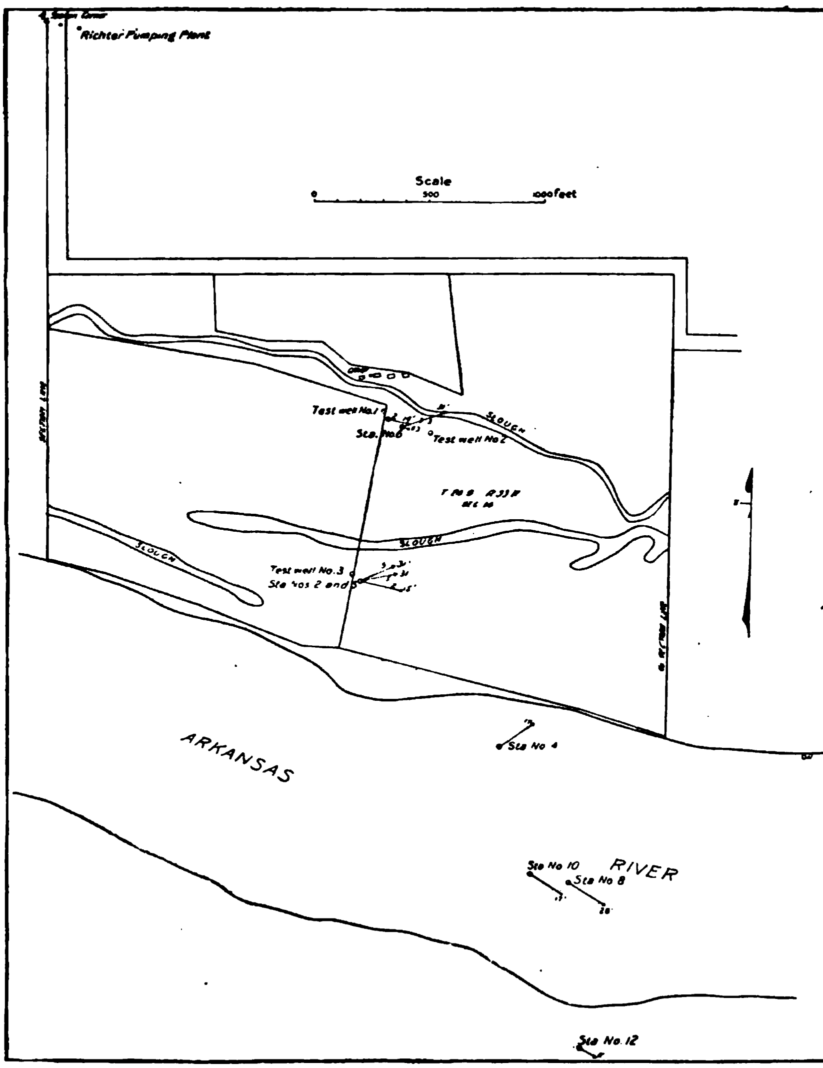


FIG. 2.—Map showing location of underflow stations and test wells at camp 1, 2 miles west of Garden, Kans. The velocity and direction of flow is shown by the length and direction of the arrows at the various stations. The depth is indicated in figures at each location.

to rise gradually for several miles farther north, this slope constituting the cultivated portion of the valley—the so-called second bottoms.

The measurements at this point were made at stations that lay, in general, in a straight line across the valley (fig. 2). Most of the measurements were made in the river channel itself, or on the low ground to the north. One test was made on the south side of the river at the foot of the sand hills and another 1 mile to the north. The velocities were determined at depths ranging from 11 to 65 feet. The results of the measurements at this location are given in Table 1.

TABLE 1.—*Underflow measurements at camp 1, 2 miles west of Garden, Kans.*

Date of test.	No. of station.	Depth of well.	Velocity of ground water.	Direction of flow, east of north.	Location and remarks.
1904.		*Feet.*	*Ft. per day.*	°	
June 30	9	16	5.3	90	1 mile north of river.
June 22	1	14	4.8	101	1,100 feet north of river.
Do	3	31	10.3	71	Do.
June 21	2	15	9.6	103	430 feet north of river.
June 24	5	31	8.0	65	Do.
June 26	5	29	8.0	77	Do.
June 25	4	17	9.0	55	In channel, 250 feet north of center.
July 6	8	28	9.6	121	In channel, 150 feet south of center.
July 4	10	17	8.2	121	Do.
July 9	12	11	4.0	120	250 feet south of river.
September 6	6	65	1.75	101	1,100 feet north of river.
September 8	40	25	1.3	104	NW. corner SW. ¼ sec. 2, T. 23 S., R. 33 W., 8¼ miles north of river.
Average			6.6	94	

Mean direction of river channel, 100° east of north.

Of the stations for which data are given in this table, No. 9 was located on the second bottoms 1 mile north of the river, No. 40 was located on the uplands 8½ miles north of the river, and No. 12 was in the sand hills south of the river. The other stations were either in the first bottoms or in the channel. Station No. 6 reached so-called "second water," or the water beneath a layer of silt which seemed quite impervious to the flow of water. The mean of all of the observed velocities was 6.6 feet a day. The average direction of the motion was 94° east of north, which may be compared to the average direction of the river valley at this point, which we have estimated to be approximately 100° east of north. On the cross section through the river channel and the first bottoms (fig. 3) are shown the depth of a number of the test wells near the river channel and the velocity of the underflow.

Except for occasional layers of silt, the gravels were very uniform in size and character of grain; a large percentage of any one sample consisted of grains larger than grains of wheat. The gravel was also found to be very uniform in lateral extent, but showed a tendency to become coarser with the depth until 32 feet was reached. At about 32 feet fine sand and silt was encountered, which seemed, as nearly as could be determined from the wells sunk in a comparatively small radius, to be horizontal in extent. Fine material was encountered at a higher level at only one place, which was near the center of the river at a depth of about 18 feet, but 50 feet upstream it was entirely absent.

A well was put down at station No. 11 in order to secure a sample of this fine material. It was found at the same level as at stations No. 6 and No. 8, and consisted of about the same kind of material, except that it contained a considerable amount of gypsum mixed with sand. This fine sand must be more or less impervious, for no water could be drawn by means of a hand pump from a well driven in the sand, and a hole washed out 8 feet below the casing remained for a considerable time unfilled with sand.

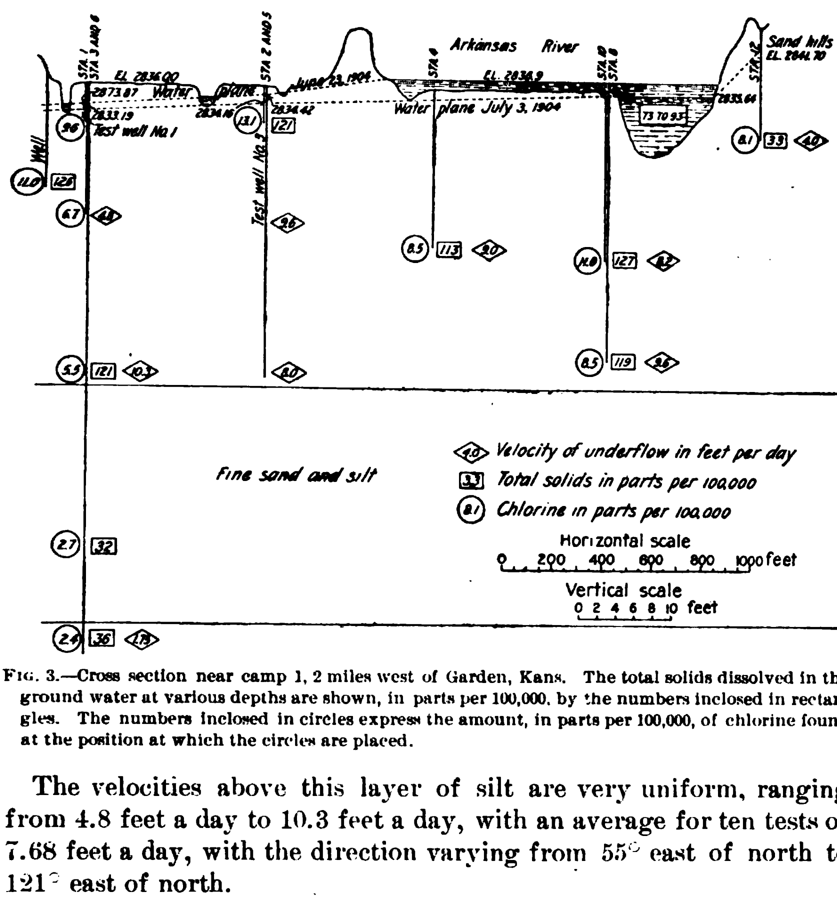


FIG. 3.—Cross section near camp 1, 2 miles west of Garden, Kans. The total solids dissolved in the ground water at various depths are shown, in parts per 100,000, by the numbers inclosed in rectangles. The numbers inclosed in circles express the amount, in parts per 100,000, of chlorine found at the position at which the circles are placed.

The velocities above this layer of silt are very uniform, ranging from 4.8 feet a day to 10.3 feet a day, with an average for ten tests of 7.68 feet a day, with the direction varying from 55° east of north to 121° east of north.

The direction of motion at these various stations, as has been stated, was in general toward the east, but several exceptions were noted from time to time. At the time field work was begun the channel of the Arkansas River was dry, as is very usual in the months from June to October. The summer of 1904, however, proved to be an exceptional one, and high floods were of constant occurrence throughout the season. One of these floods came down the river soon after the first underflow stations were established near the bank of the river. This offered an excellent opportunity of determining the influence of the river waters upon the underflow. At one underflow station, situated near the north bank of the channel of the river, 2 miles west of Garden, the direction of the flow of the ground waters was very greatly changed by the flood in the river. It was therefore possible to measure the rate at which the river contributed to the ground waters at this point. It was found that the water during the early stages of the flood flowed away from the river at the rate of 6 to 8 feet per twenty-four hours. This point can be established by consulting the record for stations No. 2 and No. 5, as given in Table 1. These stations are located at the same point. The velocity at station No. 2 on June 21, 1904, before a rain on the night of June 21, and before a flood which came down the river at 3 p. m. June 22, was 9.6 feet per twenty-four hours in a direction 103° east of north, which is substantially the direction of the river channel. After the flood the velocity at the same place (at a greater depth, however) was found to be 8 feet per twenty-four hours, in a direction 65° east of north, or at an angle of 35° away from the river channel, the flood having therefore changed the former direction of flow by about 38°. On June 26, when the flood had still further receded, a second determination of velocity showed the same rate as before, but the direction had shifted to 77° east of north, or at an angle of about 23° with the river channel.

It was not only possible to actually determine this rate of loss of water from the river by the use of the electric underflow meter, but the northerly progress of the water from the river into the gravels could be noted by observation of the changes in the temperature of the ground water as it flowed north. The river water was much warmer than the natural ground water, and the increased temperature could be followed away from the river bank. These facts are shown by the temperatures of the water recorded in Table 11. In that table will be found the following entries:

Temperature of water of river and test wells, June 20, 1904.

	°F.
River	71
Test well No. 3, 360 feet north of river	62.5
Test well No. 1, 1,100 feet north of river	59

The water taken from the other wells had a somewhat more uniform temperature, excepting in two cases—that taken from the wells at station No. 10 and station No. 8. At station No. 10, at a depth of 18 feet, the temperature was 51°; at station No. 8, 28 feet below the bottom of the river, the temperature was 48°, which was the coldest water found at any point. At these two stations the direction of the underflow was the most southerly of any found, being in each case 121° east of north.

It was also possible to partially trace inward moving ground water originating in the river by the change in the chemical composition of the water. Apparatus was at hand for determining the alkalinity, hardness, chlorine, and the total solids dissolved in the water; and this apparatus was used to secure the results just stated. A further verification of the inwardly moving ground water was found in the changed slope of the water plane during the flood periods in the river. The water plane sloped away from the river about 8 feet to the mile during the first stages of high water, and corresponded quite accurately with the observed velocities of the water. Fig. 3 shows the slope of the water plane on June 23 and July 3. Several gradients corresponding to other dates are given in Table 1.

MEASUREMENTS AT SHERLOCK, KANS. (CAMP 2).

Several underflow measurements were taken at camp 2, which was situated at Sherlock, Kans., 7 miles west of Garden. The results differed little from those found at the first set of stations at camp 1, except that more sorting of the gravels had taken place at the latter point, giving greater variety to the rate of movement. The location of the various test wells and underflow stations is marked in fig. 4. The same stations are shown in cross section in fig. 5. The details of the results are printed in Table 2. From this table it will be observed that the average velocity of the underflow for all of the stations was 8.9 feet per twenty-four hours. The mean direction of the motion was 93.5° east of north, which may be compared with the mean direction of the river valley at this point, which was computed to be 105° east of north. There was some water in the river throughout all of the time during which the tests were made, and on July 27 a heavy flood swept down the river.

TABLE 2.—*Underflow measurements at camp 2, Sherlock, Kans.*

Date of test.	No. of station.	Depth of wells.	Velocity of ground water.	Direction of flow, east of north.	Location and remarks.
1904.		*Feet.*	*Ft. per day.*	°	
July 16	13	18	5.7	64.0	700 feet north of river.
July 30	21	28	22.9	64.0	Do.
July 31	22	28	2.8	101.0	1,700 feet north of river.
July 17	14	22	9.1	75.0	In channel, 500 feet north of center.
July 23	18	21	16.0	101.0	In channel, 20 feet north of center.
July 22	17	36	3.0	103.0	In channel, 210 feet south of center.
July 18	15	22	16.7	132.0	Do.
July 29	20	26	2.2	122.0	200 feet south of river.
July 22	16	18	2.0	79.0	2,100 feet south of river.
Average			8.9	93.5	

Mean direction of river channel, 105° east of north.

By studying the results of the measurements it will be observed that station No. 22 was on the border of the second bottoms, 1,700 feet north of the north bank of the river. The velocity at this station was 2.8 feet per day, and the direction of flow was substantially the same as the direction of the river valley. This result is important, as the measurement was made on July 31, at a time when the

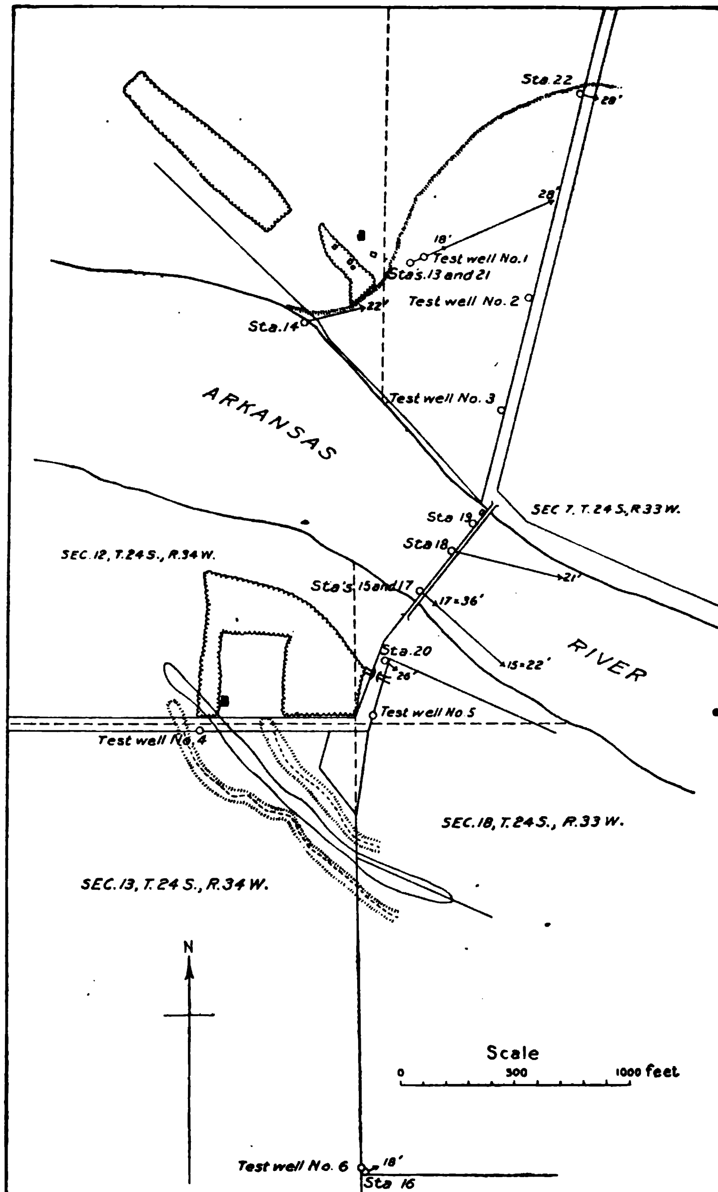


FIG. 4.—Map showing location of underflow stations and test wells at Sherlock, Kans., 7 miles west of Garden. The velocity and direction of flow of the ground water are shown by the length and direction of the arrows at the various stations. The depth is indicated in figures at each station.

flood of July 27 should have shown some influence upon the direction of flow, if it had any at all. The direction of motion at this station was in marked contrast to the direction of flow observed at stations

No. 13 and No. 21, located in the first bottoms, 700 feet north of the river. At both of the latter stations the direction of flow was 64° east of north, or in a direction making an angle of 41° northeast of the general direction of the river valley. These stations were within the immediate influence of the fluctuations of the height of the water in the river.[a]

Of the stations established in the channel of the river itself, it is interesting to note that a station located north of the center of the

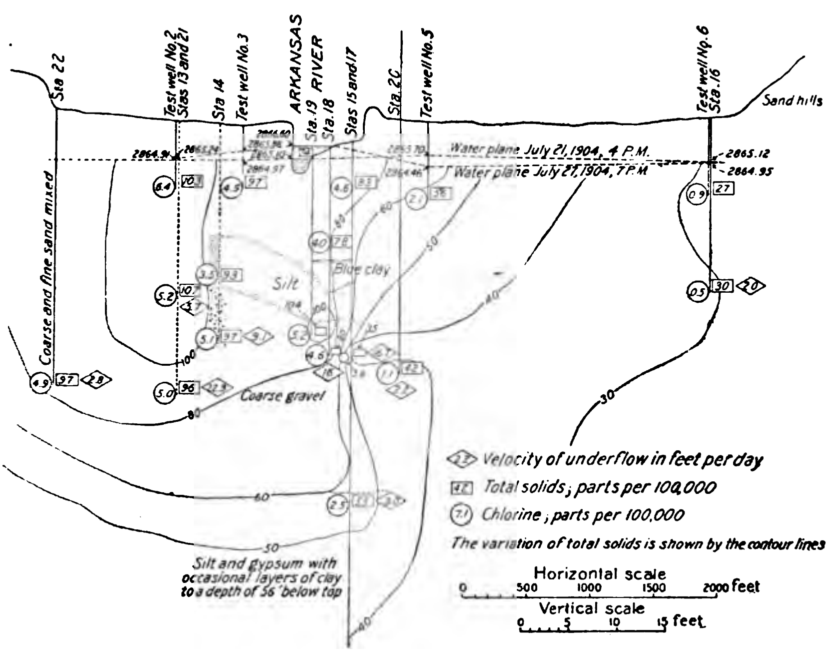


FIG. 5.—Cross section at camp 2, near Sherlock, Kans. The total solids dissolved in the ground water at various depths are shown in parts per 100,000 by the numbers inclosed in rectangles. The numbers inclosed in circles express the amount, in parts per 100,000, of chlorine found at the position where the circles are placed. The contour lines show the position of water of the same strength. The contribution of soft water from the sand hills is very apparent.

channel (station 18) showed a component of velocity northerly to the general trend of the valley, while a station south of the channel (station 15) showed a component of velocity southerly to the direction of the valley. At station No. 17, in the channel at the same point as station No. 15, but at a greater depth, the direction of the flow corresponded closely with the direction of the valley, indicating that the influence of flowing water in the river did not extend so deep. Station No. 20 was located on the first bottoms, 200 feet south of the south bank of the river. The motion at this point showed a southerly component, the direction of flow making an angle of 17° with the direction of the valley. The measurement was taken while the river was in

[a] This fact will be further illustrated at a later place in this report.

flood. Station No. 16 was located in the border of the sand hills, nearly a half mile south of the river. The direction of flow was toward the river and away from the sand hills, as should be expected on account of the excellent collecting area offered by the sand hills to the rainfall.

The fact that the influence of the river only extends to very shallow depths and that a considerable portion of the ground water originates in the sand hills is shown by the cross section (fig. 5). The contour lines in this figure correspond to equal amounts of total solids dissolved in the ground water. The soft water from the sand hills can be observed to be crowding the strong water of the underflow to the north of the valley.

MEASUREMENTS AT DEERFIELD, KANS. (CAMP 3).

Camp 3 was established near the Deerfield bridge, 14 miles west of Garden. The valley at this point lies mostly south of the channel. All of the south-side lands, to the edge of the sand hills, would probably be classed as "first bottoms." The surface of the ground on these lands is only a few feet above the river bed and the soil is unusually sandy. The topography of the sand hills south of the bottom lands is unusually well adapted for collecting the rainfall, there being several level stretches inclosed or hemmed in by the hills. A short distance south of station No. 23 there are found the remains of a former river bank, indicating that an ancient channel extended as far south as station No. 23 (see fig. 6).

On the north side of the channel the river sweeps a high bank from 6 to 10 feet above the river bed for a distance of about 3 miles. The uplands begin not more than 1 mile north of the river.

Since the channel here borders the extreme north margin of the valley the underflow measurements were made south of the river or in the channel. The results are printed in Table 3.

TABLE 3.—*Underflow measurements at camp 3, Deerfield, Kans.*

Date of test.	No. of station.	Depth of wells.	Velocity of ground water.	Direction of flow east of north.	Location and remarks.
1904.		*Feet.*	*Ft. per day.*	°	
August 6	25	16	6.3	66.0	In channel at center.
Do	24	21	12.5	67.0	In channel 400 feet south of center.
August 5	23	24	19.2	111.0	500 feet south of river.
August 8	26	36	9.2	111.0	Do.
August 9	27	24	14.8	129.0	1,050 feet south of river
August 12	28	21	1.25	74.0	1,800 feet south of river.
September 22	29	17	1.6	56.0	1.8 miles south of river.
August 17	32	31	2.2	63.0	1,800 feet south of river.
Average			8.4	84.6	

Mean direction of river channel, 70° east of north.

The average velocity of the ground water, 8.4 feet per twenty-four hours, compares accurately with the average velocities found for stations similarly located at previous camps. The mean direction does not correspond as accurately with the general trend of the river

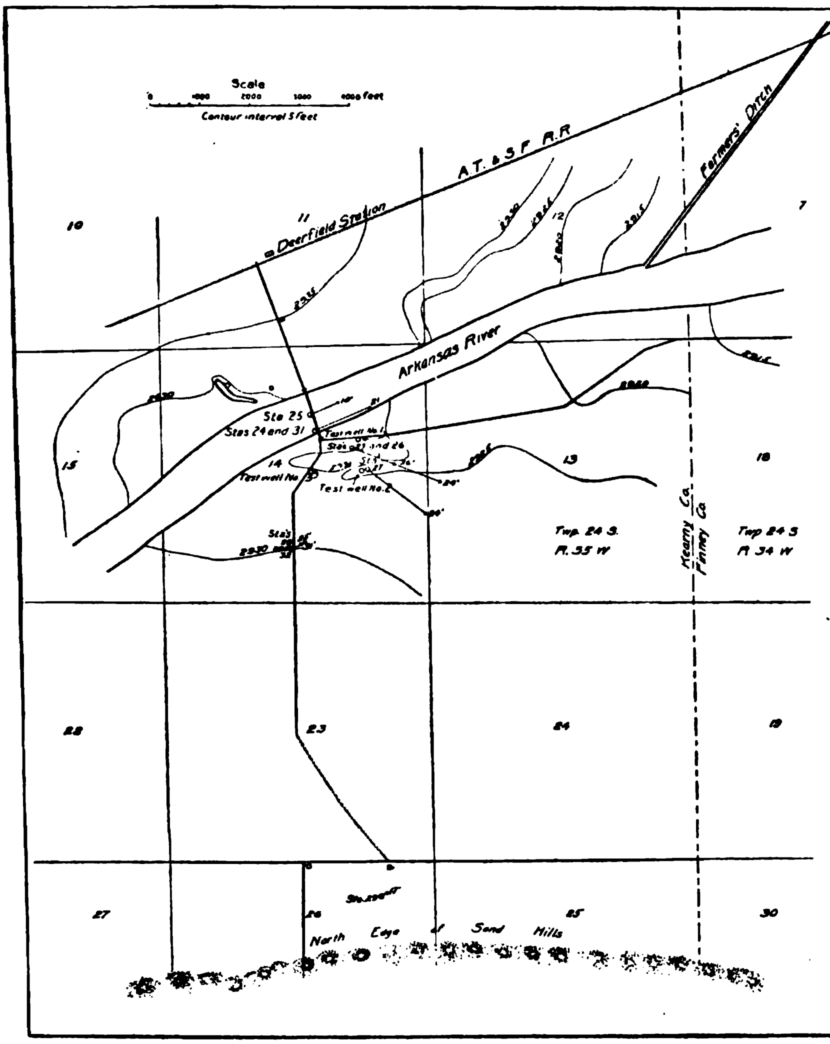


FIG. 6.—Map showing the location of underflow stations at camp 3, near Deerfield, Kans. The velocity and direction of flow of the ground water is shown by the length and direction of the arrows. The depth is indicated in figures at each station.

channel as at other stations, probably in part owing to the fact that the river has at this point a very northerly course.

It will be observed that the direction of flow at stations Nos. 23, 26, and 27, which are, respectively, 500, 500, and 1,050 feet south of the river, had a strong southerly component, the resultant direction of motion making angles in the three cases of 41°, 41°, and 59°, respectively, away from the river. These are to be contrasted with the direction of motion nearer the sand hills, at stations Nos. 29 and 32, where the direction of flow was away from the sand hills and toward the river, the direction of flow in the two cases making angles of 14° and 7°, respectively, toward the channel of the river.

MEASUREMENTS AT CLEAR LAKE, NEAR HARTLAND, KANS. (CAMP 4).

About 2½ miles southeast of Hartland, Kans., in section 13, T. 25 S., R. 37 W., there is situated a small body of water called Clear Lake. This pond is nearly circular, 320 feet in length and 280 feet across at the narrowest point. The pond is located within 500 feet of the south-side ditch, and the owners of the canal have had under serious consideration the erection of a pumping plant to take water from the pond to supply the ditch with water for irrigation. It was expected by the promoters of this scheme that the lake would act as an enormous well and would furnish a large amount of water when its level was lowered by means of large centrifugal pumps.

There have been the usual rumors current among the settlers to the effect that the pond was very deep, and that its elevation was independent of the amount of rainfall or the fluctuations in the river, which at this point is about 1 mile northwest of the pond. Investigations showed that the water in the lake was 11 feet below the water in south-side ditch. The location of the lake with reference to the ditch and the topography near it is shown on the map, fig. 7. This is a 5-foot contour map of the district surrounding the lake, made from the level of the water in the pond as datum. Mr. H. E. Hedge, engineer of the south-side ditch, furnished the field party much assistance, and especially aided them in the construction of a raft from which to take soundings, so as to make a hydrographic map of the bottom of the lake. The shores slope at an angle of about 35° to a depth of 16 feet, where there is practically a flat level floor of mud. At this depth the diameter of the lake is about 100 feet. From this it can be computed that the total volume of the lake is 483,000 cubic feet, or that the lake contains about 11 acre-feet of water. The bottom of the lake consists of an accumulation of black muck, which is very soft. A test well was sunk in the center of the lake from the raft for the purpose of determining the character of the material at the bottom, so as to settle, as far as practicable, the question of whether the lake could be used as a large well from which to secure a supply of water. In sinking a 2-inch pipe for this purpose it was found that it would sink of its own weight to a depth of 30 feet. The pipe was then forced down without driving to a depth of 40 feet, after which it was easily jetted and driven to a depth of 62 feet below the water, or 46 feet under the bottom of the lake. In clearing the material from the 2-inch pipe 75 feet of wash pipe was used, so that samples were washed up from a depth of about 12 feet below the bottom of the 2-inch well. The material washed out consisted of black mud and clay, with some quicksand.

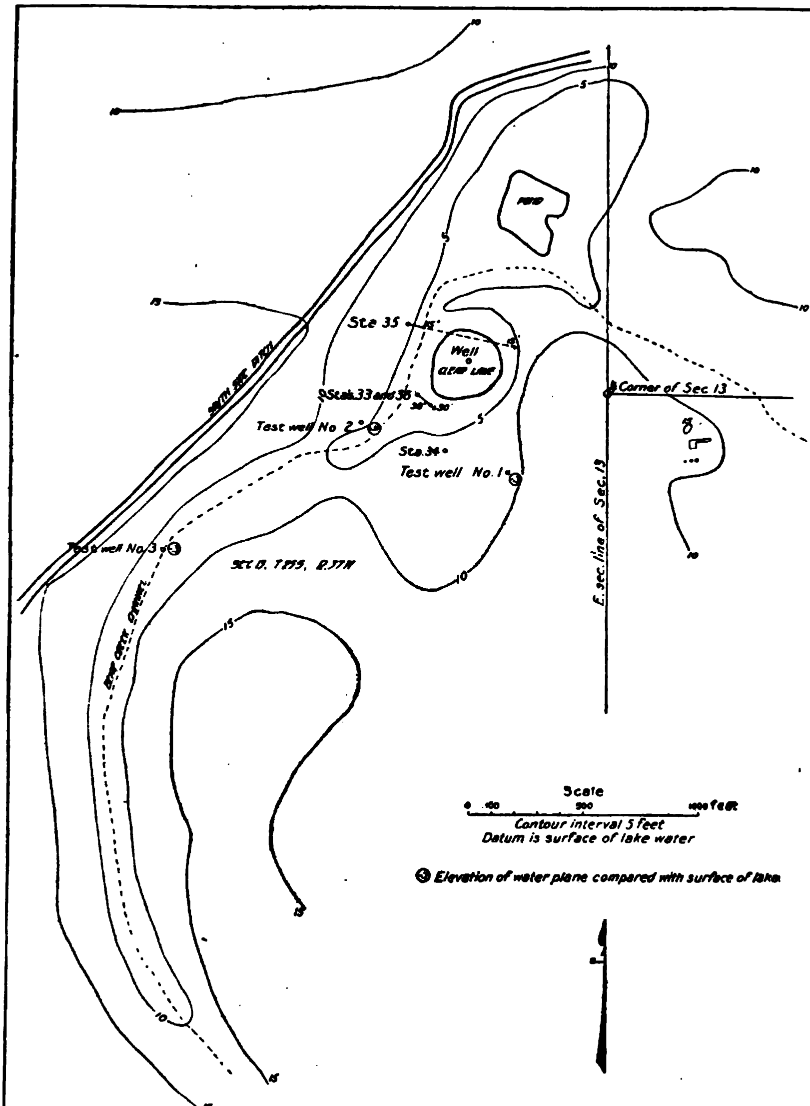


FIG. 7.—Map showing location of underflow stations and test wells near Clear Lake, Kansas.

A line of levels was run from Clear Lake to Arkansas River as nearly as practicable at right angles to the direction of the river channel. The result of this leveling showed that the river was at least 8 feet higher than the lake.[a]

[a] Field notes show that the river was quite high at the time of the observation on August 20, 1904.

This result was somewhat surprising, so that a second line of levels was run to the river along the east line of section 13 until this line intersected the river. This line of levels intersected the river at a point three-fourths of a mile below the former point. The river at this point was found to be 3 feet higher than the surface of Clear Lake. Since the river slopes about 7½ feet to the mile, this checks the former measurement that the river opposite the pond is 8 feet higher than the water in the latter.

The above observations seem to indicate that the small pond known as Clear Lake is one of the many circular depressions which are found throughout the western plains, and which have been fully described by Mr. Willard D. Johnson.[a]

This small pond is of especial interest because it is in line with the dry channel of a plains stream called Bear Creek. This stream rises in Colorado, and near the western border of Kansas has a well-marked valley, eroded to a depth of nearly 100 feet, but as it approaches Arkansas River, near the north edge of Grant County, it loses this, and its waters spread out on the plains and sink. The ordinary flow of this stream is very small, but during times of heavy rain in eastern Colorado and western Kansas it may carry a large quantity of water, which it pours out upon the high plains of northern Grant County and into the sand hills along the south side of Arkansas River. On some occasions the freshets in this stream have been so severe that the waters have nearly reached the Arkansas. There is a slight elongated depression extending through the sand hills in line with Clear Lake, which makes it possible to believe that the waters of Bear Creek have on some occasions in the past extended to the Arkansas, but so far as known there is no settler who can testify to having actually observed such an event.

It can easily be believed, from the rather remarkable character of Bear Creek, that settlers would naturally associate Clear Lake with the disappearing waters of Bear Creek, so that the story would become current that Clear Lake was merely an evidence or indication of the existence of an underground stream extending from the sand hills to Arkansas Valley itself. On this account belief in the adaptability of the lake for a supply of a large quantity of water for irrigation has been prevalent, so that an investigation of the conditions surrounding the lake has importance. There are several streams of the same type as Bear Creek in western Kansas.

Underflow stations Nos. 33, 35, and 36 were established, as shown on the map (fig. 7), for the purpose of determining the direction and magnitude of the velocity of the underground water. It was hoped to determine in this way whether or not there was any seepage at this point from the direction of Bear Creek toward Arkansas Valley. The

[a] The High Plains and their utilization: Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, pp. 609, 693-715.

direction and velocity of movement are indicated by the arrows shown in fig. 7, and the details of the measurement are given in Table 4. Station No. 33, 25 feet south of Clear Lake, gave a velocity of 5 feet a day; the direction was almost exactly across the dry channel of Bear Creek and in the general direction of Arkansas Valley. Station No. 36, located at the same place, but at a depth of 38 feet, showed a velocity of 4.3 feet in the same direction. Station No. 35, 150 feet northwest of Clear Lake, showed a velocity of 5 feet a day at a depth of 30 feet. The velocities observed at this point may have been due in part to seepage from the south-side ditch, as the direction was almost directly away from this ditch and in the general direction of the slope of the ground. Even if this be the case, it nevertheless proves that there is no seepage nor movement of ground water extending down the so-called channel of Bear Creek, for if there had been such motion the resultant velocity found would at least have shown a component of motion in the direction of the flow in the channel of Bear Creek. It would be impossible for the seepage from south side ditch to disguise completely a ground-water movement in another direction.

TABLE 4.—*Underflow measurements at camp 4, Clear Lake, near Hartland, Kans.*

Date of test.	No. of station.	Depth of wells.	Velocity of ground water.	Direction of flow, east of north.	Location and remarks.
1904.		*Feet.*	*Ft. per day.*	°	
August 19	33	30	5.0	74	25 feet southwest of Clear Lake.
August 20	35	15	3.1	101	150 feet northwest of Clear Lake.
August 21	36	38	4.3	74	25 feet southwest of Clear Lake.

An attempt was made to sink a set of wells at station No. 34, 230 feet south of Clear Lake. At this point wells were driven to a depth of 40 feet, but the material was so fine that no water could be pumped from the wells, except a very little at a depth of 16 feet. On this account no test was made.

It can easily be concluded from the tests made above that it is not feasible to use Clear Lake as a well from which a large quantity of water can be pumped for irrigation purposes. While Clear Lake undoubtedly has direct connection with the surrounding ground water and shows the level of the ground water in its neighborhood, the evidence from the character of the material encountered in stations Nos. 33, 35, and 36, and the evidence from direct observation of the flow of the water and the material encountered in the deep well sunk in the middle of the lake, show that the pond is not favorably situated for use as a source of a large supply of water for the south-side ditch.

These observations also show that no ground water reaches either Clear Lake or Arkansas River from the lost waters of Bear Creek. Any seepage water approaching Arkansas Valley from Bear Creek must take up a generally easterly movement almost immediately upon entering the sand hills.

MEASUREMENTS OF THE UNDERFLOW AT THE NARROWS OF ARKANSAS RIVER, NEAR HARTLAND, KANS. (CAMP 5).

Two miles west of Hartland, Kans., Arkansas River flows between rock bluffs, the distance between which at the narrowest portion is

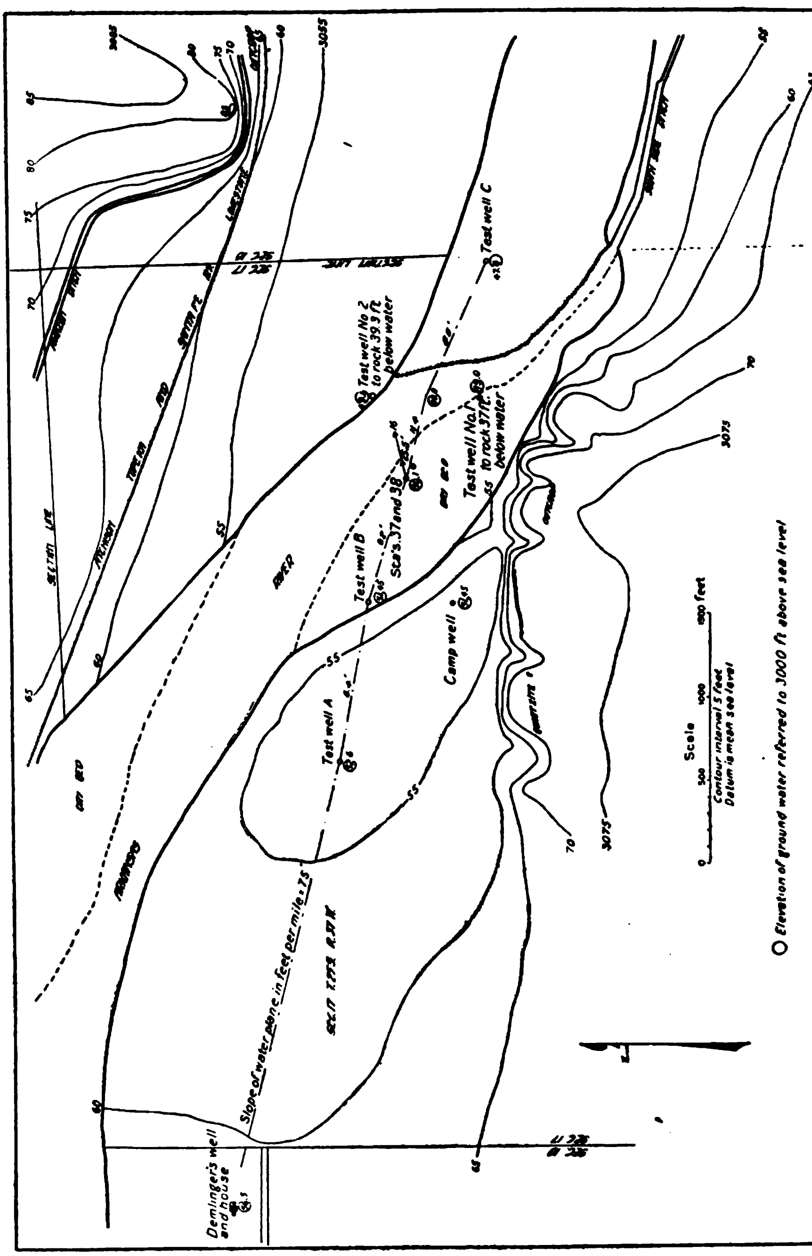


FIG. 8.—Map showing location of underflow stations and test wells near Hartland, Kans. Rock is exposed on both banks of the river at this point.

2,250 feet. The river channel occupies 900 feet of this distance, only a portion of which was utilized by flowing water on August 24, 1904.

Test wells A, B, and C were driven to shallow depths for the purpose of determining the slope of the water plane through the Narrows.

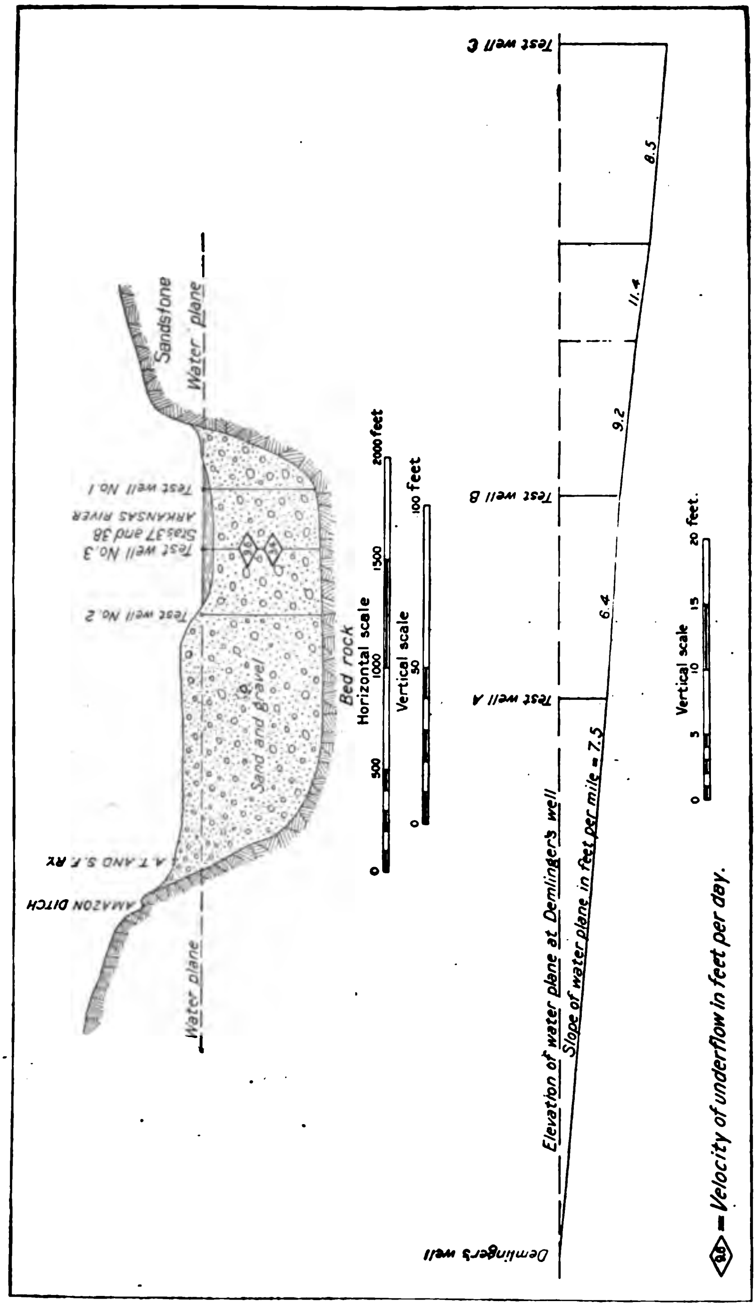


FIG. 9.—Cross section at the Narrows of Arkansas River, west of Hartland, Kans. The slope of the water plane through the Narrows is shown by the line at the bottom of the drawing.

In addition to these test wells, the elevation of the water was taken at Demlinger's well and in the wells of station No. 38, and in test wells driven for the purpose of testing for rock. These wells form a line about a mile long, as indicated on the map (fig. 8). The gradient of the water plane in the first portion of this line was 7.5 feet per mile: in the next portion it was 6.4 feet per mile, and in the next 9.2 feet per mile. Just above the Narrows the gradient was found to be 11.4 feet per mile, and in the last portion, in the Narrows itself, the slope of the water plane was 8.5 feet per mile. A profile showing these gradients is given at the bottom of fig. 9.

Test wells Nos. 1 and 2 (shown in fig. 8) were driven for the purpose of testing for bed rock. What is believed to be rock was struck at test well No. 1, at elevation 3,011.7, or 37 feet below the water plane, and at test well No. 2 rock was reached at elevation 3,009.8, or 39.3 feet below the water plane. Rock was also struck at station No. 38 at 38.75 feet below the water plane. As a diamond drill was not at hand, the evidence that bed rock was reached is, of course, not conclusive. The only test that could be applied was the evidence supplied by the drill on the wash pipe and by the way in which the 2-inch casing acted when an attempt was made to drive it.

Two measurements were made of the rate of movement of the underflow near the center of the Narrows at stations Nos. 37 and 38. The velocities determined were 9.6 feet per twenty-four hours at a depth of 16 feet and 3.4 feet per twenty-four hours at a depth of 25 feet.

TABLE 5.—*Underflow measurements at camp 5, Narrows of Arkansas River, near Hartland, Kans.*

Date of test.	No. of station.	Depth of wells.	Velocity of ground water.	Direction of flow, east of north.	Location and remarks.
1904.		*Feet.*	*Ft. per day.*	°	
August 23	37	16	9.6	77	Center of channel.
August 26	38	25	3.4	77	Do.

From the cross section of the Narrows (shown in fig. 9) an estimate can be made of the amount of water which flows through the Narrows. The total cross section of the sands, assuming the above test borings as indicating the true position of bed rock, is 75,000 square feet. Assuming one-third as the porosity of the sands and 10 feet per day as the average velocity of the ground water, the total flow through the Narrows would be 250,000 cubic feet per day, or 2.9 cubic feet per second. The actual average velocity of the underflow is undoubtedly much less than 10 feet per day, so that the above result represents the maximum that can be claimed in a high estimate.

CHAPTER II.

FLUCTUATIONS OF GROUND-WATER LEVEL.

INFLUENCE OF RAINFALL AND OF HEIGHT OF WATER IN ARKANSAS RIVER ON THE GROUND-WATER LEVEL.

During the field work of the summer several opportunities were found to observe the influence of a change of level of the water in the river upon the water plane in the adjacent bottom lands. The summer of 1904 was especially favorable for observations of this kind, as the season was an exceptional one, both in respect to the rainfall and as to the quantity of water flowing in the river. There was water in Arkansas River, in western Kansas, during nearly all of the time from the middle of June to the middle of September, and on several occasions floods of marked suddenness and great severity passed down the river. The rainfall during the same period was above the average. The record of rainfall from May 1 to October 1, as observed by the volunteer station of the United States Weather Bureau at Garden, Kans., is given in Table 6.

TABLE 6.—*Daily precipitation, Garden, Kans., May 1 to September 30, 1904.*

Date.	May.	June.	July.	August.	September.
1	0.58	0.0	0.0	0.08	0.0
2	Trace.	.25	.0	.0	.24
3	1.82	.04	.0	.0	.0
4	.75	.0	.95	.28	.0
5	.0	.0	Trace.	.0	.0
6	.0	.0	.12	.45	.0
7	.0	.0	.55	.0	.0
8	.20	.0	1.10	.0	.0
9	.0	.72	.0	Trace.	.0
10	.0	.0	.05	.0	.0
11	.0	.0	.0	.0	.0
12	Trace.	.0	1.32	.0	.0
13	.0	.30	.08	.0	.0
14	.0	.0	.0	.0	.0
15	Trace.	.0	.0	.0	.0
16	.0	Trace.	.0	.0	.0
17	.0	.19	.0	.0	.0
18	.0	.0	.0	.04	.0
19	.0	.0	.0	.32	.0
20	.0	.0	Trace.	.0	.0
21	.03	Trace.	Trace.	.0	.0
22	.85	.94	[a]1.42	.0	.0
23	.0	.0	.0	.0	.0
24	.0	.0	.0	.0	.0
25	.0	.0	.06	.11	.0
26	.0	.0	.0	.0	.0

[a] Much less at Sherlock, Kans.

TABLE 6.—*Daily precipitation, Garden, Kans., May 1 to September 30, 1904*—Continued.

Date.	May.	June.	July.	August.	September.
27	0.03	0.0	0.0	0.0	0.10
28	.04	.20	.0	.0	1.55
29	.0	.0	.0	.0	1.10
30	.0	.0	.0	.09	.10
31	.0	....	Trace.	Trace.	....
Total	4.30	2.64	5.65	1.32	3.39

Total for five months, 17.30.

Observations of the water plane were made very systematically during the various stages of the water in the river by Mr. Wolff, who was in charge of the party making the field observations. The results of these observations are given in the accompanying diagrams, which Mr. Wolff has constructed from the field notes. The first underflow determinations were made at the camp located about 2 miles west of Garden, Kans., on the ranch of Mrs. M. Richter, which is referred to in the text as camp 1. At this camp a number of shallow test wells were put in place for the special purpose of observing the position of the water plane. These test wells are shown on the map (fig. 2), from which it will be observed that test wells Nos. 1 and 2 were located north of the river bank at a distance of about 1,070 feet; test well No. 3 was closer to the river, at a distance of about 360 feet from the north bank. A large well located on the ranch of Mrs. Richter, and used for irrigation, was also used for the purpose of keeping track of the fluctuations of the water plane. The location of this well is shown on the map (fig. 2) near the quarter-section corner in the upper right-hand corner of the map. As will be observed, this well is situated a considerable distance upstream from test wells Nos. 1, 2, and 3; hence the water in it stood much higher than that in the test wells, since the water plane slopes eastward at the rate of about 7½ feet per mile. The land in which test wells Nos. 1, 2, and 3 are situated is what is commonly called in that locality "first bottoms." Immediately north of test wells Nos. 1 and 2 the "second bottoms" begin, the land here being some 3 to 5 feet higher than in the "first bottoms." Two sloughs shown on the map were grass covered, but contained more or less water either during high stages of the river or after heavy rains. In fig. 10 the elevations of water in Arkansas River from June 16 to July 11, 1904, and the elevations in test wells Nos. 1, 2, and 3 and in Mrs. Richter's well are represented graphically. The elevations are expressed in feet above mean sea level, as determined from the United States Geological Survey permanent bench marks in the valley. The detailed observations at these stations are printed in Table 7, in which the elevations are given in feet above mean sea level. The observation of the height of the river was made from a gage rod set up in the river and observed from the bank with a level. Observations were made morning and evening during the period covered by the table. There were occasional omissions of observation of river height, due to the absence of the level from camp.

TABLE 7.—*Elevation of ground water in the Arkansas River and in test wells near camp 1, 2 miles west of Garden, Kans.*

[Wells Nos. 1 and 2 are 1,070 feet north of river; well No. 3 is 360 feet north of river. Datum is 2,800 feet above mean sea level.]

Date.	Time.	Elevation of water in well No. 1.	Hydraulic gradient per mile from well No. 2 to well No. 1.	Elevation of water in well No. 2.	Hydraulic gradient per mile from well No. 1 to well No. 3.	Elevation of water in well No. 3.	Elevation of water in river.	Barometric pressure in inches of mercury.
1904.		*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Inches.*
June 16	12 m	33.97					36.7	28.60
Do	6 p. m	33.86						26.54
June 17	6 a. m	33.90						26.62
Do	12 m	33.87						26.55
Do	6 p. m							
June 18	6 a. m	33.98						26.63
Do	6 p. m	33.76	6.4	33.53	6.3	34.61	36.2	26.50
June 19	6 p. m	33.75	4.7	33.58	5.9	34.55		26.45
June 20	6 a. m	33.89	8.1	33.60	5.3	34.61	36.1	
Do	6 p. m					34.41	36.0	
June 21	6 a. m	33.82			4.8	34.47	36.0	26.47
Do	12 m	33.59	6.4	33.36	5.5	34.33		
Do	6 p. m	33.77	7.2	33.51	4.5	34.38		
June 22	6 a. m	34.45	8.9	34.13	4.2	35.02		
Do	12 m							
Do	6 p. m	33.94			8.8	35.12	37.7	
June 23	6 a. m	34.05	6.7	33.81	7.6	35.07	36.9	26.27
Do	6 p. m	33.87			8.0	34.95	36.9	
June 24	6 a. m	34.00	6.9	33.75	7.1	34.95	36.5	
Do	6 p. m	33.77	6.7	33.53	6.8	34.69	36.3	26.35
June 25	6 a. m	33.93	8.1	33.64	5.1	34.61	36.2	
June 26	6 a. m	33.93			3.6	34.42	35.9	
Do	12 m	33.77	6.1	33.55				
June 27	6 a. m	33.93	7.7	33.67	3.9	34.45	35.9	
Do	6 p. m						35.9	
June 28	6 p. m						35.9	
June 29	6 a. m						35.8	
Do	6 p. m						35.8	
July 1	6 a. m						35.7	
Do	6 p. m						35.7	
July 2	6 a. m						35.7	
Do	6 p. m						35.7	
July 3	6 a. m						35.7	
Do	6 p. m	33.19	6.7	32.95	7.2	34.16	35.6	
July 4	6 a. m						35.6	
July 5	6 a. m						35.6	
July 6	6 a. m						35.6	
July 7	6 a. m	33.99	7.2	33.73	2.72	34.35	35.6	
Do	6 p. m						35.6	
July 8	6 a. m	34.53	7.2	34.27	3.64	35.02	35.7	
July 9	6 a. m	33.88	8.1	33.59	1.71	34.11	35.7	
July 10	6 a. m						35.6	
July 11	6 a. m	33.68	8.1	33.39	3.15	34.10	35.8	

From the morning of June 21 until noon of June 22, which are left blank in the table, there was no material change in the height of the river. The water in the river slowly sank during the period covered from noon of June 16 to noon of June 22. The record shown in fig. 10 begins on June 16. The levels in the various wells remained substantially stationary from that date until June 22. During the night of June 21 a heavy rain fell, which is given on the official record at Garden as 0.94 of an inch. The test wells on the morning of June 22 showed marked changes in the elevation of the ground water, due to the rain of the previous night. Well No. 1 rose 0.68 of a foot; well No. 2 rose 0.62 of a foot; well No. 3 rose 0.64 of a foot, while the Richter well rose 0.05 of a foot before noon of June 22, and by the morning of June 24 had risen 0.10 of a foot. The river remained stationary until 3 p. m. of June 22, when a flood consisting of an abrupt wave swept down the river, causing a rise of 1.7 feet. Notwithstanding this rise in the river, the water in test wells Nos. 1 and 2, 1,070 feet from the river, fell during the interval between the morning and evening of June 22, while test well No. 3, which was situated within 360 feet of the river bank, was only 0.1 higher at 6 p. m. of June 22 than it was at 6 a. m. on the same day. These results show that the heavy rain of the night of June 21 raised the water in all of the test wells, but that the flood of the afternoon of June 22 raised the water only in the well nearest the river. The river gradually receded from the high-water mark reached on the afternoon of June 22, and all of the test wells gradually fell. There was no rain until July 4, except a slight shower on June 28. Test wells Nos. 1, 2, and 3 showed a tendency to fall, although the water in the river was from 2 to 3 feet higher than the water in the wells during all of this period.

The rise in the water plane from 6 p. m. of June 21 to 6 a. m. of June 22, amounting to a rise of 0.68 foot in test well No. 1 and 0.62 foot in test well No. 2, was due, as stated above, to a heavy rain which fell during the night. From the data at hand it is possible to express the magnitude of the contribution to the underflow as so many cubic feet of water for each mile of the river valley. If this contribution be supposed to extend uniformly over a given period of time, then the addition to the ground water may be expressed as a continuous flow of so many cubic feet of water per second for each linear mile of the river valley. Thus, in the present case, if we suppose that the rainfall of the night of June 21 fell uniformly during the twelve hours from 6 p. m. to 6 a. m., we can readily compute that the observed increased amount of ground water was equivalent for each mile of valley along the river to a continuous flow of water amounting to 23.8 cubic feet per second. To put this in other words, we can say that if the sands of the valley had contributed to the river by seepage all of the water which the rain added to these same sands, the seepage would amount to a continuous flow into each mile of the river of 23.8 cubic feet per second, maintained for twelve hours.

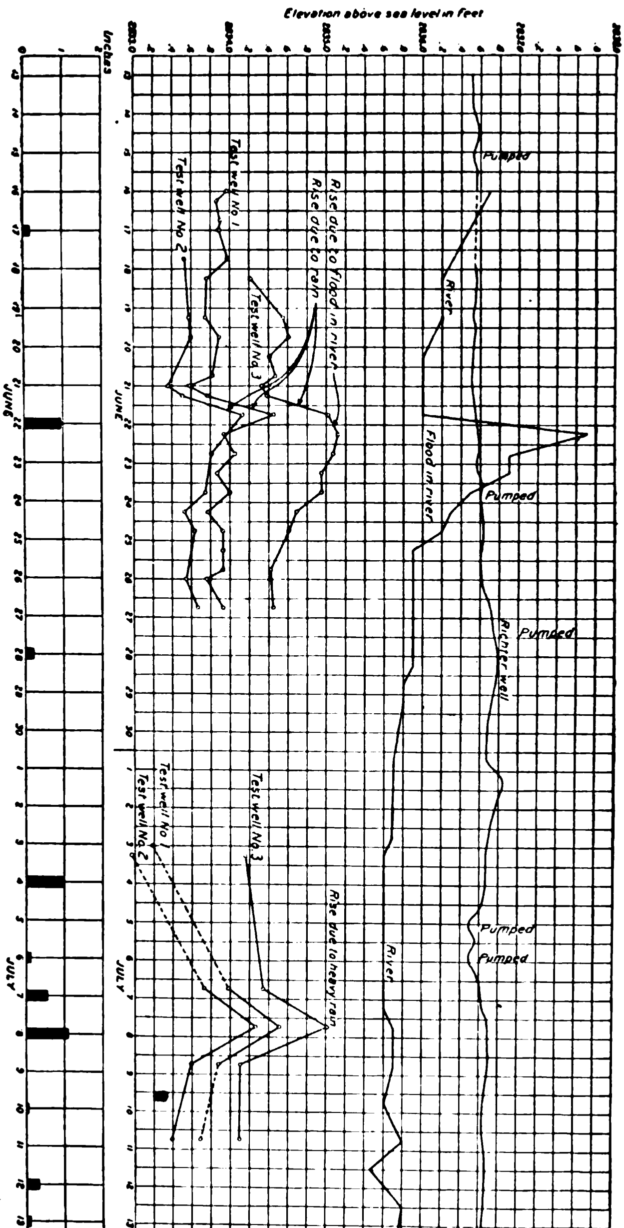


FIG. 10.—Elevation of water in Arkansas River and test wells, Garden, Kans., from June 16 to July 11, 1904. Test well No. 1 is 1,070 feet north of river; test well No. 2 is 190 feet east of test well No. 1; test well No. 3 is 860 feet north of river; Richter well is 2,500 feet north of river.

In a similar way, if the water contributed to the ground by the flood in the river from 3 p. m. to 6 p. m. of June 22 be considered as spread uniformly over twelve hours, it can readily be computed that the gain by the ground due to this cause represents a seepage loss for each mile of the river of 6.4 cubic feet of water per second. It can readily be seen, therefore, that the rainfall contributed a much greater volume of water to the underflow than was contributed by the flood in the river.

The average rise of the ground water during the night of June 21 was such that it would require a rainfall, without run-off, of 2.2 inches to fully account for it. The rainfall recorded at Garden for the night of June 21 was 0.94 inch. The difference between the measured rise of the ground water and the rainfall is explained by the fact that there is almost no run-off from the level lands of the river valley, so that nearly all of the drainage is underground by means of the deposits of sands and gravels. The seepage of this drainage is in part toward the low-water plane along and near the river channel. At such a place the amount of rise in the ground water would naturally be higher than could be accounted for by the localized rainfall.

After the high water of June 22 the river gradually fell until, on the morning of June 27, it had reached an elevation of 2,835.9 feet, which was 0.1 foot lower than its elevation on the morning of June 22. The water in the test wells gradually fell during the same period, the corresponding loss of ground water being given in Table 8 as a continuous flow of water expressed in cubic feet per second for 1 mile of river valley. By the morning of June 27 nearly all of the water contributed to the sands of the valley by the rain of June 21 and the flood of June 22 had disappeared. The gain and loss can be expressed as follows, in the form of a balance sheet:

TABLE 8.—*Loss and gain of ground water per mile of river valley, 1904.*

I.—FROM RIVER TO WELL NO. 1, 1,070 FEET NORTH OF RIVER, GARDEN, KANS.

Time.	Gain in ground water per mile of river valley.	Remarks.
	Sec. feet.	
June 18, 6 p. m., to June 21, 6 p. m.	− 0.93	No change in elevation of river water, and only slight change in elevation of water in well No. 1 until June 22.
June 21, 6 p. m., to June 22, 6 a. m.	25.8	Due to rainfall of 0.94 inch.
June 22, 6 a. m., to June 22, 6 p. m.	7.3	Due to rise in river.
June 22, 6 p. m., to June 23, 6 a. m.	− 5.4	
June 23, 6 a. m., to June 24, 6 a. m.	− 3.1	
June 24, 6 a. m., to June 27, 6 a. m.	− 2.7	
July 3, 6 p. m., to July 7, 6 a. m.	2.3	Due to rain. No change in elevation of river water.
July 7, 6 a. m., to July 8, 6 a. m.	22.4	Due to rain night of July 7. No change in elevation of river water.
July 8, 6 a. m., to July 9, 6 a. m.	−14.6	Rate of loss during 24 hours after precipitation of 1.2 inches of night of July 7.
July 9, 6 a. m., to July 11, 6 a. m.	− 1.0	

TABLE 8.—*Loss and gain of ground water per mile of river valley, 1904*—Continued.

II.—FROM RIVER TO WELL NO. 2, 900 FEET NORTH OF RIVER, SHERLOCK, KANS.

Time.	Gain in ground water per mile of river valley.	Remarks.
	Sec. feet.	
July 15, 9 a. m., to July 20, 7.30 a. m	− 2.0	
July 20, 7.30 a. m., to July 25, 6 a. m	− 1.0	
July 27, 11 a. m., to July 27, 1 p. m	54.0	
July 27, 1 p. m., to July 27, 3 p. m	72.0	
July 27, 3 p. m., to July 27, 5 p. m	65.0	
July 27, 5 p. m., to July 27, 7 p. m	37.0	
July 27, 7 p. m., to July 28, 6 a. m	1.5	
July 28, 6 a. m., to August 1, 6 a. m	− 1.8	

III.—FROM RIVER TO WELL NO. 5, 550 FEET SOUTH OF RIVER, SHERLOCK, KANS.

July 18, 7 a. m., to July 20, 7 a. m	− 1.35	
July 20, 7 a. m., to July 25, 7 p. m	− .54	
July 25, 7 p. m., to July 27, 11 a. m	− .20	
July 27, 11 a. m., to July 27, 1 p. m	63.8	
July 27, 1 p. m., to July 27, 3 p. m	28.9	
July 27, 3 p. m., to July 27, 5 p. m	13.4	
July 27, 5 p. m., to July 27, 7 p. m	1.34	
July 27, 7 p. m., to July 29, 8 a. m	− .22	
July 29, 8 a. m., to August 1, 8 a. m	− .92	

IV.—FROM WELL NO. 5 TO WELL NO. 6, 2,600 FEET SOUTH OF RIVER, SHERLOCK, KANS.

July 18, 7 a. m., to July 20, 12 m	− 2.6	
July 20, 12 m., to July 25, 8 a. m	− 1.5	
July 25, 8 a. m., to August 1, 8 a. m	− .5	

V.—FROM RIVER TO WELL NO. 2, 1,730 FEET SOUTH OF RIVER, DEERFIELD, KANS.

August 4, 9 a. m., to August 9. a. m	− 0.51	
August 6, 9 a. m., to August 8, 7.30 a. m	5.26	
August 8, 7.30 a. m., to August 9, 9 a. m	1.82	
August 9, 9 a. m., to August 10, 7.30 a. m	− .61	

Summary of loss and gain of ground water per mile of river valley.

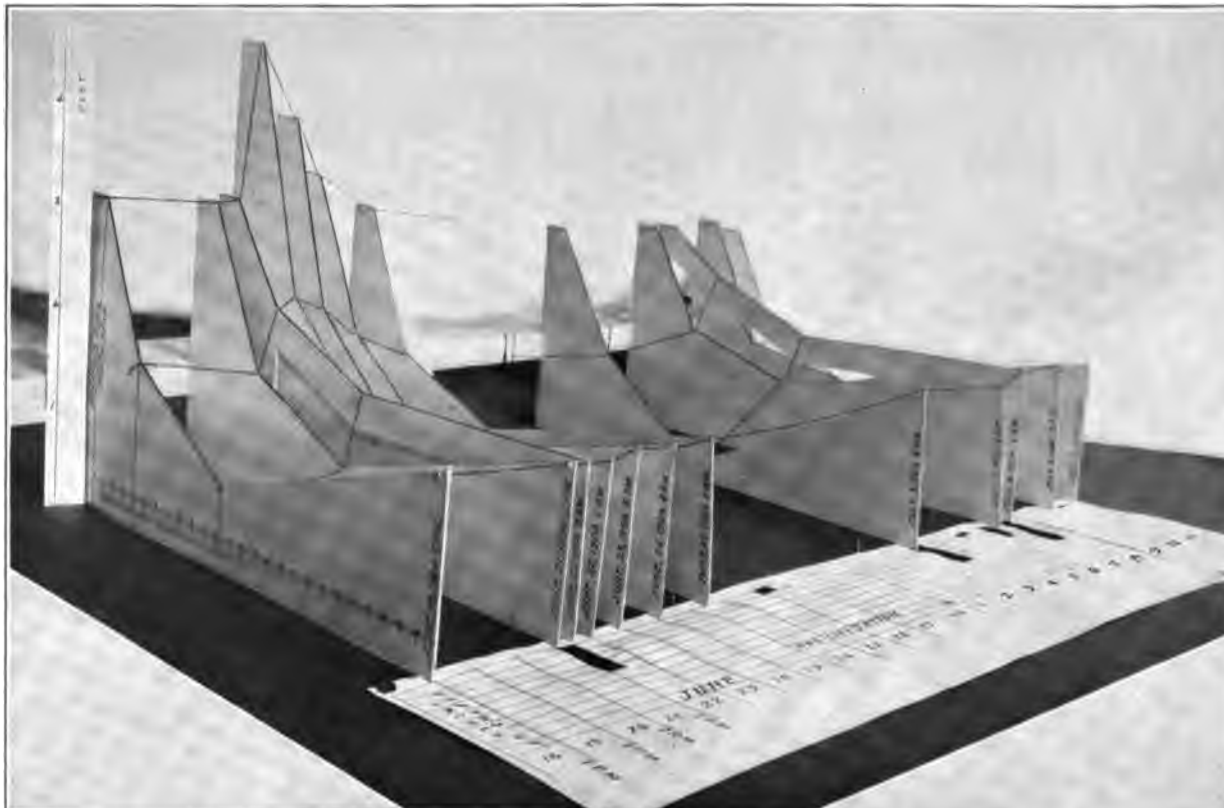
	Cubic feet.	Acre-feet.
GAIN.		
Rain of night of June 21. From 6 p. m., June 21, to 6 a. m., June 22, 12 hours, at 23.8 cubic feet per second	1,030,000	23.6
Flood of afternoon of June 22. From 6 a. m., June 22, to 6 p. m., June 22, 12 hours, at 7.3 cubic feet per second	315,000	7.2
Total gain	1,345,000	30.8
LOSS.		
6 p. m., June 22, to 6 a. m., June 23, 12 hours, at 5.4 cubic feet per second	233,000	5.4
6 a. m., June 23, to 6 a. m., June 24, 24 hours, at 3.1 cubic feet per second	268,000	6.1
6 a. m., June 24, to 6 a. m., June 27, 72 hours, at 2.7 cubic feet per second	700,000	16.1
Total loss	1,201,000	27.6
Net gain	144,000	3.2

In Pl. I there is shown a view of a model designed to illustrate the changes in ground-water levels which have just been discussed. This model shows, by cardboard cross sections, the level of the water in Arkansas River and in three wells north of the river on various dates in June and July, 1904. These are the same wells and the same data given in Table 8 and represented graphically in fig. 10. The height of the river is represented at the left end of each cardboard section and the position of the surface of the ground water in the three wells appears at the appropriate distances to the right, the wells being indicated by vertical lines and by the right end of the card. The well represented by the right end of each cardboard section is located about 2,500 feet north of the north bank of the river.

The surface of the ground water is represented in the model by the straight lines forming the top of each piece of cardboard. Of course the actual surface did not consist of a broken line, as shown, but of a curved line passing smoothly through the angles of the broken line. The representation of the ground-water surface as straight lines between the various wells introduces no substantial error in the results, and it illustrates the characteristic changes with greater fidelity than curved lines, whose forms, in any case, could be known only approximately.

It can readily be observed from this diagram that the river and water plane remained substantially stationary from June 18 to June 21. The influence of the heavy rain of the night of June 21 is shown on the third cardboard section by the more elevated water plane of the next morning, the river remaining stationary during this interval. The fourth cardboard cross section (6 p. m., June 22) shows the river flood, which began at 3 p. m. June 22. This cross section shows that the water plane sank, notwithstanding this heavy flood, except at the well nearest the river. The river gradually fell, the water plane also falling at the same time. The model shows the water plane at its lowest observed position on July 3. The section shown in the model for July 7 illustrates the influence of the rains falling from July 3 to July 7 in raising the water plane. The greatest rise in the water plane observed at any time is shown in the model by the third section from the end, that corresponding to the morning of July 8. This rise was due to a rain of more than 1 inch on the night before. As in the previous instances, the water plane rapidly fell away after the rise. It is important to bear in mind that the height of the river remained almost constant from July 3 to 9.

These same changes are also shown in fig. 10, where a curve is given for the changing height of water in each well and the river. In using this diagram or the table it is important to know that it is usually necessary to compare evening observations with evening observations, and not with morning observations. Owing to changes in tempera-



CARDBOARD MODEL OF CHANGES IN WATER PLANE NEAR CAMP 1.

ture and barometer there are diurnal periodic changes in the position of the water plane, and these fluctuations are such that it is always more satisfactory to compare observations taken at corresponding times of the day, unless the intermediate changes are very violent. The morning level of the ground water is normally higher than the

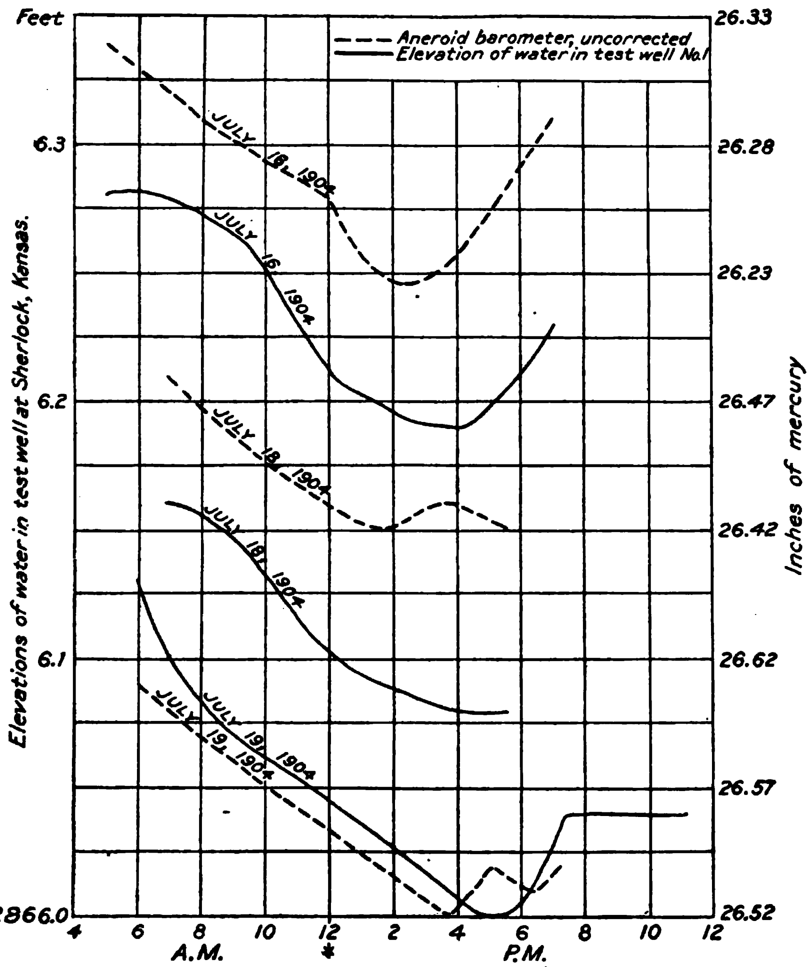


FIG. 11.—Curves of barometric pressure and height of water plane, showing correspondence between the fluctuations of the barometer and the water plane as observed on several dates at Sherlock, Kans. The dotted lines give the diurnal variations in the barometric pressure; the full lines show the elevations of the water in test well No. 1.

evening level, the fluctuations in the wells discussed above being indicated very clearly by some of the lines in fig. 10, especially those showing the June fluctuations in test wells Nos. 1, 2, and 3.

Some results showing the correspondence between the barometric pressure and the ground-water elevation were sought for at camp 1, near Sherlock, Kans. The data obtained are depicted graphically in fig. 11. The results were not what were expected, as the influence of the barometric pressure should be to raise the ground water as the barometer falls.[a] This indicates that the low position of the ground water in the afternoon of each day is probably a temperature effect, due to the decrease in the capillarity of the water with the temperature. The ground water at test well No. 1, Sherlock, and in test wells Nos. 1, 2, and 3, Garden, was within 3 feet of the surface of the ground and the difference in temperature of day and night was very great.

In fig. 10 the level of water in the Richter well, 2,500 feet north of the river, is compared for a period of about thirty days with the elevation of the water in Arkansas River. The total variation of the water plane, as shown by the levels observed in the well twice daily during the thirty-day interval, did not exceed 2 inches. This shows that the influence of the river upon the ground water dies out to practically nothing in a distance of one-half mile. The influence of the rainfall upon the water in the well is traceable by a comparison of the rainfall record and the well curve, but it is uncertain whether any connection can be detected between the elevation of the river and the well curve. The influence of occasional pumping upon the ground-water level is quite pronounced.

The observations given above indicate the following conclusions:

1. The level of the ground water shows a marked tendency to remain at a level lower than the channel of the river at a point about one-fourth mile north of the river channel.

2. The elevation of the water plane is very sensitive to the amount of rainfall, the rise in the water plane (due to a rain) in the first bottoms being greater than can be accounted for by the localized precipitation.

3. High water in the river has much less effect upon the level of the ground water than the rainfall, its influence being confined to a distance of a few hundred feet from the river channel.

4. The water plane falls at a very rapid rate after its elevation has been increased by rainfall or by a flood in the river.

5. The fact that the water plane lies for a considerable distance at a level lower than the river channel, even when there is water in the river for an extended length of time, and the rapid way in which the ground water sinks after its rise due to heavy rain, establishes the fact that the underground drainage through the sands and gravels beneath the river valley is more than sufficient to carry off all of the rainfall without run-off into the river channel.

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[a] Slichter, C. S., Motions of underground waters: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, p. 73.

FLUCTUATION OF GROUND-WATER LEVEL AT SHERLOCK, KANS.

Observations of changes of level of ground water near Sherlock, Kans., were made during the period extending from July 15 to August 3, 1904. For this purpose a number of test wells were driven, the location of which is shown in fig. 4. Of these test wells, No. 2 was 900 feet and No. 3 was 400 feet north of the river; No. 5 was 550 feet and No. 6 was 2,500 feet south of the river. The complete record of observations taken in the field is given in Table 9. The principal results presented by this table are shown graphically in fig. 12. As shown by this diagram, Arkansas River gradually fell from July 15 until July 27. At this time the water in the river had reached a very low stage, the flowing water occupying a width in the channel of about a rod and a depth of about 6 inches.

TABLE 9.—*Elevation of water in river and in test wells, Sherlock, Kans.*

[Datum: 2,800 feet above mean sea level.]

Date.	Time.	Elevation of water in well No. 1.	Hydraulic gradient per mile from well No. 2 to well No. 1.	Elevation of water in well No. 2; 900 feet north of river.	Hydraulic gradient per mile from well No. 2 to well No. 3.	Elevation of water in well No. 3; 400 feet north of river.	Hydraulic gradient per mile from well No. 3 to river.	Elevation of water in river.	Hydraulic gradient per mile from well No. 5; 550 feet south of river.	Elevation of water in well No. 5; 550 feet south of river.	Elevation of water in well No. 6; 2,500 feet south of river.	Hydraulic gradient per mile from well No. 6 to well No. 5.	Elevation of water in well No. 4; 1,000 feet south of river.	Hydraulic gradient per mile from well No. 5 to well No. 4.	Barometer pressure of mercury, uncorrected.
1904.		Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Inches.
July 15	9 a. m.	66.36	9.1	65.50	2.4	65.73	2.9	65.95							26.34
July 16	5 a. m.	66.28	8.8	65.45	4.5	65.68									26.32
Do	9.30 a. m.	66.26						65.85							
Do	10.30 a. m.	66.24													26.27
Do	12 m.	66.21	9.8	65.28	3.1	65.57									26.26
Do	1.30 p. m.	66.20													26.23
Do	3.30 p. m.	66.19													26.23
Do	7 p. m.	66.23													26.29
July 17	8 a. m.	66.23						65.75							26.45
Do	11.30 a. m.	66.21	9.5	65.31	2.2	65.52									26.35
July 18	7 a. m.	66.16	9.3	65.23	2.1	65.48	2.4	65.60	3.3	65.94	65.32	1.7	67.07	7.9	26.48
Do	10.45 a. m.	66.12	9.8	65.19	2.5	65.43				65.93	65.28	1.8	67.00	7.5	26.44
Do	12 m.	66.10	9.7	65.18	2.6	65.43									26.43
Do	1.30 p. m.	66.09	9.8	65.16	2.7	65.42				65.90	65.26	1.7	66.97	7.4	26.42
Do	3.30 p. m.	66.08	9.9	65.14	2.8	65.41				65.89	65.25	1.7	66.95	7.4	26.44
Do	5.30 p. m.	66.08	9.6	65.16	2.6	65.41	3.2	65.65	2.2	65.88	65.24	1.7	66.95	7.4	26.42
July 19	6 a. m.	66.13						65.55							26.61
Do	8 a. m.	66.08													26.59
Do	10 a. m.	66.06													26.57
Do	3.40 p. m.	66.01													26.52
Do	5 p. m.	66.00	9.9	65.06	2.1	65.26					65.17				26.54
Do	6.15 p. m.	66.01													26.53
Do	7.15 p. m.	66.04						65.45							26.54
Do	9.15 p. m.	66.04													
Do	11.15 p. m.	66.04													

July 20	7.30 a. m	65.99	9.5	65.09	.9	65.18	.9	65.25	5.4	65.81			66.96	8.1	26.62
Do	12 m	65.93								65.79	65.17	1.7			26.54
July 21	4 p. m	65.86	10.0	64.91	.6	64.97	.4	65.10		65.70	65.12	1.6	66.82		26.46
Do	7 p. m	65.87						65.00							26.50
July 22	2 p. m									65.76	65.19	1.5	66.97		
Do	5 p. m							65.15			65.16				
July 23	3 p. m							65.05	5.9	65.66	65.11	1.5	66.81	8.1	
July 24	6 p. m	65.79	9.9	64.85	.3	64.88	1.6	65.00							
July 25	7 a. m	65.82	9.4	64.93	.2	64.95	.7	65.10							
Do	10 a. m			64.83	.5	64.88									
Do	6 p. m	65.74	10.0	64.79	.5	64.84	2.1	65.00	5.1	65.58	65.03	1.4	66.67	7.9	
July 26	9.30 a. m							65.05		65.52	65.02	1.4	66.69	8.1	
Do	5 p. m							65.00		65.49	64.99	1.4	66.64	8.0	
July 27	11 a. m	65.70	10.0	64.75	.8	64.83	1.7	65.00		65.46	64.97	1.3	66.62	8.1	26.51
Do	1 p. m			64.73	1.7	64.89	14.6	66.00	4.0	65.42					
Do	3 p. m	65.80	9.8	64.87	4.4	65.29	14.6	66.40	9.2	65.44	64.95	1.3	66.57	7.9	
Do	5 p. m	65.91	8.9	65.07	6.4	65.68	12.3	66.60	11.2	65.44					
Do	7 p. m	66.04	8.5	65.24	6.8	65.88	10.8	66.60	10.9	65.46					
July 28	6 a. m	66.21	8.0	65.45	5.3	65.95	5.3	66.35							
July 29	8 a. m	66.25	9.0	65.40	5.1	65.88	5.5	66.30	5.8	65.70	64.89	2.2	66.78	7.5	
July 30	8 a. m	66.12	9.0	65.27	3.1	65.56	4.5	65.90							
July 31	8 a. m									65.71					
August 1	8 a. m	66.10	9.2	65.23	2.9	65.50	2.6	65.70	.2	65.72	64.79	2.5	66.79	7.5	
August 2	6 p. m							65.45							
August 3	7 a. m							67.10							

During this same period of fall in the river there was no rainfall except on July 22 and a very light rain on July 25. The rain of July

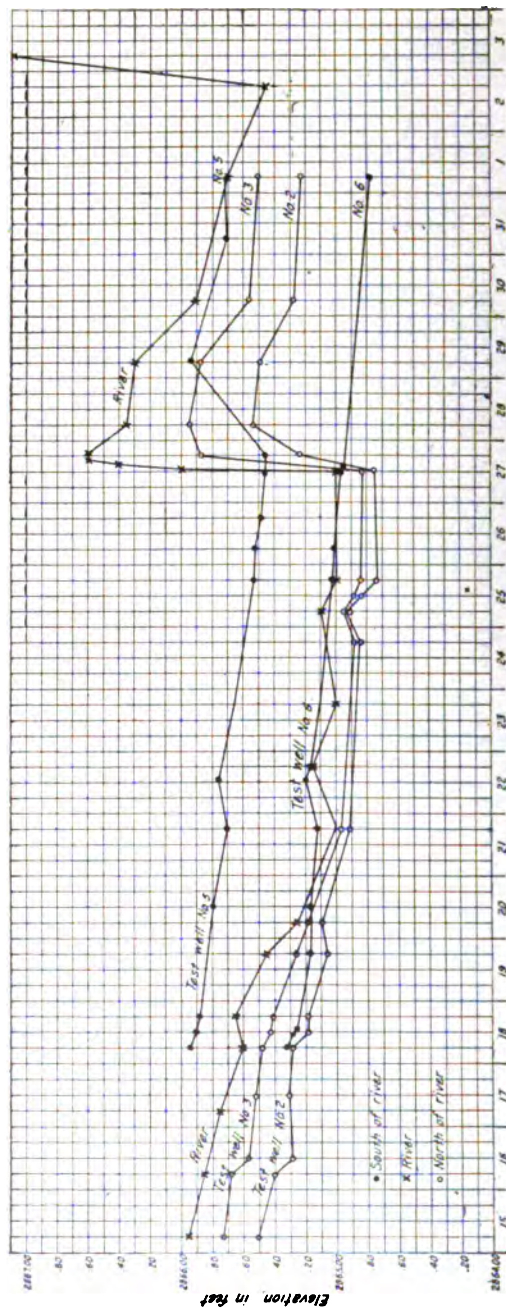
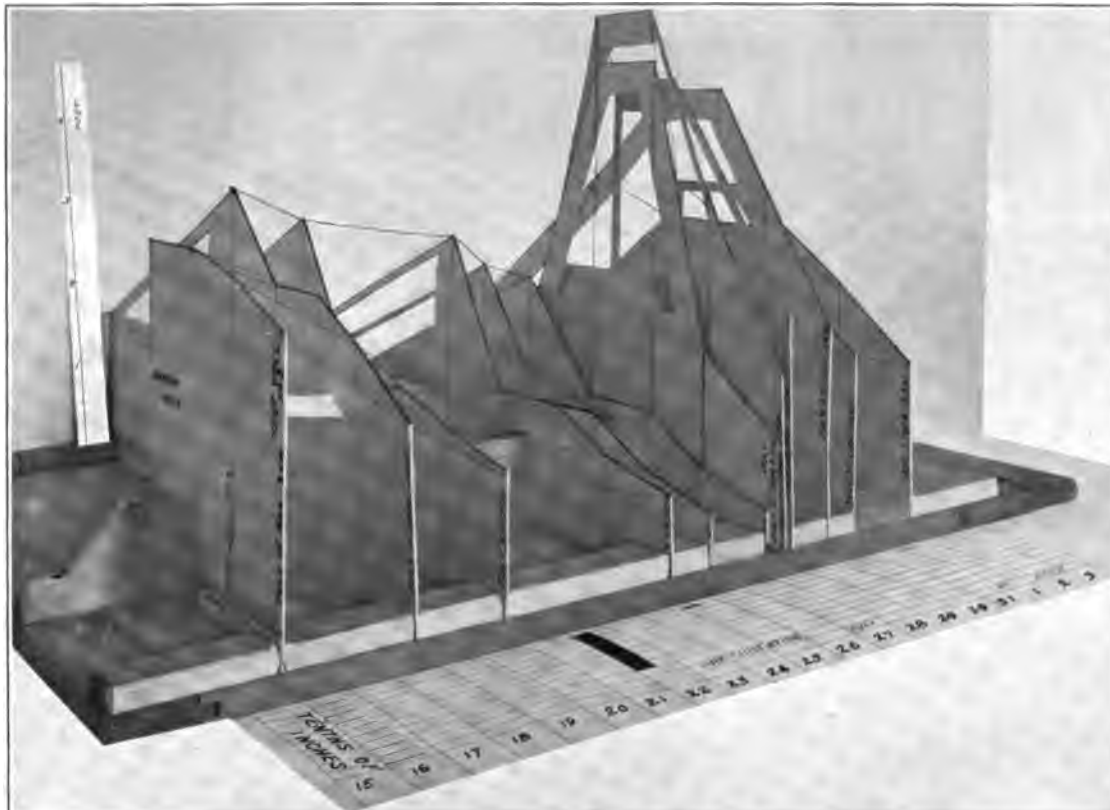


FIG. 12.—Elevation of water in test wells and in Arkansas River for the various dates from July 15 to August 3, 1904, at Shelrock. Preceding the entries in this diagram considerable rain had fallen. There were several heavy rains between July 8 and 13. These rains undoubtedly account for the high position of the water plane in all of the wells at the beginning of the period for which the diagram is constructed. Test well No. 2 is 900 feet north of river; test well No. 3 is 400 feet north of river; test well No. 5 is 550 feet south of river; test well No. 6 is 2,500 feet south of river.

22 was measured at Garden by the volunteer observer of the United States Weather Bureau as 1.42 inches, but the rainfall at Sherlock was very much less. During this period of fall of level of the water



CARDBOARD MODEL OF CHANGES IN WATER PLANE NEAR SHERLOCK, KANS.

in the river the test wells north of the river fell at corresponding rates. The total fall in the river amounted to 0.95 of a foot; the fall in test well No. 3, 400 feet north of the river, during the same period was 0.9 of a foot; in test well No. 2, 900 feet north of the river, 0.77 of a foot; in test well No. 5, 550 feet south of the river, 0.5 of a foot; and in test well No. 6, 2,500 feet south of the river, 0.3 of a foot. On July 27, between 11 a. m. and 5 p. m., the river rose 1.6 of a foot, restoring the level of water in the river to the height of July 15 plus 0.6 of a foot. This sudden rise in the river was not accompanied by rainfall in the neighborhood of Sherlock. Its influence upon the various test wells is shown by fig. 12. The immediate effect upon test wells Nos. 2 and 3, north of the river, was very apparent. Between 11 a. m. and 7 p. m. test well No. 3, 400 feet north of the river, rose 1.05 feet, and test well No. 2, 900 feet north of the river, rose 0.49 of a foot. By the next morning at 6 a. m. the river had fallen 0.25 of a foot; test well No. 3, 400 feet north of the river, had risen about 0.1 of a foot, and test well No. 2, 900 feet north of the river, had risen 0.23 of a foot. The river continued to fall very slowly, on the morning of July 29 having fallen only about one-half of 0.1 of a foot from its elevation on July 28; the water in test wells Nos. 2 and 3 had dropped about the same amount, and on August 1, at 8 a. m., when the river had fallen 0.6 of a foot below its elevation of July 29, test wells Nos. 3 and 2 had dropped 3.6 and 1.8 feet, respectively. During this same period of time the water plane south of the river acted very differently from that observed on the north side of the river. The water in test well No. 6, 2,500 feet south of the river, fell continuously from July 18 to August 1, notwithstanding the flood of July 27; and that in test well No. 5, 550 feet south of the river, fell from July 18 until July 27, the total fall amounting to 0.47 of a foot. No observation was made at this test well on July 28, but by the morning of July 29 the water had risen 0.45 of a foot. On August 1 it had fallen 0.2 of a foot below its level on the morning of July 29, in sympathy with the general fall of the water in the river. It can be seen from this that the elevation of the water in the various test wells showed all varieties of change during the flood in the river. The wells within 900 feet of the river fluctuated quite accurately with the changing level in the river itself, while the water in the test well one-half mile from the river seemed to show no effect of the flood in the river during the period of observation.

In explanation of the gradual fall in the test wells from July 18 to July 27, it must be remembered that the position of the water, as found on July 18, was high on account of the heavy rains which fell during the first twelve days of July. From July 4 to July 13, inclusive, 3.27 inches of rain were caught at the rain gage at Garden, Kans.; the rainfall at Sherlock, Kans., was probably as great, so it is very likely that the level of the water found in the test wells on July 15 and 18 was high owing to the previous rains. In fig. 13 the results of the flood of July 27 are shown in greater detail than in the previous diagram.

A photograph of a cardboard model showing the changing positions of the water plane at Sherlock is reproduced in Pls. II and III. The top of each cardboard corresponds to a cross section of the water plane taken across the valley on a certain date, the right side of each card corresponding with the north side of the valley, the left side corre-

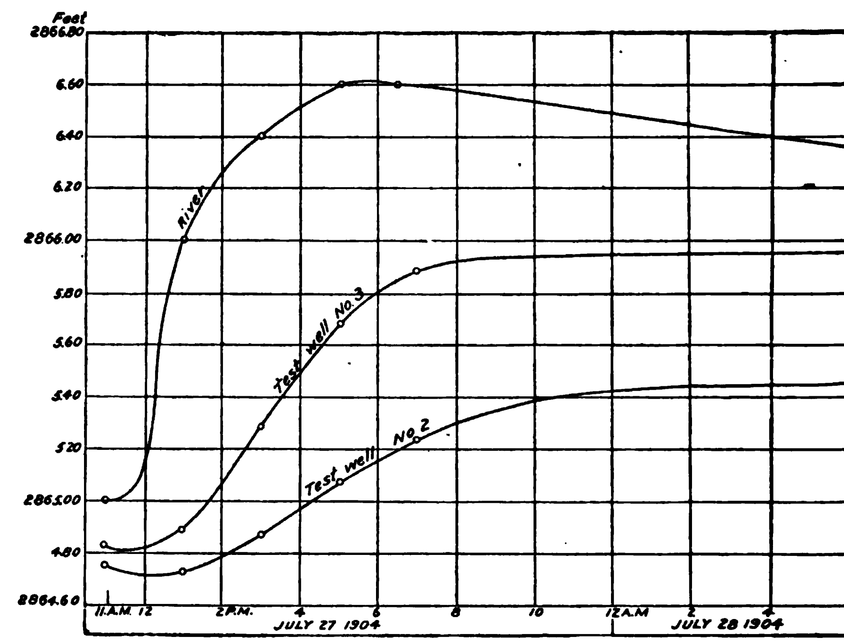
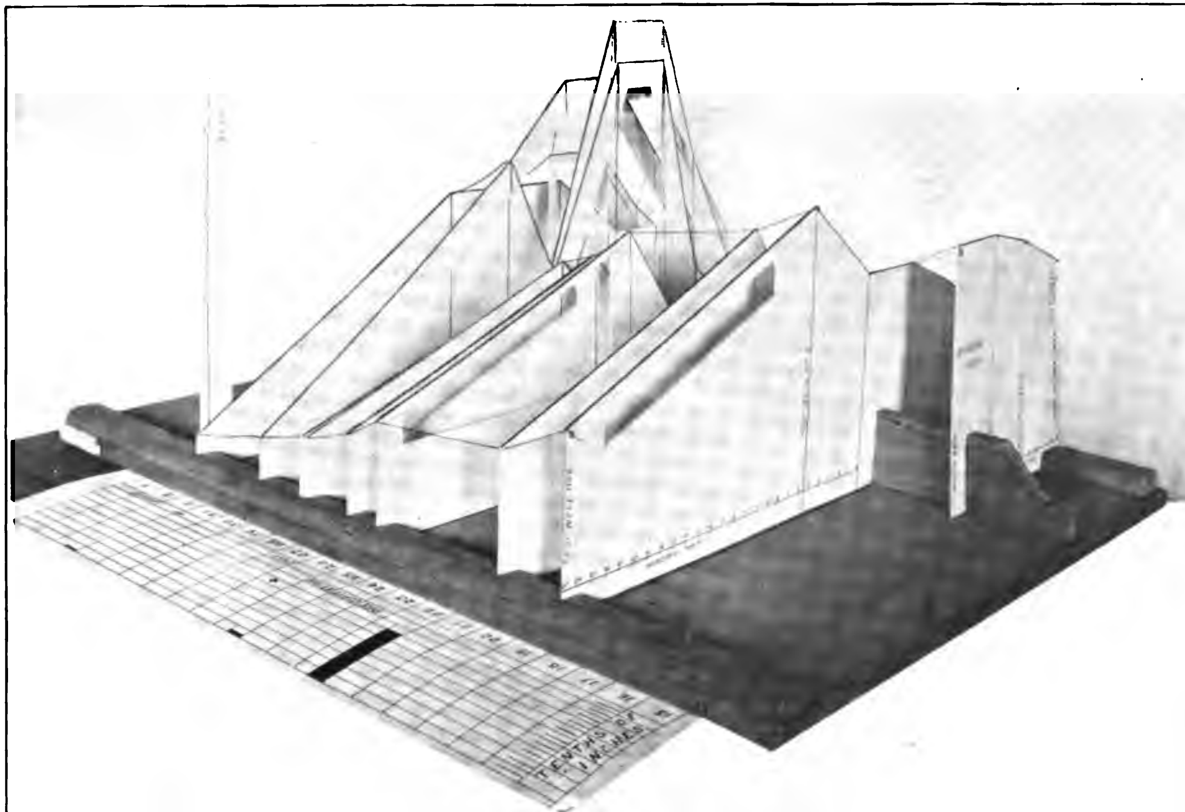


FIG. 13.—Elevation of water in Arkansas River and in two test wells near Sherlock, Kans., for various hours during the flood of July 27, 1904. The vanishing influence of the flood with increasing distance from the river is clearly brought out by the diagram. Test well No. 2 is 900 feet north of north bank of river; test well No. 3 is 400 feet north of north bank of river.

sponding with the south side of the valley. The location of each test well is shown by a vertical line, and the position of the channel of the Arkansas is indicated by the level segment of each card near the middle of each section. The model shows to the eye the way in which the river and the water in all of the test wells gradually fell from July 13 to July 27, and it also illustrates the influence of the flood of July 27 upon the wells near the river. It also shows that the level of water in well No. 6, one-half mile south of the river, was not influenced by the flood in the river, but continued to fall during the entire period. The decreasing influence of the river on the water plane with the distance from the river is brought out clearly by the diagram (fig. 13).

It is apparent from this model, as well as from the one shown for camp 1, that there is a marked tendency for the ground water near the river, especially on the north side, to remain at a lower level than



CARDBOARD MODEL OF CHANGES IN WATER PLANE NEAR SHERLOCK, KANS.

the water in the river itself. At the time the data presented by the model were obtained, there had been water in the river for six or seven weeks and the amount of rainfall had been above the average. These facts indicate that the underground drainage through the sands and gravels is more than sufficient to drain off the precipitation, without return seepage into surface streams and without run-off from the surface of the ground.

The various amounts of ground water gained or lost by each mile of the valley along the river at Sherlock from July 15 to August 1, 1904, is expressed in Sections II, III, and IV of Table 8 (p. 31). For the purpose of making the results as definite as possible the gain or loss for each mile of valley is given as a continuous flow of water expressed in cubic feet per second. Thus, according to the table, the strip of ground between the river bank and test well No. 2, 900 feet north of the river, extending along the stream for a distance of a mile, lost water from July 15 to to July 20 at a rate equivalent to a steady flow of water equal to 2 cubic feet per second. During the flood on July 27 this same strip of country absorbed water from the river during the first two hours of flood at the rate of 54 cubic feet per second. The rate of gain during the three following periods of two hours each was 72, 65, and 32.4 second-feet, respectively. During the eleven hours from 7 p. m., July 27, to 6 a. m., July 28, the rate of gain fell to 1.5 second-feet, after which the ground lost water. These results, and similar results for the south side of the river, are given in the table. Putting all of these results together we can compute the amount of water furnished to the sands by the flood in the river as follows, the computation applying to 1 mile of the river valley only:

Water furnished to sands near Sherlock, Kans., by flood of Arkansas River.

	Cubic feet.
North of river:	
July 27, 11 a. m. to 1 p. m., 2 hours, at 54 cubic feet per second	389,000
July 27, 1 p. m. to 3 p. m., 2 hours, at 72 cubic feet per second	525,000
July 27, 3 p. m. to 5 p. m., 2 hours, at 65 cubic feet per second	467,000
July 27, 5 p. m. to 7 p. m., 2 hours, at 32.4 cubic feet per second	234,000
July 27, 7 p. m. to 6 a. m. July 28, 11 hours, at 1.5 cubic feet per second	59,500
Total gain	[a] 1,674,500
South of river:	
July 27, 11 a. m. to 1 p. m., 2 hours, at 63.8 cubic feet per second	459,000
July 27, 1 p. m. to 3 p. m., 2 hours, at 28.9 cubic feet per second	208,000
July 27, 3 p. m. to 5 p. m., 2 hours, at 13.4 cubic feet per second	96,500
July 27, 5 p. m. to 7 p. m., 2 hours, at 1.34 cubic feet per second	9,650
Total gain	773,150
July 27, 7 p. m. to 8 a. m. July 28, loss at 0.22 cubic foot per second	10,296
Net gain	[b] 762,854
Total gain both sides of river	[c] 2,437,354

[a] Equals 38.4 acre-feet. [b] Equals 17.6 acre-feet. [c] Equals 56 acre-feet.

The gain of 56 acre-feet took place on land having an area of 175 acres.

The above results show the gain between test well No. 2, 900 feet north of the river, and test well No. 5, 550 feet south of the river. There was some gain in ground water in the lands north and south of these boundaries, but the data are not at hand for the computation. The susceptibility of the adjoining lands in receiving seepage water from the river was greater on the north side than on the south side of the river.

FLUCTUATION OF GROUND-WATER LEVEL AT DEERFIELD, KANS.

Observation of the ground-water level was made at camp 3, near Deerfield, in three test wells. The location of these test wells appears on the map, fig. 6. The water in the river occupied but a small part of the river channel during most of the time during which these observations were made, and therefore the distances of the test wells from the edge of the flowing water are given in fig. 19, in preference to the distances from the river bank. Test well No. 1 was 1,100 feet, and well No. 2, 1,730 feet south of water in the river. Test well No. 3 was 1,100 feet south of the river, but 1,000 feet upstream from test well No. 2.

TABLE 10.—*Elevation of water in river and test wells at Deerfield, Kans.*

Date.	Time.	Elevation of water in well No. 1, 1,100 feet from river.	Hydraulic gradient, per mile, from well No. 1 to well No. 2.	Elevation of water in well No. 2, 1,730 feet from river.	Hydraulic gradient, per mile, from well No. 2 to well No. 3.	Elevation of water in well No. 3, 1,100 feet from river.	Hydraulic gradient, per mile, from river to well No. 3.	Elevation of water in river.
1904.		*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
August 4	9 a. m	2,923.02	0.25	2,922.99	8.7	2,924.57	−1.10	2,924.80
August 5	do	2,923.14	.17	2,923.12	9.0	2,924.75	1.44	2,924.45
August 6	do	2,923.21	.50	2,923.27	8.8	2,924.87	2.98	2,924.25
August 8	7.30 a. m	2,923.23	.50	2,923.29	8.4	2,924.82	−1.58	2,925.10
Do	10 a. m	2,923.23	.34	2,923.27	8.5	2,924.83		
Do	12 m	2,923.23	.42	2,923.28	8.5	2,924.83		
Do	4.30 p. m	2,923.23	.25	2,923.26	8.7	2,924.84		
August 9	9 a. m	2,923.27	.17	2,923.29	8.8	2,924.89	−1.49	2,925.20
Do	2.30 p. m	2,923.29	.08	2,923.28				
August 10	7.30 a. m	2,923.32	.00	2,923.32	8.7	2,924.91	1.73	2,924.55

The chart given in fig. 14 shows that a flood on August 7 in the river had no influence upon the water level in any of the wells, although frequent observations were made to detect such influence. The diagram likewise shows the effect of the rain in raising the ground water as shown by all of the wells from August 4 to August 7. During this same interval the river was falling, while the ground water was rising. The rainfall was measured at camp by catching rain in a tin bucket and correcting for difference in area between top and bottom of bucket. The observed rainfall on August 4 and August 5 amounted to about 1.75 inches. The water in the various test wells rose by the following amounts between August 4 and August 6: Test well No. 1, 0.17 foot, or 2.02 inches; test well No. 2, 0.29 foot, or 3.48 inches; and test well No. 3, 0.30 foot, or 3.60 inches. If we assume that the soil had a porosity of 33⅓ per cent, these observed changes in the level of the water plane are equivalent to actual increments of 0.7, 1.16, and 1.2 inches, respectively. These amounts will average

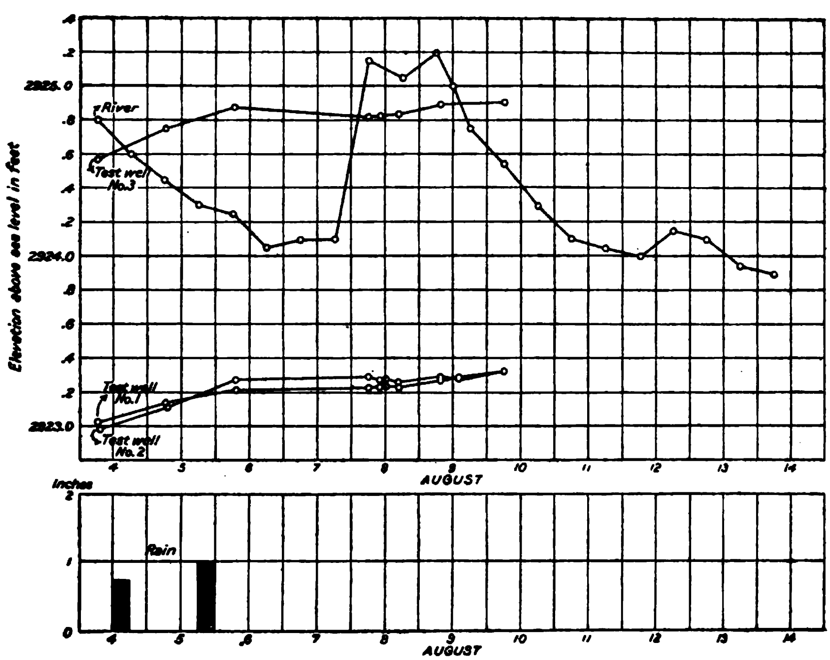


FIG. 14.—Elevation of water in Arkansas River and test wells at Deerfield, Kans., August 4 to 14, 1904. Test well No. 1 is 1,100 feet south of stream. Test well No. 2 is 1,730 feet south of stream. Test well No. 3 is 1,100 feet from stream and 1,000 feet from test well No. 2.

almost exactly 60 per cent of the rainfall for the two days, August 4 and August 5, 1904. This result gives very direct proof of the excellent quality of the catchment area furnished by the sandy bottom lands on the south side of the river at Deerfield.

EVAPORATION EXPERIMENTS NEAR DEERFIELD.

The table of meteorological data below has value in showing that a considerable amount of stored ground water is lost in the first bottoms of Arkansas River by evaporation. Although these measurements extend over only a very brief period, they are sufficient to establish the fact that the loss of ground water by evaporation is about ten times as great where the water is within 1 foot of the surface of the ground as it is where the water lies at a depth of 3 feet. The pumping plants that materially lower the ground water in the bottom lands will thus save a considerable amount of water that now goes to waste in evaporation and in supplying the rank growth of wild grasses that flourish in the first bottom lands. It is safe to say that this savable loss amounts on the average to a foot of water for each acre of first bottoms for the months of July and August alone.

The following is a record of observations of evaporation from three tanks filled with natural soil in which the water plane was kept at a constant depth, compared with the evaporation from a tank of open water. The tanks were located in the bottom lands of Arkansas Valley, near the head gates of the Farmers' ditch. The soil is a sandy loam changing to coarse sand at a depth of about 3 feet.

Meteorological records at Deerfield, Kans., from July 3 to September 8, 1905.

Week of—	Rainfall in inches.	Vapor pressure.	Per cent of relative humidity.	Velocity of wind in miles.	Evaporation in inches.				
					Open water.	1 foot to water, soil cultivated.	1 foot to water; soil uncultivated.	2 feet to water.	3 feet to water.
July 3-9 [a]	0.11	....	....	16.50	....	....	....	....	0.13
July 9-16	0.0	.440	47.3	15.89	....	....	....	....	0.13
July 16-23	0.08	.482	50.2	16.13	2.53	....	....	....	0.25
July 23-30	1.24	.560	61.2	19.78	2.39	....	....	....	1.40
July 30-Aug. 6	1.50	.568	63.9	12.05	1.80	....	....	....	0.05
Aug. 6-13	0.38	.478	54.8	13.62	2.45	1.48	1.78	0.65	0.43
Aug. 13-20	0.05	.580	57.3	13.26	2.22	1.34	1.21	0.60	0.17
Aug. 20-27	0.0	.520	49.3	19.58	3.04	1.14	1.38	0.49	0.08
Aug. 27-Sept. 3	0.03	.395	41.4	17.19	3.19	0.92	1.51	0.49	0.12
Sept. 3-8 [a]	0.71	.489	60.4	14.54	1.21	0.70	0.87	0.60	0.04

[a] Week incomplete.

CHAPTER III.

CHEMICAL COMPOSITION OF THE WATERS OF THE UNDERFLOW.

Chemical tests of the ground waters were made wherever possible during the process of the work. Portable field apparatus was at hand which could be used in making a few simple tests. The determinations made included titrations for chlorine, alkalinity, and hardness. Total solids were determined by means of the Whitney electrolytic bridge. The curve of total solids used in this case was obtained by evaporating a sample of water containing 95.9 parts per 100,000 total solids. The results of the test are brought together in Table 11, and the curve used for the determination of the total solids is printed as fig. 15 (p. 47).

TABLE 11.—*Analyses of ground water in the Arkansas Valley, western Kansas.*

WEST OF GARDEN, KANS.

Date.	Chlorine (parts per 100,000).	Alkalinity as $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.	Depth of well.	Location.
1904.					°*F.*	*Feet.*	
June 16	4.61	14.0	21.35	....	....	....	River water.
June 20	....	....	....	....	71.0	....	Do.
June 28	5.31	31.0	30.9	93	67.0	....	Do.
July 11	5.85	13.75	49.1	105	88.0	....	Do.
July 6	21.79	22.9	25.3	49	55.0	10	Windmill south of river.
July 5	8.10	16.5	....	83	58.0	9	Station 12.
June 28	8.51	19.0	34.7	119	48.0	28	Station 8.
Do	11.00	22.0	37.6	127	51.0	17	Station 10.
June 21	8.51	17.0	33.9	113	58.5	17	Station 4.
June 20	....	24.5	38.65	....	....	15	Station 2.
Do	6.72	15.0	43.20	....	52.0	15	Station 1.
June 18	4.96	20.0	40.51	....	....	32	Station 3.
July 8	6.00	16.1	39.5	121	52.0	32	Station 6, well A.
Do	3.05	11.4	13.9	37	52.0	58	Do.
July 9	2.70	12.9	21.6	32	55.0	48	Station 6, well B.
Do	1.67	11.9	14.6	36	53.0	56	Do.
July 6	4.59	14.7	33.5	105	55.0	30	Station 11.
Do	5.42	20.4	38.8	114	52.0	16	Mrs. Richter's well at camp.
June 15	13.50	23.0	53.5	....	....	5	Do.
June 16	12.80	19.5	47.9	....	....	5	Do.
Do	9.60	22.0	48.0	....	....	3-4	Test well No. 1.
June 20	21.30	20.5	48.0	....	59.0	3-4	Do.
Do	6.72	17.5	36.0	....	62.5	3-4	Test well No. 3.
June 23	18.12	24.0	39.1	121	60.0	3-4	Do.
June 28	11.00	20.0	39.5	126	52.0	12	New well (camp).

TABLE 11.—*Analyses of ground water in the Arkansas Valley, western Kansas*—Cont'd.

WEST OF GARDEN, KANS.—Continued.

Date.	Chlorine (parts per 100,000).	Alkalinity as $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.	Depth of well.	Location.
1904.					°F.	Feet.	
June 16	8.88	19.5	39.9	....	....	12	New well (camp).
Do	10.62	22.5	43.7	....	....	14	Station 1.
July 7	.78	13.1	10.7	6	65.0	....	Sand hills, sec. 36, T. 24 S., R. 34 W.
Do	.67	13.6	11.2	6	57.0	....	Do.
September 22	2.06	19.9	25.6	35	....	16	Sec. 2, T. 23 S., R. 33 W.
1905.							
January 24	4.2	19.2	53.3	86	....	25	Poor farm.
Do	2.1	11.4	31.1	57	....	20	Shultz.
Do	5.1	18.0	82.0	119	....	40	L. C. Working.
Do	4.1	20.5	31.2	102	....	36	A. Robinson.
Do	3.4	18.1	27.9	68	....	a 13	Foreman.
Do	11.4	19.2	39.3	76	....	115	Faye.
Do	17.6	22.7	45.9	150	....	85	M. McClurken.
Do	1.2	18.5	21.8	26	....	a 30	Frank Kolbus.

GARDEN, KANS.

Date.	Chlorine (parts per 100,000).	Alkalinity as $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.	Depth of well.	Location.
1904.							
September 22	0.92	14.1	25.6	16	....	130	Atchison, Topeka and Santa Fe R. R. well.
Do	.85	15.9	30.0	16	....	110	Carter's well.
Do	3.96	20.3	69.2	80	....	16+40	City waterworks well.
1905.							
January 24	1.6	18.8	29.5	42	....	78	S. L. Leonard.

SHERLOCK, KANS.

Date.	Chlorine (parts per 100,000).	Alkalinity as $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.	Depth of well.	Location.
1904.							
July 16	4.04	13.20	27.70	73.0	71.0	....	River water.
July 22	3.85	13.90	37.90	74.0	73.0	....	Do.
July 16	.89	21.20	13.09	27.0	63.0	8	Test well No. 6.
July 19	.50	17.50	4.64	30.0	58.5	18	Station 16.
July 16	.58	21.50	2.38	56.0	60.0	8	Test well No. 4.
July 26	1.10	17.85	26.20	42.0	56.0	26	Station 20.
July 18	3.62	16.75	27.30	35.0	57.0	22	Station 15.
July 21	2.46	21.30	28.10	55.0	56.2	36	Station 17.
July 30	4.61	19.45	44.70	83.0	65.0	10	Near station 17.
July 22	4.58	15.90	40.60	80.0	56.0	22	Station 18.
Do	4.05	15.90	42.90	78.0	57.0	14	Do.
July 23	5.20	17.45	46.30	104.0	53.0	20	Station 19.
July 16	3.47	14.65	30.00	93.0	57.7	18	Station 14.
Do	5.10	15.75	31.10	97.0	55.0	22	Do.
July 15	5.18	15.50	....	107.0	54.0	18	Station 13.
July 27	4.97	15.25	48.5	96.0	55.5	28	Station 21.
Do	4.90	16.25	50.6	97.0	57.5	28	Station 22.
July 19	.96	16.85	20.0	21.0	....	....	Sec. 30, T. 24 S., R. 34 W.
Do	.17	19.00	25.9	37.0	....	....	Sec. 20, T. 24 S., R. 34 W.
September 22	2.24	21.30	29.9	44.0	....	40	Sec. 30, T. 22 S., R. 33 W.

a To water.

TABLE 11.—*Analyses of ground water in the Arkansas Valley, western Kansas*—Cont'd.

DEERFIELD, KANS.

Date.	Chlorine (parts per 100,000).	Alkalinity as $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.	Depth of well.	Location.
1904.					°F.	Feet.	
September 22..	1.49	15.1	31.2	22.0	66.0	10	NE. quarter sec. 26, T. 24 S., R. 35 W.
August 6.......	2.60	14.7	28.9	49.0	....	24	SW. quarter sec. 24, T. 24 S., R. 35 W.
August 10......	2.45	17.7	32.7	74.0	60.0	12	Near station 28.
August 9.......	5.00	15.7	51.2	95.0	56.0	24	Station 27.
August 4.......	7.60	16.2	55.2	117.0	59.5	12	Well at camp.
Do.........	6.64	15.3	57.0	114.0	58.0	25	Station 23.
August 8.......	5.11	16.0	48.4	90.0	57.0	37	Station 26.
August 4.......	8.61	17.7	65.9	117.0	59.5	6	Test well No. 1.
August 5.......	5,32	15.5	44.4	108.0	59.0	21	Station 24.
August 6.......	5.39	16.7	48.3	106.0	57.0	16	Station 16.

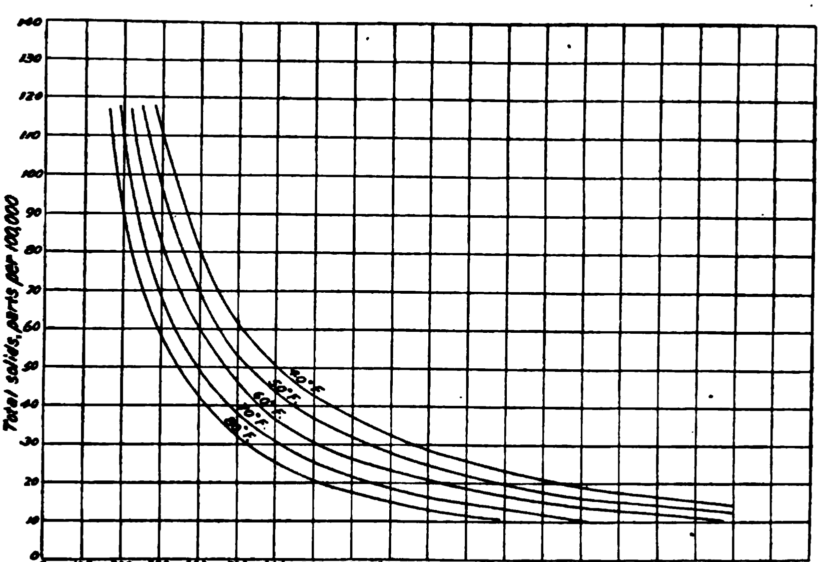


FIG. 15.—Curve for Whitney electrolytic bridge used in converting resistance in ohms into total solids for ground waters of Arkansas Valley.

A comparison of the results of the tests at various stations shows a marked decrease in the quantity of dissolved solids in the water with the depth at which the sample was taken. In forcing down test wells at almost any point in the bottom lands of Arkansas River the increasing softness of the water can be noted almost from foot to foot. At a considerable depth, say from 60 to 100 feet or more, there are found waters which are popularly called in this region "second" or "third" waters, which are very much softer than the water obtained from shallow wells. At points located in the sand hills south of the river there are places where shallow wells furnish water much softer than the so-called second or third waters found in the vicinity of Garden.

The total solids in the ground water determined at wells in the first camp, 2 miles west of Garden, varied from 121 parts per 100,000 for water taken 4 feet below the water plane to 103 parts per 100,000 for water taken at 6 feet, and 80 parts per 100,000 for water taken at 14 feet. Water taken from the railroad well, 130 feet deep, at Garden, showed total solids of 16 parts per 100,000. Water in the sand hills south of the river at a depth of 9 feet showed 33 parts per 100,000 total solids, and another well, deeper, but of unknown depth, showed 6 parts per 100,000 total solids. The tendency of the ground water near the surface in the bottom lands of the river to run high in solids seems to indicate that this increased hardness is due to the loss of the ground water by evaporation. The water plane in these bottom lands lies close to the surface of the ground and is subject to frequent fluctuations due to rain and changes of conditions in the river itself. These changes are sufficient to account for a large excess of dissolved solids in the surface waters, and it is believed that no other explanation is necessary. As the ground water moves downstream, the various filaments of moving water must thread themselves around the grains of sand and gravel, continually dividing and subdividing the water as it moves through the capillary pores. The effect of this action is to slowly work the concentrated water near the surface down to greater depths, forming a ground water of graduated strength. Every layer of silt, clay, or other impervious material which possesses a considerable area acts as a partition, separating the moving ground water into layers which do not mix, except where the impervious strata give out. This results in layers of water of distinct difference in total solids, which are locally known as "first," "second," and "third" water, etc.

In the following table (Table 12) the various samples of ground water are classified by depth of the wells, and the averages of the different determinations are tabulated. From this arrangement a comparison is possible between the waters of different depths, in which the errors due to special peculiarities of particular wells are partly eliminated. Some of the well water taken from stock or domestic wells showed marked pollution, but all such samples have been included in the table.

TABLE 12.—*Quality of ground water in Arkansas River Valley, as determined from the averages of classified samples.*

Classification.	Chlorine (parts per 100,000).	Alkalinity $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.
Wells under 10 feet deep:					°F.
Average of 11 samples	10.32	20.84	38.53	75.80	60.50
Probable error	1.45	.434	3.76	10.47	.735
Error per cent.	14.05	2.08	9.77	13.83	1.21
Wells 10 to 20 feet deep:					
Average of 18 samples	7.77	18.55	40.13	96.73	56.15
Probable error	.829	.520	1.321	4.87	.795
Error per cent.	10.66	2.80	3.30	5.04	1.42
Wells 20 to 30 feet deep:					
Average of 14 samples	4.96	16.28	40.95	91.00	55.50
Probable error	.335	.251	1.989	5.162	.552
Error per cent.	6.76	1.54	4.85	5.68	.995
Wells 30 to 40 feet deep:					
Average of 10 samples	4.62	17.62	38.00	92.75	55.05
Probable error	.397	.862	2.312	9.5	.74
Error per cent.	8.60	4.89	6.08	10.23	1.34
Wells 40 to 70 feet deep:					
Average of 6 samples	2.47	12.07	16.70	35.00	53.33
Probable error	.28	.298	1.659	1.031	.596
Error per cent.	11.33	2.47	9.93	2.95	1.12
Wells over 70 feet deep:					
Average of 4 samples	1.12	16.27	28.37	24.67	....
Probable error	.160	.924	.939	5.854	....
Error per cent.	14.29	5.67	3.31	23.7	....
Sand hills wells:					
Average of 9 samples	1.24	16.41	18.21	26.86	61.25
Probable error	.222	.587	2.32	4.05	1.07
Error per cent.	17.9	3.57	12.73	15.06	1.75

The above table is not free from objection, since the waters of the first bottoms, second bottoms, etc., have all been grouped together. The water in the first bottoms is softer than that in the second bottoms, owing to the ease with which both the rainfall and the softer water from the river contribute to its supply. In Table 13 all wells north of the river, less than 40 feet in depth, have been classified as first-bottom, second-bottom, and upland wells, and the averages of the various groups have been taken.

TABLE 13.—*Quality of ground water in wells north of Arkansas River Valley and less than 40 feet in depth, as determined from the averages of classified samples.*

Classification.	Chlorine (parts per 100,000).	Alkalinity $CaCO_3$ (parts per 100,000).	Degree of hardness (parts per 100,000).	Total solids (parts per 100,000).	Temperature.
First-bottom wells:					° *F.*
Average of 38 samples	6.86	18.18	42.81	93.75	56.67
Probable error	.447	.309	1.672	3.318	.387
Error per cent	6.52	1.7	3.91	3.54	.683
Second-bottom wells:					
Average of 7 samples	4.04	18.27	47.64	89.43	52.0
Probable error	.280	.819	5.40	5.938	([a])
Error per cent	6.93	4.48	11.3	6.65	....
Upland wells:					
Average of 3 samples	1.83	19.90	76.80	35.0	....
Probable error	.216	.545	1.673	3.5	....
Error per cent	11.8	2.74	2.18	10.0	....

[a] One observation.

CHAPTER IV.

ORIGIN AND EXTENT OF THE UNDERFLOW.

ORIGIN.

The investigations which have been explained in the preceding pages of this report indicate that the water of the Arkansas underflow has its main source in the rainfall upon the sand hills south of the river and upon the bottom lands and uplands north of the river.

The average annual rainfall in the vicinity of Garden is about 20 inches. A very large portion of this passes into the level and porous soil, so that the actual contribution to the underflow must be considerable. As previously stated in this paper there is a ground water district along the river that remains lower than the river, whether the same be flowing or not, in which region the rise in the ground water after a rain is more than can be accounted for by the localized precipitation. This fact indicates not only that the underground drainage at this point is contributed to by rainfall on distant catchment areas, but that the underflow constitutes a separate drainage system which is more than sufficient to take care of the rainfall. Determinations made in the sandy flats south of the river at Deerfield (see Chap. II) show that the rise in the water plane, observed after a rain storm, amounts to as much as 60 per cent of the water that fell. This fact verifies what is quite obvious to a careful observer, that there is no run-off from the lands adjacent to Arkansas River in the region under discussion.

The total depths of the deposits of sand and gravels at Garden is not known very exactly. A deep well was sunk at Garden in 1888, which, according to a partial log printed in the local newspaper, showed that rock was reached at a depth of 311 feet. Every indication drawn from the behavior of the ground water shows that the gravels must extend to a considerable depth, so that it is safe to assume that the well log just referred to gives a correct notion of the depth to rock. However, as one approaches the western boundary of Kansas, bed rock comes near the surface, which fact, even if no other evidence were at hand, would show that no portion of the ground water could originate in Colorado. The former popular belief in a Colorado source of the ground water has practically disappeared, although a few settlers still adhere to it. During the summer of 1904 one resident of Finney County informed the writer that the water in his well was invariably roily after a rain storm during the preceding night in Colorado. This corresponds to nearly passenger-train speed for the flow of ground water. The story may be regarded as about the sole surviving ghost of the numerous extravagant beliefs which were formerly current among the settlers.

The region near Garden, Kans., is peculiarly the area properly called the High Plains. The land is level and completely covered in its natural condition with a short compact sod of buffalo grass. Johnson and other writers on this region have remarked the complete lack of run-off from this portion of the plains area. The precipitation falls mostly during the summer months and is sufficient in amount to maintain a luxuriant sod, which not only protects the soil against erosion, but prevents, by the obstruction offered by the grass, the escape

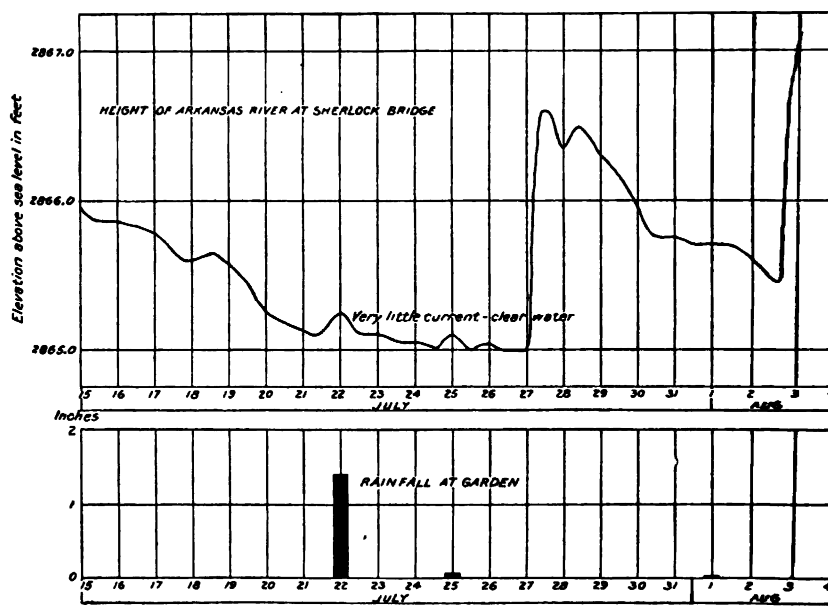


FIG. 16.—Elevation of water surface of Arkansas River at Sherlock Bridge, compared with rainfall record at Garden, Kans.

of the water in flowing torrents. In consequence the rainfall is completely taken care of by absorption into the ground and by evaporation and use by the vegetation. Eastward from the High Plains region rainfall is greater, and the sod is not able to prevent the formation of rills and eroded channels, so that much of the water runs off into surface streams. Westward from the High Plains district, as Colorado is approached, the rainfall decreases and in consequence vegetation becomes so scant that it is not able to protect the surface of the ground from erosion even from a diminished rainfall. Hence it is that both to the east and west of the High Plains there is a marked run-off, but in the plains district proper the rains are disposed of by absorption.

The above facts are well shown by the results previously discussed in this paper. The summer of 1904 was one of unusually ample rainfall in the plains, and many floods came down the river. The river was carefully watched by the field party and its elevation noted. Figs. 16 and 17 show the elevation of the river at Sherlock and Deerfield bridges, respectively, compared with the rainfall at Garden. A similar diagram for camp 1, near Garden, is given in fig. 10. A study of these diagrams shows practically no influence of the rainfall upon

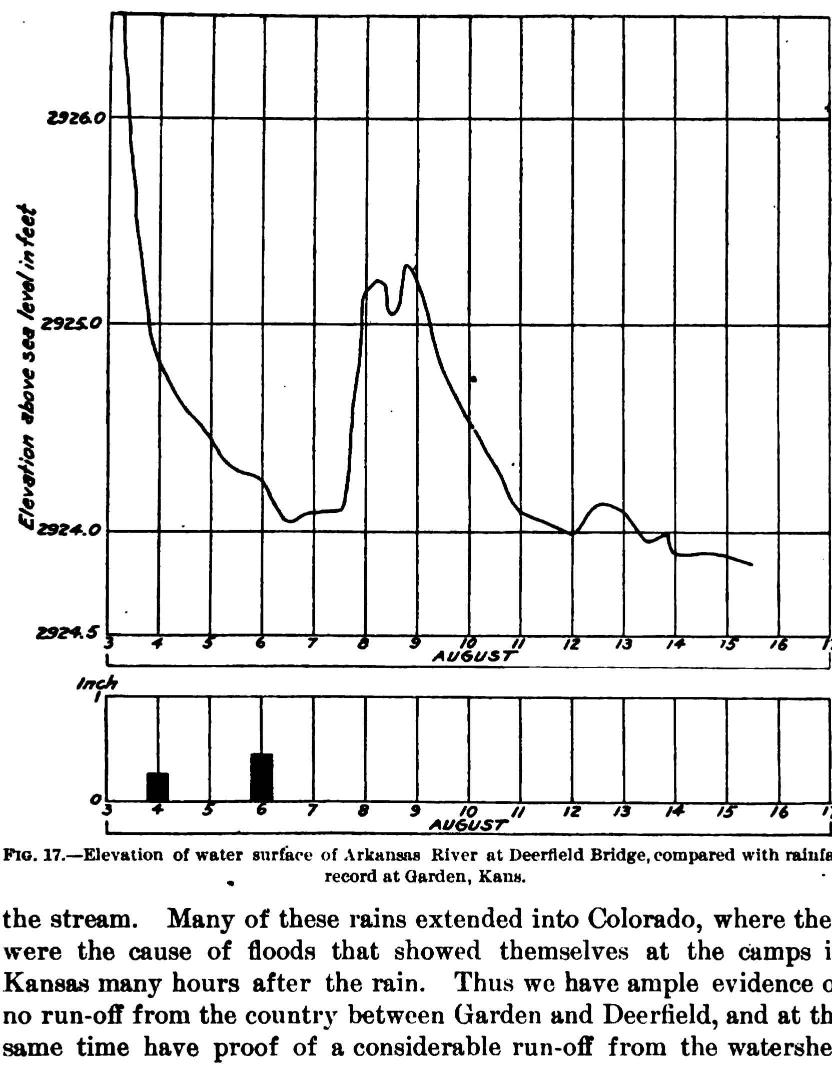


FIG. 17.—Elevation of water surface of Arkansas River at Deerfield Bridge, compared with rainfall record at Garden, Kans.

the stream. Many of these rains extended into Colorado, where they were the cause of floods that showed themselves at the camps in Kansas many hours after the rain. Thus we have ample evidence of no run-off from the country between Garden and Deerfield, and at the same time have proof of a considerable run-off from the watershed toward the western limit of Kansas and in Colorado.

The few instances in which small surface streams are formed near the Colorado line—like the plains streams known as Bear Creek and White Woman Creek—are no exception to the statement above that there is no run-off into the Arkansas in the High Plains district, for these streams entirely disappear as surface streams before the Arkansas is reached. Their waters, less the evaporation, are ultimately joined to the underflow. The situation may be summarized in the following words: The underground drainage in this region is so enormous, and the water passes through the gravel so freely, that there is no surplus water left to form surface streams, or to form a perennial supply for Arkansas River. If the gravels of the plains near Garden were less deep, it is entirely conceivable that the Arkansas River would be a perennial spring-fed stream at this point.

The large contribution to the underflow, which is made by the rainfall upon the sand hills south of the river, is clearly demonstrated by the course of the contours in fig. 5. In this diagram the soft water from the south side of the river can be observed to be pressing the hard water of the first bottoms northward toward the left side of the river valley.

Annual precipitation at Dodge and Garden, Kans.

Year.	Dodge.	Garden.	Year.	Dodge.	Garden.
1875	10.78	....	1890	11.72	....
1876	15.40	....	1891	32.34	27.[illegible]
1877	27.89	....	1892	19.66	....
1878	17.96	....	1893	10.12	....
1879	15.43	....	1894	12.60	11.4[illegible]
1880	18.12	....	1895	20.31	....
1881	33.55	....	1896	19.87	....
1882	13.14	....	1897	21.58	....
1883	28.50	....	1898	31.46	2[illegible].7[illegible]
1884	30.36	....	1899	28.45	20.[illegible]
1885	23.71	....	1900	20.76	19.[illegible]
1886	19.35	....	1901	16.06	18.3[illegible]
1887	15.71	....	1902	17.70	19.6[illegible]
1888	22.94	....	1903	15.27	20.6[illegible]
1889	19.17	....	1904	17.19	21.0[illegible]

NORTH AND SOUTH LIMITATIONS.

A noteworthy feature of the underflow is the lack of any natural north or south limitation to the easterly moving stream. There are important changes from place to place in the north and south slope of the water plane, but none are of sufficient consequence to materially modify the dominant influence of the easterly gradient of 7 to 8 feet to the mile. The velocities found at the edge of the sand hills to the south of the river, and at a distance as high as 9 miles from the channel of the river, are about the same as those found near the bed of the river in similar material. There is nothing surprising in this except that the stratification of the sand and gravel on the High Plains is such that there is no natural north or south limitation to the eastward-moving ground waters.

CHAPTER V.

SUMMARY OF TESTS OF SMALL PUMPING PLANTS IN THE ARKANSAS VALLEY.

GENERAL RESULTS.

Table 14 shows the results of tests of a number of pumping plants used for irrigation in Arkansas Valley between Garden and Lakin, Kans. Most of the entries in the table explain themselves.

The fuel used in most of the plants is gasoline, the current price of which during the summer of 1904 was 22 cents a gallon, a cost that is almost prohibitive, even when pumping water from the most excellent wells found in the valley.

TABLE 14.—*Tests of small pumping plants, Arkansas Valley, Kansas.*

1	2	3	4	5	6	7
Owner of plant.	Location.	Kind of pump.	Horse-power of engine.	Fuel used.	Price of fuel per gallon.	Total lift.
						Feet.
D. H. Logan	Garden, Kans.	No. 3 centrifugal	6	Gasoline	$0.22	22.1
Mrs. M. Richter	do	Menge	10	do	.20	15.5
C. E. Sexton	do	2 vertical 6 by 16 cylinder.	1½	do	.22	15.06
Nathan Fulmer	Lakin, Kans	Chain and bucket	7	do	.21	17.0
J. M. Root	do	do	2½	do	.22	15.8
King Bros	Garden, Kans.	No. 4 centrifugal	14			63.0
Waterworks	do	2 duplex steam				
I. L. Diesem	do	No. 4 centrifugal	10	Gasoline	.12½	22.13
L. E. Smith	do	No. 3 centrifugal	6	do	.12½	17.60
H. B. Holcomb	Sherlock, Kans	No. 14 centrifugal	80	Coal	[a] 4.00	23.0
H. S. Kipp	Garden, Kans.	2 horizontal 5 by 5 cylinders.	3½	Gasoline	.12½	21.7
J. R. McKinney	do	No. 4 centrifugal	5	do	.12½	21.47

[a] Price per ton.

TABLE 14.—*Tests of small pumping plants, Arkansas Valley, Kansas*—Continued.

1	8	9	10	11	12	13	14
Owner of plant.	Distance water is lowered	Yield of well per minute.	Specific capacity of well per minute.	Area of percolating or strainer surface.	Specific capacity per square foot of strainer per minute.	Cost of fuel per acre-foot of water.	Cost of fuel per 1,000 feet-gallons.
	Feet.	*Gallons.*	*Gallons.*	*Sq. feet.*	*Gallons.*		*Cents.*
D. H. Logan	6.85	272	42.2	107.0	0.394	$2.93	[illegible]
Mrs. M. Richter	5.3	394	73.0	266.5	.27	2.90	[illegible]
C. E. Sexton	3.0	91	30.3	57.2	.53	3.75	[illegible]
Nathan Fulmer	6.35	540	85.0	334.0	.254	1.37	[illegible]
J. M. Root	4.16	215	51.7	210.0	.246	2.78	[illegible]
King Bros	20.3	183	9.0	85.0	.106	....	....
Waterworks	5.48	290	77.0	247.0	.31	....	....
I. L. Diesem	6.72	363	54.0	151.0	.356	2.10	[illegible]
L. E. Smith	2.16	198	91.6	70.7	1.290	1.67	[illegible]
H. B. Holcomb	9.60	2,300	240.0	1,876.0	.128	[a] .85	[illegible]
H. S. Kipp	2.83	96	34.0	45.3	.75	1.09	[illegible]
J. R. McKinney	8.39	420	50.0	116.0	.42	1.20	[illegible]

[a] Including cost of labor and lubricating oil.

SPECIFIC CAPACITY.

The numbers in column 10 express the readiness with which the well furnishes water to the pump. The numbers in each case were found by dividing the numbers in column 9 by the corresponding numbers in column 8; these numbers, therefore, express the amount of water the well would furnish if the water level was lowered but 1 foot. These numbers constitute what the writer has called the "specific capacity" of the well, and are large in the case of a good well and small in the case of a poor well.

The water-bearing gravels are usually from 9 to 15 feet below the surface of the ground, and good wells can be very cheaply constructed. There is no quicksand or hardpan or other troublesome material above the water-bearing gravels. The well tubes or strainers are usually 12 to 20 inches in diameter, and are made of slotted galvanized iron. For the most part the wells are of the very best design and possess a remarkably high specific capacity; the writer knows of few places where better ones can be constructed.

The usual construction consists of a dug well, 6 to 10 feet in diameter, excavated several feet below the level of ground water, with a number of "feeders" or tubular wells penetrating the bottom of the well. No better construction can be suggested for small plants. The only modification in detail that seems likely to better the present excellent results would be the use of galvanized-iron strainers with larger slots than are at present in use. This would be practicable at some of the wells. Heavy pumping would remove much of the fine material that now remains in contact with the present well strainers.

In column 12 there are given the same magnitudes as are expressed in column 10, reduced in each case to 1 square foot of well strainer. The numbers in this column express, therefore, the amount of water in gallons per minute furnished by 1 square foot of well strainer under a head of 1 foot of water. They are a numerical expression of the degree of coarseness of the material in which the well is placed.

These numbers are almost the same for all of the well plants, when proper allowance is made for difference in construction. At the Richter, Fulmer, and Root plants, there are large dug wells with several feeders in the bottom. The numerous feeders interfere with each other somewhat, keeping the specific capacity lower than it would otherwise be. At the Logan and Sexton plants the construction is different. The Logan well is constructed of 20-inch casing, through the bottom of which are two 4-inch feeders extending 26 feet below the bottom of the 20-inch casing. The 20-inch casing is perforated for 10 feet at the bottom. At the Sexton plant there is a 12-inch well 22 feet deep, and a 10-inch well 31 feet deep, both perforated 10 feet from the bottom.

COST OF PUMPING.

While the cost of water at these various pumping plants may at first glance seem high, and the results not especially encouraging, yet a more careful inspection shows that the facts are really highly favorable. It must be remembered that the cost of pumping is based upon a 22-cent price of gasoline. This price is almost prohibitive, but fortunately there exist several possible ways of cutting down very materially the cost of power, and on this point the following suggestions are offered:

In the first place, the cost of pumping can be reduced by the use of crude oil in place of the gasoline. Crude oil from Kansas fields should be laid down at Garden at from 3 to 4 cents a gallon. The crude oil requires a special device, which must be used in connection with the gasoline engine, called a generator, in which the crude oil, or part of it, is converted into a gas before it is led into the engine cylinder. By the use of such a generator the cost of fuel can be lowered to a point about equivalent to a 5 cents a gallon price for gasoline. The crude-oil generators will work best on engines of 12 to 30 horsepower.

If plants of from 20 to 50 horsepower are constructed, as I believe will inevitably be the case in the near future, the cheapest power will probably be found in the use of coal in small gas-producer plants in connection with gas engines.[a] These small gas-producer plants are largely automatic in action and can be operated by anyone. With hard coal or coke or charcoal at $8 per ton, the cost of power would be less

[a] See test of producer-gas plant, Chapter VI.

than one-half cent per horsepower for one hour, or only one-fifth of the cost of power from gasoline at 22 cents a gallon. The writer anticipates no difficulty, therefore, in keeping the cost of water below 60 to 75 cents an acre-foot for fuel, or below $1.25 to $1.50 per acre-foot for total expense. Hundreds of such plants have been put in use in England during the past ten or more years, and they are in charge of unskilled labor. These gas-producer plants are used in England for a great variety of purposes, such as power for agricultural machinery, and for small electric-light plants for country estates, etc. They are used in as small units as 5 horsepower.

In this country the producer-gas plants have been in use for several years, and at the present moment they are fast taking the place of steam power in new plants. The cost of a producer plant and gas engine is about the same as the cost of a steam engine and boiler of same size when everything is included, but the cost of power from the producer-gas plant is very much less than that obtained from small steam engines.

In producer plants, ranging upward from 100 horsepower, a style of plant may be installed in which soft coal or lignite may be successfully used. This still further cuts down the cost of power. In fact, large plants of this type furnish the cheapest artificial power that has yet been devised. The saving is not only in fuel, but also in labor, as one man is capable of running a 300-horsepower plant.

That part of the operating expense which is properly chargeable to fuel cost can be accurately determined. Column 13, Table 14, expresses the cost per acre-foot of water recovered. In column 14 is given the cost of fuel for lifting 1,000 gallons of water 1 foot. For the purpose of comparison, these results are expressed in fractional parts of a cent. It should be noted that the cost given in the table is based upon a 22-cent price for gasoline. There is no doubt but that producer-gas plants in moderate-sized units would enable irrigation by pumping in the bottom lands of Arkansas River to be highly profitable.

No allowance has been made for interest, depreciation, and labor. These expenses, if included, would about double the cost per acre-foot.

CHAPTER VI.

DETAILS OF TESTS OF PUMPING PLANTS.

TEST OF PUMPING PLANT OF D. H. LOGAN, GARDEN, KANS.

This plant is located in the northeast corner of sec. 13, R. 33 W., T. 24 S., and is in the northwest corner of the city of Garden. The outfit consists of a 6-horsepower Fairbanks, Morse & Co. horizontal gasoline engine connected by a belt to a No. 3 centrifugal pump. The well is constructed of 20-inch galvanized-iron casing 32 feet long, perforated 10 feet up from the bottom, inside of which are two 4-inch feeders 28 feet long, perforated their entire length, and extending 26 feet below the bottom of the 20-inch casing, making a total depth of 58 feet. The pump has been in operation since April, 1902, and the engine since April, 1903. The water was measured by the use of a fully contracted weir with a length of crest of 0.66 foot.

The engine was started at 9 o'clock and the weir was ready for water at about 10.30. The water was turned on weir and the head read until it became constant at 1 p. m. In order to determine the expense of pumping, all of the gasoline was used out of the reservoir, then 1 gallon was poured in and the length of the run noted to be one hour and thirty-two minutes, or two-thirds gallon per hour. As the engine is a 6-horsepower one, this equals 0.111 gallon, or 0.445 quart of gasoline per horsepower hour.

The average corrected head on the weir was found to be 0.440 foot. Using weir formula

$$q=c\,\tfrac{2}{3}\,\sqrt{2\,g}\,b\,H^{\frac{3}{2}},$$

where $b=0.66$, whence $c=0.592$, the discharge is found to be

$$q=0.6045 \text{ second-foot}=272 \text{ gallons per minute.}$$

Data of Logan pumping plant, Garden, Kans.

	Feet.
Average depth to water while pumping	18.6
Normal depth to water	11.75
Amount lowered by pumping	6.85
Elevation of well platform	2,835.26
Distance water was raised above platform	3.5
Lift, or total distance water was raised	22.1

Total area of well strainer, 107 square feet.

The fuel cost of pumping was, therefore, 0.9 cent per 1,000 gallons of water recovered, or $2.93 per acre-foot. The cost of 1,000 foot-gallons (1,000 gallons raised 1 foot) was, therefore, 0.0406 cent, or one twenty-fifth cent.

The specific capacity of the well is 42.2 gallons a minute, or 0.394 gallon for each square foot of well strainer.

The engine ran at a speed of 350 revolutions a minute, exploding 143 times a minute. The diameter of engine pulley is 16 inches and of pump pulley 10 inches. This gives a speed of 560 revolutions a minute to the pump.

The size of the pond was 40 feet by 60 feet, mostly covered with a green scum, which would prevent evaporation. As to seepage, the pond falls 8 inches in twelve hours at night. The pond being 2,400 square feet in area, the observed seepage represents a loss of 16.68 gallons per minute, which should be added to the capacity of pump and well, but not to the effective capacity for Mr. Logan.

There is a windmill at a well 20 feet north of the one pumped by the gasoline engine—a 12-foot airometer connected to a 10-inch pump of 12-inch stroke. After the weir measurements were completed the windmill was thrown into gear. There was a brisk wind from the south and the pump threw a good quantity of water, but no appreciable lowering of the water in the gasoline-engine well 20 feet away was detected. The rise of the water in the well was obtained twice.

Below are the two sets of observations:

Rise of water after cessation of pumping in Logan well, Garden, Kans.

FIRST TRIAL—WINDMILL NOT RUNNING.

Time.	Depth to water.	Time.	Depth to water.
	Feet.		*Feet.*
55 seconds	[a] 18.60	2 minutes and 8 seconds	12.35
1 minute and 5 seconds	16.05	2 minutes and 22 seconds	12.35
1 minute and 20 seconds	14.55	2 minutes and 33 seconds	12.25
1 minute and 37 seconds	12.95	2 minutes and 48 seconds	12.15
1 minute and 55 seconds	12.50		

SECOND TRIAL—WINDMILL RUNNING.

Time.	Depth to water.	Time.	Depth to water.
24 minutes and 30 seconds	([a])	25 minutes and 48 seconds	12.55
24 minutes and 35 seconds	18.0	26 minutes	12.45
24 minutes and 45 seconds	16.5	26 minutes and 23 seconds	12.35
24 minutes and 48 seconds	14.35	26 minutes and 58 seconds	12.25
25 minutes and 10 seconds	13.10	27 minutes and 15 seconds	12.25
25 minutes and 26 seconds	12.90	27 minutes and 30 seconds	12.25
25 minutes and 38 seconds	12.56		

[a] Stopped pumping.

The curves showing the rate of rise of water in the Logan well after pumping ceased are given as curves 1 and 2 in fig. 18. Curve 2 is the one which was produced when the windmill was pumping from a well 20 feet away. The comparison of this curve with curve 1, which was produced when the neighboring well was not used, is very interesting, showing, as it does, a less rapid rise when the neighboring well was in use. To find the specific capacity for the Logan well from these curves we must substitute the values of the various constants in the formula

$$c=17.25 \frac{A}{t} \log \frac{H}{h} \text{ gallons per minute.}$$

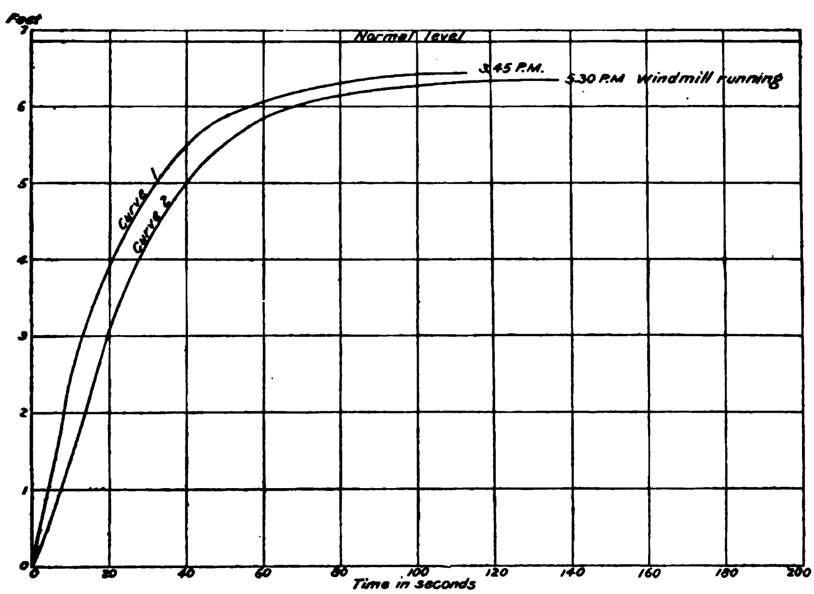


FIG. 18.—Rising curves for Logan well. Curve 2 taken when neighboring well was being pumped by windmill. Curve 1 taken when windmill was shut off.

The value of the area, A, of cross section of the well casing is 2.17 square feet, and H, the amount the water is lowered by the pump, is 6.85 feet. The amount of depression, h, of the water level below the natural level at any time can then be selected from the curve, and the specific capacity readily computed. If t be taken to be 40 seconds, or $\frac{2}{3}$ of a minute, h will be found from the curve to be equal to $6.85-5.5=1.35$ feet, hence

$$c = 17.25 \times \tfrac{3}{2} \times 2.17 \times \log \left(\frac{6.85}{1.35}\right) \text{ gallons per minute} = 39.5 \text{ gallons per minute.}$$

The yield of the well for the maximum depression, 6.85 feet, must then be

$$6.85 \times 39.5 = 270 \text{ gallons per minute.}$$

The curve of rise of water forms one of the best methods of determining the yield of a well. Such curves can readily be obtained. Well data should always include measurements of the amount of lowering of the water surface by the pumps, and it is only necessary to continue these measurements after the pumps have stopped to secure sufficient data to estimate the specific capacity and total yield of the well. This avoids the necessity of constructing a weir or other method of measuring the water discharge. The accuracy is sufficiently great for the purpose for which such data are used. The method can be used only in cases where an internal suction pipe extends into the well casing with sufficient room around it to permit a sounder to be lowered to the water surface. If there is no foot valve or other means for preventing the water from returning to the well after pumping ceases, the rising curve may still be used for the determination of the specific capacity, provided that only the portion of the curve be used which was formed after the water had completely returned to the well from the pump.

TEST OF THE RICHTER PUMPING PLANT, NEAR GARDEN, KANS.

This plant is located in the northwest corner of SW. ¼ sec. 14, R. 33 W., T. 24 S. The upper part of this well is cased with part of the old standpipe from Garden. The casing is 10 feet in diameter and extends down 20 feet. In the bottom of this part of the well are placed four 8-inch galvanized-iron feeders, arranged symmetrically about the center; each feeder is 25 feet long, perforated its entire length, and extends about 2½ feet above the bottom of the large part of the well.

The pump used is a Menge pump, which operates on the principle of a screw propeller of a steamship. It bores the water out and up a square wooden penstock or pump shaft. There are two of these propellers mounted one above the other on vertical iron shaft inside the penstock. The top of the iron shaft carries the belt pulley and has a shoulder bearing which takes the thrust of the pump as a pull above. This pump is made in New Orleans.

The pump is run by a 10-horsepower Otto gasoline engine, which runs at a speed of 300 revolutions per minute. The circumference of the drive pulley is 5.25 feet, and of the driven pulley 2.65 feet, making the pump run at 595 revolutions per minute. The screws are boxed up and under water when the pump is not in operation. A small pond was constructed at the end of the discharge trough and a fully contracted rectangular weir of length of crest of 1.2 feet was used to measure the discharge. The measurements for head were taken 6 feet away from the weir, and boards were interposed between the discharge trough and weir to cut down the velocity, which might tend to give erroneous results. The average corrected head on the weir was 0.371 foot. Using the weir formula

$$q=c\tfrac{2}{3}\sqrt{2\,g}\,b\,\mathrm{H}^{\frac{3}{2}}$$

and taking c from Merriman's tables as 0.603,

$$q=0.876 \text{ second-foot}=394 \text{ gallons per minute.}$$

Using a small Price acoustic water meter in the discharge trough, by measuring the velocity at different places and also by integrating, the discharge was found to be 0.76 second-foot, or 342 gallons per minute. The water in the flume was so shallow that this determination is of little value. By putting chips in the discharge trough and catching the time with a stop watch, the surface velocity was found to be 1.565 feet per second. This number multiplied by 0.8 gives an average velocity of 1.25 feet per second and a discharge of 0.884 second-foot, or 397 gallons per minute.

An attempt was made to determine the amount of gasoline used. The reservoir was filled full and the engine run for 1 hour and 36 minutes, or 1.6 hours. All the gasoline we had, 9¼ quarts, did not then fill the tank. This was at noon, July 6. On the morning of July 7, 9½ quarts were required to completely fill the reservoir, a total of 18¾ quarts or 37½ pints for the run of 1.6 hours for a 10-horsepower engine. The makers claim their engines use one pint per horsepower hour. This would require in this case 16 pints, or less than half of what was actually measured, if the engine developed its full horsepower. A leak in the tank or feed pipe is clearly indicated, so this amount, while being of value to the owner of the plant, is valueless so far as comparative cost of pumping is concerned.

Two observations of the rising curve were obtained which plot well together. The lower part of the curve is not accurate, because of the water in the penstock dropping back into the well when pumping ceases.

Data of Richter pumping plant, near Garden, Kans.

	Feet.
Elevation of the ground at well	2,846.0
Average elevation of water in well	2,836.8
Average elevation of water in well when pumping	2,831.5
Elevation of discharge from penstock	2,847.0
Lift	15.5
Average amount water is lowered by the pump	5.3

Number of explosions of engine, 126.5 per minute.

Total area of surface of well strainers and all percolating surfaces, 226.5 square feet.

The curves of rise for this well were obtained on two different occasions and are shown as curves 1 and 2 in fig. 19. They plot together very well. To find the specific capacity of the well from the curve, we note the following values of the constants in the formula for specific capacity:

$$c=17.25\ \frac{A}{t}\ \log\frac{H}{h}\ \text{gallons per minute.}$$

The area, A, of cross section of the well casing, less the amount occupied by obstructions, is 76.79 square feet. The amount, H, that the water is lowered by the pump is 5.3 feet. The amount of depression, h, of the water surface below the natural level at any time can be selected from the curve. From the curve, at the close of ten minutes, h equals 5.3 less 4, or 1.3 feet.

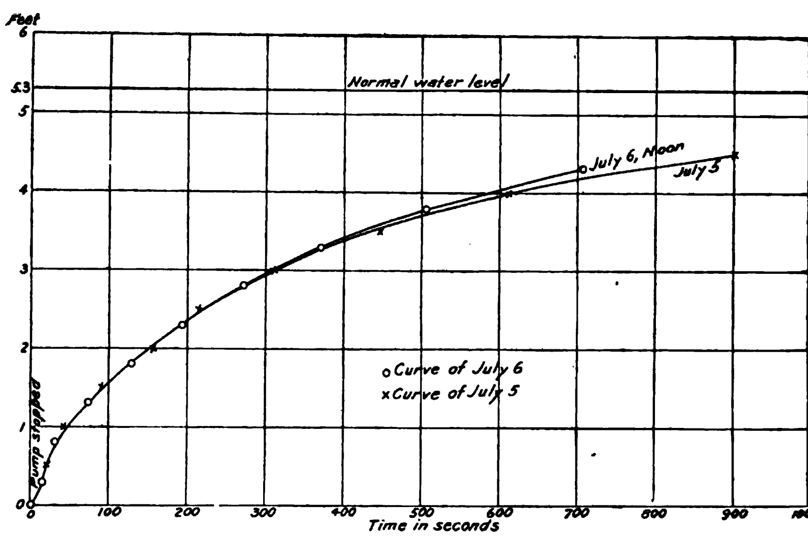


FIG. 19.—Rising curves for Richter well, near Garden, Kans.

Hence the specific capacity

$$c=17.25\times\frac{76.79}{10}\ \log\frac{5.3}{1.3}=81\ \text{gallons per minute.}$$

Multiplying by 5.3, the head under which pumping took place, the total yield of the well is $81\times5.3=430$ gallons per minute.

The above determination of the specific capacity is inaccurate, since the first portion of the rising curve does not show the true rate of rise of water in the well. The penstock of the propeller pump holds 37.7 cubic feet of water, which immediately returns to the well when the pump is stopped. This amount of water is sufficient of itself to raise the level in the well by 0.465 foot. For this reason, only that portion of the rising curve should be used which is not influenced by the returning water from the penstock. Thus, if we use that part of the curve from $t=100$ seconds to $t=600$ seconds, we will eliminate the inaccurate portion. Making this modification, the data are changed to

$$H=3.75 \text{ feet; } h=1.30 \text{ feet; } t=8\tfrac{1}{3} \text{ minutes.}$$

Computing the specific capacity on this basis, we obtain

$$c=73 \text{ gallons a minute.}$$

Multiplying this by 5.3, the total estimated yield is 388 gallons a minute, which checks remarkably with 394 gallons a minute obtained.

The area of the strainer and bottom of the well is 266.5 square feet. The above specific capacity divided by 266.5 gives 0.341 gallon per minute as the specific capacity per square foot of percolating surface.

The engine ran at a speed of 300 revolutions and exploded 125 times per minute. This would indicate that it was working at about 83 per cent of its rated capacity. Assuming that such was the case, and that it would then use 83 per cent of the fuel necessary to run it at its full rated power (10 horsepower), we have 8.3 pints as the probable amount of gasoline used per hour by the engine during the test. This, at 20 cents per gallon, would make a cost of 21 cents per hour. This assumption makes the cost of water 0.89 cent per 1,000 gallons, $2.90 per acre-foot, and one-seventeenth cent per 1,000 foot-gallons.

TEST OF PUMPING PLANT OF C. E. SEXTON, NEAR GARDEN, KANS.

This plant is located at about the center of sec. 13, R. 33 W., T. 24 S., and is 1 mile west of Garden. It consists of two pumps of 16-inch stroke, with 6-inch pistons, connected to a walking beam and driven by 1½-horsepower Fairbanks, Morse & Co. vertical gasoline engine. The east well has a 12-inch casing 22 feet deep, and the west well a 10-inch casing 31 feet deep, both casings being perforated for a distance of 10 feet up from the bottom. The pump rods are 2 by 4 timbers.

The two pumps discharge into an artificial pond or reservoir, and the flow was measured with a weir at the outlet of the reservoir. The weir was fully contracted with a length of crest of 0.66 foot.

The height of water on the weir was measured by placing a stick on the head of the nail and marking the water line on the stick with a pencil, then measuring with a pocket tape; in the absence of a hook gage this was the best method that suggested itself.

The weir heights taken as a measure of the discharge of the pump are those obtained after the water level in the reservoir had become stationary, as indicated by an absence of systematic variation of the weir heights. As evaporation would make the results too small, the following data are important:

The size of reservoir is 50 feet by 90 feet, or 4,500 square feet: trees border the north and south sides, with high grass along the banks; brisk wind was blowing from southwest; temperature of air was 80°, temperature of water, 52°; there was sunshine until about [illegible] p. m., when it became cloudy and the wind moderated.

The east well threw a much smaller stream than the west well, probably due to a leak in the suction pipe, and consequent pumping of air. No air was pumped by the west pump.

Measurements to the water surface in the east well were made at five-minute intervals, but no soundings were obtained in the west well. The number of strokes of each pump averaged 24.5 per minute during the test; the number of explosions of the gasoline engine averaged 106.2 per minute. The battery used with the engine not working satisfactorily, a gasoline torch was used for ignition. Gage readings of distance to water in well were made downward from a point on the well platform whose elevation above sea level was 2,836.69.

Data of Sexton pumping plant, near Garden, Kans.

	Feet.
Distance to water when level is normal	8.8
Distance to water when pumping	11.86
Amount water level was lowered	3.06
Elevation	2,827.9
Distance water was raised above point on platform	3.2
Total distance water was raised (11.86+3.2)	15.06

Total area of well strainers, 57.2 square feet.

The reservoir has been in use for some time and the seepage was probably quite small, a small enough per cent to be negligible. There was no leakage around the weir, or elsewhere.

The gasoline tank was filled at the start, and when the run was completed the amount needed to refill was measured, thus getting the amount used by the engine, which was 11 quarts for a run of 9 hours and 37 minutes, or 1.14 quarts per hour, making a trifle over three-fourths quart per horsepower hour. The average corrected weir height was 0.206 foot.

Using the formula for a contracted weir

$$q=c\tfrac{2}{3}\sqrt{2g}\,b\,H^{\frac{3}{2}},$$

and taking from Merriman's Hydraulics the value of the constant c for $b=0.66$ and $H=0.206$ as 0.611, we have for the discharge

$$q=0.202 \text{ second-foot}, =91 \text{ gallons per minute.}$$

With gasoline at 22 cents per gallon, or 5½ cents per quart, the expense of an hour's run, not counting gasoline used for ignition tube, is

$0.0625 per hour, or $0.0115 per thousand gallons of water pumped, or $3.75 per acre-foot. The lift being 15.06 feet, the cost per 1,000 foot-gallons is 0.076 cent, or about one-thirteenth cent per 1,000 gallons raised one foot.

On July 8 the rise of water in the east well was taken by means of a thin pine board stuck down between the casing and pump. The intervals of time were measured with a stop watch. The pine strip was lowered into the well until the water was reached, after which the board was drawn up, the wet line marked, the time recorded, and the board replaced, the observations being repeated as fast as possible. The distances marked on the strip were measured later.

Rise of water after cessation of pumping in Sexton well, near Garden, Kans.

Time.	Rise.
	Feet.
8 seconds	0.46
20.5 seconds	2.44
56.6 seconds	2.92
82 seconds	3.01
104.6 seconds	3.02
134.5 seconds	3.03

The rising curve plotted from these data was of little use in determining the specific capacity of the wells, both on account of an unknown amount of water returned to the well by leakage of the pump, and because of the unknown amount of lowering of the water in the west well.

TEST OF PUMPING PLANT OF NATHAN FULMER, LAKIN, KANS.

This plant is in the center of NE. ¼ sec. 10, R. 36 W., T. 25 S., Kearney County, 3 miles south of Lakin, Kans. The well consists of a wooden casing, 6 feet in diameter and 10 feet deep, sunk with the top flush with the surface of the ground. Inside of this cylindrical casing and extending 9½ feet below the bottom of it is a tapered wooden curbing 10 feet long, 4 feet in diameter at the top, and 5 feet in diameter at the bottom. This curb was given the tapering form in order to lessen the friction on the sides in sinking the well. The total depth of the two large curbs is 19½ feet. Arranged in a circle in the bottom of the main well, about 5 inches from the edge, are 7 feeders. Four of these feeders are 7 inches and 3 are 8 inches in diameter. The length of each feeder is 23 feet 4 inches. The feeders extend down to within 3 or 4 inches of an underlying clay or silt and 8 inches above the bottom of the large well. The total depth of the well is 42 feet. The feeders are made of No. 20 galvanized sheet iron with three-eighths-inch perforations arranged in circles from three-fourths of an inch to 2 inches apart.

The material encountered in sinking the well, according to Mr. Fulmer, was, first, 4 feet of clay, then sand, which became coarser with the depth. The bottom stratum consists of a mixture of fine sand and gravel, some of the latter being the size of a hen's egg. Water was found at a depth of 8 feet.

A local make of chain and bucket pump, known as the Pittman pump, is used in this well. It consists of an upper shaft and submerged lower shaft around which run the two sprocket chains to which are attached the galvanized iron buckets, each with a capacity of 12.5 gallons. The buckets, 33 in number, are hung between the chains and are of such shape that when they come over at the top of the circuit they discharge the water readily into the discharge trough. allowing very little to run back into the well. To aid in starting the water down the trough a number of horizontal guide vanes are placed therein with a slope away from the descending buckets in such a way that the water is started down the trough with very little splashing back into the well. These pumps are of a recent design, and are made in Kearney County. There are three such pumps in operation, one run by a windmill near Garden, and one owned by Mr. Root, a test of which is described in this report (pp. 70-73).

The power is supplied through the proper gearing by a Howe gasoline engine, built by the Middletown Machine Company, which develops about 7 horsepower at 285 revolutions per minute. The engine is cooled by water taken from the discharge trough. The supply of gasoline is put in a rectangular sheet-iron tank, 2.4 feet by 2.6 feet by 1 foot high, which is placed in the ground outside the engine house. The ratio of the gearing between the engine and bucket chain is such that 175½ revolutions of the engine produce 1 revolution of the bucket chain, or 5⅛ revolutions of the engine to each bucket discharge.

The discharge trough empties into a reservoir from which the seepage is quite rapid. As there was no chance to put a weir between the pump and the reservoir, and since one placed at the outfall of the reservoir would measure only a portion of the water entering the reservoir, the amount of water pumped was measured by counting the number of revolutions of the bucket chain and computing the capacity of several buckets to secure an average value. The average capacity was found to be 12.52 gallons. The computed discharge. obtained by counting the revolutions of the bucket chain and noting the time, was 561 gallons per minute. It was estimated that the buckets lacked about 0.05 foot of being full, this being about 4 per cent of the measured capacity of the buckets. Also, during the run. 22 buckets came up empty, caused by the failure of the valve in the bottom to work, which amounts to a loss of one-fourth of 1 per cent of the total discharge. Reducing the observed 561 gallons by 4 per cent gives 540 gallons per minute as the corrected discharge of the well. The water level was lowered 6.35 feet below the normal. The lift to the discharge trough was 17 feet. The engine ran at 240 revolutions and averaged 64 explosions per minute.

The amount of gasoline used was determined by measuring the depth of gasoline in the tank at intervals and noting the time at each measurement; then by plotting a curve the average rate per hour of lowering of the gasoline in the tank was obtained, and, the horizontal cross section of the tank being known, the amount of gasoline used per hour was computed to be 0.65 gallon. The cost of gasoline was 21 cents per gallon in barrel lots, making the expense of running the

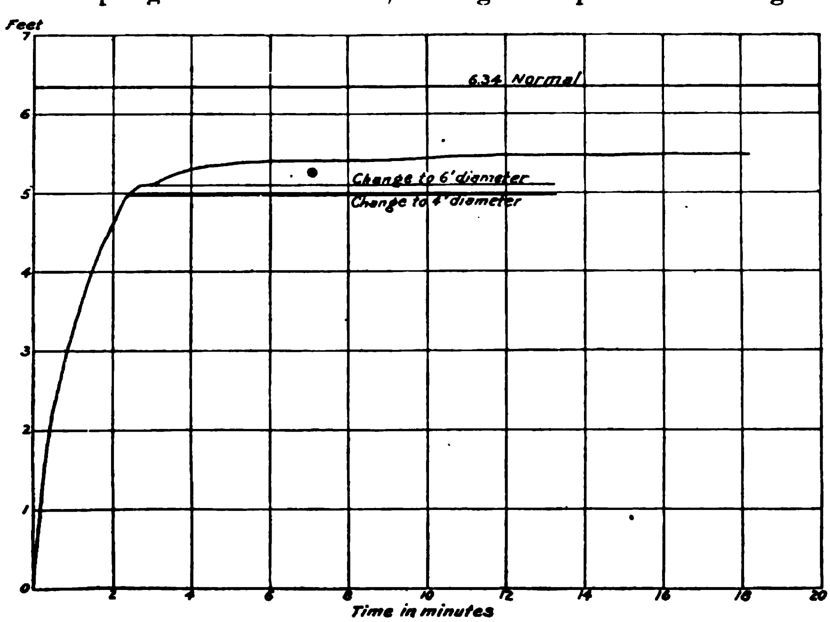


FIG. 20.—Rising curve for the Fulmer well, Lakin, Kans.

engine 13.65 cents per hour. The cost of water per acre-foot was therefore $1.37. The cost of water per 1,000 gallons was 0.42 cent, and the cost per 1,000 foot-gallons was one-fortieth of a cent.

A reservoir 100 feet wide by 240 feet long is used in connection with the plant. This reservoir was made by digging out the inside and using the material to form the banks. This produced a very porous bottom and much trouble has been experienced from seepage. To remedy this the bottom was puddled thoroughly by plowing and harrowing, then putting in chaff and straw and herding cattle and horses in the bottom for several days, but the surface of the water still drops about 6 inches per day.

One observation of the rising curve of this well was made (fig. 20).

It will be noticed that there is an irregularity in the curve corresponding to a depth of about 1½ feet below the normal water level, caused by the sudden change in cross section from 12¼ square feet to 28¼ square feet at the top of the lower casing. From the rising curve the specific capacity may be obtained from the following formula:

$$c=17.25\,\frac{A}{t}\log\frac{H}{h}$$

At a point 4 feet above lowest position of water level the average area A of the well from 0 to this point is 17 square feet. $t=1.6$ minutes; $H=6.35$ feet; $h=2.35$ feet.

Then $c=80$ gallons per minute. This, multiplied by 6.35, the amount the water was lowered by pumping, gives 508 gallons, which is within 6 per cent of the observed discharge.

The total area of percolating surface, 7 feeders, and the bottom of the well, is 334 square feet. The above specific capacity divided by 334 gives 0.24 gallon per minute per square foot of percolating area.

The amount of water recovered can not be increased without lowering the pump, as a glance at the diagram will show, the water level being now lowered slightly below the lower shaft.

The Fulmer plant was installed in the spring of 1903 and has been in operation since April of that year. The cost of the entire plant is as follows:

Cost of Fulmer plant, Lakin, Kans.

	Cost of material.	Labor.		Total cost.
		Time, in days.	Cost.	
Well:				
Material and lumber	$18.50	[a] 3	$6.00	$24.50
Digging		[a] 45	90.00	90.00
Seven feeders at $8.40 (24 feet each, at 35 cents a foot)	58.80			58.80
Reservoir, man and team, at $3.50 a day		40	140.00	140.00
Pump, made by Mr. Fulmer, market price about	260.00			260.00
Engine:				
Cost in Kansas City	328.50			347.12
Freight	18.62			
Shed, 8 by 22 by 7 feet	35.00	[a] 5	10.00	45.00
Incidentals				34.58
Total cost	719.42		246.00	1,000.00

[a] Labor, $2 a day.

Mr. Fulmer uses water from the south-side ditch, and only about 15 acres of cantaloupes and fruit trees are irrigated. The capacity of the plant is about 100 acres.

TEST OF PUMPING PLANT OF J. M. ROOT, LAKIN, KANS.

This plant is located at the southeast corner of northwest ¼ sec. 4, R. 36 W., T. 25 S., Kearney County, 3 miles southwest of Lakin, Kans.

The well consists of a wooden casing, 6 feet in diameter and 12 feet long, sunk with the top flush with the ground. Inside of and below this is a 10-foot casing, 4½ feet in diameter at the top and 5½ feet at the bottom, sunk until the top is 2 feet above the bottom of the upper casing, making the total depth of the main well 20 feet. In the bottom of this main well are sunk 5 feeders in a circle about 10 inches from the edge of the lower casing. The feeders are 8 inches in diameter; two of them are 24 feet long and three are 18 feet long. The 24-foot feeders project 2 feet above the bottom, while the 18-foot feeders project only 1 foot. These feeders are made of No. 20 galvanized iron, and the perforations are the same as in Fulmer's well, previously described.

The material encountered in sinking the well was, first, about 1 foot of sand, then about 17 feet of black dirt, followed by 1 foot of yellow clay and 2 feet of sandy clay. There is no record of the material encountered in sinking the feeders.

The Pittman pump is used in this well and is of the same pattern as that described in connection with the Fulmer plant. The buckets are smaller, having a capacity of 6.3 gallons, and the bucket chain has places for 40 buckets, 24 of which were in place at the time of the test. The vacant places were left at regular intervals around the chain, but the effect was to give the chain a swinging motion, which caused the slopping out of a great deal of water. The valves in the bottoms of the buckets also leaked excessively.

Power is furnished by a vertical 2½-horsepower two-cycle Weber gasoline engine with throttle governor, built by the Weber Gas and Gasoline Engine Company, Kansas City, Mo. The engine is cooled by a small tank and exploded by an autosparker. The ratio of the gearing between the engine and the bucket chain is such that 257 revolutions of the drive wheel produce 1 revolution of the bucket chain, or 6.4 revolutions of the engine to each bucket raised, if the buckets are all on the chain.

There is no reservoir used with this plant. The discharge was measured with a fully contracted weir, with a length of crest of 1 foot. The average head observed was 0.2805 foot, giving the following discharge by the Francis formula:

$$\begin{aligned} q &= 3.33\,(b - 0.2\text{ H})\,\text{H}^{\frac{3}{2}} \\ &= 3.33\,(1.0 - 0.056)\,0.2805^{\frac{3}{2}} \\ &= 3.33 \times 0.944 \times 0.1485 \\ &= 0.4675 \text{ second-foot} \\ &= 210 \text{ gallons per minute.} \end{aligned}$$

By the formula given by Merriman for fully contracted weir of length of crest of 1 foot the discharge is computed to be 218 gallons per minute. The following computations are based on a discharge of 215 gallons per minute: As the water level was lowered 4.16 feet the specific capacity is 51.7 gallons per minute. The lift was 15.8 feet. The engine averaged 488 revolutions per minute, exploding at every revolution.

The amount of gasoline used for a three-hour run was exactly 6 quarts, or at the rate of 0.5 gallon per hour. This gasoline cost 22 cents per gallon, making the cost of fuel 11 cents per hour. The cost of water is 0.855 cent per 1,000 gallons, $2.78 per acre-foot, and one-nineteenth cent per 1,000 foot-gallons. The lack of economy in this plant is in the engine, which is old and in poor condition, and in the buckets, the valves of which leak badly. Also the water was lowered so far that the buckets did not start up full, and the swinging motion of the chain spilled a great deal. The owner has never been able to keep the plant running for more than half an hour at a time, and it took as long to put the plant in order as it did to make the test.

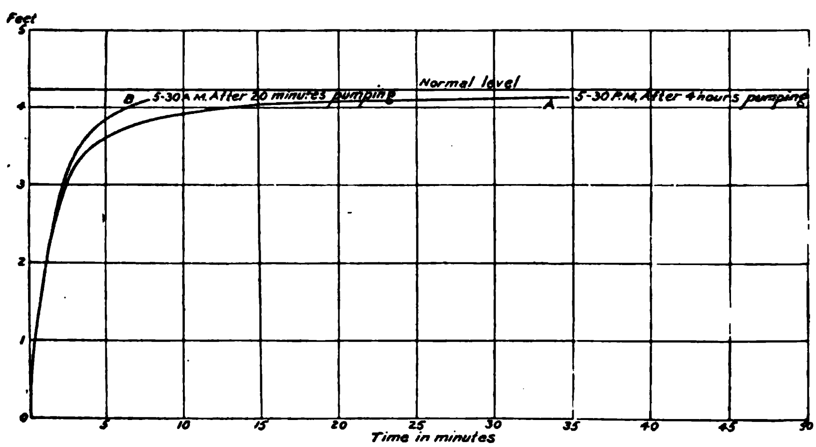


FIG. 21.—Two rising curves for the Root well: Curve A, after four or five hours of pumping; curve B, after only twenty minutes of pumping.

Two rising curves of this well were obtained, which make an interesting comparison (see fig. 21).

Curve A was taken late in the afternoon, after about four or five hours' pumping, and curve B was taken after about twenty minutes' pumping, when the water was lowered to the same depth as during the preceding afternoon.

Curve B is much steeper than curve A, showing that the water flowed into the well faster. This can be explained by the fact that during the short period of pumping (twenty minutes) the cone of influence had not extended as far as in the first case, and there was therefore less unsaturated soil to fill with water and a steeper slope of the ground-water surface.

The specific capacity of the well, determined from these curves, using the method described heretofore, is 62.5 gallons per minute. This multiplied by 4.16, the amount of lowering of the well by the pump, gives 260 gallons per minute, which is 19 per cent above the observed discharge. The percolating surface—area of feeders plus bottom of well—is 210 square feet, and dividing the specific capacity determined from the discharge by 210 we get 0.246 gallon per minute as the specific capacity per square foot of percolating area. This large error is probably caused by the steep slope given to the rising curve by the leakage of the water from the buckets. The pump must be lowered before a greater quantity of water can be recovered, as the water at present is lowered to the level of the lower shaft.

This plant has not been utilized for irrigation as yet, but its use is contemplated for irrigating about 20 acres of beets, cantaloupes, melons, and garden truck.

The Root plant was installed in the spring of 1904, being completed in the latter part of May. Its total cost was as follows.

Cost of Root pumping plant near Lakin, Kans.

	Cost of material.	Labor.		Total cost.
		Time in days.	Cost.[a]	
Well:				
Lumber	$27			$27
Feeders	42			42
Labor—				
Prospecting for location, digging big hole		2	$4	4
Making big curb		9	18	18
Sinking big curb		12	24	24
Sinking feeders		17	34	34
Pump	100			100
Engine	100			100
Installing			6	6
Shed:				
Lumber, nails, and window	33			33
Paint and painting	4			4
Labor			10	10
Total	306		96	402

[a] Labor, $2 a day.

TEST OF WELL AT KING BROTHERS' RANCH, GARDEN, KANS.

This well is located near the west side of sec. 30, R. 33 W., T. 22 S., about 12 miles northwest of Garden, Kans.

The well consists of a shaft, about 5 feet square, sunk 41.4 feet, to within 1.2 feet of the water level. From the bottom of this shaft a 15-inch, perforated, galvanized-iron casing extends down to a depth of 40.5 feet from the normal surface of the ground water.

It was put down by King Brothers to determine the amount of ground water which could be recovered at this point from a single well and its influence on other wells.

Fifteen feet from the first well a second well was sunk to a depth of 91 feet, 7.9 feet lower than the first well. This second well was put down for the purpose of determining the effect on the water plane of lowering the water in the first well by pumping.

A No. 4 Byron-Jackson centrifugal pump was placed at the bottom of the shaft of the first well and connected by a long belt running over 2 idle wheels to a 14-horsepower thresher engine on the surface of the ground. The discharge was measured by a fully contracted weir with a crest of 1 foot. The head at the time of maximum discharge, when the water in the well was as far down as the pump could lower it, was 0.25 foot, corresponding to a flow of 183 gallons per minute. This maximum rate was very difficult to maintain for any length of time, because of the temporary manner in which the machinery was installed. The belt was liable to slip and allow the water to rise several feet; also the idle wheels at the top of the shaft over which the belt ran were poorly mounted, and at times a stop was necessary to cool off a hot box at that place.

The above discharge was measured when the water level in the well was lowered 20.3 feet by the pump. Dividing the discharge by the distance gives the specific capacity of the well, or the amount of water furnished for 1 foot of lowering, as 9 gallons per minute. The total percolating area of well strainer exposed to the water was 85 square feet. From this it appears that the specific capacity of the well strainer is 0.106 gallon per square foot per minute.

As this was a test of the capacity of the well only, and not of the pumping plant, no indicator cards nor other device was used to get the efficiency of the plant, and no measure was made of the coal burned. The mechanical efficiency would undoubtedly have been low, as there was a constant slipping of the belt, and the idle wheels were home-made, running in wooden bearings, which were smoking constantly.

The maximum lowering of the water in the main well was 20.2 feet, and the corresponding depression of the water plane, 15 feet away, as indicated by the test well, was 3.5 feet. This shows the steep slope of the water plane and the comparatively small radius of the base of the cone of influence.

Readings were taken of the water level in the main well and the test well, and the discharge was noted at intervals. The accompanying curve, fig. 22, shows rising curves for the main well and the test well plotted together. A study of the curve brings out several facts that might well be expected. The rise of the test well lags slightly behind that of the main well. The curve of the main well shows an irregularity due to the caving in of material around the strainer.

King Brothers contemplate sinking 20 of these wells in a north and south line. They propose to connect them all with a tunnel just above the water plane and lay a main suction pipe in this tunnel, with branches tapping all the wells. The pumps will be located in the shaft already dug, and connected by a belt to the power plant on the surface.

The owners paid 40 cents a foot for sinking the wells and furnished one man. The price paid for the 15-inch, No. 16, iron casing was $1 a foot. They contemplate using wooden casing in the remainder of

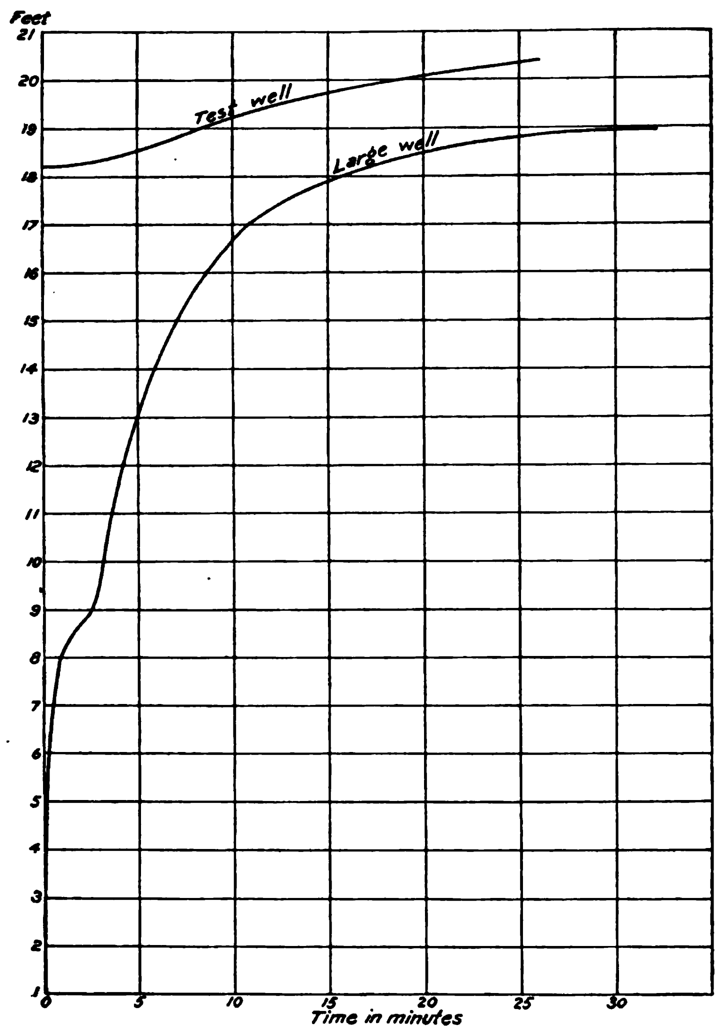


FIG. 22.—Rising curves for main well and test well, King Brothers' plant, Garden, Kans.

the wells. This will be made of pine lumber 1 inch by 3 inches. It will take 16 such boards to make the circular casing, at a cost of $3.50 per hundred linear feet of lumber. One man, at $1.50, can make and perforate about 25 feet of this casing in a day. This would make the cost of wooden casing 62 cents per foot.

After the tunnels and wells are dug King Brothers purpose to contract for the installation of a compound Corliss engine and centrifugal pump at about $9,500. They expect the plant to raise 4,000 gallons of water per minute, with a 60-foot lift, this being at the rate of 2,000,000 foot-pounds per minute on 4,800 pounds of coal per twenty-four hours. If the coal contain 12,500 British thermal units, and if the boiler efficiency be assumed at 75 per cent, engine 13 per cent, and belt 90 per cent, the pump would be required to have an efficiency of 70 per cent to realize the above expectation. These figures require that the plant turn out 5.9 per cent of the energy in the fuel in the form of useful work.

TEST OF CITY WATERWORKS WELL, GARDEN, KANS.

The first test began at 4.25 a. m. June 28, 1904, when one pump was started. The second pump was started at 5.50 a. m. The hydrants used in flushing the sewers were opened at about 7.30 a. m. and closed at 10.35 a. m. The east pump was stopped at 11.15 a. m.; the west pump was operated constantly all day. On account of the flushing of the sewers an exceptionally large amount of water was pumped during this test.

Gage heights in the well were read every five minutes, and the number of cycles of each pump was recorded every tenth minute for the ten preceding minutes. The pumping machinery consists of two compound steam duplex pumps, with cylinders 8 inches by 12 inches, which are very old and worn. The test was continued until 12.40 p. m. At 8 p. m. the test was again taken up, this being the time when the sprinkling of lawns is stopped. The cycles of the engine were counted and well heights taken as before.

Pumping is stopped at 9 p. m. Sprinkling of lawns is allowed from 7 to 11 a. m. and from 4 to 8 p. m. Most of the rise of water in the well occurs before 9 p. m., when the pump is stopped. A plug was made for the feeder and inserted July 7, but it did not fit tight enough to stop the flow. The rising curve was taken July 7 in the evening and also July 8, when the plug was driven down so as to be watertight. July 11 the rising curve was again taken when the water was lower.

The well is 16.2 feet inside diameter and 20 feet deep. The bottom is about 8.9 feet below the normal level of the ground water. There is a 10-inch feeder in the bottom of the well, which extends to a depth of 42 feet below the ground and about 3 feet above the bottom of the large well. It is open at the bottom and perforated 10 inches up from the bottom. The water level is about 11 feet below the ground level.

Data of city waterworks well, Garden, Kans.

	Feet.
Elevation of top of well roof	2,837.26
Distance to top of gage	10.35
Distance, top to 0	13.25
	23.60
Elevation, water normal	8.72
Normal elevation of water	2,822.38
Ground level	2,832.00

Elevation of bottom of well 2,813.66=0 of gage.

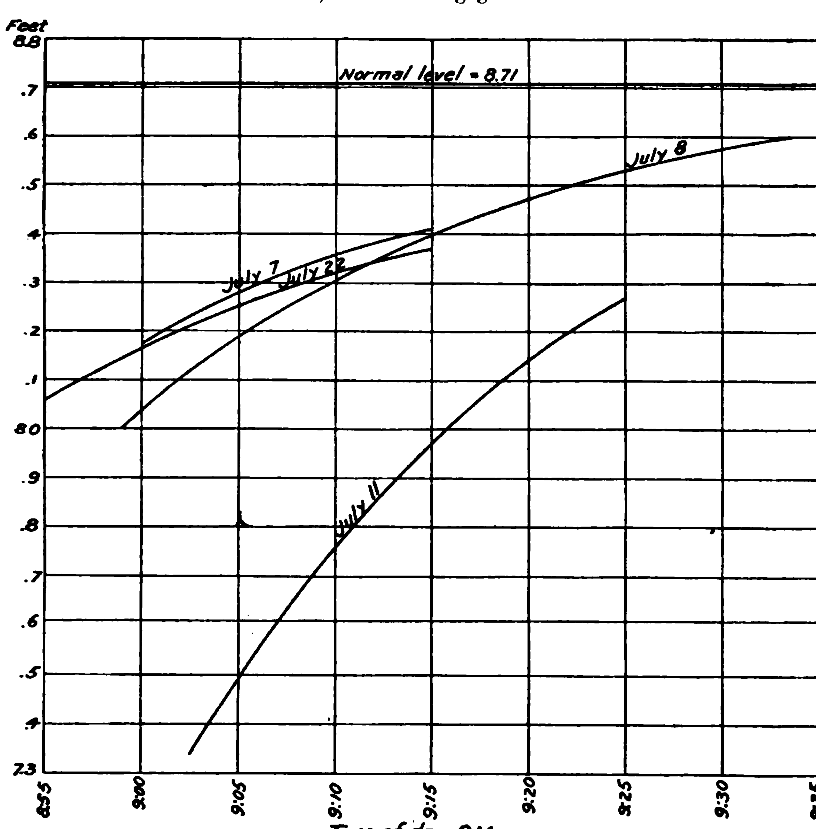


FIG. 23.—Rising curves for city waterworks well, Garden, Kans.

The rising curves obtained for this well on June 28, July 7, 8, and 11 are reproduced in fig. 23. Fig. 24 gives the engine cycles and elevation of water in the well for several hours of heavy pumping on June 28, 1904, while the sewers were being flushed. The displacement in the two cylinders of one of the pumps amounts to 1.362 cubic feet. The curve in fig. 24 enumerates the cycles of pumps, so that the total discharge of the pumps can be obtained, if no allowance be made for slip, by multiplying the number of cycles by 1.362. The discharge, computed in this way, amounts to 685 gallons a minute.

The amount of slip is enormous. Using the rising curve for June 28, we may place $H=0.65$, $h=0.46$, and $A=204.5$ square feet in the formula for specific capacity. This gives a specific capacity

$$c=53 \text{ gallons a minute.}$$

This result is much below the normal on account of the excessive amount of pumping on that day, due to the flushing of sewers. The maximum amount of lowering of the water in the well was 5.48 feet,

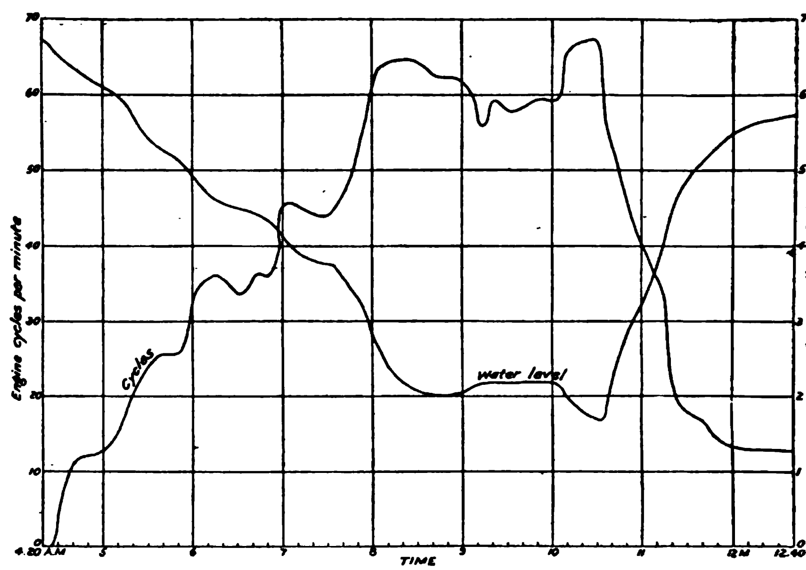


FIG. 24.—Elevation of water in city well, Garden, Kans., and engine cycles of steam pump during heavy pumping while flushing sewers, June 28, 1904.

which occurred at 10.30 a. m., when the pump cycles numbered 67 per minute. Multiplying 5.48 by 53, the total discharge at that time is found to be 290 gallons a minute. The slip of the pump must therefore have amounted to 57 per cent at this time.

Using the rising curve of July 7, the specific capacity of the well is found to be 67 gallons a minute. The following table shows the specific capacities computed for the several dates:

Specific capacity of city waterworks well, Garden, Kans., 1904.

Rising curve.	Specific capacity per minute.
	Gallons.
June 28, 8.55 to 9.05 p. m	53
July 7, 9 to 9.10 p. m	67
July 8, 9 to 9.10 p. m	73
July 11, 9.05 to 9.15 p. m	80

These results furnish interesting comparisons. The low specific capacity on June 28 was obtained after the prolonged and excessive pumping for flushing of the sewers. There was a light rain on the night of July 6, and a heavy rain on the night of July 7, which influenced both the consumption of water in the city, and in a slight degree the amount of ground water available.

On July 8 and 11 the feeder in the bottom of the well was plugged. The plug did not leak, but the casing of the feeder must have leaked badly since no influence upon the specific capacity of the well can be detected.

In Water-Supply Paper No. 67,[a] a rising curve for this same well is given, as observed by Johnson in 1900. From that curve it is possible to compute the specific capacity of the well in 1900. The following determinations are based upon various intervals after pumping has stopped, as indicated in the table. The specific capacity of a well always appears to be lower than its true value, if the very last portion of the rising curve be used, since at this period a large fraction of the water is being utilized in filling up the ground around the well.

Specific capacity of city waterworks well, Garden, Kans., 1900.

Intervals.	Specific capacity per minute.
	Gallons.
0-10 minutes	73.0
0-20 minutes	71.0
0-30 minutes	66.4
0-50 minutes	63.0
20-40 minutes	59.0
40-50 minutes	55.0

These results seem to be identical with those obtained in 1904.

The average specific capacity (77 gallons a minute) as determined in 1904 indicates that the maximum yield of the well, if the water in the well be lowered 8 feet, is 615 gallons a minute. The pumps in use at present can not pump much more than half of this amount of water on account of the worn condition of pistons and cylinders.

The total percolating surface of the well bottom and strainer of the feeder is 247 square feet. From this it can be deduced that the specific capacity of the well is 0.31 gallon a minute per square foot of percolating surface.

[a] Slichter, C. S., The motions of underground waters: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, p. 68.

TEST OF HOLCOMB'S PUMPING PLANT.

A very important attempt to recover water from the underflow was begun in the 1904 season by the owners of the Riverside stock ranch, about 7 miles west of Garden, Kans. A well 200 feet long and 5 feet wide, excavated to a depth of 9 to 10 feet below the water plane was constructed of sheet piling, and 11 galvanized-iron feeders were inserted in the bottom of the wells to a depth of about 20 feet. A 75-horsepower Corliss engine with condenser, a 90-horsepower boiler, and a No. 15 Byron-Jackson centrifugal pump were put in position at the north end of the well. Foundations for the engine and pump and buildings to cover the machinery were constructed in a very substantial manner. As soon as the engine and pump are in satisfactory working order it is purposed to sink a large number of additional feeders in the bottom of the well in the expectation of increasing its capacity to 6,000 gallons per minute. The approximate cost of the plant is about $8,000 for machinery and $4,000 for the well. Trinidad slack coal is used for fuel at a cost of $4 to $4.50 per ton.

The construction of this pumping plant has attracted very wide attention and if it proves to be a success it will mean a great deal for the progress of irrigation in the bottom lands of Arkansas Valley. There is some question whether 6,000 gallons per minute can be obtained from the present well, even with a very large number of additional feeders, but it will be entirely practicable to increase the length of the well without very much additional expense. The present well, with ten 20-foot feeders, 16 inches in diameter, would furnish about 6,000 gallons per minute, if we can rely upon a specific capacity of one-third gallon per minute for each square foot of strainer. This would require, however, the lowering of the natural level of the water to a distance of 10 feet, which is somewhat more than would be best for the most economical running of the plant. [a]

Both suction and discharge pipe of the centrifugal pump are made of No. 16 galvanized iron, riveted and soldered. A 20-inch flap valve is placed at the upper end of the discharge pipe, dispensing with the use of a foot valve. The pump is primed before starting by opening a 1-inch valve in a lead pipe from the main pump to the air pump. When the proper vacuum is shown by the gage the 1-inch valve is closed and the engine started.

A test run of the plant was made for five days, from July 18 to 23, 1905. The engine was started at the lowest speed at which it would work the pump satisfactorily. After running at this rate for forty-eight hours the speed was increased until nearly the full capacity of the well had been reached.

The amount of water pumped during the test averaged about 2,300 gallons a minute, or 5 cubic feet of water a second. This is equivalent

[a] Actual test of the plant shows that this amount of water can not be recovered without extending the well.

to a daily discharge of 10 acre-feet, or a sufficient amount of water to cover 10 acres of land 1 foot deep. As is well known, the present well is not sufficiently large to supply the pump and engine with all of the water that they are designed to handle; in fact, the pump and engine are capable of handling three times the amount of water at present available for long-continued runs. It is expected that by clearing out the feeders at present in the well, and by enlarging the well, the capacity of the plant will be greatly increased; but even at the present low rate of delivery, and consequent rather low efficiency of the machinery, the cost of water delivered is comparatively low.

The average amount of coal consumed was 2,450 pounds per twenty-four hours, or about 1¼ tons per day. At $4 a ton the daily cost of coal was $5 per twenty-four hours. The cost of labor for the day and night man, each at $1.25 per day, makes the cost for coal and labor $7.50 per twenty-four hours. The cost of lubricating oil and miscellaneous supplies may be estimated at $1 a day, making a total cost of $8.50 per twenty-four hours. At this rate the cost of water was 85 cents per acre-foot, not including interest on the plant nor any allowance for depreciation and repairs on the machinery and well. If these latter items be included, the cost of water would be very materially increased.

It seems, however, unfair to estimate these charges at the present time, as the expense of erecting the plant was incurred on the basis of securing a very considerably larger amount of water than is at present delivered; for that reason the interest charges would be very high, if charged against the present amount. It seems very probable that if the supply of water from the well is sufficiently increased the plant will ultimately be capable of delivering water into the ditch at a cost not to exceed $1 per acre-foot, including a moderate charge for interest and depreciation on machinery, but not including any profit.

The following tables show the fuel consumed and the data obtained during the test. The well was not of sufficient size to supply the pump with water, and toward the end of the run difficulty was experienced in operating the plant. Occasionally the water became so low that air would be taken into the suction pipe, and the plant would have to be stopped to prime the pump. In order to secure proper returns, it will be necessary to enlarge the well to about three times its present capacity, otherwise the engine and pump will be entirely too large for the well.

Consumption of coal at test of Holcomb pumping plant.

	Pounds.
July 20, 6 a. m. to 6.30 p. m	1,[illegible]
July 20, 6.30 p. m. to July 21, 6.20 a. m	1,[illegible]
July 21, 6.20 a. m. to 6 p. m	1,[illegible]
July 21, 6 p. m. to July 22, 6 a. m.[a]	1,[illegible]
July 22, 6 a. m. to 6.30 p. m	1,[illegible]
July 22, 6.30 p. m. to July 23, 6.30 a. m.[b]	1,[illegible]
July 23, 6.30 a. m. to 2.40 p. m	[illegible]
	8,[illegible]

Data of test of Holcomb pumping plant.

[From 7.54 a. m., July 18, to 2.40 p. m., July 23.]

Date.	Hour.	Discharge of flume.		Depth of water below its initial position.	Speed in revolutions per minute.	
		Cubic feet per second.	Gallons per minute.		Engine.	Pump.
				Feet.		
July 18..	7.54 a. m	0	0	0	0	[illegible]
July 18..	8 a. m	13.38	5,980	4.40	75	[illegible]
July 18..	8.45 a. m	7.42	3,330	7.25	73	[illegible]
July 18..	9.30 a. m	7.38	3,310	7.27	73	[illegible]
July 18..	11.20 a. m	6.29	2,820	7.44	73	[illegible]
July 18..	2.25 p. m	5.87	2,630	7.61	73	[illegible]
July 18..	4.25 p. m	5.32	2,380	7.61	73	[illegible]
July 18..	5 p. m	4.89	2,190	7.62	73	[illegible]
July 19..	7.15 a. m	4.67	2,090	7.80	73	[illegible]
July 19..	9.30 a. m	4.72	2,110	7.74	72	[illegible]
July 20..	7 a. m	5.30	2,380	7.87	74	[illegible]
July 20..	7.45 a. m	4.67	2,090	8.40	75	[illegible]
July 21..	6 a. m	5.82	2,610	8.50	74	[illegible]
July 21..	8 a. m	5.68	2,540	9.50	77	[illegible]
July 21..	8.20 a. m	5.30	2,380	9.53	77	[illegible]
July 22..	7 a. m	5.03	2,250	9.30	76	[illegible]
July 23..	.....do	5.15	2,310	9.60	77	[illegible]

TEST OF PRODUCER-GAS PUMPING PLANT NEAR ROCKY FORD, COLO.

The future of irrigation in the bottom lands of Arkansas Valley will be greatly influenced by the cost of power for pumping water. One of the possible ways of cutting down this cost is by the use of producer-gas in gas engines, as mentioned in Chapter V.

A 35-horsepower producer-gas plant has been installed by Mr. A. W. Shelton, about 6 miles northeast of Rocky Ford, Colo. It consists of a 40-horsepower Pintsch suction gas producer, a 35-horsepower single-cylinder gas engine, and a 12-inch Byron-Jackson vertical-shaft centrifugal pump. The water is pumped from a canal through 15-inch concrete tile (200 feet of intake and 300 feet of discharge) to an eleva-

[a] Stopped 35 minutes. [b] Stopped 36 minutes.

tion of 16 feet above the level of the water in the canal, called the first discharge, and at another point to an elevation of 28 feet above the level of the water in the canal, called the second discharge.

A test was made of this plant, extending from December 2 to 6, 1905. The results of the test of the engine and producer-gas apparatus are given herewith. Unfortunately the cement-discharge pipe gave out when the plant was first started, so that water could not be pumped during the test, and the hydraulic data for this plant are therefore not available.

In connection with the generation of the gas a vaporizer, scrubber, and purifier are used. Water is evaporated at atmospheric pressure to generate the steam required in the producer. The vaporizer is located directly on top of the producer. The scrubber is of the form in which a spray of water trickles down through coke, the water running out at the bottom of the scrubber. After being scrubbed the gas passes through a purifier box, next through a gas governor, and then to the engine.

The engine used was one of the Olds gasoline type, somewhat modified for the use of producer-gas. The engine governor was not the one belonging to the engine.

A belt was connected from a 30-inch pulley on the engine to a pulley on the vertical shaft of the centrifugal pump. A clutch at the engine shaft allowed the pump to be disconnected at will.

A small pulley, fastened to the shaft opposite the pulley end, carried a belt which drove a 3 by 5 inch "Baker" feed-water pump. This pump, making about 45 revolutions a minute, drew water from the well and discharged it into a 3 by 8 feet by 30 inches storage tank near the roof of the building and above the producer. A pipe leading from the bottom of this tank furnished all the water used to operate the plant, viz, water for the engine jacket, steam, and scrubber. During the brake tests it also supplied cooling water for the brake. The water, after being used, passed through the seals and then into the well from which it was drawn.

Preliminary brake tests of gas engine at pumping plant of A. W. Shelton, near Rocky Ford, Colo., December 4, 1905.

Test.	Net load, in pounds	Speed, in revolutions per minute.	Explosions per minute.	Mean effective pressure, in pounds per square inch.	Brake horsepower.	Indicated horsepower.	Mechanical efficiency (per cent).	Maximum pressure of explosion, in pounds per square inch.	Maximum pressure of compression, in pounds per square inch.
1	151	199	99.5	44.0	26.7	34.0	78.5	225	140
2	169	200	100.0	45.6	30.0	35.4	84.9	235	133
3	174	199	99.5	50.0	30.7	38.7	79.4	235	148

Diameter of piston .... 14 inches.
Length of stroke .... 20 inches.
Length of brake arm .... 56 inches.

Brake constant $=\frac{2\times\pi\times56}{12\times33000}=$ .... .00088

Engine constant $=\frac{20\times\pi\times14\times14}{12\times4\times33000}=$ .... .00778

Brake horsepower= .... Net load $\times$ Speed $\times$ .00088

Indicated horsepower= .... Mean effective pressure $\times$ Explosions $\times$ .00778

Mechanical efficiency $=\frac{\text{Brake horsepower}}{\text{Indicated horsepower}}$.

Indicator spring, pounds per square inch, =160 for Test No. 1, 250 for Test No. 2, and 250 for Test No. 3.

Of the gas producer a test of three hours' duration was made. The gas governor was not in operation. The producer was filled with coal at the beginning, as well as at the end of the run. As the engine was not operating well the load had to be taken off for a time.

Test of gas producer at pumping plant of A. W. Shelton, near Rocky Ford, Colo., December 4, 1905.

Time.	Net brake load.	Revolutions per minute.	Explosions per minute.	Temperature °F.					Coal.
				Jacket water.			Engine room.	Outside air.	
				Entering.	Leaving.	Range.			
P. m.	*Pounds.*								*Pounds.*
1.35	181	204	102	45	154	109	60	50	....
1.50	181	208	104	45	160	115	60	50	....
2.05	181	....	....	....	166	....	60	49	....
2.20	181	206	103	45	165	120	64	48	....
2.35	181	206	103	45	165	120	64	48	21
2.50	181	206	103	45	208	163	64	47	24
3.05	Load partly off.	204	102	45	208	163	....	46	....
3.20		202	101	45	....	....	....	46	....
3.35		....	....	....	....	....	....	45	....
3.50		....	....	....	....	....	....	44	17
4.05	181	218	117	45	....	....	....	43	18
4.20	181	220	110	45	....	....	....	42	....
4.35	181	....	....	45	....	....	....	41	21
Av.....	125	207	105	45	175	132	62	46	....
Total..	....	....	....	....	....	....	....	....	104

Summary of test of gas producer at pumping plant of A. W. Shelton, near Rocky Ford, Colo., December 4, 1905.

Duration of test .... 3 hours.
Net brake load (maximum) .... 181 pounds.
Net brake load (average) .... 125 pounds.
Revolutions per minute (average) .... 207.
Explosions per minute (average) .... 105.
Temperature of water entering jacket (average) .... 45° F.
Temperature of water leaving jacket (average) .... 175° F.
Range of jacket-water temperature .... 132° F.

Temperature of engine room (average) .... 62° F.
Temperature of outside air (average) .... 46° F.
Total coal consumed .... 104 pounds.
Pressure maximum explosion .... 258 pounds per square inch.
Pressure maximum compression.......................... 145 pounds per square inch.
Pressure, suction at exit of producer .... 2 inches water.
Pressure, suction at exit of scrubber...................... 2.125 inches water.
Pressure, suction at exit of purifier....................... 2.25 inches water.
Pressure, mean effective................................ 51.6 pounds per square inch.
Indicated horsepower ($51.6\times105\times.00778$)= .... 42.2.
Brake horsepower (maximum)=($181\times207\times.000888$)=... 33.2.
Brake horsepower (average)=($125\times207\times.000888$)= ..... 22.9.
Mechanical efficiency (maximum) $\frac{33.2}{42.2}$................. 78.9 per cent.
Mechanical efficiency (average) $\frac{22.9}{42.2}$..................... 54.2 per cent.
Pounds of coal per brake horsepower per hour, based on maximum brake horsepower $\left(\frac{104}{3\times33.2}\right)$.............. 1.05.
Pounds of coal per brake horsepower per hour, based on average brake horsepower $\left(\frac{104}{3\times22.9}\right)$ .... 1.51.

Test of gas engine at pumping plant of A. W. Shelton, near Rocky Ford, Colo., as shown by sample indicator card.

Duration of test (10 a. m. to 5 p. m.)................... 7 hours.
Rated horsepower of engine............................. 35.
Weight of engine....................................... 11,000 pounds.
Mean effective pressure (average of 54 cards) [a] .... 43.7 pounds per square inch.
Indicator spring... 160 pounds per square inch.
Load on brake .... 175 pounds.
Revolutions per minute................................... 201.6.
Explosions per minute.................................... 100.8.
Brake horsepower ($175\times201.6\times.000888$)................ 31.4.
Indicated horsepower ($43.7\times100.8\times.00778$) .... 34.3.
Mechanical efficiency $\left(\frac{31.4}{34.3}\right)$ [a] .... 91.5 per cent.
Kind of producer... Pintsch.
Producer rated horsepower................................ 40.
Temperature of water entering jacket..................... 49.8° F.
Temperature of water leaving jacket .... 165.8° F.
Range of jacket-water temperature .... 116° F.
Temperature of outside air .... 46.8° F.
Temperature of engine room .... 72.0° F.
Pressure of maximum explosion .... 260 pounds per square inch.
Pressure of maximum compression....................... 150 pounds per square inch.
Pressure of maximum steam.............................. Atmospheric.
Pressure of maximum suction at producer exit........... 2.2 inches water.
Pressure of maximum suction at scrubber exit........... 2.2 inches water.
Pressure of maximum suction at purifier exit .... 2.4 inches water.

[a] The high mechanical efficiency is probably due to an error in the indicated horsepower. The mean effective pressure appears to be too low. The reducing motion used was made of wood and had become considerably worn when this test was made.

Data concerning coal used in test of producer-gas pumping plant of A. W. Shelton, near Rocky Ford, Colo.

Kind	Colorado anthracite, Floresta mine.
Cost at plant per ton	$6.
Size	Pea.
Total quantity fired	325 pounds.
Total refuse (clinkers, ash, and unburned coal)	66 pounds.
Total clinkers	5 pounds.
Total unburned coal	43 pounds.
Total ash (siftings)	18 pounds.
Calorific value of coal per pound	13,850 B. T. U.
Pounds of coal per brake horsepower per hour, as fired and uncorrected for unburned coal in refuse $\left(\frac{325}{7\times31.4}\right)$	1.48.
Pounds of coal per brake horsepower per hour (corrected for unburned coal in refuse)	1.24.

Approximate analysis of coal used at producer-gas pumping plant of A. W. Shelton, near Rocky Ford, Colo.

	Per cent.
Moisture	2.2
Volatile matter	7.6
Fixed carbon	83.8
Ash	6.4

Water used per hour in producer-gas pumping plant of A. W. Shelton, near Rocky Ford, Colo.

	Pounds.
By jacket	1,200
By brake	930
By scrubber (approximately)	1,300
By vaporizer	16

Efficiencies at various loads of producer gas pumping plant of A. W. Shelton, near Rocky Ford, Colo.; test of December 6, 1905.

Time.	Net brake load (lbs.).	Revolutions per minute.	Explosions per minute.	Jacket water. Temperatures. Inlet.	Outlet.	Range.	Pounds per hour.	Mean effective pressure (pounds per square inch).	Indicated horsepower.	Brake horsepower.	Mechanical efficiency (per cent).
9.50	27	205	45.0	44	104	60	1,260	44.1	15.4	4.9	30.0
10.13	50	203	52.2	44	107	63	1,570	41.1	17.9	9.0	50.2
10.42	75	203	59.4	42	112	70	1,300	44.9	20.7	13.5	65.2
11.06	100	203	69.3	42	117	75	1,340	44.1	23.8	18.0	75.6
11.30	125	203	83.0	42	126	84	1,340	43.7	28.2	22.5	79.8
12.25	150	199	96.0	42	136	94	1,570	42.4	31.6	26.5	83.8
1.18	175	201	100.5	42	145	103	1,570	39.9	31.2	31.2	....
1.40	200	189	95.0	42	140	98	1,570	39.9	29.5	33.6	....
2.00	215	178	89.0	42	148	106	....	41.3	28.6	33.9	....
....	a225	....	....	....	....	....	....	....	....	....	....

a Engine would not carry load.

Analysis of gas from gas producer at pumping plant of A. W. Shelton, near Rocky Ford, Colo.

	Per cent.	B. T. U. per 100 cubic feet of gas at 60° F.
CO_2	6.3	
O	1.0	
CO	23.2	7,513
CH_4	0.0	
H	18.6	6,107
N	50.9	
	100	13,620

It will be observed from the results obtained in the test that 1.24 pounds of coal per hour produced 1 brake horsepower. At $6 a ton the cost of fuel was therefore three-eighths of a cent per brake-horsepower hour. At this rate power was obtained at a cost for fuel equivalent to gasoline at 3 cents per gallon. One-half cent per brake-horsepower hour for labor and five-eighths cent per brake-horsepower hour for supplies, depreciation, and repairs should cover all other charges. The total cost of power should not exceed, therefore, 1½ cents per brake-horsepower hour, or about $4.50 per day of ten hours, for the present plant. In this length of time the plant should furnish about 8 acre-feet of water on the 16-foot lift, or at a cost of about 58 cents per acre-foot.

The first cost of the pumping plant in round numbers, was $3,300 for the producer, engine, and pump; $200 for the building, and $1,500 for the intake and discharge pipe and flumes.

INDEX.

CLASSIFICATION OF THE PUBLICATIONS OF THE UNITED STATES GEOLOGICAL SURVEY.

[Water-Supply Paper No. 153.]

The serial publications of the United States Geological Survey consist of (1) Annual Reports, (2) Monographs, (3) Professional Papers, (4) Bulletins, (5) Mineral Resources, (6) Water-Supply and Irrigation Papers, (7) Topographic Atlas of United States—folios and separate sheets thereof, (8) Geologic Atlas of the United States—folios thereof. The classes numbered 2, 7, and 8 are sold at cost of publication; the others are distributed free. A circular giving complete lists may be had on application.

Most of the above publications may be obtained or consulted in the following ways:

1. A limited number are delivered to the Director of the Survey, from whom they may be obtained, free of charge (except classes 2, 7, and 8), on application.
2. A certain number are delivered to Senators and Representatives in Congress for distribution.
3. Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be had at prices slightly above cost.
4. Copies of all Government publications are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted by those interested.

The Professional Papers, Bulletins, and Water-Supply Papers treat of a variety of subjects, and the total number issued is large. They have therefore been classified into the following series: A, Economic geology; B, Descriptive geology; C, Systematic geology and paleontology; D, Petrography and mineralogy; E, Chemistry and physics; F, Geography; G, Miscellaneous; H, Forestry; I, Irrigation; J, Water storage; K, Pumping water; L, Quality of water; M, General hydrographic investigations; N, Water power; O, Underground waters; P, Hydrographic progress reports. This paper is the twelfth in Series K and the fiftieth in Series O, the complete lists of which follow (PP=Professional Paper; B=Bulletin; WS=Water-Supply Paper):

SERIES K, PUMPING WATER.

WS 1. Pumping water for irrigation, by H. M. Wilson. 1896. 57 pp., 9 pls. (Out of stock.)
WS 8. Windmills for irrigation, by E. C. Murphy. 1897. 49 pp., 8 pls. (Out of stock.)
WS 14. New tests of certain pumps and water lifts used in irrigation, by O. P. Hood. 1898. 91 pp., 1 pl. (Out of stock.)
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WS 41. The windmill; its efficiency and economic use, Pt. I, by E. C. Murphy. 1901. 72 pp., 14 pls. (Out of stock.)
WS 42. The windmill, Pt. II (continuation of No. 41). 1901. 73-147 pp., 15-16 pls. (Out of stock.)
WS 91. Natural features and economic development of Sandusky, Maumee, Muskingum, and Miami drainage areas in Ohio, by B. H. Flynn and M. S. Flynn. 1904. 130 pp.
WS 117. The lignite of North Dakota and its relation to irrigation, by F. A. Wilder. 1905. 59 pp., 8 pls.
WS 136. Underground waters of Salt River Valley, Arizona, by W. T. Lee. 1905. 196 pp., 23 pls.
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SERIES O, UNDERGROUND WATERS.

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WS 6. Underground waters of southwestern Kansas, by Erasmus Haworth. 1897. 65 pp., 12 pls. (Out of stock.)
WS 7. Seepage waters of northern Utah, by Samuel Fortier. 1897. 50 pp., 3 pls. (Out of stock.)

WS 12. Underground waters of southeastern Nebraska, by N. H. Darton. 1898. 56 pp., 21 pls. (Out of stock.)

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WS 31. Lower Michigan mineral waters, by A. C. Lane. 1899. 97 pp., 4 pls. (Out of stock.)

WS 34. Geology and water resources of a portion of southeastern South Dakota, by J. E. Todd. 1900. 34 pp., 19 pls.

WS 53. Geology and water resources of Nez Perces County, Idaho, Pt. I, by I. C. Russell. 1901. 86 pp., 10 pls. (Out of stock.)

WS 54. Geology and water resources of Nez Perces County, Idaho, Pt. II, by I. C. Russell. 1901. 87-141 pp. (Out of stock.)

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WS 57. Preliminary list of deep borings in the United States, Pt. I, by N. H. Darton. 1902. 60 pp. (Out of stock.)

WS 59. Development and application of water in southern California, Pt. I, by J. B. Lippincott, 1902. 95 pp., 11 pls. (Out of stock.)

WS 60. Development and application of water in southern California, Pt. II, by J. B. Lippincott, 1902. 96-140 pp. (Out of stock.)

WS 61. Preliminary list of deep borings in the United States, Pt. II, by N. H. Darton. 1902. 67 pp. (Out of stock.)

WS 67. The motions of underground waters, by C. S. Slichter. 1902. 106 pp., 8 pls. (Out of stock.)

B 199. Geology and water resources of the Snake River Plains of Idaho, by I. C. Russell. 1902. 192 pp., 25 pls.

WS 77. Water resources of Molokai, Hawaiian Islands, by W. Lindgren. 1903. 62 pp., 4 pls.

WS 78. Preliminary report on artesian basins in southwestern Idaho and southeastern Oregon, by I. C. Russell. 1903. 53 pp., 2 pls.

PP 17. Preliminary report on the geology and water resources of Nebraska west of the one hundred and third meridian, by N. H. Darton. 1903. 69 pp., 43 pls.

WS 90. Geology and water resources of a part of the lower James River Valley, South Dakota, by J. E. Todd and C. M. Hall. 1904. 47 pp., 23 pls.

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WS 102. Contributions to the hydrology of eastern United States, 1903, by M. L. Fuller. 1904. 522 pp.

WS 104. Underground waters of Gila Valley, Arizona, by W. T. Lee. 1904. 71 pp., 5 pls.

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B 252. Preliminary report on the geology and water resources of central Oregon, by I. C. Russell. 1905. 138 pp., 24 pls.

WS 120. Bibliographic review and index of papers relating to underground waters published by the United States Geological Survey, 1879-1904, by M. L. Fuller. 1905. 128 pp.

WS 122. Relation of the law to underground waters, by D. W. Johnson. 1905. 55 pp.

WS 123. Geology and underground water conditions of the Jornada del Muerto, New Mexico, by C. R. Keyes. 1905. 42 pp., 9 pls.

WS 136. Underground waters of Salt River Valley, Arizona, by W. T. Lee. 1905. 196 pp., 24 pls.

B. 264. Record of deep-well drilling for 1904, by M. L. Fuller, E. F. Lines, and A. C. Veatch. 1905. 106 pp.

PP 44. Underground water resources of Long Island, New York, by A. C. Veatch, C. S. Slichter, Isaiah Bowman, W. O. Crosby, and R. E. Horton. 1906. 394 pp., 34 pls.

WS 137. Development of underground waters in the eastern coastal plain region of southern California, by W. C. Mendenhall. 1905. 140 pp., 7 pls.

WS 138. Development of underground waters in the central coastal plain region of southern California, by W. C. Mendenhall. 1905. 162 pp., 5 pls.

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WS 145. Contributions to the hydrology of eastern United States; M. L. Fuller, geologist in charge. 1905. 220 pp., 6 pls.

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WS 149. Preliminary list of deep borings in the United States. Second edition, with additions, by N. H. Darton. 1905. 175 pp.

PP 46. Geology and underground water resources of northern Louisiana and southern Arkansas, by A. C. Veatch. 1906. — pp., 51 pls.

WS 153. The underflow in Arkansas Valley in western Kansas, by C. S. Slichter. 1906. 90 pp., 3 pls.

The following papers also relate to this subject: Underground waters of Arkansas Valley in eastern Colorado, by G. K. Gilbert, in Seventeenth Annual, Pt. II; Preliminary report on artesian waters of a portion of the Dakotas, by N. H. Darton, in Seventeenth Annual, Pt. II; Water resources of Illinois, by Frank Leverett, in Seventeenth Annual, Pt. II; Water resources of Indiana and Ohio, by Frank Leverett, in Eighteenth Annual, Pt. IV; New developments in well boring and irrigation in eastern South Dakota, by N. H. Darton, in Eighteenth Annual, Pt. IV; Rock waters of Ohio, by Edward Orton, in Nineteenth Annual, Pt. IV; Artesian well prospects in the Atlantic coastal plain region, by N. H. Darton, Bulletin No. 138.

Correspondence should be addressed to

THE DIRECTOR,

UNITED STATES GEOLOGICAL SURVEY,

WASHINGTON, D. C.

MAY, 1906.

O

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CHARLES D. WALCOTT, DIRECTOR

THE GEOLOGY AND WATER RESOURCES OF THE EASTERN PORTION OF THE PANHANDLE OF TEXAS

BY

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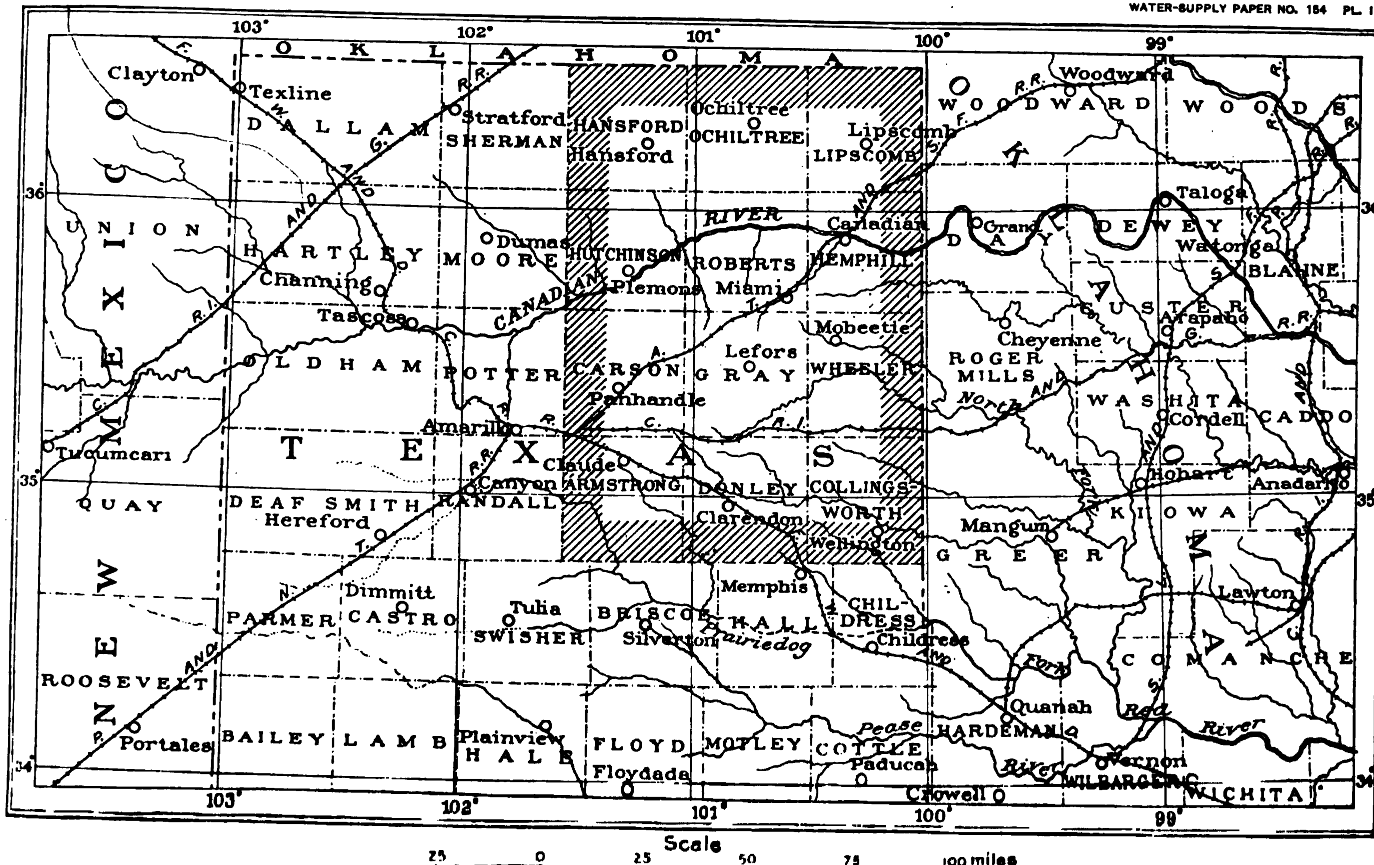
ILLUSTRATIONS.



MAP OF THE TEXAS PANHANDLE AND ADJACENT REGIONS, SHOWING THE AREA TREATED IN THIS REPORT.

GEOLOGY AND WATER RESOURCES OF THE EASTERN PORTION OF THE PANHANDLE OF TEXAS.

By Charles N. Gould.

INTRODUCTION.

Area covered.—The area described in this report lies in the northeastern part of the Texas Panhandle, and includes the following 12 counties: Lipscomb, Ochiltree, Hansford, Hutchinson, Roberts, Hemphill, Wheeler, Gray, Carson, Armstrong, Donley, and Collingsworth, each of which is approximately 30 miles square. It is an area 90 miles east and west and 120 miles north and south, situated south of the center of the Great Plains. The total area is approximately 10,800 square miles. It extends from 100° to 101° 35′ west longitude and from 34° 45′ to 36° 30′ north latitude. On the north and east it is adjoined by Oklahoma.

Sources of data.—The field work upon which this report is based was done during the years 1903 and 1904. During the former season little more was accomplished than a general reconnaissance in the region adjacent to Canadian River, through Carson, Hutchinson, Roberts, and Hemphill counties to the Oklahoma line, thence south through Hemphill, Wheeler, and Collingsworth counties as far as Elm Fork of Red River. On this trip the writer was assisted by Messrs. Charles T. Kirk, Chester A. Reeds, Charles A. Long, and Pierce Larkin, students in the University of Oklahoma. During the field season of 1904 the writer made an examination of the area to which this report relates, assisted by Prof. E. G. Woodruff. Most of the counties were studied in detail, excepting on the broader plains areas, of which only a reconnaissance was made. The well records were mostly secured from farmers and ranchmen by correspondence. Professor Woodruff has assisted in the preparation of the manuscript, "Topography" and "Water conditions by counties" being principally his work.

TOPOGRAPHY.

GENERAL FEATURES.

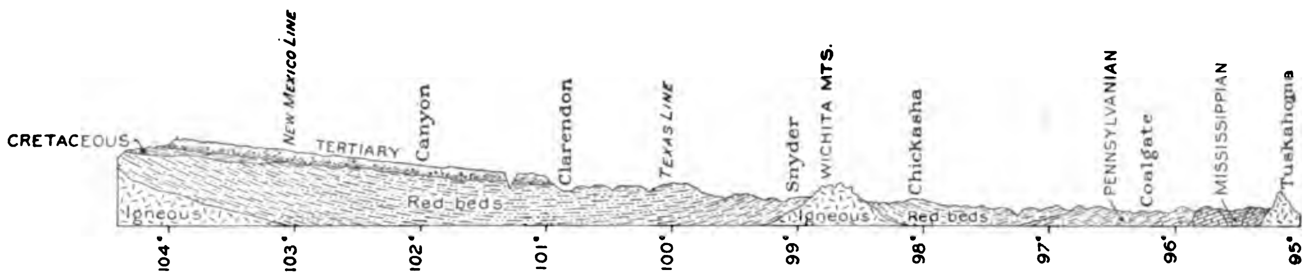
The region here described lies in the southern part of the Great Plains. Its general slope is to the east, with only a slight gradient to the south. The topography is properly divisible into two classes—the High Plains and the eroded plains—with local modifications produced by dune sands. A third and more local phase is found in the river flood plains. The location is shown on Pl. I, and the general features are indicated in the two general cross sections of the Great Plains shown on Pl. II, and on Pl. III, which includes the general region of the High Plains.

HIGH PLAINS.

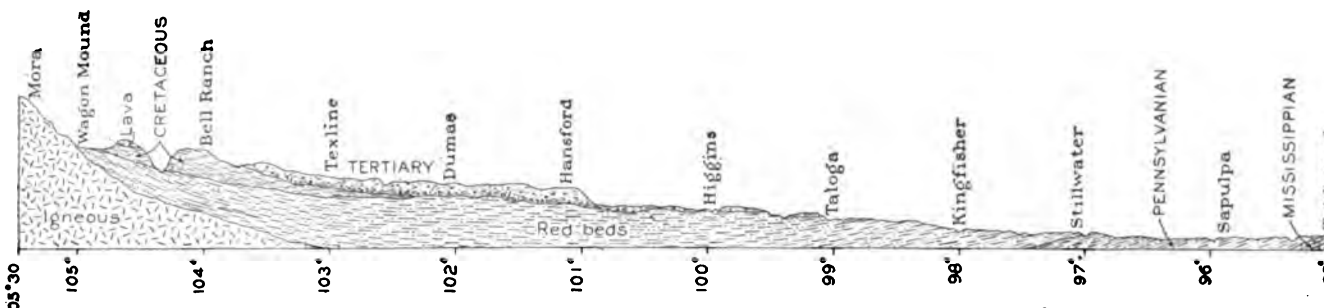
Surface features.—The region here treated as the High Plains includes not only the northern portion of the Llano Estacado or Staked Plains of Texas and New Mexico, but also the high, level plains in the region north of Canadian River. It seems probable that this area was once a great plain extending far to the east, with moderate slope covered by the deposits of the meandering rivers which were flowing from the mountains and depositing their load of sediment. By this deposition of material the stream beds were filled and the water forced to a new channel. By continued shifting of streams, irregular layers were deposited with much less uniform bedding than those of marine deposition. It is thought that the material composing the High Plains was laid down in this way upon the red beds, the basal formation in this region.

In later times the High Plains have been extensively cut into by stream erosion, until at present, in the region under discussion, the original level surface remains only in those localities more remote from the larger valleys. From a geological standpoint the erosion of the High Plains has been rapid and is still vigorously in progress. In the region comprised in this report High Plains constitute portions of the following counties: Western Lipscomb, most of Ochiltree and Hansford, southwestern Hemphill, southern Roberts, northwestern Hutchinson, western Gray, nearly all of Carson, and portions of Donley and Armstrong, including the greater part of the region mapped as Tertiary on Pl. V, an area of approximately 4,000 square miles.

In general the surface of the High Plains is so nearly level that railroads require little or no grading, and wagon roads go directly from point to point. With a surface so nearly level drainage is wholly undeveloped. Rain water can not run off, but either evaporates or collects in broad, shallow depressions, known in some locali-



CROSS SECTION OF THE HIGH PLAINS ALONG LATITUDE 34°45′



CROSS SECTION OF THE HIGH PLAINS ALONG LATITUDE 36°

GEOLOGIC SECTIONS ACROSS A PORTION OF NORTHWESTERN TEXAS.

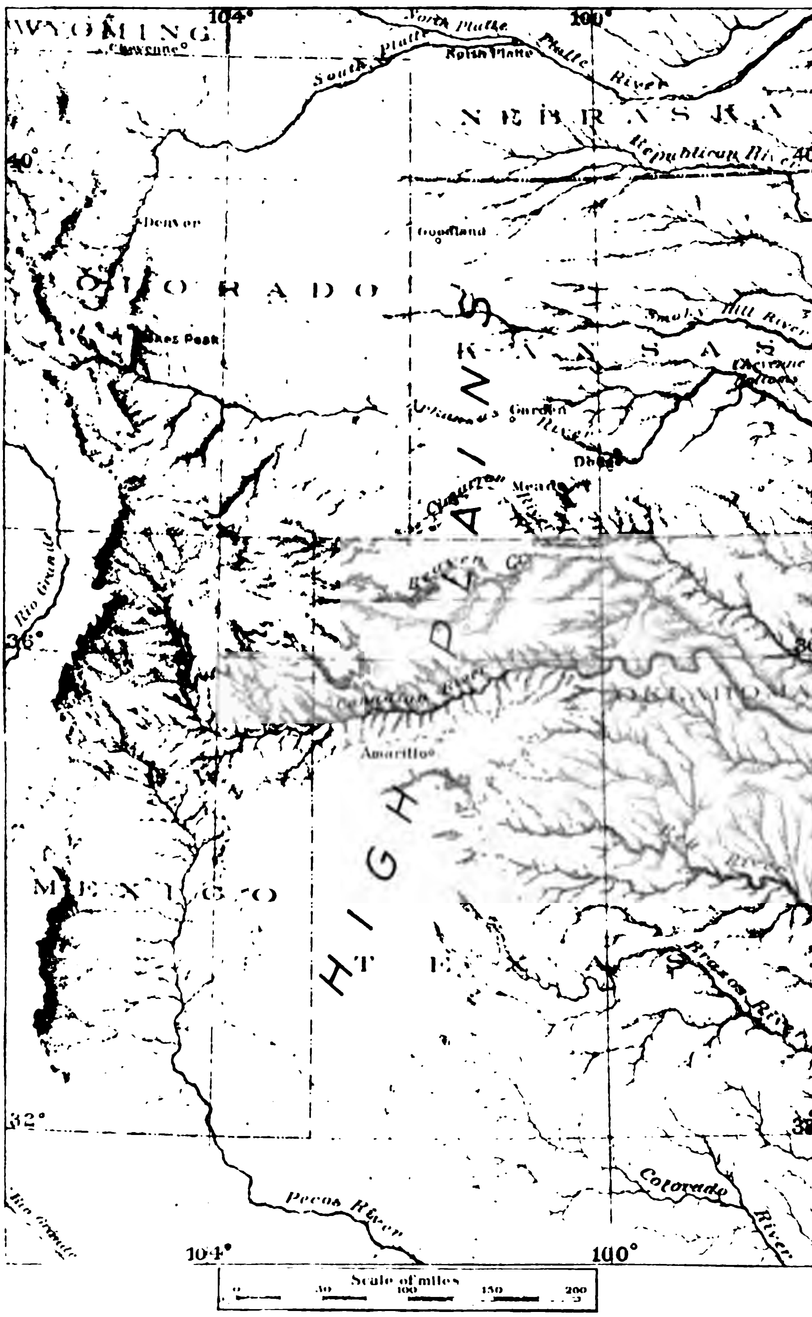
have a sparse vegetation, since the very rapid erosion prevents most kinds of plants from obtaining a foothold. Bunch grass, yucca, and dwarf mesquite are often present. Such a region is most difficult to traverse, and in localities where the breaks are conspicuous it can be crossed with a wagon only at infrequent intervals over specially selected routes. This escarpment is most typical along the Canadian and in Palo Duro Canyon, in Armstrong County.

ERODED PLAINS.

Interstream highlands.—From the High Plains the escarpment forms a descent to the lower level of the eroded plains, which occupy the entire eastern part of the region to which this report relates. From the eroded plains the Tertiary and Pleistocene rocks, which compose the High Plains, have been entirely removed, and the streams, both large and small, are now cutting deep valleys into the subjacent red beds. This part of the Panhandle is a rolling plain, which is now being eroded rapidly, yet without the conspicuous badland forms that mark the escarpment. The streams are confined almost entirely to rather deep, steep-sided valleys; the plains between are rolling and well drained.

Standing on this plain not far from the escarpment are outlying hills, generally conical, but often elongated, and joined into irregular ridges. They sometimes attain a height of 100 to 200 feet. These hills have resulted from the thickening and hardening of certain of the upper members of the red beds, usually ledges of sandstone, gypsum, or dolomite, which resisted erosion and protected the relatively softer clays and shales beneath. A line of such hills extends from near Shamrock, in Wheeler County, southwest to beyond Memphis, the county seat of Hall County. South from Shamrock the ridge reaches its maximum width near the post-office of Dozier, at which place the range is 10 miles wide. Here it consists of a number of isolated mesa-like hills rising 100 feet above the eroded plains and capped by a ledge of sandstone, described under "Geology," 6 to 14 feet thick. The most typical of these hills are Rocking Chair Mountain, north of Elm Fork; Antelope Hills, northeast of Dozier; the Dozier Mounds, southeast of Dozier, and Flat Top, northwest of Dozier. The range is interrupted in northern Collingsworth County by Salt Fork of Red River, but again becomes conspicuous in the southwestern portion of the county, where the creeks are rapidly trenching the valleys between the mesas and bringing the hills into strong relief.

Valleys and canyons.—Crossing the eroded plains at intervals are a number of streams which have their rise on the High Plains, and, after cutting through the escarpment, find their way into the larger



THE HIGH PLAINS.

rivers which receive the drainage of the Panhandle. For the most part these streams have carved valleys averaging 3 miles wide and 100 to 200 feet deep in the eroded plains. These streams have already been mentioned, and they will be discussed in more or less detail under "River plains."

SAND HILLS.

The sand hills form an important topographical feature of the Panhandle. In size the hills range from small mounds to ridges 30 to 40 feet high; in shape they are oval, crescent, or elongated, but when parallel they are separated by trough-like depressions. The hills extend in various directions, although, in certain localities, those ranging S. 15° E. appear to predominate. Within the sand-dune regions are broad, shallow, basin-like depressions which are probably large blow-outs covering 1 to 10 acres. There are a few localities containing migratory dunes. One such is on the south side of Canadian River, in western Roberts County, where the dunes are approaching the river. Another is north of Prairie Dog Fork of Red River, in southwestern Donley County.

The sand composing these dunes is derived from two sources, chiefly from the sandstone ledges of either the red beds or the Tertiary disintegrating in place, or from the river sand which in times past has been transported from farther west. These make two classes of sand hills, both of which are frequently found in the same region. The subject is treated more fully under "Geology."

Sand hills occur chiefly in the escarpment region or along the streams, as in western Lipscomb and northern Roberts and Hemphill counties, in Wheeler County south of Mobeetie, and in Donley and Collingsworth counties along the south side of Prairie Dog Fork. A typical sand hill is shown in Pl. IV, *A*.

RIVER PLAINS.

Canadian Valley.—The north central part of the Panhandle of Texas is traversed by Canadian River, which rises in the mountains of New Mexico and in its eastward course crosses the region under discussion in a valley 5 to 20 miles wide cut deeply into the High Plains. The sides of this gorge constitute a portion of the escarpment, with its bad-lands structure of short, sharp ridges, often destitute of vegetation, separated by V-shaped valleys. The flood plan, 1 to 5 miles wide, occupies the bottom of the gorge, 600 feet below the level of the High Plains. The river runs over a sandy bed varying in width from a half mile to more than a mile. It is constantly shifting, excavating sand in one place and depositing it in another.

Wolf Creek Valley.—Wolf Creek has cut a wide valley in the High Plains in the northeastern portion of this region. It rises in western Ochiltree County, at an elevation of 3,300 feet, and descends to 2,350 feet at the Oklahoma line, 45 miles east—a gradient of 21 feet per mile. The width of the valley varies from 1 to 4 miles, and the breaks which adjoin it are less rugged than those along the Canadian. Sand hills occur along this creek and its tributaries.

Washita River Valley.—The headwaters of the Washita, which in Oklahoma becomes a river of considerable size, rise in Gray County, Tex., in a small creek not differing from many others in this part of the plains. It flows eastward in a valley 1 to 3 miles wide across northern Wheeler County and finally passes from Texas into Roger Mills County, Okla.

North Fork of Red River Valley.—North Fork of Red River rises among the High Plains in the southeastern part of Carson County, and flows east in a broad bend to the north, passing from the State almost directly east of its starting place. It flows in a narrow, sand-choked valley, with sand dunes flanking its south side and with red-beds bluffs guarding it on the north for a considerable part of its course. This river descends from an elevation of 3,000 feet on the plains to 2,050 feet at the State line, making a descent of 950 feet in a passage of 60 miles, or 16 feet per mile.

Elm Fork of Red River Valley.—Elm Fork of Red River, which becomes a stream of considerable importance in Greer County, Okla., is in the Panhandle a mere creek, the greater portion of whose bed is entirely dry during the summer. It rises in the escarpment in northwest Collingsworth County and, flowing southeast in a deep valley cut in the eroded plains, makes its exit from the State 35 miles from its source.

Salt Fork of Red River Valley.—Salt Fork of Red River rises on the High Plains in northern Armstrong County at an elevation of 3,250 feet, crosses the escarpment, cuts a valley in the eroded plains, and after a tortuous course passes from the State in the southeastern part of Collingsworth County at an elevation of 1,900 feet, a descent of 15 feet per mile. It flows in a sand-filled valley, and at times of low water the river is a narrow ribbon upon a sand bed half a mile wide.

Prairie Dog Fork of Red River Valley.—Prairie Dog Fork of Red River crosses Armstrong County in Palo Duro Canyon (not to be confused with Palo Duro Creek, in Hansford County), which is 5 miles wide and which has been cut 875 feet through the Tertiary and the red-beds rocks. The river flows in a narrow valley at the bottom of this gorge, the sides of which present an alternate precipitous and terraced structure according to the nature of the beds. In Armstrong County there are 20 miles of this canyon.

A. SAND HILLS BLOWN FROM CANADIAN RIVER.

B GYPSUM LEDGE, SHOWING BANDED STRUCTURE.

Minor stream valleys.—Sweetwater Creek, in northern Wheeler County; Spillers Creek, in Collingsworth; Mulberry Creek, in Armstrong; Mammoth Creek, in Lipscomb; Palo Duro and Coldwater creeks, in Hansford; Kit Carson and White Deer creeks, in Hutchinson; Red Deer Creek, in Hemphill, and McClellan Creek, in Gray County, are the largest streams of secondary importance. These smaller streams all form a part of the three major drainage systems, the North Fork of Canadian, the Canadian, and the Red River. Most of these minor streams are periodic, although numerous springs at the base of the Tertiary feed many of the smaller creeks, thus rendering them perennial.

GEOLOGY.

GENERAL RELATIONS.

The general geologic features of the Texas Panhandle are not complex. Most of the rocks belong to two great systems—the Permian and the Tertiary—and there are small amounts of Quaternary deposits, all of which lie nearly level. The lowest formations exposed consist of extensive deposits of red clays and shales known as the red beds, most of which are of Permian age. The greater part of the upper formations are made up of sands, clays, and conglomerates belonging to the Tertiary system. Covering these two members in many places are beds of sand, gravel, and alluvium of Quaternary age. On the geologic map (Pl. V) the distribution of these formations is shown. The relative age and general character of the various deposits are given in the following table:

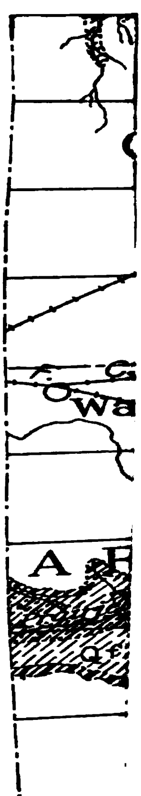
Geologic formations of the Texas Panhandle.

System.	Name.	Predominant characters.
Quaternary	Alluvium	Loam, sand, and gravel.
	Sand hills	Sand, chiefly in dunes.
	Tule formation	Sand, clay, and gravel.
Tertiary	Blanco formation	Clay, sands, and conglomerates.
	Goodnight formation	
	Loup Fork formation	
Triassic	Dockum formation	Clays, sandstones, and conglomerates.
Carboniferous (Permian).	Quartermaster formation.	Red sandy clay and soft sandstone.
	Greer formation	Red clay, with gypsum and dolomite.

A typical section of the Permian, Triassic, and Cenozoic strata in Palo Duro Canyon, 15 miles south of Claude, Armstrong County, Texas, is as follows:

Typical section in Palo Duro Canyon, Texas.

System.	Formation.	Character.	Thickness in feet.
Tertiary		Tertiary clays, varying in color from almost white to pink; usually with calcite concretions and a few pebbles; occasional harder bands forming terraces.	200
Unconformity.			
Triassic	Dockum	Gray to brown or reddish sandstone. Soft and friable, cross bedded; often changing into conglomerate with lenses of blue and red clay. Variegated clays, maroon, wine-colored, drab, gray, bluish, and red, with ledges of sandstone, sometimes becoming hard enough to form an escarpment; often simply a gray arenaceous shale.	270
Unconformity.			
Carboniferous (Permian).	Quartermaster	Red clay shale, with bands of harder clays, sometimes forming a sandstone, and occasional bands of white or gray clay or sandstone, weathering into characteristic buttes or mounds. Seams of satin spar in the lower part.	275
	Greer	Red clay shale, with ledges of massive white or purple gypsum, interstratified with bands of clay and sandstone.	180
		Total	925



The following sections made on Palo Duro, Tule, and Mulberry canyons in the southwestern part of the region here discussed, where all formations are best exposed, indicate the relative thickness (in feet) of the various beds:

Geologic sections in Palo Duro, Tule, and Mulberry canyons, Texas Panhandle.

System.	Formation.	Palo Duro Canyon. Silverton-Clarendon road.	Palo Duro Canyon. Silverton-Claude road. South side.	Palo Duro Canyon. Silverton-Claude road. North side.	Tule Canyon. Silverton-Claude road.	Mulberry Canyon. Silverton-Clarendon road.
		Feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*
Tertiary and Quaternary.	....	380	220	200	180	240
Triassic	Dockum ...	110	175	260	165	Eroded.
Carboniferous (Permian).	Quartermaster.	805	280	210	50	160
	Greer	175	195	190	Not exposed.	100
Total	....	970	870	860	395	500

PERMIAN RED BEDS.

GENERAL STATEMENTS.

The oldest rocks found on the surface in the Panhandle of Texas are the Permian red beds. These rocks occupy a considerable part of the Great Plains from southern Kansas across Oklahoma and Texas as far as New Mexico and Arizona, and outcrop along the eastern flank of the Rocky Mountains as far north as the Black Hills of South Dakota.

In Oklahoma, where the Permian red beds are typically exposed, they have been divided by the writer into five formations, as follows:[a]

Carboniferous	Permian	Quartermaster.
		Greer.
		Woodward.
		Blaine.
		Enid.
	Pennsylvanian.	

The rocks typically exposed around Chandler, now known to be Pennsylvanian, consist of red shales and red or gray sandstones. The Enid formation is composed largely of red clay shales, with an occasional ledge of soft sandstone. The Blaine is characterized by massive

[a] Gould, Chas. N., General geology of Oklahoma. Second Bien. Rept. Oklahoma Geol. Survey, 1902, pp. 42–58. Revised in Water-Sup. and Irr. Paper No. 148, U. S. Geol. Survey, 1905, p. 39.

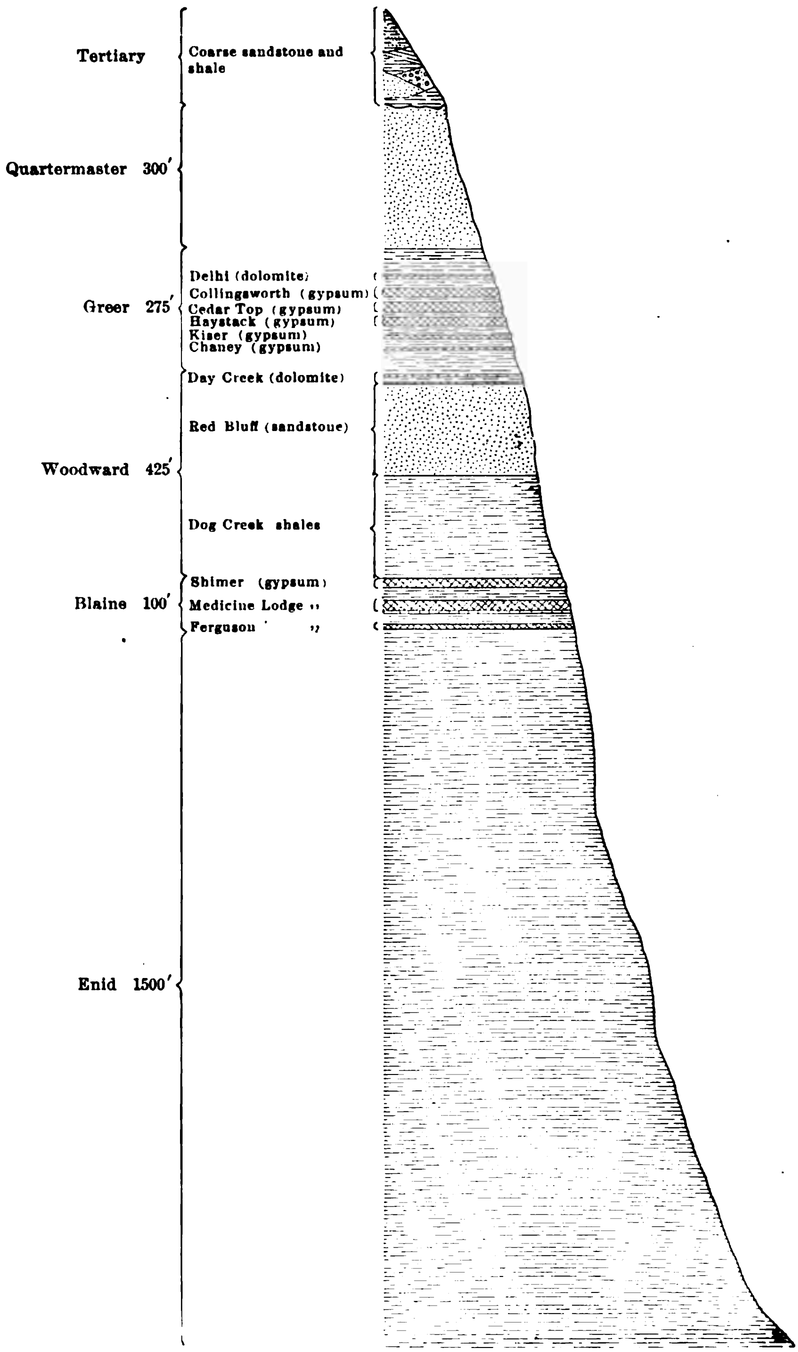


FIG. 1.—Generalized section of Oklahoma red beds. In the above legend "Delhi" should read Mangum, and "Red Bluff" should read Whitehorse.

ledges of white gypsum interbedded with red shales. The Woodward is made up of red shales and sandstones and a ledge of white dolomite. The Greer is also a gypsum formation, in which the ledges are interstratified with red shales. In the Quartermaster the rocks consist chiefly of red shales and clays, with ledges of soft sandstone. Fig. 1 shows the general character and relative thickness of the red beds as exposed in Oklahoma.

Professor Cragin classified the red beds in Kansas and northern Oklahoma, but he did not examine the lower nor the upper members. He divided the portion that he studied into the Salt Fork and Kiger divisions, each consisting of a number of formations.[a]

In comparing this author's classification with the one used by Professor Cragin it may be said that, in general, the Enid, Blaine, and Woodward formations correspond to his Salt Fork and Kiger. Neither the rocks near Chandler nor the Greer nor the Quartermaster formations, as they are now known, were described by Professor Cragin.

Professor Cummins divided the red beds into the Wichita, Clear Fork, and Double Mountain formations, without, however, sharply differentiating them.[b] Doctor Adams, who studied the lower members of the Texas red beds, found that the divisions made by Professor Cummins were unsatisfactory and recommended that they should not be retained.[c] He has also shown that the Wichita beds in Texas, like those near Chandler, in Oklahoma, are Pennsylvanian in age.[d] From the best available information it seems probable that the Wichita beds are approximately the equivalent of those near Chandler, the Clear Fork beds include about the same rocks as the Enid, Blaine, and Woodward formations, and that the Double Mountain beds are practically the same as the Greer and Quartermaster formations. The following table expresses the conditions:

Relationship of formation classifications.

Cummins's classification.	Classification of the writer.	Cragin's classification.
Double Mountain beds	Quartermaster	
	Greer	
Clear Fork beds	Woodward	Kiger division.
	Blaine	Salt Fork division.
	Enid	
Wichita beds		

[a] Cragin, F. W., Permian system of Kansas: Colorado Coll. Studies, vol. 6, 1896, p. 3.

[b] Cummins, W. F., Rept. on the geology of northwestern Texas: Second Ann. Rept. Texas Geol. Survey, 1890, pp. 400–402.

[c] Adams, George I., Stratigraphic relations of the red beds to the Carboniferous and Permian in northern Texas: Bull. Geol. Soc. America, vol. 14, 1903, pp. 191–200.

[d] Ibid., pp. 195–199.

THE PERMIAN IN THE PANHANDLE REGION.

Of the formations of the Permian red beds discussed above, only the Greer and Quartermaster are exposed in the Panhandle of Texas.

Greer formation.—The Greer formation, the lowest member of the red beds found in the Panhandle, has its type exposure in Greer County, Okla., along Elm Fork of Red River, a few miles east of the Texas line. It is here composed of 150 to 200 feet of brick-red clays and shales interstratified with ledges of white, bluish, and pinkish gypsum, with an occasional ledge of magnesian limestone and dolomite. In many places, however, the gypsum beds are entirely wanting or occur as single ledges, while in other localities there are six or more well-marked beds, ranging from 1 to 30 feet in thickness, besides one or two ledges of irregular gray, honeycombed, magnesian limestone, 1 to 3 feet thick. A number of localities in Collingsworth County, Tex., may be cited where these definite gypsum layers occur, but extensive study in the region has shown that all of them are more or less lenticular and do not persist for any considerable distance. Indeed it is not an uncommon occurrence for two or more of these ledges to merge locally by the thinning out of the intervening clays, while at a short distance beyond the gypsums themselves become thin and disappear. In very few parts of the red beds is the tendency to form lenses better exemplified than in the Greer formation. Along North Fork of Red River, just east of the Panhandle line, the writer has named the following members of the Greer: Chaney, Kiser, Haystack, Cedartop, and Collingsworth gypsums and Mangum dolomite.[a] The sequence of the beds is shown in fig. 2.

In view of the facts as presented above, it appears better not to indicate by name those lenses which persist only for a short distance and which can not be correlated in adjoining regions; hence in the present paper no attempt will be made to subdivide the Greer formation.

Because of the lenticular nature of the beds it is not always possible to locate the exact limits of the various subdivisions of the red beds. In general, however, the upper limit of the Greer is placed either at the top of the highest prominent gypsum ledge or at the top of the ledge of magnesian limestone or dolomite, which appears 10 to 20 feet above the highest ledge of solid gypsum.

The gypsum members of the Greer formation abound in caves and sink holes. In Pl. VI, *A*, is reproduced a photograph of an opening on the surface of a gypsum cave in western Oklahoma. The soft shales which underlie the ledges are easily eroded, and the

[a] Gould, Chas. N., General geology of Oklahoma: Second Bien. Rept. Okla. Geol. Survey, 1902, pp. 55-56.

A. GYPSUM CAVE.

B. SPRING ISSUING FROM A CAVE IN GREER GYPSUM.

gypsum is gradually dissolved by water. It is not uncommon to find a prairie stream of considerable size which disappears in a sink hole. In such case, however, it usually comes again to the surface at no great distance as a spring issuing from a cave (Pl. VI, *B*). These sink holes are of various shapes, with the oblong and circular forms predominating. The oblong sink holes often terminate

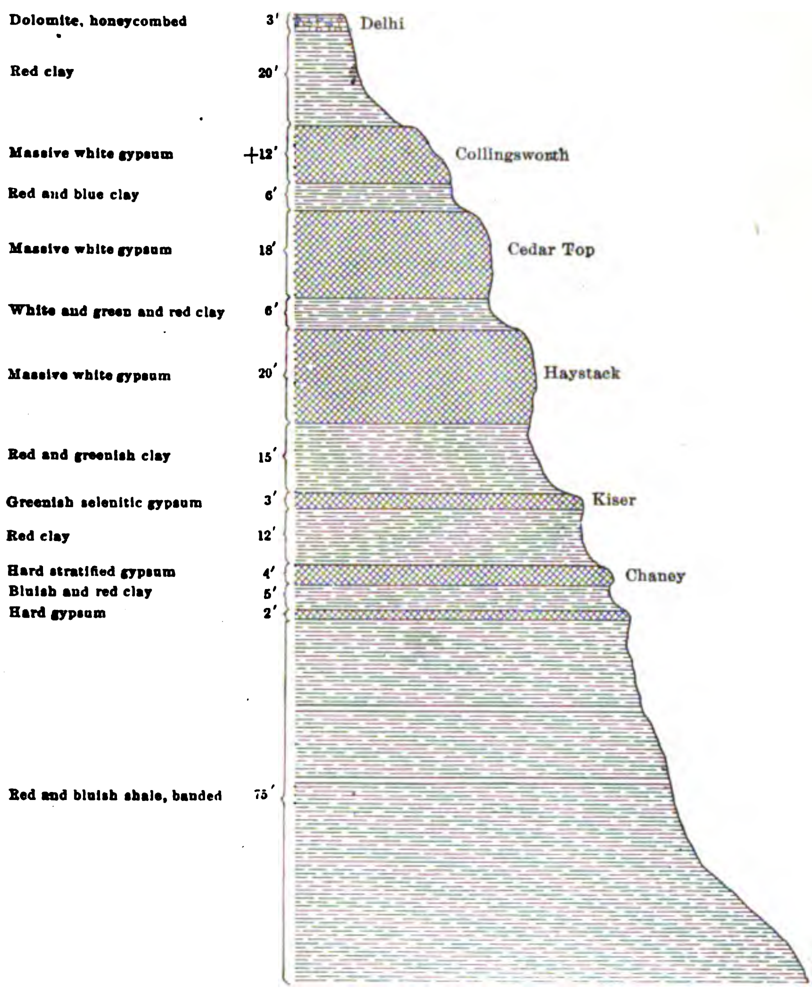


FIG. 2.—Section showing members of Greer formation on Elm Fork of Red River at Salton, Okla. In above legend "Delhi" should read Mangum.

abruptly in caves at one end, while the circular ones exhibit a conical hole in the center, through which the water escapes most freely. These sink holes vary in depth from a few inches to 20 feet or more and are 10 to 100 feet in diameter. In general they are irregularly distributed; in some cases, however, they seem to occur in chains, evidently connected by an underground passage, thus marking the beginning of a drainage channel. These sink holes are probably formed by the subterranean drainage which dissolves the gypsum and clay below the surface, forming caves which eventually collapse and become stream beds. North of Shamrock, 2 miles from North Fork of Red River, there is a typical sink-hole region in which hundreds of these openings occur in an area of a few square miles.

In various places the members of the Greer exhibit a marked peculiarity of stratification not usually found in the rocks of the plains. The gypsum ledges are here often distinctly laminated, as shown on Pl. IV, *B*. Steep local dips, both anticlines and synclines, are often observed along the sides of a cliff.

A striking peculiarity of the Greer formation is the erratic dip of the gypsum. Frequently in tracing a ledge along a small stream it is found that within a distance of perhaps half a mile the stream descends 50 feet, while the gypsum along the bluff still retains the same height above the water channel. On the opposite side of the stream the same ledge may be traced along another branch until less than a mile away it is 75 feet higher than at the main stream. In other words, the dip of the ledge is toward the stream on both sides, though the ledge is continuous. This peculiarity of dip gives the appearance of irregularly folded strata, yet there has been no general folding whatever. The phenomenon is not easy to understand. Perhaps the most plausible explanation is that the shales have been removed from beneath the gypsum ledges, permitting the latter to sink along the streams into the semblance of a local dip. This fact is exemplified on Pl. VII, *A*.

The Greer formation being the lowest member of the red beds in the Panhandle naturally outcrops low in the stream valleys. It is well exposed along the branches of Red River, particularly on Elm, Salt, and Prairie Dog forks. On Elm Fork it outcrops along the valley of the stream from the Oklahoma line as far west as Shamrock, in southern Wheeler County. Between Elm Fork and Salt Fork the Greer forms the plain as far west as the post-office of Dozier, while along the north side of Salt Fork gypsum ledges appear in the bluffs at intervals, finally disappearing in Donley County a few miles east of the center. South of Salt Fork a strip of sand hills covers the red beds, so that the Greer formation is not exposed north of the divide between Salt and Prairie Dog forks of Red River.

It is in the valley of Prairie Dog Fork of Red River and its tributaries, Spillers and Mulberry creeks, and in Palo Duro Canyon, in Collingsworth, Donley, and Armstrong counties, that the Greer formation attains its typical development in the Panhandle. In this locality it is exposed along the bluffs of the main creeks and caps the slopes of the smaller side canyons that are dissecting the red-beds plain. In Palo Duro Canyon, in particular, the gypsums of the Greer are con-

A UNDERMINING OF GYPSUM LEDGES.

B. EROSION IN THE QUARTERMASTER SANDSTONE IN PALO DURO CANYON.

Tertiary cliffs in the distance.

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umber of narrow ravines have been cut out. In
ometimes pass into caves and sink holes, and after
d for half a mile or more reappear in a deep can-
hite gypsum cap the bluffs and wind in sinuous
ie streams.

ormation.—Resting conformably upon the Greer
of rocks, consisting for the most part of soft, red
dy clays and shales. To this formation the name
been applied, the name being derived from a
Custer counties of Oklahoma, along which the
y exposed. In the lower part of the formation the
shales, usually red, but sometimes containing green-
ers of clay and often (particularly near the base) a
ount of gypsum, which is usually in the form of
satinspar or of rounded concretions. At a higher
hales become more arenaceous and not infrequently
idated sandstone, which is rather thin bedded and
into small rectangular blocks. These harder members
ermaster formation often weather into long, narrow
d more or less conical mounds, varying in height from
t, as shown in Pl. VII, *B*. These conical mounds some-
alone, but more often they appear in groups; occasion-
are hundreds of them on a single quarter section.

dstone members are further characterized by marked and
iar irregular dips and folds. Strata are often seen dipping
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to all points of the compass, even on a small area. These
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Quartermaster formation.—Resting conformably upon the Greer are 250 to 300 feet of rocks, consisting for the most part of soft, red sandstones and sandy clays and shales. To this formation the name Quartermaster has been applied, the name being derived from a creek in Day and Custer counties of Oklahoma, along which the rocks are typically exposed. In the lower part of the formation the rocks are chiefly shales, usually red, but sometimes containing greenish bands or layers of clay and often (particularly near the base) a considerable amount of gypsum, which is usually in the form of white or pink satinspar or of rounded concretions. At a higher level the red shales become more arenaceous and not infrequently form a consolidated sandstone, which is rather thin bedded and prone to break into small rectangular blocks. These harder members of the Quartermaster formation often weather into long, narrow buttresses and more or less conical mounds, varying in height from 10 to 50 feet, as shown in Pl. VII, *B*. These conical mounds sometimes occur alone, but more often they appear in groups; occasionally there are hundreds of them on a single quarter section.

The sandstone members are further characterized by marked and very peculiar irregular dips and folds. Strata are often seen dipping at an angle of 20 to 40 degrees, but the dip is irregular, varying in direction to all points of the compass, even on a small area. These local dips often produce slopes which have the character of those formed by normal faults or by general folding. The cause of this phenomenon is not well understood, but apparently the erratic dips are caused by the erosion of some of the subjacent gypsum members of the Greer formation.

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Throughout the greater part of this region the Quartermaster formation is overlain unconformably by the Tertiary or Quaternary deposits. In the localities where the Dockum beds are present the upper limit of the formation is located at the line where the color of the shales changes from brick red to maroon or wine color.

In general the Quartermaster formation outcrops in a belt 1 to [illegible] miles wide at the base of the High Plains. It appears in the southern part of Wheeler County, between Shamrock and Dozier, occupies the northwestern part of Collingsworth County, and follows along the north side of Salt Fork as far west as Clarendon. In the southern part of Collingsworth County the Quartermaster is exposed south and west of Wellington, the county seat. It forms the dissected plain between Memphis and Giles in southern Donley and eastern Armstrong counties. Along Palo Duro Canyon, in southwestern Armstrong County, it exhibits a maximum thickness of 300 feet and forms the top of the intracanyon terrace, just above the Greer gypsum ledges.

The red beds are exposed along Canadian River in northern Carson and southern Hutchinson counties. The most typical exposures are along Dixon and Antelope creeks, in Carson County, where 250 feet appear in vertical section. They contain some beds of dolomite and gypsum. These beds do not seem sufficiently uniform and persistent to warrant giving them definite names, yet they are more extensive than similar beds that occur in other portions of the formation. It is probable that a detailed study will reveal that these beds extend farther to the west along Canadian River. Plicated structure, noted elsewhere, is exemplified in this region. The following section was made in southwestern Hutchinson County, 2 miles from the mouth of Antelope Creek:

Section of red beds on Antelope Creek, Carson County, Tex.

	Feet
Red clays, with sandy shale	[illegible]
Gray sandstone	[illegible]
Red clay	8
Gray dolomite, weathers out in blocks which are scattered over talus slope (this ledge forms a terrace)	[illegible]
Red clay	13
Gypsum, bluish in places; a fairly persistent uniform ledge	[illegible]
Red clay, lower portion covered	[illegible]

For lithological reasons these beds as a whole are considered [illegible] belonging to the Quartermaster formation.

Near the middle of the Quartermaster formation, as exposed [illegible] Collingsworth and Hall counties, there is a ledge of rather hard, [illegible] or pinkish, more or less oolitic sandstone, which on weathering gives rise to a number of flat-topped buttes and ridges. Of these the most typical are Rocking Chair Mountains (Pl. VIII, *A*), southwest [illegible]

A. ROCKING CHAIR MOUNTAINS.

A hill capped by the Dozier sandstone.

B. SANDSTONE MEMBER OF THE DOCKUM FORMATION IN PALO DURO CANYON.

Shamrock; Antelope Butte, near the head of Elm Fork of Red River; Dozier Mounds, near Dozier post-office, and 'Possum Peaks, Twin Mounds, and Ragged Top, a few miles farther west of Dozier. Between Salt and Prairie Dog forks of Red River, in the vicinity of Memphis, in Hall County, these buttes are conspicuous. Hogback Butte, 8 miles south of Memphis, is a noted landmark. These buttes persist for an unknown distance south of Salt Fork of Red River.

From the sandstone on Antelope and Dozier mounds, Dr. J. W. Beede identifies fossils belonging to the following genera: *Dielasma*, *Schizodus*, *Allorisma*, *Pleurophorus*, *Edmondia*, *Aviculopecten*, *Leiopteria*, *Capulus?* (*Lepetopsis?*), *Loxonema*, *Strophostylus*, *Murchisonia*, *Pleurotomaria*, and *Worthenopsis;* indicating the Permian age of the sandstone.

TRIASSIC RED BEDS.

Dockum formation.—The upper part of the Texas red beds was described by Professor Cummins under the name of Dockum beds,[a] and afterwards by Drake.[b] This formation, which is composed largely of clays, sandstones, and conglomerates, underlies practically all of the Staked Plains of Texas and southeastern New Mexico. According to Drake,[c] the Dockum beds average 200 feet in thickness, and may be divided into three members, as follows: (1) A lower bed of sandy clay 0 to 150 feet thick, (2) a central bed or beds of sandstone, conglomerate, and some sandy clay 0 to 235 feet thick, and (3) an upper bed of sandy clay and sandstone 0 to 300 feet thick.

Along Palo Duro Canyon in Armstrong and Briscoe counties, where this formation was studied by the writer, it is difficult to divide it into recognizable members. The formation abounds in local unconformities with clay, sandstone, and conglomerate lentils, with cross-bedded structure, and other features indicative of shallow-water deposition. In places the lower portion is made up of red, maroon, or wine-colored clays, while at higher horizons there are more or less lenticular sandstones and conglomerates, as shown in Pl. IX, *B*. On weathering, the sandstones of the Dockum beds give rise to unique erosion forms; the harder members protect the softer shales beneath and produces pillars, chimneys, toadstools, and other unusual figures, some types of which, exposed in Tule Canyon, 6 miles northwest of Silverton, are shown in Pl. IX, *A*.

The lithologic characters which justify the separation of the Dockum beds from the Permian are, (1) the gray and brown color of the sandstones and conglomerates and the abundance of the latter;

[a] Cummins, W. F., First Ann. Rept. Texas Geol. Survey, 1899, pp. 189–190; Second Ann. Rept., 1900, pp. 424–428.

[b] Drake, N. F.; Stratigraphy of the Triassic formations of northwest Texas; Third Ann. Rept. Texas Geol. Survey, 1901, pp. 227–247.

[c] Ibid., pp. 229–233.

(2) the maroon, wine-colored, and yellow shales and clays, and (3) the extensive cross-bedding and local unconformities of the various members. Whether or not the Dockum formation is conformable throughout with the subjacent Quartermaster formation is still an open question. There is often local unconformity between the two formations, but on the other hand there are localities in which the brick-red shales and argillaceous sandstone of the Quartermaster grade so imperceptibly into the wine-colored shales and gray-brown conglomerates of the Dockum that the closest search fails to reveal the line of separation between them.[a]

Concerning the age of the Dockum formation it may be said that vertebrate fossils, found in these rocks and described by Cope,[b] as well as certain new forms of *Unios* named by Simpson,[c] indicate that the beds belong to the Triassic. In all, seven species of vertebrates and fohr of pelecypods have been secured from this formation.

TERTIARY AND QUATERNARY FORMATIONS.

REFERENCE LIST OF PUBLICATIONS.

For extended discussions of the Tertiary rocks of various parts of the Great Plains the reader is referred to the following publications:

Cummins, W. F., Notes on the geology of northwest Texas: Fourth Ann. Rept. Texas Geol. Survey, 1893, pp. 190–203.

Dumble, E. T., Cenozoic deposits of Texas: Jour. Geol., vol. 2, No. 6, 1894, pp. 549–563.

Hay, Robert, Water resources of a portion of the Great Plains: Sixteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1895, pp. 569 et seq.

Haworth, E., Physical properties of the Tertiary: Univ. Geol. Survey Kansas, vol. 2, 1897, pp. 247–284. Underground waters of southwestern Kansas: Water-Sup. and Irr. Paper U. S. Geol. Survey No. 6, 1897.

Darton, N. H., Report on the geology and water resources of Nebraska west of 103d Mer.: Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, pp. 719–785. Also in Prof. Paper U. S. Geol. Survey No. 17, 1903.

Darton, N. H., Rept. on the geology of the central Great Plains: Prof. Paper U. S. Geol. Survey No. 32, 1905.

Johnson, Willard D., The High Plains and their utilization: Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, pp. 601–741. Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, pp. 631–669.

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[a] Since the above was written opportunity has been afforded for studying these beds in the western part of the Panhandle of Texas, both on upper Palo Duro Canyon and along the valley of Canadian River. The writer finds that in this region the Triassic is everywhere separated by a pronounced unconformity from the subjacent Permian red beds and that it is clearly divisible into two formations, each consisting of well-marked members. These formations and members will be described in a forthcoming water supply and irrigation paper.

[b] Cope, E. D., Vertebrate remains from the Dockum Terrane of the Trassic system: Fourth Ann. Rept. Texas Geol. Survey, 1903, pp. 11–17.

[c] Simpson, C. T., Descriptions of four new Triassic Unios from the Staked Plains of Texas: Proc. U. S. National Mus., vol. 18, No. 1072, 1896, pp. 381–385.

A. EROSION FORMS IN THE DOCKUM SANDSTONE IN TULE CANYON.

B. SANDSTONE AND SHALE MEMBER OF THE DOCKUM FORMATION.

Showing lenticular nature of the strata.

STRATIGRAPHY.

General statement.—After the deposition of the Permian and Triassic red beds in the Panhandle region the area was elevated and for a long period of time the land was extensively eroded. Farther south and west extensive deposits of Cretaceous rocks rest on the red beds, but in the part of the Panhandle under discussion Cretaceous formations are absent.

Resting uncomformably upon the eroded surface of the red beds throughout the region described in this paper are extensive deposits of the Cenozoic age—Tertiary or Quaternary—which make up the rocks of the High Plains. These formations, which consist largely of loosely consolidated clays, sands, and conglomerates, typically white, but varying locally into gray, buff, brown, or other colors, constitute the "Tertiary grit" and the "Tertiary marl" or "mortar beds" of the Kansas geologists. In Nebraska, Mr. Darton subdivides the beds of approximately this age into the Arikaree and the Ogalalla. In the Panhandle of Texas Professor Cummins has distinguished four horizons, basing his classification upon the evidence afforded by vertebrate fossils obtained in the different beds and identified by Professor Cope.[a]

The following table sets forth the names of the members as used by Professor Cummins, the geologic age, and the number of species Professor Cope found in each:

Vertebrate fossils distinguishing four horizons in the Panhandle of Texas.

Period.	Epoch.	Formation.	Number of species.
Quaternary	Pleistocene	Tule (*Equus* beds)	10
Tertiary	Pliocene	Blanco	16
	(Transition)	Goodnight	8
	Miocene	Loup Fork	17

Loup Fork formation.—The term "Loup Fork" has long been used to include a series of rocks, usually considered later Miocene in age, which are extensively exposed on the Great Plains, particularly in Colorado, Nebraska, Kansas, Oklahoma, Texas, and New Mexico. The rocks consist largely of sands, clays, and conglomerates, the latter made chiefly of smooth water-worn pebbles presumably derived from the Rocky Mountains. The thickness of the deposits varies, but the maximum is several hundred feet. The Loup Fork beds constitute the lowest Tertiary formation known to exist in the Panhandle. According to Professor Cummins these beds do not extend

[a] Fourth Ann. Rept. Texas Geol. Survey, pt. 8, 1893, pp. 18-86.

farther south along the eastern edge of the Llano Estacado th[illegible] the Prairie Dog Fork of Red River.[a] On Mulberry Creek, 12 miles west of Clarendon, where Cummins and Cope obtained the fossils identified by the latter, the Loup Fork beds are 30 feet thick a[illegible] "composed of alternating beds of bluish and almost pure white sand."[b]

Goodnight formation.—This division, named by Professor Cummins from the town in Armstrong County, Tex., consists of calcareous and arenaceous clays, sands, and heavy conglomerates. Lithologically, it is practically impossible to differentiate these beds from those of the Loup Fork or Blanco, and it is only by means of fossils contained in them that the beds are known to be of different age. Professor Cope identified eight vertebrates from these beds and assigned them to an age intermediate between the Loup Fork and the Blanco. Professor Cummins states that the Goodnight beds have extensive development south of Mulberry Creek. The maximum thickness as given by him is approximately 150 feet.[d]

Dall, on the authority of Dumble, has called these beds Palo Duro. He classes them as transitional between the Miocene and Pliocene, and says: "These beds, identified in western Texas by Scott as transitional, also had the absurd name of Goodnight applied to them." Certainly no one who has ever been in that portion of the Panhandle would consider the name of Goodnight as absurd, for it is the name of one of the largest of the old-time cattle ranches, as well as of a good-sized town, the seat of a flourishing college.

Blanco formation.—Professor Cummins gave the name Blanco beds to those Tertiary rocks which rest unconformably upon the Dockum conglomerate at the type locality of the latter—i. e., at Dockum, Dickens County, Tex. Vertebrate fossils from that region have been identified by Professor Cope, who states that "the horizon is more strictly and nearly Pliocene than any of the lacustrine terranes hitherto found in the interior of the continent."[f] The rocks consist of alternating layers of sand, clay, and diatomaceous earth, approximately 160 feet in thickness.

Tule formation.—These beds, described by Professor Cummins and by Professor Cope, were assigned by the latter to the *Equus*-bed

[a] Cummins, W. F., Notes on the geology of northwest Texas: Fourth Ann. Rept. Texas Geol. Survey, 1893, p. 203.

[b] Ibid., p. 204.

[c] Cope, E. D., Vertebrate fauna of the Loup Fork beds: Fourth Ann. Rept. Texas Geol Survey, pt. 8, 1893, p. 46.

[d] Cummins, op. cit. pp. 201–202.

[e] Dall, Wm. H., Table of North American Tertiary horizons, etc.: Eighteenth Ann. Rep't U. S. Geol. Survey, pt. 2, 1898, p. 338.

[f] Cope, E. D., Vertebrate fauna of the Blanco beds: Fourth Ann. Rept. Texas Geol Survey, 1893, p. 47.

[g] Cummins, op. cit. pp. 199–200.

horizon of the early Pleistocene, on account of vertebrates from Tule Canyon in Swisher County. In general, the statement made by Professor Cope that "*Equus* beds form the superficial formation of the country at various points on the Staked Plains and about its eastern escarpment,"[a] may be considered as accurate. However, the *Equus* beds are by no means confined to the top of the Llano Estacado, but occur in other localities as well, notably north of Canadian River. These rocks consist of coarse sand, clay, and gravel, with variable thickness.

Age of beds.—It is the experience of the writer, after ten seasons spent in studying these deposits in Kansas, Oklahoma, Texas, and New Mexico, that it is practically impossible to separate either the Tertiary or Pleistocene deposits of the plains into mappable formations. From the bottom of the Loup Fork to the top of the *Equus* beds the general character of the rocks changes so constantly and with such extreme irregularity that they can not for the most part be differentiated in the field. Sections made at about twelve points in eastern Colorado, western Kansas, western Oklahoma, and in the Panhandle of Texas show such a marked similarity of structure that without the evidence of fossils it is impossible to determine whether the rocks belong to the Miocene, the Pliocene, or the *Equus* beds. Even Professor Hay, who studied these rocks in Kansas and applied to them the descriptive terms "Mortar beds," "Tertiary grit," "Tertiary marl," etc., did not succeed in differentiating them into definite horizons. If it were possible to distinguish formations stratigraphically, the matter of classification would be greatly simplified, but in the light of present knowledge, it seems not only inexpedient but even impossible to differentiate them structurally. In view of these facts, therefore, the general term Tertiary will be used to include the Loup Fork, the Goodnight, the Blanco, and in most cases also the Tule or *Equus* beds. The *Equus* beds are classed with Tertiary chiefly, as stated above, because these beds can not be distinguished in the field, nor, indeed, by any other means than that of vertebrate fossils, which are present only in scattered localities.

ORIGIN OF THE TERTIARY DEPOSITS.

With regard to the origin of the Tertiary deposits of the Great Plains two general theories have been advanced. The earlier geologists who studied these rocks considered them lacustrine in origin; Professor Marsh, for instance, described a great Pliocene lake covering practically the entire Great Plains area, in which deposits 1,500 feet thick were laid down.[b] Professor Cummins, in speaking of the

[a] Cope, E. D., Vertebrate fauna of the Blanco beds: Fourth Ann. Rept. Texas Geol. Survey, 1893, p. 75.
[b] Marsh, O. C., Amer. Jour. Sci., vol. 9, Jan., 1875, p. 52.

Goodnight beds, says, "They seem to have been deposited in a lake much more extensive to the south than the Loup Fork, which latter seems to have had its southern termination here" (at Mulberry Canyon).[a] Professor Cope has already been quoted regarding "Lacustrine terranes." Professor Hay accepted the lake theory, although he did not account for the formation of these supposed bodies of water.[b] Later investigations, however, have led to the opinion that it is to fluviatile rather than to lacustrine agencies that we must look for the origin of the Tertiary deposits.

Professor Haworth, in discussing the Kansas Tertiary,[c] observes: "The relative positions of the sand, the gravel, and the clay of the Tertiary over the whole of Kansas * * * correspond much better to river deposits than to lake deposits. The irregularity of formation succession, the limited lateral extent of the beds of gravel, sand, and clay, and the frequent steepness of the cross-bedding planes, all correspond to river deposits. * * * The materials themselves have many indications of river deposits and a very few of lake deposits."

Mr. Johnson, in his report on "The High Plains and Their Utilization," expresses the opinion that "The structure, an uneven network of gravel courses and elongated beds of sand penetrating a mass of silt and sand-streaked clay, is the normal product of desert-stream work under constant desert conditions. The coarse material is not regarded as the product of necessarily strong-running streams and the fine material of sluggish streams, in alternating epochs of humid and dry climate or of high and low inclination of slope, but as the simultaneous product of branching streams of the desert habit, here running in a channel and there spreading thinly."[d]

The only point at issue among these writers seems to be whether the cause of the deposition of the material by the streams is to be sought in climatic changes which produced alternate periods of aridity and humidity, or in deformation movements of the earth's crust by which the eastern part of the Great Plains was elevated and the gradient of the streams lessened. With regard to this matter the writer does not express an opinion. The subject has been discussed by Johnson, to whose article the reader is referred.[e]

GENERAL CHARACTER OF THE TERTIARY DEPOSITS.

It has been stated already that the greater part of the rocks consists of clays, sandstones, and conglomerates with clays predominat-

[a] Cummins, W. F., Notes on the geology of northwest Texas: Fourth Ann. Rept. Texas Geol. Survey, 1893, p. 201.

[b] Hay, Robert, Water resources of a portion of the Great Plains: Sixteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1895, p. 571.

[c] Haworth, E., Physical properties of the Tertiary: Univ. Geol. Survey Kansas, vol. 2, 1897, p. 283.

[d] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, 655.

[e] Ibid., chap. 2, p. 612–656.

A. EDGE OF TERTIARY ESCARPMENT.

Showing alternation of hard and soft beds.

B. PECULIAR WEATHERING OF TERTIARY CLAY IN PALO DURO CANYON.

ing. In color the clays are normally white, so white that when exposed they are frequently spoken of as "gyp" cliffs or "chalk" cliffs, although they contain neither gypsum nor chalk. However, the color of the clays is not invariably white; it often grades into the various other light tints. In structure the clay is usually so soft that it may be crushed with the fingers; but, on the other hand, the more calcareous members are frequently indurated and make a fair quality of limestone. Occasionally beds are found full of white calcareous lumps or concretions, which give to the rock a mottled appearance. The lime often cements the clay together in the form of elongated concretions, which, on weathering, have a resemblance to stalactites, as shown in Pl. X, *B*, and form what one author calls "pipy" concretions.[a]

Sand beds and ledges of conglomerate also constitute a considerable part of the Tertiary and Quaternary. The sand is usually in smooth, rounded, white or yellowish grains and the material is of quartz. The conglomerate is made up typically of smooth water-worn pebbles, usually composed of quartz, granite, porphyry, and other igneous rocks, varying in size from sand grains to bowlders as large as a peck measure. These pebbles most commonly occur in beds or layers sometimes as much as 25 feet thick, but often they are intermingled with fine sand and sometimes sprinkled through the clay members.

In a number of localities the gravel beds at the immediate base of the Tertiary contain considerable numbers of water-worn *Gryphæa* shells of lower Cretaceous age. It has been stated that at the present time there are no Cretaceous rocks exposed between the red beds and the Tertiary deposits in this part of the Panhandle, but that extensive Cretaceous deposits are found along the southern and western edges of the Llano Estacado. Whether these shells were derived from the lower Cretaceous rocks in place, or were transported by streams from beds farther west, it is impossible to determine, but in the light of available data the latter supposition seems probable.

The relative proportion of the different rocks enumerated above varies with the locality, but it is probable that three-fourths of the Tertiary and Pleistocene material exposed along the eastern edge of the Staked Plains is some form of clay, silt, or marl, the other one-fourth being sand or conglomerate. Farther north, in Kansas and Nebraska, the proportion of coarser material is relatively larger, often being more than one-half.

In all places on the plains, so far as known, these materials are arranged in a heterogeneous manner—the clays, sand, pebbles, silt, conglomerate, and other forms of rock occurring indiscriminately and without similarity of position. In one place a section of a hill

[a] Darton, Nelson H., Report on the geology and water resources of Nebraska west of 103d Mer.: Prof. Paper U. S. Geol. Survey No. 17, 1903, p. 25.

shows nothing but clay and silt; half a mile away beds of sandstone and gravel occur; and still farther away the section reveals little besides sand and conglomerate. (Pl. X, *A*, *B*, exhibits typical Tertiary structure.)

SAND HILLS.

There are two general classes of sand hills in the Panhandle, those derived from the disintegration of rocks in place and those blown by the winds from some stream channel. The sand hills of disintegration occur usually either along the base of the escarpment at the foot of the High Plains or along the divide between two river systems. The material of which these sand hills are composed has been largely derived in place by the disintegration of Tertiary rocks. As the clay and silt which make up a considerable part of the Tertiary deposits were removed by the action of water the sand and gravel remained behind and the finer materials have been shaped by the wind. In each of the four eastern counties of the Panhandle there are considerable areas of sand hills which have been formed in this manner. In Lipscomb and Hemphill counties sand hills occur along Wolf Creek and on the divide between that stream and Canadian River. Much of Wheeler County is covered with sand hills, particularly in the region between Sweetwater Creek and North Fork of Red River. In Collingsworth County there is a region 10 to 15 miles wide south of Salt Fork, extending entirely across the county, composed wholly of these sand hills, and in southeastern Donley County there are large areas covered with sand hills.

In the second class of sand hills are those formed of wind-blown sand derived from the stream channels. In the Panhandle region they seem to occur indiscriminately on both the north and south sides of the various rivers, usually along the flood plain between the channel of the stream and the bluffs. Hills of this character, which are composed of fine white or yellowish quartz grains, are usually barren of vegetation, as shown in Pl. IV, *A*. They are present along practically all the larger streams, particularly along Canadian River and Salt Fork of Red River. Migrating dunes are not uncommon.

ALLUVIUM.

Along all the large streams in this region there have been deposited materials of greater or less thickness that have been brought by the streams from higher levels. In the valley of Canadian River is a broad belt of bottom land made up largely of alluvium, which here consists chiefly of fine sand and clay mixed with decayed organic matter and occasional coarser gravels, the whole constituting a sandy loam. As much of the clay is derived from the red beds, the loam often partakes of a reddish color. Along all the small streams emptying into Canadian River are bottom lands or flood plains composed of practically the same material, and there are deposits along the various tributaries of Red River. Along North Fork and Salt Fork the bottom lands are from half a mile to a mile wide.

WATER RESOURCES.

UNDERGROUND WATERS.

GENERAL CONDITIONS.

The underground waters of the Panhandle of Texas may be discussed under two general heads—red-beds waters and Tertiary waters. Under the latter head is included water from the sand hills. The water of the red beds occurs chiefly on the eroded plains at the foot of the escarpment in the southern and eastern part of the region. The water of the Tertiary is found on the High Plains and in the escarpment regions—that is, on the greater part of the area under discussion. The water of the red beds is limited in amount and usually impregnated with mineral salts, particularly gypsum ($CaCO_4$) and common salt ($NaCl$), so that it is often unfit for general use, while the Tertiary water is uniformly abundant and almost always pure and wholesome. So different both in quality and quantity are the waters from these two horizons that it seems best to discuss them separately.

WATER FROM THE RED BEDS.

Character.—Wherever the Permian red beds are exposed the water is unsatisfactory in quality, although it ordinarily is plentiful. Water from the red beds generally contains appreciable amounts of mineral salts, which in many cases are so abundant as to render it unfit for general use. To all this salt-impregnated water the common term "gyp" water is applied. In point of fact, however, much of the water does not contain any considerable per cent of calcium sulphate. A number of other mineral salts are found in the red beds, the most abundant of which are sodium chloride, sodium sulphate, sodium carbonate, magnesium carbonate, magnesium sulphate, calcium chloride, and sodium borate, in about the order named. In some instances all of these salts are found in the water of a single well, but in most cases only two or three of them appear in appreciable quantities.

It must not be understood, however, that all the water from the red beds is bad, for there are numerous localities where soft and pure water is found. Especially is this true of the localities where the water is obtained in the Quartermaster formation, which, as has been stated, consists largely of soft sandstones and sandy shales. In this formation but little gypsum occurs, and the proportion of the other mineral salts enumerated above is not as great as in the rocks of the Greer formation. The general statement may be made, however, that water from the red beds is not good water.

Occurrence.—Water in the red beds is usually found under one of two conditions: first, in sandstones or sandy clay beds, and second, in underground veins, either joints in the clay or in gypsum caves. As has been stated, the red beds consist largely of red clay shale with occasionally interbedded members of sandstone and gypsum. In part the clays are composed of very fine-grained material which is practically impervious to water. Frequently they contain a high proportion of sand, in which case the interstices between the sand and clay particles are sufficiently wide for the seepage of water, and it is from beds of this character that perhaps the greater part of the wells obtain their permanent supply. In many parts of the red beds, especially in the Quartermaster formation, the arenaceous clay beds become a true sandstone and the relatively large spaces between the sand grains afford ready passage for water.

Many of the wells, however, find their supply not in sand nor even in sandy shales, but, if the testimony of drillers is given credence, in joints in the red clay. Those who have had experience in drilling wells agree in stating that while the greater part of the water in the red beds is found in sand, many of the wells penetrate nothing but the red clay. It is not uncommon for the drill to strike a so-called vein in the clay, in which the flow is so strong that the water rises many feet before the tools can be lifted. It has been said that the red beds abound in sink holes and caves, and that from many of the caves springs issue. In a number of cases the drill has been known to penetrate these caverns, which thus become reservoirs for the wells. The depth of wells in the red beds varies from 20 to 190 feet, averaging 60 feet.

WATER FROM TERTIARY ROCKS.

Character.—Almost without exception the water obtained in the Tertiary and sand-hills deposits of the Great Plains is good. Analyses of water from a number of wells and springs in these formations in Nebraska, Kansas, Oklahoma, and Texas have almost invariably shown that the water contains little or no harmful mineral salts. There are to be found small amounts of calcium sulphate, calcium chloride, calcium carbonate, magnesium carbonate, and sodium bicarbonate in the water of some of the wells, but the average amount of mineral salts in 7 samples was but 15 grains per gallon. The water on the plains is almost universally soft, pure, and wholesome, suitable for household and stock use.

Occurrence.—In order to appreciate the underground-water conditions of the High Plains, an understanding of the rocks from which the water is obtained is necessary. As has been shown under "Geology," pages 25–31, the Tertiary deposits, several hundred feet thick, which cover this region, consist chiefly of alternating layers of clay, sand, and gravel. It is generally believed by geologists that the material which comprises these rocks was derived largely from the Rocky Mountains, and that it was spread out in the beds of streams which in time past flowed from the mountains and were lost on the plains. These streams left deposits, now of sand, now of gravel or clay, and now of pebbles, which, in time, were covered by other deposits, sometimes of the same, but more often of other material. This process was continued until several hundred feet of alternating beds of the various kinds of rocks were deposited. From this it will be understood that the greater part of the beds must necessarily be irregularly lens-shaped in cross section, and in most cases will not be found continuous over large areas. In some places the greater part of the thickness consists of clay or silt, while in other localities sand and gravel predominate. In general, it is observed that the deposits near the base of the Tertiary have a greater proportion of the coarser material, consisting of sand and gravel beds, and that at a higher level they have clays and silts in greater abundance.

Most of the water of the High Plains is known as "sheet water." This is a term almost universally used in the western part of the United States to indicate any fairly constant supply of water at a more or less uniform depth beneath the surface. The term "underflow" is sometimes used to indicate practically the same phenomenon. The general impression seems to be that at some depth beneath the surface there is a regular "sheet" or "lake" of water, which if tapped by a well will yield a constant supply. In some places two or even three "sheets" are supposed to exist, and the expression "first sheet" and "second sheet," or "first water" and "second water" are common. Another prevalent notion is that the water in these "sheets" is constantly flowing, stream-like, beneath the surface, an idea disclosed by the expression "the underflow is to the south," or "the underflow is east." Much of this theory, however, is erroneous and not based on valid conceptions of the conditions found in the nature and relations of the water-bearing beds. Rounded grains of sand and gravel do not lie close enough together to fill all of the space, but have interstices between them. These pores or spaces are minute reservoirs for the water which, in its passage through such materials, seeps from one of these minute reservoirs to the next, and thus very slowly flows along underground. This movement is called "underflow," but it is not nearly so rapid as popularly supposed. Experiment has shown that even along stream beds, such as the Arkansas River in western Kansas, the rate of underflow does not average more than 10 feet a day.[a] On the High Plains, where the gradient is exceedingly low, it is doubtful if the water moves more than this distance in a year. This is a point, however, upon which there are practically no data, and estimates may be misleading.

Dry ground, according to the theory just advanced, is ground the pores of which contain no water, while wet or saturated ground is that in which the pores are filled. Since water tends to sink to the lowest levels, there is in most regions a certain, but variable, thickness of beds filled with water in what is technically known as the "zone of saturation." The upper surface of this zone of saturation is called the "water table," and this is often identical with the popular phrase "sheet water." Since water moves so slowly underground, this water table often becomes approximately similar in contour to

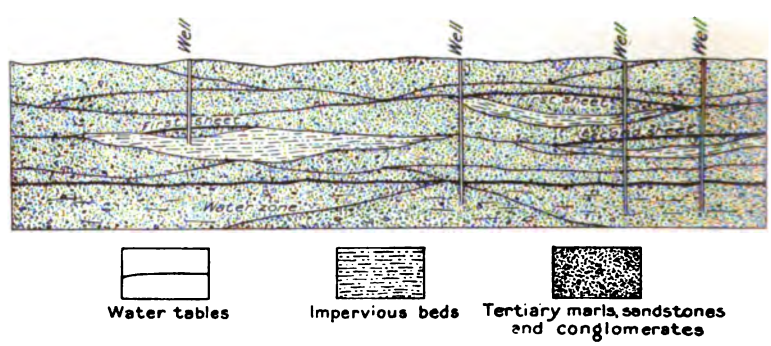


FIG. 3.—Ideal section of Tertiary, showing first and second sheet water.

the surface of the ground, being high on the divides and low near the streams where the water may escape in springs.

Attention has been called to the conditions under which these beds were laid down. As originally deposited they must have had an irregular outline and surface, especially when laid down in swamps or lakelets. Where the material is clay or very fine sand its interstices are very minute and practically impervious to water. Such fine deposits are often overlain by sand and gravel in basins or channels, and these in turn by other fine-grained deposits in varying succession, so that the alternation in water-bearing and impervious beds is most irregular, as shown in fig. 3. If such deposits are penetrated by a well the first sand encountered will supply water, the quantity of which depends upon the size of the water-bearing deposit, its coarseness of grain, the height of its edges, etc.; in the next coarse sand bed a second water stratum is found, and so on until finally the main water table is penetrated. This may be considered a probable

[a] Slichter, Charles S., The motions of underground waters: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, pp. 41-43.

explanation of the " first and second water," " first and second sheet," etc. It also possibly accounts for conditions similar to the one found near Groom, Gray County, where records obtained from a relatively small area show well depths ranging from 300 to 360 feet, except in one well where water is obtained at 228 feet. This shallower well probably finds its source of supply in one of these buried basins.

Wells throughout the Tertiary area usually secure water at depths varying from 20 to 500 feet. On the High Plains the average of twenty wells, taken at random from half a dozen counties, was 258 feet. The deepest wells are found along the line of the Santa Fe Railroad on the high divide south of Canadian River, in Carson and Gray counties, where the wells are from 350 to 500 feet deep. On the High Plains in Hansford, Ochiltree, and Lipscomb counties, north of Canadian River, the average depth is 240 feet. In Armstrong County, along Prairie Dog Fork of Red River, the average depth is less than 200 feet. In certain parts of the region, notably in Hansford and Carson counties, the driller sometimes fails to obtain a water supply, and instances are reported where the entire thickness of the Tertiary has been penetrated without finding an adequate amount. It is the experience of drillers that if the "red clay" (evidently red-beds clay) is encountered without finding a sufficient amount of water, it is useless to go deeper.

SOURCE OF THE UNDERGROUND WATER.

Local precipitation is the source of the underground water of the High Plains. The rainfall at Amarillo, Tex., a few miles west of the region here discussed, averaged 21.94 inches annually for a period of twenty years.

Rainfall on the surface of the earth is disposed of chiefly by evaporation, run-off, and sinking, or seepage into the ground. It is estimated that in general the amount of water disposed of in each of the three ways is about equal, but the relative amounts in different regions depend upon several local conditions. For instance, on a steep slope the greater part runs off; in a warm, arid climate the greater part evaporates, while in loose soil the greater part soaks in.

On a considerable part of the Great Plains, where the surface is level and the drainage systems undeveloped, there is no run-off, and the rainfall is either absorbed by the ground or evaporates. After a rain the water which does not evaporate immediately or is not absorbed by the ground accumulates in broad, shallow depressions on the surface, known as "buffalo wallows," or "lakes," and there remains until it evaporates. Johnson estimates that not more than 3 or 4 inches annually soak into the ground, an amount which would not saturate more than about 1 foot of sandy strata.[a] This estimate

[a] Johnson, Willard D., The High Plains and their utilization: Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 646.

FIG. 4.—East-west section of High Plains, showing position of water table.

of the amount of water absorbed seems rather small. but in all probability not more than 6 inches of rainfall are added to the ground water each year.

THE WATER TABLE.

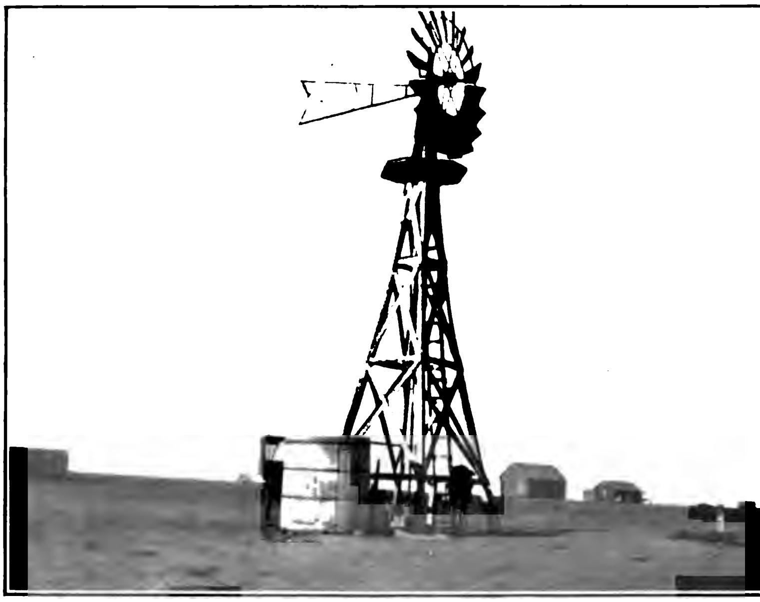
As stated on page 34, the "water table" or "water plane" is the subsurface plane beneath which the ground is saturated with water; in other words, the level at which the top of the ground water stands. It varies constantly from place to place, from year to year, and even from day to day. It is supplied chiefly from rainfall and is lowered when the water is removed, as, for instance, in the case of springs, by artesian wells, or by heavy pumping. Ordinarily it is at a considerable distance below the surface, but occasionally it reaches the surface level, as in springs, swamps, or marshes.

On the High Plains the water table is located at the upper point of saturation of the pervious beds. Well records show that this water level for the High Plains, as a whole, averages approximately 250 feet below the surface. So far as known, this level is fairly constant, the amount of water taken away by springs and wells being approximately equaled by the amount added each year by precipitation. Fig. 4 shows an east-west section of the plains and the position of the water table.

USE OF WINDMILLS.

As the Panhandle is chiefly a grazing country, most of the wells have been put down for the purpose of furnishing water for cattle. On the greater number of the larger ranches wells are located 2½ to 3 miles apart, and windmills are almost universally used to bring the water to the surface.

Often upon the prairie the only object in view to indicate that the locality is inhabited is a solitary windmill. These mills are placed upon towers 20 to 30 feet high, constructed of wood or steel. All types of factory-made turbines are used, but the steel mill with a wheel 8 to 10 feet in diameter seems to be most effective for general purposes. Larger wheels, some even 20 feet in diameter, are employed to elevate the water for the entire supply of some of the larger towns, as, for instance, Panhandle and Ochiltree. (Pl. XI, *A*.) On the



A. WINDMILL AND TANK AT OCHILTREE, TEX.

B. TYPICAL WINDMILL AND TANK.

range the water is pumped into large steel or wooden vats or into shallow basins (locally called "tanks") excavated in the ground or formed by damming a shallow draw. These tanks are of various sizes, but ordinarily they have a capacity of several thousand barrels and are very serviceable, since the soil is of such nature that when thoroughly compact and saturated it permits but little seepage. Typical views of windmills and tanks are shown in Pl. XI, *A*, *B*. The mill is allowed to operate continuously, and is visited occasionally by a rider for purposes of repairs or oiling. Wind is such a constant factor on the plains that little concern is felt regarding the power to raise the water, and in very few instances are provisions made, or are other means necessary, for lifting it. In a few instances gasoline engines are installed for use in case of emergency. In the Panhandle a week seldom passes without wind to drive the mills so that they will supply sufficient water for the stock.

DEEP-SEATED WATERS.

The project of obtaining artesian water in various parts of the Panhandle, particularly in the red-beds areas at the foot of the High Plains, is often considered. In general, the arguments advanced in favor of the project are based on the mistaken idea that there is an underground source of supply from the High Plains or the Rocky Mountains.

From what has been stated already it will be understood that the water supply of the High Plains is derived wholly from the rainfall, and while the part which sinks into the ground and is added to the ground water may amount to 5 or 6 inches a year, the geologic structure is not favorable for artesian conditions. Between the Tertiary and the underlying red beds there is everywhere a pronounced unconformity, and the rocks of the red beds beneath this unconformity are composed chiefly of impervious clays and shales, through which the water can not pass readily. These conditions, then, preclude the probability of artesian water supply having its source on the High Plains.

That the Rocky Mountains are a source for an artesian supply through the red beds is also improbable. These red beds, which are covered by the Tertiary of the High Plains, reappear in New Mexico beyond the western escarpment of the plains, and are exposed along the eastern base of the mountains at a higher altitude than in the region east of the plains escarpment. Some of these beds are coarse-grained, and doubtless they contain water in some places, but whether they could be reached by deep wells and would yield water in the Panhandle region remains to be determined. Their general relations are shown in Pl. II and fig. 3. In the eastern part of the Panhandle these beds must be very deep seated, probably more than 2,000 feet, and the drill has never reached this depth in the red beds anywhere in this part of the plains. The only place where the red beds have been well explored is along their eastern margin in eastern Oklahoma, where, however, artesian water was not found.

At Childress, Tex., 20 miles south of the southeastern corner of the region discussed, the Fort Worth and Denver City Railroad has drilled a well to a depth of 1,300 feet in search of water for engines and shops. From the surface to the bottom of the well the drill passed through nothing but red clay shales containing a few ledges of sandstone and gypsum. Several horizons of salt water were encountered, but no fresh water was obtained.

At a number of points in Oklahoma wells have been drilled in search of coal, oil, gas, and water, but so far artesian supplies have never been found. At Fort Reno the Government sunk a well to the depth of 1,400 feet in search of water for the post, but none was secured. Near Oklahoma City a well 2,050 feet deep passed out of the red beds at 1,550 feet. No artesian water was found.

From all data at hand the conclusion must be drawn that the chances are very poor for finding artesian water in the red beds under the plains. The red beds present difficulties to very deep drilling which usually have been insurmountable, and if artesian water does exist in the lower members of this series it is doubtful if it can be reached at a cost which would be generally profitable. However, it is to be hoped that at some time the experiment will be tried of testing all of the red-bed strata.

SPRINGS.

There are in the region under discussion two general classes of springs—those from the red beds and those from the Tertiary and sand hills. Both in amount of flow and in character of water these springs differ considerably, and for that reason it is thought best to describe the two classes separately.

RED-BEDS SPRINGS.

Springs in the red beds are of infrequent occurrence, and those present are rarely strong. They may be classified, according to the character of their water, as salt springs, gypsum springs, and fresh-water springs.

Salt springs.—Along the branches of Red River, Prairie Dog, Salt, and Elm forks, there are a number of weak salt-water springs, often little more than seeps. The horizon from which the water comes is usually near the base of the Greer formation. On Elm Fork of Red River in western Greer County, Okla., 5 miles east of the Texas line.

there are two salt plains of considerable size, fed by a number of strong salt springs, the combined flow of which approximates hundreds of thousands of gallons of salt water a day. No springs as strong as these are found in any part of the Panhandle. Some salt springs occur, however, but so far as known the salt brine of Texas springs is not used, and it is not probable that the salt water of the Panhandle will ever be utilized, on account of the much larger amounts near at hand in Oklahoma.

Gypsum springs.—Practically all the gypsum springs in the Panhandle issue either from beneath or in close proximity to the massive gypsum ledges that make up a considerable part of the Greer formation. Such springs occur along Elm Fork of Red River in Collingsworth County and on the branches of Prairie Dog Fork in Collingsworth, Donley, and Armstrong counties. Sometimes these springs are mere wet-weather seeps, but in a number of cases they are strong, boiling, perennial springs derived from underground streams, flowing from beneath ledges of white gypsum.

Fresh-water springs.—The greater number of the fresh-water springs of the red beds issue from the Quartermaster formation, which, as has been stated, consists largely of sandstone and sandy shale, with but little gypsum or other mineral salts. The conditions are ideal for springs, provided there is a source of supply, and in a region of greater rainfall a large number might be expected to exist. In the Panhandle, however, the number is small. In the Quartermaster formation there are very few bold flowing springs. This is due to the peculiar lithologic character of the rocks, mostly soft sandstones and sandy clays, which, as stated on page 21, usually weather into peculiar rounded knobs and buttresses and into narrow canyons. It is in the latter that the springs occur, and it is not uncommon to find at the head of a little canyon an outcropping ledge of sandstone, beneath which the water seeps out of the bank. The flow is rarely strong, but it is often very persistent, and the water usually accumulates to form a tiny rill in the bottom of the canyon. Ranchmen and farmers frequently take advantage of the soft rock to hollow out a small basin, in which the water collects, often in quantities sufficient to supply a farmhouse, or even to furnish water for a number of cattle.

Springs are occasionally found in the Dockum beds, issuing from beneath ledges of sandstone or from under the conglomerate members. These springs are usually weak and unimportant and so far as noticed not utilized. This latter fact may be attributed chiefly to their inaccessibility, for the Dockum is exposed only along the steep escarpment at the foot of the High Plains.

TERTIARY SPRINGS.

Throughout the High Plains region the Tertiary deposits yield numerous springs, which are always of good water and have long been most advantageous to the settlers and travelers. Camps, forts, farms, and even cities have been located with reference to the proximity of a Tertiary spring or spring-fed creek. In the Texas Panhandle there are thousands of such springs. They are found, usually in great numbers, in every one of the twelve counties described in this report.

The source of supply of the Tertiary springs is chiefly in the ground water, otherwise called the "sheet water," or "underflow," of the High Plains, and they are usually found where deep canyons have been cut into the highlands.

Not infrequently springs occur at the line of contact between the Tertiary deposits and the clay strata of the upper part of the red beds. This condition is due to the ready seepage of water through the Tertiary sands to the top of the impervious red beds, where it flows laterally until it reaches the surface. Many of these contact springs do not issue from a single opening, but the water finds its escape along a zone of seepage extending sometimes for hundreds of yards along the side of a cliff. In such cases the amount of water discharged at any one place is not large, but the aggregate is often considerable.

Excellent springs frequently occur in the sand hills at the contact of the sand and the relatively impervious underlying strata. The flow from these sand-hill springs is seldom strong, but the water is pure and wholesome. Springs of this type occur chiefly in the sand-hill regions of the four eastern counties.

In a region where the underground supply is scanty the water that issues from springs is necessarily limited in amount. Very few of the springs discharge half a second-foot of water, and perhaps the greater number of them will not average one-tenth of that amount. The water usually flows but a short distance and then disappears in the sand. Where there are a number of strong springs in a locality the water unites to form a small creek, which is sometimes perennial, but usually intermittent.

STREAMS.

CLASSIFICATION OF DRAINAGE.

The drainage of this region flows into Mississippi River. The water from the northern part of the area flows into either the Canadian or the North Fork of the Canadian, tributaries of Arkansas River, while the water from the southern part reaches Red River. The drainage may be classified as follows:

North Fork of Canadian drainage.—Coldwater, Palo Duro, and Wolf creeks are tributary to North Fork of Canadian River.

Canadian drainage.—Canadian River flows northeast across this region into Oklahoma, traversing Hutchinson, Roberts, and Hemphill counties. It receives as tributaries a number of small creeks which rise on the plains both north and south of the river, cutting their way through the escarpment and entering the river nearly at right angles. The width of the basin from watershed to watershed averages not more than 35 miles.

Red River drainage.—Five main branches of Red River either rise in or pass through this part of the Panhandle. Beginning on the north they are as follows: (1) Washita River, which in Oklahoma and Indian Territory becomes a stream of considerable size, rises in southwestern Hemphill County and flows east; (2) North Fork has its source in Gray County and flows east across Wheeler County into Oklahoma; (3) Elm Fork has its origin in northwestern Collingsworth County and flows southeast; (4) Salt Fork rises in northern Armstrong County and flows east across Collingsworth County before reaching Oklahoma; (5) Prairie Dog Fork rises on the High Plains far to the west, and in this region flows through Palo Duro Canyon across the southwest corner of Armstrong County. These four branches, North, Elm, Salt, and Prairie Dog forks join at the southeast corner of Greer County, Okla., forming Red River, a tributary of Mississippi River.

STREAMS IN DETAIL.

Coldwater Creek.—In its upper course this stream is known as Rabbit Ear Creek, from the fact that it rises near the Rabbit Ear Mountains, two volcanic peaks in northeastern New Mexico. It flows southeast across Dallam and Sherman counties, then turning northeast crosses the northwest corner of Hansford County, passes into Beaver County, Okla., and empties into Beaver Creek at the town of Hardesty. In its course through Hansford County it has cut a canyon 1 to 3 miles wide and approximately 100 feet deep into the Tertiary rocks of the High Plains. The stream is fed by Tertiary springs. Consequently its water is fresh.

Palo Duro Creek.—This stream flows diagonally across Hansford County from southwest to northeast. It is a typical High Plains stream. Rising on the level prairie, it soon begins to cut a trench, which becomes deeper and wider until in Hansford County it is a canyon 1 to 3 miles wide and 100 feet below the level of the plains. Only in parts of its course is there water the year around. At Hansford, the county seat, the stream is dry except after heavy rains, but

15 miles downstream running water appears. This stream also empties into Beaver Creek in Beaver County, Okla.

Wolf Creek.—Wolf Creek rises on the High Plains a few miles southwest of Ochiltree, the county seat of Ochiltree County, and flows east across Ochiltree and Lipscomb counties into Woodward County, Okla. At old Fort Supply it joins Beaver Creek, forming North Fork of Canadian River. In its upper course it has cut a narrow canyon with precipitous bluffs. Farther down it passes out of the High Plains and enters the sand-hills region, where the bed is wide and sandy. The creek is fed by small branches—Camp, Willow, Cottonwood, Plum, Mammoth, and others—the water of which comes from Tertiary springs among the sand hills. Wolf Creek has the reputation among the cattlemen of being the most constant stream in the Panhandle.

Canadian River.—The largest stream in the Panhandle of Texas, Canadian River, has its headwaters among the high peaks of the Rocky Mountains in northern New Mexico. In its upper course it receives a number of tributaries which are fed by mountain springs. After leaving the mountains it flows southeast first across a plain composed of upper Cretaceous rocks, then for nearly 100 miles through a canyon 500 to 800 feet deep in the Dakota sandstone, finally reaching the red-beds plain in the region north of Tucumcari, N. Mex. At this point it changes its direction to the northeast and so flows out of New Mexico and across the Panhandle of Texas into Oklahoma, where it again turns southeast, finally joining Arkansas River in the eastern part of Indian Territory. Of the 700 miles of its course only about 100 miles are included in the part of the Panhandle under discussion. Across this region it flows in a broad curve, convex to the north, crossing southeastern Hutchinson, northern Roberts, and middle Hemphill counties before passing into Oklahoma. Throughout this distance the river has cut a broad canyon in the High Plains. In places the headlands between the tributary creeks approach almost to the river, but at most points the flood plain, usually a sandy flat, is 2 to 4 miles wide. The channel of the river itself is a sand bed averaging three-quarters of a mile in width.

Canadian River is perhaps more treacherous than any other stream of the plains. The stream is either dry or a raging torrent. The river may have been dry for weeks at a time, when suddenly, without warning, a wall of water several feet high rushes down the channel, sweeping everything before it, and for a number of days the river continues high, then gradually subsides. following this period of abnormal flow the sand in the stream becomes "quicksand," or loose sand which appears firm but gives way suddenly under foot, rendering the stream extremely dangerous to cross. Many a herd of

A

B

FRESHET ON RED DEER CREEK AT MIAMI, TEX.

cattle has been mired in Canadian River, and every year loaded wagons and even teams are abandoned. The cause of the sudden and rapid rises is not yet fully understood, but most of them are caused by heavy rains near the head of the stream.

Such sudden rises are not confined to the Canadian River, or to the larger streams, for the small streams exhibt the same phenomena, though on a much diminished scale. Pl. XII, *A*, *B*, shows a rise which occurred on Red Deer Creek, a tributary of the Canadian, at Miami, Roberts County, August 16, 1904. The town lies in a rather narrow valley cut by the stream into the High Plains. There had been no rain at Miami for several weeks and the bed of the creek was a dry sand flat. A heavy rain occurred at the head of the creek a few miles southwest, and two hours later the water came down the stream channel, a narrow tongue of white foam, as shown in Pl. XII, *A*. This was followed by a wall of turbid, yellow water that filled the banks of the stream. In half an hour the flood was at its highest, a seething, foam-capped torrent. By next morning the water had disappeared except from a few pools in the channel, as shown in Pl. XII, *B*, and by noon even these were dry.

Canadian River does not receive any large tributaries in its course across the plains. In three counties which it crosses in the region to which this report relates there are a number of small creeks, none more than 25 miles long, emptying into the river. Of these the most important are Spring, Kit Carson, Dixon, Antelope, Blue Bear, Walnut, Buffalo, White Deer, and Red Deer, all of which rise on the High Plains and cut their way through the escarpment before reaching the river.

Washita River.—Only the upper course of Washita River is in Texas, where it is a small creek, not differing from scores of others which take their rise in the escarpment and sand-hill regions. It flows east across the southern part of Hemphill County. Farther east in Oklahoma the valley of the Washita lies almost entirely in the red beds, and it is there known as the muddiest stream of the plains. In Hemphill County, Tex., however, it has not yet cut through the Tertiary, and is here a clear, fresh-water stream.

North Fork of Red River.—This, the northernmost of the four branches which make up the Red River, rises on the High Plains, in Carson County, breaks through the escarpment in Gray County, and flows northeast into Wheeler County, then southeast into Oklahoma. East of the Gray-Wheeler county line the stream has cut through the Tertiary deposits and into red beds, which are here exposed along its north bank, while on the south side sand hills occur. Across Gray and Wheeler counties the bed of North Fork is sand choked and has a surface flow only part of the year. The chief tributaries of North Fork are McClellan Creek, which drains southern Gray County, emptying near the Wheeler County line, and Sweetwater Creek, which drains northern Wheeler County and passes into Oklahoma before joining the main stream. All of these streams are fed by Tertiary springs, and even where the surface sand is dry water may usually be obtained by digging a few feet.

Elm Fork of Red River.—Northern Collingsworth and southern Wheeler counties are drained by Elm Fork, which rises along the escarpment, but soon reaches the red beds, across which it flows for the greater part of its course in Texas. The water of the upper branches is derived from the Tertiary springs in the sand hills along the escarpment, but as soon as the river enters the red-beds formations, gypsum and salt water flow into it, until by the time it reaches the Oklahoma line the water has lost its purity. Shortly after entering Greer County it receives water from a number of salt springs, and from that point is considered to contain the saltiest water of any stream of the plains.

Salt Fork of Red River.—Salt Fork is a typical stream of the plains. It rises far out on the Llano Estacado in southern Carson County, crosses northeastern Armstrong County, and flows entirely across Donley and Collingsworth counties before reaching the Oklahoma line. In its upper course it is but a shallow draw in the level prairie, but eastward it soon flows in a trench, and 10 miles from its source this deepens into a canyon with cliffs of white Tertiary beds. Up to this point the stream receives no water except the run-off, but a few miles lower in its course it reaches the Tertiary springs level and has a surface flow the greater part of the year. Eastward the bed widens and becomes sand choked, until in central Donley County, north of Clarendon, it cuts through the lower members of the Tertiary and enters the red beds. From that point almost to Mangum, in Greer County, Okla., it flows between red-beds bluffs on the north side and sand hills on the south side. It is a sandy, treacherous stream, dangerous to cross except when low.

Prairie Dog Fork of Red River.—This stream flows southwestward across Armstrong County in Palo Duro Canyon, which has been discussed under "Topography," page 12. This canyon is perhaps the most notable canyon in the High Plains. Near its mouth the walls are approximately 1,000 feet high, composed of red beds in the lower part and of Tertiary deposits above. Several creeks are tributary to this stream, the chief of which, Spillers and Mulberry creeks, drain parts of southern Collingsworth and Donley counties. These streams do not differ materially from others in this region. Spillers Creek rises in the sand hills of Collingsworth County and flows southeast across the red beds into Childress County

A. BUFFALO WALLOW.

B. LAKE ON HIGH PLAINS.

before joining Prairie Dog Fork. Mulberry Creek rises on the High Plains and has cut a deep canyon entirely through the Tertiary to a depth of several hundred feet into the red beds.

DRAINAGE OF THE HIGH PLAINS.

From what has been said it will be understood that there is a considerable portion of the Panhandle which has no developed drainage; in other words, from a great part of the High Plains there is no run-off. The headwaters of the various small streams tributary to Canadian or Red rivers have cut into the slope of the escarpment, but so far, except in a few isolated localities, the flat upland has not yet been invaded and remains still uneroded. It is graphically described by Johnson,[a] who says the plains are the remnants of an old débris apron, unscored by drainage, yet standing in relief.

Scattered at irregular intervals on this flat surface are saucer-shaped depressions, in which water collects. In size these depressions vary from the ordinary "buffalo wallow," a few feet across (Pl. XIII, *A*), to lakes hundreds of rods in diameter (Pl. XIII, *B*). In a few instances, particularly in localities near the edge of the plain, the basins are deep and bowl shaped, as shown on Pl. XIV, *B*. Often the lakes are perennial and afford an abundant supply of stock water the year round; others are ephemeral, being filled by rains but soon becoming dry, while still others contain water part of the year. These lakes occur with no regularity. In some localities on the High Plains there are none of these basins for miles, while in other sections there are scores of them in a single township. Pl. XV represents the conditions on the High Plains in parts of Carson and Gray counties.

Many of the larger depressions have extensive drainage basins, which sometimes collect the water from a number of square miles. Small prairie streams receive the run-off from the outer part of the basin and lead to the lake. It is not infrequent in traversing the High Plains to encounter a sag in the surface along which storm water is carried to a near-by lake. These small stream beds, however, rarely exceed a mile or two in length. In view of the admirable treatment of the subject by Johnson,[b] there seems no need to enter upon a discussion of the origin of these lakes. The writer agrees that the "innumerable hollows in the High Plains surface, large and small alike, are due to ground settlement rather than to some process either of original construction or of subsequent erosion."[c]

The influence of these lakes upon the settlement of the country has been important, for on the High Plains the matter of water supply is

[a] Johnson, W. D., The High Plains and their utilization: Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 626.
[b] Ibid., pp. 695–711.
[c] Ibid., p. 702.

vital. In the early history of the Panhandle, before wells had been sunk, these lakes constituted the only source of supply, and thus it happened that the early cow camps were located beside some permanent body of water. In a number of instances a town grew up at the site of the cow camp, and to-day some of the largest county seats—for example, Clarendon, Claude, and Panhandle—owe their location to the presence of such basins. Although windmills are now used to draw water for household use, as shown in Pl. XI, *A*, *B*, a great part of the stock water still comes from the lakes.

IRRIGATION.

NEED OF IRRIGATION.

The Panhandle of Texas is located in the semiarid belt of the Great Plains. The annual rainfall averages approximately 20 inches, but the greater part of this amount is from dashing rains. During certain seasons there is little or no rain. The soil is extremely fertile, and if water were present it is capable of producing abundant crops. At various times on the High Plains farming has been attempted, and often with success for a few years, but usually seasons of drought have ensued and the effort has been abandoned. In general, practically all the crops that have been raised successfully on the High Plains are such forage plants as kaffir corn, sorghum, and milo maize, which are able to mature with a minimum of moisture. At the foot of the plains, particularly along some of the stream valleys, the culture of corn, oats, cotton, and alfalfa is now being attempted with considerable success. Crops are frequently abundant for several successive years, but occasionally fail during periods of drought.

It will be readily understood that in a region with climatic conditions such as those in the Panhandle, irrigation is necessary for successful farming. This fact has long been recognized and a number of desultory attempts have been made to irrigate small tracts, but nothing approaching a large system has ever been projected.

POSSIBLE METHODS OF IRRIGATION.

It is proposed in the following pages to discuss four possible modes of irrigation which might be put into operation in the Panhandle of Texas, viz, (1) irrigation from streams, (2) springs, (3) storm water, and (4) wells.

Irrigation from streams.—It has been shown above that the larger streams of this region practically all flow in broad, shifting, sand-choked channels, contained between low, sandy banks, and that the water varies constantly, the stream being at one time a rushing torrent, at another nothing but a dry sand bed. Only one of these rivers—the Canadian—has its headwaters in the mountains; all the

A. ORCHARD AND GARDEN AT CLAUDE, TEX.

B. JACOB'S WELL, IN A DEEP BASIN NEAR EDGE OF HIGH PLAINS.

others take their rise on the High Plains and are fed by local rains or by springs.

There are no Government gaging stations in the Panhandle of Texas, and no accurate data are available regarding the amount of flow in the various rivers. It is known, however, that enough water passes down the streams each year, particularly during times of flood, to irrigate considerable areas of valuable land. In most cases, however, there would be great difficulty in storing these flood waters. In the first place, so far as known, there are no available dam sites along the larger streams. The broad, shallow channel, sometimes filled to a depth of 100 feet with fine sand, precludes the construction of masonry dams. Besides this, in most cases material for dams is rare, or, indeed, entirely wanting. There are few hard rocks in this region except an occasional local ledge of sandstone or dolomite in the red beds and some indurated Tertiary limestone along the bluffs. Again, the sandy nature of the soil along the streams presents difficulties in the way of the construction of ditches.

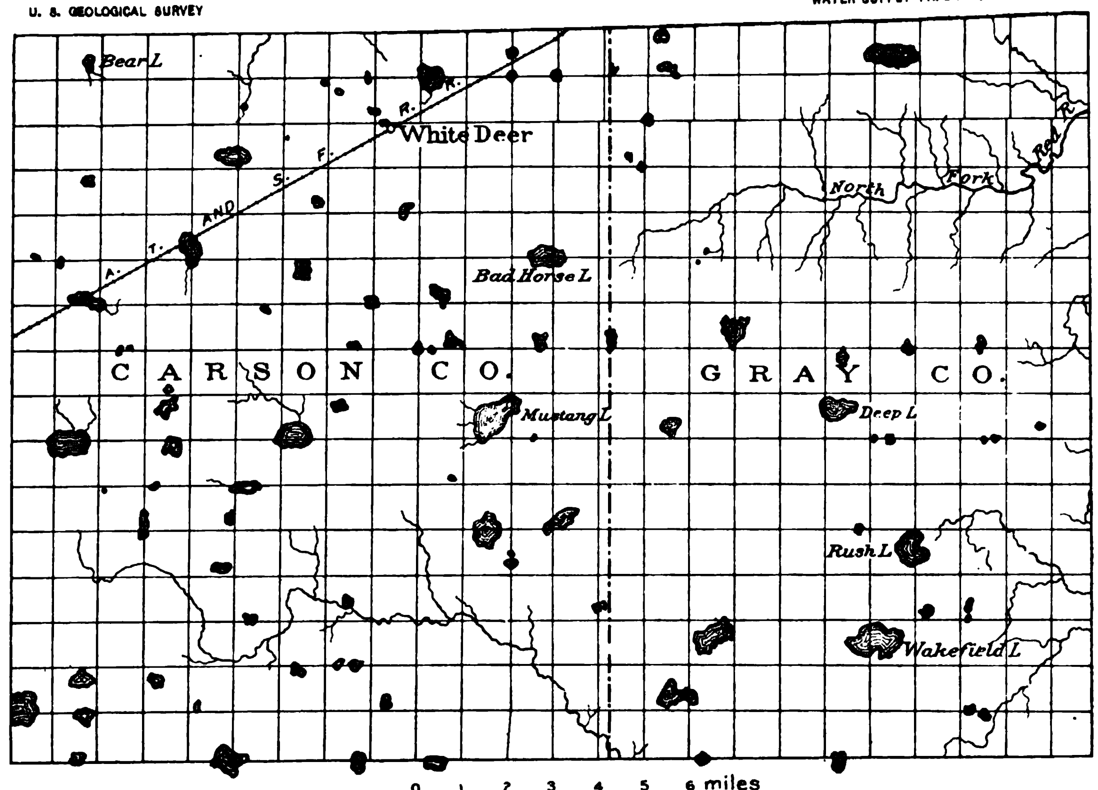
Along some of the smaller streams irrigation has been carried on with more or less success. Along Palo Duro Creek, near the post-office of Mulock, in northeastern Hansford County, Robinson Brothers have a plant in operation from which 35 acres are irrigated. The difficulty at this place has been in securing a suitable site for the dam, and the scarcity of material with which to construct it. In former years several dams here have been washed out during times of high water. Other similar plants are projected farther down Palo Duro Creek; one at Range, Okla., 12 miles below Mulock, has been in successful operation for a number of years. In eastern Wheeler County the water of Sweetwater Creek was formerly utilized to irrigate a tract of 60 acres, but in the last few years the project has been abandoned. There are a number of smaller streams where small plants sufficient to irrigate 10 to 25 acres might be successfully installed. Particularly are there opportunities for such projects along Mammoth, Wolf, and Sweetwater creeks, the upper branches of the various forks of Red River, and some of the short tributaries of Canadian River.

Irrigation from springs.—Since only springs that have a considerable flow can be utilized for this purpose, irrigation from springs must, at best, be confined to limited areas. In the Panhandle it is rather an unusual occurrence for a spring to be located where the water may be led off to irrigate a tract of land, but in a few cases there are springs which are so located that they might be thus utilized. So far as known there is no irrigation directly from a single spring in this region, but there are localities in which a small creek, formed by a number of springs uniting, might be deflected from its channel and carried by a ditch over a tract of land. Examples of this condition might be cited along the smaller tributaries of Wolf and Sweetwater creeks and Canadian River.

Irrigation from storm waters.—Much of the rainfall of the Panhandle occurs as dashing showers at irregular intervals, chiefly during the spring and summer months. After a shower the water on the High Plains accumulates in shallow sags which empty into broad, shallow "lakes;" while among the breaks and at the foot of the plains it passes into the streams. In numerous places the sags on the High Plains or the dry channels among the breaks have been dammed, forming reservoirs, known locally as "tanks," to hold stock water, and in a few instances a ditch has been led out from one of these artificial ponds to irrigate a few square rods of garden or orchard. While it is obvious that irrigation of this character can never be practiced on a large scale, it is nevertheless possible for hundreds of families in the region to be provided with home-grown vegetables and fruit by irrigation from storm waters.

Irrigation from wells.—In the discussion of the subject of artesian water in another part of this report, the conclusion was reached that the probabilities for artesian supply in the Panhandle are not good. On the other hand, however, ordinary wells, which are common in all parts of the region, usually supply considerable amounts of water, often more than is needed for stock water and domestic use, and the surplus might well be used for irrigation. The chief difficulty in the way of the utilization of well water for these purposes is the matter of expense in lifting the water to the surface. In this region wind power is almost universally used for this purpose. In localities where wells are shallow, as, for instance, along stream valleys or among the sand hills, it has been found profitable to use water from wells for irrigating areas of considerable size. In the greater part of the Panhandle, however, the water is too deep to be used in this way. As has been stated, the average depth of the wells on the High Plains is over 200 feet, while on the eroded plains the wells average nearly 100 feet in depth. It is obvious that under such conditions little more can be done than to irrigate a garden or an orchard, and so far as has been observed this is all that is ever attempted. Pl. XIV, *A*, page 46, shows an orchard and garden at Claude, which is irrigated from a well over 250 feet deep. Examples similar to this are not uncommon.

In the sand-hill regions, where the water is not so deep, there are a number of instances of small plots irrigated with water obtained from a well. On the red-beds plain there is less irrigation by this means, partly because the gypsum water is not suitable for irrigation, but chiefly because the need of irrigation is not realized.



MAP SHOWING LOCATION OF LAKES ON A PORTION OF THE HIGH PLAINS.

FUTURE OF IRRIGATION.

Taking into account the local facts it seems very doubtful if there will ever be any extensive irrigation in the region under discussion. The supply of water is not sufficient for this purpose except along the larger streams, where the conditions are such that dams can not be constructed. Small streams, springs, artificial ponds, and wells supply water for limited irrigation, sufficient often to raise vegetables and fruit for a family, but not more. As time goes on and the region is more thickly settled, these small plants will increase in number. There is little to warrant the hope that the water supply in the Panhandle will ever increase, and unless some more efficient means than the ordinary windmill be secured to lift the water from deep wells to the surface it is extremely improbable that anything like extensive works can ever be installed. On the other hand, it is obvious that only a very small part of the available water is now being utilized. It is possible that the future will witness in this region thousands of small pumping plants, each capable of supplying sufficient water to irrigate a garden and an orchard.

WATER CONDITIONS BY COUNTIES.[a]

LIPSCOMB COUNTY.

Topography.—Lipscomb County is in the northeastern corner of the Panhandle. Its surface is a level plain trenched from west to east through the middle by the valley of Wolf Creek. This valley is like a great sloping groove, 250 feet below the level of the High Plains at the west line of the county and 500 feet below at the east line. The High Plain along the northern line of the county is being cut into by branches of Beaver Creek. Wolf Creek Valley separates the county into two areas of plains, one forming the table-land between Wolf Creek and Beaver Creek drainage basins, the other lying between Wolf Creek and Canadian River. Sand hills are present in the northeastern part of the county and between Wolf Creek and the southern line.

Geology.—The rocks of the surface in Lipscomb County are en-

[a] There are no Federal public lands in Texas. This State, when it came into the Union by annexation, retained its public lands, and the general system of township and range lines, by which the lands of the greater part of the United States are surveyed, is not employed. No regular section lines exist, but various-sized tracts are laid off, usually in blocks of square miles as they were selected by the land-grant railroads or purchased by individuals. Some of the earliest surveys employed the Spanish vara, which equals 33.385 inches (1,897.7 + varas equal 1 mile). With such a system of survey the roads are irregularly distributed, no correction lines exist, and the location of points by township and range is impossible. In the eastern part of the Panhandle, however, the counties are uniformly 30 miles square, each containing 900 square miles, a condition which tends to obviate much of the difficulty otherwise encountered in attempting to map the region.

tirely of Tertiary and Quaternary age. Gray sandstones and clays predominate and form conspicuous bluffs along the larger streams.

Water supply.—The two largest streams in the county are Wolf Creek and its tributary Mammoth Creek. Wolf Creek enters from Ochiltree County near the center of the west line of the county and flows directly east; Mammoth Creek rises in the northwest part of the county and flows southeast, joining Wolf Creek in Oklahoma a few miles east of the State line. These creeks, as well as a number of other smaller tributaries of Wolf Creek, are spring fed, but in their lower courses flow through sand-choked beds. Ordinarily the amount of water is small, flowing but 1 or 2 second-feet, and at places entirely disappearing under the sand. The streams are subject to sudden rises of a few hours' duration, at which time the creeks flow several hundred second-feet. In this county the drainage is so well developed that few lakes exist on the High Plains. Lipscomb has the reputation among cattle men of being the best watered county in the Panhandle. The sand strata, both that of the Tertiary and in the sand-hill regions, furnishes an abundance of good water, issuing in the form of numerous springs, which reach the surface along the streams. Although their flow is seldom large, these springs are constant in volume and usually perennial in their character. The water is pure and wholesome and almost always free from notable amounts of salts. It is these springs which feed the numerous tributaries of Wolf Creek and render the water so abundant in the county.

Wells on the uplands in the county range from 130 to 333 feet in depth, with an average of 150 feet. In the valleys a maximum depth of 50 feet usually secures an abundance of water, but along Wolf Creek and some of the smaller streams many of the best wells are not more than 20 feet deep. Of the well records secured in Lipscomb County, the average depth was 121 feet.

OCHILTREE COUNTY.

Topography.—Ochiltree County is in the northern part of this region, lying almost wholly on the High Plains and having uniform plains topography. Near the center it is trenched by the head branches of Wolf Creek, in a valley which gradually deepens to the east. Some of the small side branches of Canadian River which head in the southern part of this region are actively eroding the plains, forming rugged breaks. A few of the small branches of Beaver Creek, which head in extreme northern Ochiltree County, have caused but little erosion.

Geology.—The rocks are entirely Tertiary and Quaternary, and, with the exception of the regions of the breaks near the streams, the surface is flat. Along the bluffs there are ledges of Tertiary clay and sand.

Water supply.—There are two drainage systems in this county. The eastern portion is drained by Wolf Creek, which has its source in the central part of the county, where it is a small fresh-water stream, fed by perennial springs, and flows through a wide gorge in the Tertiary rocks. Canadian River drains the southern portion, and branches of this stream, which have their origin in Tertiary springs along the breaks, flow south into Roberts County. A few minor branches of Beaver Creek drain the northern part.

The High Plains surface is entirely without drainage, and shallow lakes are abundant and often of relatively large size, sometimes covering 100 acres or more. Ochiltree County has an abundance of good well water, but ordinarily it is found at a considerable depth. In the southern part of the county, near the breaks, the depth to water exceeds 400 feet, while farther north water is obtained in abundance from 150 to 300 feet. In the Wolf Creek Valley the wells are shallow, many of them finding good water at 50 to 100 feet. The average depth of twenty-four wells in Ochiltree County is 245 feet.

HANSFORD COUNTY.

Topography.—Hansford County is in the northwestern part of the region to which this report relates. It includes a portion of the High Plains, trenched by two streams, Palo Duro Creek, which rises in the extreme southwestern portion of the county and passes into Oklahoma near the northeast corner, and Coldwater Creek, also known as Rabbit Ear Creek, which enters the county near the center of the west line and flows across the northwest corner, passing into Beaver County, Okla. The greater portion of the county retains its original plains features, while a lesser part consists of valleys and breaks. The entire county presents a gradual slope to the east. The highest point in this part of the Panhandle is attained in this county, just south of the center, along the west line—an altitude of 3,750 feet.

Geology.—Nearly all of the surface rocks of Hansford County are of Tertiary and Quaternary age. In the extreme northeast corner Palo Duro Creek has cut through the Tertiary and exposes the underlying red beds. Along Coldwater and Palo Duro creeks bluffs of hard Tertiary rocks occur, but for the most part nothing appears on the surface except ledges of soft Tertiary marl exposed along prairie draws.

Water supply.—With the exception of a small portion at the south which drains into Canadian River, all the waters of this county find their way into the Beaver Creek drainage system. Palo Duro Creek is a small stream which rises in the southwestern part of the county, and gradually deepens its valley in its passage northeast until at the county line it has attained a depth of approximately 300 feet below the level of the High Plains. Its numerous lateral branches receive the drainage from a considerable area, and at times of heavy rains these discharge their waters into a main trunk, which for a short time becomes a torrent. In its central and lower course there are fertile valleys which afford good farming land, especially adapted to alfalfa culture. Several small irrigation works have been constructed along the lower part of Palo Duro Creek. The difficulty in maintaining these plants is that the dams, built of rough stone uncemented, wash out in times of freshets. Coldwater Creek crosses the northwest corner of the county, entering from Sherman County, Tex., and passing northeast into Beaver County, Okla. It is a small stream flowing through a well-developed gorge. Lakes similar to those in other parts of the region occur upon the High Plains. Hansford County has an abundance of good well water at depths varying on the High Plains from 190 to 300 feet, and rarely does a well fail to encounter an ample supply. In the valleys springs occur, the water from which is like water from the Tertiary beds, pure and wholesome. The average depth of thirteen wells in this county is 235 feet.

HUTCHINSON COUNTY.

Topography.—Hutchinson County lies in the western part of the region to which this report relates. The surface of its northern part is High Plains. The central and southern part is occupied by the canyon of Canadian River and the breaks on either side formed by short tributary creeks.

Geology.—Along Canadian River and along the small streams flowing into it in the central and southwestern part of the county there are extensive exposures of Permian red beds, consisting of red clays and shales with interbedded dolomite and gypsum members. These rocks have been provisionally referred to the Quartermaster formation. The remainder of the county is composed of typical Tertiary and Quaternary deposits, the former being exposed as bluffs along the streams and the latter as sand hills and alluvium.

Water supply.—The entire drainage of this county flows into Canadian River, which crosses the county from southwest to northeast. On the north side there are several small streams, the chief of which are Kit Carson and Coldwater creeks, while from the south flow White Deer, Spring, Bear, Dixon, and Antelope creeks. Thus the greater part of Hutchinson County is well drained. It has an abundance of good water, and, with the exception of some wells in the Canadian Valley and in its tributary creeks, which find their supply in the red beds, the water is wholesome and free from injurious salts. West from Plemons a number of strong springs occur at the line of contact between the red beds and the Tertiary. On the

High Plains water is obtained at depths ranging from 136 to 320 feet, and in the valleys at less than 20 feet. The well records collected show an average depth of 243 feet.

ROBERTS COUNTY.

Topography.—Roberts County lies in the north central part of the region embraced in this report. The surface of the southern portion is High Plains trenched to the east by the gorge of Red Deer Creek. The northern portion is occupied by the valley of Canadian River, with its broad flood plain bordered by a region of breaks on either side. The breaks are so extensively dissected by the smaller stream canyons that the northern portion of Roberts County is one of the most rugged localities in the Panhandle.

Geology.—The surface rocks are mostly of Tertiary age, with the usual sand hills along the streams. In the northwestern portion of the county Canadian River has cut down into the red beds which are exposed in the bluffs along the north bank. Alluvium deposits occur along this river and its tributaries.

Water supply.—The drainage belongs entirely to the Canadian River system. This river flows through a broad flood plain occupied occasionally by low sandy marshes. The waters of a considerable portion of this country reach the river by parallel streams rising in the south central part and flowing north. Few streams enter from the north side, and those which do are short, steep, and intermittent. Red Deer Creek, a tributary of Canadian River, rises in the southern portion of the county and flows northeast into Hemphill County. Ordinarily this stream has no surface flow, but it becomes a raging torrent when there are sudden storms about its head. Tertiary springs occur along the breaks and canyons and supply a number of small creeks. Wells in the High Plains area are 150 to 350 feet deep, and in the valleys water is obtained at from 0 to 20 feet.

HEMPHILL COUNTY.

Topography.—Hemphill County is in the eastern portion of the Panhandle. Its topography is varied, for Washita River rises in the southwestern corner and its northern portion is crossed by Canadian River. These two river systems have removed practically all of the original High Plains level and reduced the region to broad valleys with undulating surfaces between. Along Canadian River is a wide, sandy flood plain, occupied by sand hills in scattered areas. Sand hills also occur in the north, central, and eastern parts of the county.

Geology.—The rocks are chiefly Tertiary deposits, sand hills, and wash. Small areas of red beds are exposed along the Canadian and

Washita rivers in the eastern portion of the county. Along the streams there are bluffs and outliers of white Tertiary rocks, but the greater part of the county consists of rugged breaks and undulating sand hills.

Water supply.—The drainage system is well developed. The water from more than half of this county finds its way into Canadian River through a number of short, swift streams, many of which are perennial, having their source in Tertiary springs. The southern portion of the county is drained by Washita River, a stream which becomes a prominent river in Oklahoma, but is only a small creek in the Panhandle. Its water, being derived from the Tertiary springs, is fresh and free from the injurious salts so common in the large rivers. Springs are not uncommon in this county, and the water obtained from both springs and wells is almost uniformly soft and pure. The depth at which water is found in Hemphill County varies greatly. In the north, south, and southwest the Tertiary beds of the High Plains furnish fresh water at depths ranging from 100 to 332 feet. In the northern sand-hill regions water occurs at depths of 75 to 150 feet. In Canadian and Washita valleys wells are less than 20 feet deep. Records of nineteen wells at various places in this county show an average depth of 94 feet.

WHEELER COUNTY.

Topography.—Wheeler County lies in the eastern part of the Panhandle. The surface is a part of the eroded plains, except small areas in the northwest and southwest. The region is a rolling plain dissected by two principal streams—Sweetwater Creek and North Fork of Red River—with their tributaries. There are extensive sand-hill regions, one of which, 5 to 10 miles in width, and being widest near the center of the county, extends along the south side of Sweetwater Creek almost the entire length of the county. An area of very prominent sand dunes occupies part of the northeastern corner of the county. The hills are mostly low ridges from one-eighth of a mile to 1 mile long and 10 to 20 feet high. Broken ridges and knolls occur everywhere and blow-outs are common. A third sand-hill region is found in the southwestern part of the county, between North Fork and the headwaters of Elm Fork, and a fourth region is in the extreme southeastern part.

Geology.—Red beds belonging to the Greer and Quartermaster formations appear along North Fork of Red River and the branches of Elm Fork in the southern part of the county. Gypsum, dolomite, and red shales of the Greer formation and the red sandy shales and thin sandstones of the Quartermaster formation may be seen in the vicinity of Shamrock, and red bluffs outcrop along the north side of North Fork entirely across the county. The greater part of the surface rocks, however, consist of sand derived from the Tertiary deposits and of alluvium along the valleys.

Water supply.—The drainage of this county is through three streams, Sweetwater Creek and North Fork and Elm Fork of Red River. The first named, farthest to the north, is a small perennial stream which rises just beyond the limits of the county and in the eastern part was formerly used to some small extent for irrigation. North Fork of Red River crosses this county from west to east a little south of the center. Ordinarily it is a small stream flowing in a sand-choked bed and receives no important tributaries in this county. The waters of this river are highly impregnated with calcium sulphate and sodium chloride. The extreme southern part of the county drains to Elm Fork of Red River. As might be expected in a region of sand hills, there are in Wheeler County a number of fine springs. Six miles southwest of Mobeetie is Anderson's spring, which boils up out of the sand and runs off down a little canyon. It is one of the strongest springs in the Panhandle and flows perhaps 1 second-foot. It is said to be artesian in character and the water if confined will rise 15 feet. Other noted springs in the county are Nasby Spring (which fills a 3-inch pipe), Broncho Spring, and Stanley Spring (both of which have a very strong flow). Well water from the Tertiary and sand hills is obtained through the greater part of the county at depths of 80 to 200 feet. In the red-beds region, in the southern part, wells are not so deep, rarely exceeding 50 feet, and the water is usually not good, containing a considerable percentage of mineral salts. Records of nineteen wells in this county show an average of 71 feet.

GRAY COUNTY.

Topography.—Gray County occupies the south central part of the region here discussed. The surface is a portion of the High Plains cut into by two streams—North Fork of Red River, which flows through a gorge crossing the county from west to east, and McClellan Creek, flowing in a similar gorge from southwest to northeast, joining North Fork near the eastern line of the county. Wide breaks border the gorge of these two principal streams.

Geology.—With the exception of a small area of red beds near the mouth of McCellan Creek along the eastern line, the rocks of Gray County are entirely Tertiary and Quaternary. High white cliffs are exposed along the edges of the High Plains, and along the breaks and streams there are alluvial deposits and sand hills.

Water supply.—The greater part of the drainage is through North Fork of Red River and McClellan Creek. The former stream, which rises in Carson County just west of the Gray County line and flows east, drains only a limited region at the south through a few short tributaries, none of which are more than 5 miles in length. The southern portion of the county is drained by McClellan Creek, a branch of North Fork, which rises west of the south central part of the county. Springs from the Tertiary and sand hills occur in a number of places along the streams. Many are from small seeps, but several have an estimated flow of 40 gallons per minute. On the plains good water is obtained at depths ranging from 100 to 280 feet. In the valleys depths to water do not exceed 35 feet. Along North Fork in the eastern part of the county a few wells afford gypsum water; otherwise the supply in this county is pure and wholesome. Records of eleven wells show an average depth of 166 feet.

CARSON COUNTY.

Topography.—Carson County lies in the western part of the region. With the exception of Ochiltree County, Carson contains a larger proportion of the High Plains than any other county here described. The northern part is dissected by tributaries of Canadian River and a very small portion of the eastern part is occupied by the headwaters of Salt Fork of Red River. With these exceptions its surface is the uniform level of the High Plains, dotted at intervals by shallow lakes.

Geology.—In the extreme northwestern portion of the county along the canyons of Antelope and Dixon creeks, tributaries of Canadian River, there are exposures of the red beds, consisting of red clays and shales with ledges of gypsum and dolomite. With this minor exception, the rocks are Tertiary and Quaternary. Along the breaks high Tertiary cliffs are present, but the flat, upland Tertiary constitutes the greater part of the rocks of the county.

Water supply.—The drainage of Carson County is into two systems, Canadian River and Red River, between which is a great flat table-land divide. The former stream receives the water from the northern part of the county through a number of creeks—the most important being White Deer, Spring, Dixon, and Antelope—all of which have their rise near the central line of the county and flow north. The headwaters of Salt Fork of Red River occupy a few square miles in the southwestern part of the county. Most of this county, however, has no drainage other than that which finds its way into the shallow lakes on the level upland and disappears by seepage and evaporation.

In the southern part of Carson County the water table seems to be very deep, for while a few wells secure permanent flows at depths of less than 250 feet, many of them are obliged to penetrate 400 to 450 feet for an adequate supply; but water when found is both abundant and pure. Among the breaks and along the creeks in the northern part of the county the wells range from 50 to 200 feet. Few springs occur, those which are found being near the base of the Tertiary in the northwestern part of the county.

ARMSTRONG COUNTY.

Topography.—Armstrong county is in the southwestern part of the region. It is a level plain cut by three canyons trending southeast. The central and northwestern part of the county has the uniform surface of the High Plains. The northeastern portion is trenched by the upper course of Salt Fork of Red River, which has its source near the center of the north line of the county. Mulberry Creek Canyon crosses the county from northwest to southeast. In the southwestern portion is Palo Duro Canyon, through which flows Prairie Dog Fork of Red River. Twenty-five miles of this gorge, 875 feet deep and 5 miles wide, lies in Armstrong County. The sides of the canyon are frequently precipitous and exhibit typical banded structure so rough that the canyon is passable by wagon only along selected routes.

Geology.—The best geological sections obtainable in the Panhandle are found along Palo Duro Canyon in Armstrong County, where all the formations discussed in this report are exposed. The Greer and Quartermaster formations of the Permian red beds are particularly well exposed in this canyon. The Dockum formation outcrops halfway up the escarpment, and Tertiary clays, sand, and conglomerate lie along the upper part of the bluffs. The level upland in other parts of the county exhibits the ordinary Tertiary and Quaternary rocks.

Water supply.—Northeastern Armstrong County is drained by the headwaters of Salt Fork of Red River, and Mulberry Creek receives the drainage from the central and southeastern parts of the county. Prairie Dog Fork of Red River in Palo Duro Canyon, in the southwestern part, has no large tributaries in the county, and although it flows through a great canyon the stream itself ordinarily has little or no surface flow, but, like other streams of the plains, is subject to rapid rises after heavy rains near its head. The drainage of a large portion of the county is undeveloped, and the shallow lakes which occur at frequent intervals often reach considerable size. Springs are not common, but a few are found at the base of the Tertiary and among the red beds. On the High Plains water is obtained in abundance in wells ranging in depth from 120 to 320 feet. Few wells have been sunk in the red beds, but those that have been dug usually find water of rather poor quality at depths ranging from 20 to 100 feet. Records of eighteen wells in Armstrong County show an average depth of 207 feet.

DONLEY COUNTY.

Topography.—Donley County lies in the southern part of the region. The northern and western portions are level High Plains. On the eroded plains which occupy the eastern and southern parts of the county the surface is rolling and dissected by many streams. Even on the High Plains the streams occupy well-marked courses and have so dissected the surface that only in a few instances is the upland sufficiently level to permit the water to collect in lakes. The western extension of the sand-hill region, which crosses Collingsworth County south of Salt Fork of Red River, finds its terminus in the escarpment at the base of the High Plains in Donley County.

Geology.—Both red beds and Tertiary rocks are exposed in Donley County. The red beds, including both the Greer and Quartermaster formations, outcrop along the streams, particularly along North Fork of Red River in the region northeast of the center of the county and along Mulberry Creek in the southwestern part. Along the latter stream the Dockum beds occur. Tertiary rocks constitute the High Plains in the northern and western parts of the county: while the sand hills derived largely from Tertiary deposits occupy considerable areas in the central and southern portions. Alluvium is found along the stream valleys.

Water supply.—The drainage of the county is through two branches of Red River—Salt Fork, which crosses the county, and several smaller branches of Prairie Dog Fork, which rise in the county and flow south. Salt Fork flows almost due east across the center of the county. It is a small stream with a sand-choked bed and a flow of but a few second-feet, the water being free from disagreeable salts. The southern part of Donley County is drained by the branches of Salt Fork, the chief of which is Mulberry Creek, rising in Armstrong County and flowing across the southwest corner of Donley County. It is an ordinary stream of the plains, with a wide, sand-choked channel and ordinarily little water, but at times of heavy rainfall it assumes the proportions of a river. Springs are found in both the sand-hill regions and among the red beds. On the High Plains and in the escarpment region wells are from 40 to 250 feet deep. In the valleys and lower portions of the county water is obtained at 20 to 165 feet. The Tertiary and sand-hill water is good, while that found in the red beds is usually bad. Records from thirty wells show an average depth of 152 feet.

COLLINGSWORTH COUNTY.

Topography.—Collingsworth County forms the southeastern portion of the region here discussed. This county, which is almost wholly in the eroded plains, presents the most diverse topography of all here described. It is trenched from northwest to southeast by three stream systems—Elm and Salt forks of Red River, and Spillers Creek, a branch of Prairie Dog Fork. Just west of the center and extending entirely across the county, trending slightly west of south, are the Dozier Mounds, composed of hard ledges of sandstone underlain by stratified clays, shales, and sandstones. In the southwestern corner of the county these hills are deeply dissected by streams, rendering the region very rugged. On the south side of Salt Fork is a sand-hill region ranging from 2 to 9 miles in width, extending from northwest to southeast entirely across the county.

Geology.—The greater part of the rocks of Collingsworth County belong to the Greer and Quartermaster formations of the red beds. Along the various streams ledges of gypsum and dolomite outcrop, while at a higher level soft sandstones occur. The extreme northwestern part of the county is in the escarpment region, and Tertiary and Quaternary sand hills appear south of Salt Fork of Red River.

Water supply.—Elm Fork of Red River rises in the northwestern portion of the county and flows southeast, leaving the county 9 miles from the northern limit, thus draining the entire northern portion. Salt Fork of Red River crosses the county from west to east in a tortuous course near its center and drains the middle portion of the county. Spillers Creek, a branch of Prairie Dog Fork of Red River, has its source in Donley County and flows southeast across Collingsworth, draining the southwestern part. Except in the sand-hill regions south of Salt Fork, good water is difficult to obtain for the reason that the county is underlain by red beds, which contain large quantities of gypsum and other mineral salts. There are a number of springs of good water among the sand hills. Springs occur in the red beds also, but the water often contains salt or gypsum and is not suitable for general domestic use. Wells in the sand hills are 10 to 220 feet in depth and in the red beds 40 to 190 feet. Records from twenty wells in Collingsworth County show an average depth of 105 feet.

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[Water-Supply Paper No. 154.]

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WS 78. Preliminary report on artesian basin in southwestern Idaho and southeastern Oregon, by I. C. Russell. 1903. 53 pp., 2 pls.

PP 17. Preliminary report on the geology and water resources of Nebraska west of the one hundred and third meridian, by N. H. Darton. 1903. 69 pp., 43 pls.

WS 90. Geology and water resources of a part of the lower James River Valley, South Dakota, by J. E. Todd and C. M. Hall. 1904. 47 pp., 23 pls.

WS 101. Underground waters of southern Louisiana, by G. D. Harris, with discussions of their uses for water supplies and for rice irrigation, by M. L. Fuller. 1904. 98 pp., 11 pls.

WS 102. Contributions to the hydrology of eastern United States, 1903, by M. L. Fuller. 1904. 522 pp.

WS 104. Underground waters of Gila Valley, Arizona, by W. T. Lee. 1904. 71 pp., 5 pls.

WS 110. Contributions to the hydrology of eastern United States, 1904; M. L. Fuller, geologist in charge. 1904. 211 pp., 5 pls.

PP 32. Geology and underground water resources of the central Great Plains, by N. H. Darton. 1904. 433 pp., 72 pls.

WS 111. Preliminary report on underground waters of Washington, by Henry Landes. 1904. 85 pp. 1 pl.

WS 112. Underflow tests in the drainage basin of Los Angeles River, by Homer Hamlin. 1904. 55 pp., 7 pls.

WS 114. Underground waters of eastern United States; M. L. Fuller, geologist in charge. 1904. 285 pp., 18 pls.

WS 118. Geology and water resources of east-central Washington, by F. C. Calkins. 1905. 96 pp., 4 pls.

B 252. Preliminary report on the geology and water resources of central Oregon, by I. C. Russell. 1905. 138 pp., 24 pls.

WS 120. Bibliographic review and index of papers relating to underground waters published by the United States Geological Survey, 1879–1904, by M. L. Fuller. 1905. 128 pp.

WS 122. Relation of the law to underground waters, by D. W. Johnson. 1905. 55 pp.

WS 123. Geology and underground water conditions of the Jornada del Muerto, New Mexico, by C. R. Keyes. 1905. 42 pp., 9 pls.

WS 136. Underground waters of the Salt River Valley, by W. T. Lee. 1905. 194 pp., 24 pls.

B 264. Record of deep-well drilling for 1904, by M. L. Fuller, E. F. Lines, and A. C. Veatch. 1905 106 pp.

PP 44. Underground water resources of Long Island, New York, by A. C. Veatch and others. 1906.

WS 137. Development of underground waters in the eastern coastal plain region of southern California, by W. C. Mendenhall. 1905. 140 pp., 7 pls.

WS 138. Development of underground waters in the central coastal plain region of southern California, by W. C. Mendenhall. 1905. 162 pp., 5 pls.

WS 139. Development of underground waters in the western coastal plain region of southern California, by W. C. Mendenhall. 1905. 105 pp., 7 pls.

WS 140. Field measurements of the rate of movement of underground waters, by C. S. Slichter. 1905. 122 pp., 15 pls.

WS 141. Observations on the ground waters of Rio Grande Valley, by C. S. Slichter. 1905. 83 pp., 5 pls.

WS 142. Hydrology of San Bernardino Valley, California, by W. C. Mendenhall. 1905. 124 pp., 13 pls.

WS 145. Contributions to the hydrology of eastern United States; M. L. Fuller, geologist in charge. 1905. 220 pp., 6 pls.

WS 148. Geology and water resources of Oklahoma, by C. N. Gould. 1905. 178 pp., 22 pls.

WS 149. Preliminary list of deep borings in the United States. Second edition, with additions, by N. H. Darton. 1905. 175 pp.

PP 46. Geology and underground water resources of northern Louisiana and southern Arkansas, by A. C. Veatch. 1906.

WS 153. The underflow in Arkansas Valley in western Kansas, by C. S. Slichter. 1906.

WS 154. The geology and water resources of the eastern portion of the Panhandle of Texas, by C. N. Gould. 1906. 64 pp., 15 pls.

The following papers also relate to this subject: Underground waters of Arkansas Valley in eastern Colorado, by G. K. Gilbert, in Seventeenth Annual, Pt. II; Preliminary report on artesian waters of a portion of the Dakotas, by N. H. Darton, in Seventeenth Annual, Pt. II; Water resources of Illinois, by Frank Leverett, in Seventeenth Annual, Pt. II; Water resources of Indiana and Ohio, by Frank Leverett, in Eighteenth Annual, Pt. IV; New developments in well boring and irrigation in eastern South Dakota, by N. H. Darton, in Eighteenth Annual, Pt. IV; Rock waters of Ohio, by Edward Orton, in Nineteenth Annual, Pt. IV; Artesian well prospects in the Atlantic coastal plain region, by N. H. Darton, Bulletin No. 138.

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THE DIRECTOR,

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WASHINGTON, D. C.

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DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

FLUCTUATIONS OF THE WATER LEVEL IN WELLS, WITH SPECIAL REFERENCE TO LONG ISLAND, NEW YORK

BY

A. C. VEATCH

WASHINGTON
GOVERNMENT PRINTING OFFICE
1906

CONTENTS.

ILLUSTRATIONS.

FLUCTUATIONS OF THE WATER LEVEL IN WELLS, WITH SPECIAL REFERENCE TO LONG ISLAND. NEW YORK.

By A. C. Veatch.

INTRODUCTION AND SUMMARY.

In connection with the investigation of the geology of Long Island by the United States Geological Survey in the summer of 1903, a few observations were made on the fluctuation of the water level in wells, both with direct-reading and self-recording gages. In the consideration of these data, as well as those collected at the same time by the New York City commission on additional water supply, it has seemed desirable to enter into a general discussion of the fluctuation of water in wells.

Some of the results of this study may be briefly summarized as follows:

1. The most important and characteristic of the natural ground-water fluctuations is the regular annual period. This is a relatively uniform curve, with a single maximum and minimum, on which the fluctuations of shorter periods, as a rule, form but minor irregularities. This curve does not generally resemble the rainfall curve. Were the rainfall uniform throughout the year, the ground water would still show a regular yearly period and the maximum would occur early in the year in the North Temperate Zone. The effect of irregularities in the rainfall is to move the time of occurrence of this maximum either forward or back.

2. The water from single showers is generally delivered gradually to the ground-water table, and even where noticeable fluctuations are produced, these do not commonly make important irregularities in the regular annual ground-water curve.

3. Single showers may, by transmitted pressure through the soil air, produce instantaneous and noticeable rises in the water in wells and notably increase the stream discharge without contributing either to the ground water or directly to the surface flow.

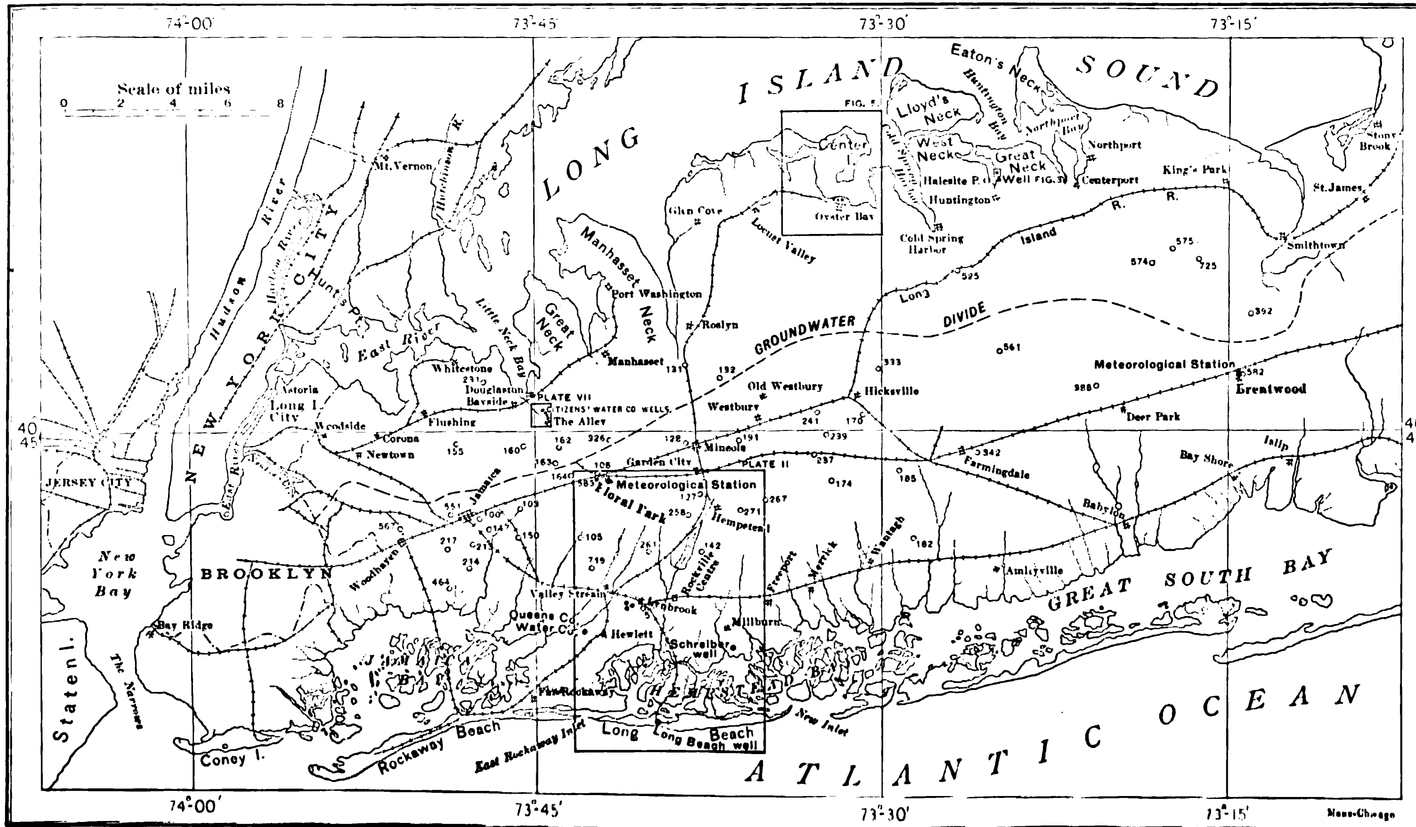
4. The amount contributed to the ground water can not be satisfactorily estimated by the rise and fall of the water in wells, because the same amount of rainfall under the same geologic and climatic conditions, in beds of the same porosity, will produce fluctuations of very different values. Near the ground-water outlet the total yearly range may be but a few inches, while near the ground-water divide it may be 50 or 100 feet. When an attempt is made to calculate the amount of water received from single rains, the results are not reliable, because in the cases which are usually taken, such as sharp, quick rises, it is impossible to tell how much of the rise is due to transmitted pressure and how much to direct infiltration.

5. Because of the increase in stream flow due (1) to transmitted pressure from rains, (2) to changes in barometric pressure, and (3) to increase in area of ground-water discharge, with the elevation of the ground-water table, it is not possible to

correctly separate the quantity of water in the stream discharge contributed by spring flow from that contributed by direct surface run-off. There are many reasons for believing that in humid regions "flood flows" contain large percentages of ground water.

6. Tidal fluctuations in wells are very often produced by a plastic deformation due to the loading of the tides, and the occurrence of such fluctuations in wells does not in itself indicate a connection between the water-bearing strata and the sea.

7. Temperature changes may produce marked fluctuations (1) by changes in capillary attraction—such fluctuations are perceptible only at the surface of the zone of complete saturation, are not transmitted to deeper levels, and vary directly with the temperature; (2) by changes in viscosity or rate of flow—fluctuations due to this cause vary inversely with the temperature, and show in deep wells by transmitted pressure.



SKETCH MAP OF WESTERN LONG ISLAND, SHOWING LOCATION OF DETAILED MAPS (PLS. II AND VII AND FIGS. 2, 4, AND 5) AND POSITION OF WELLS REFERRED TO IN FIG. 13.

PART I.

LONG ISLAND OBSERVATIONS.

INTRODUCTORY OUTLINE OF THE HYDROLOGIC CONDITIONS.

The conditions on Long Island, New York, are particularly favorable for the study of the fluctuations of water in wells. The geologic and topographic conditions are such that it may be affirmed that the underground water is derived wholly from the rain which falls on the surface of the island, and the problems involved are, therefore, not unduly complicated, as they are in many regions, by the possibility of the influx of water from other areas. In addition to this comparatively complete ground-water isolation, the island is of such a size—120 miles long and 20 miles wide—that ground-water phenomena can attain a relatively complete development, and the geologic structure of the water-bearing beds, while not complicated, is sufficiently varied to produce several differing conditions.

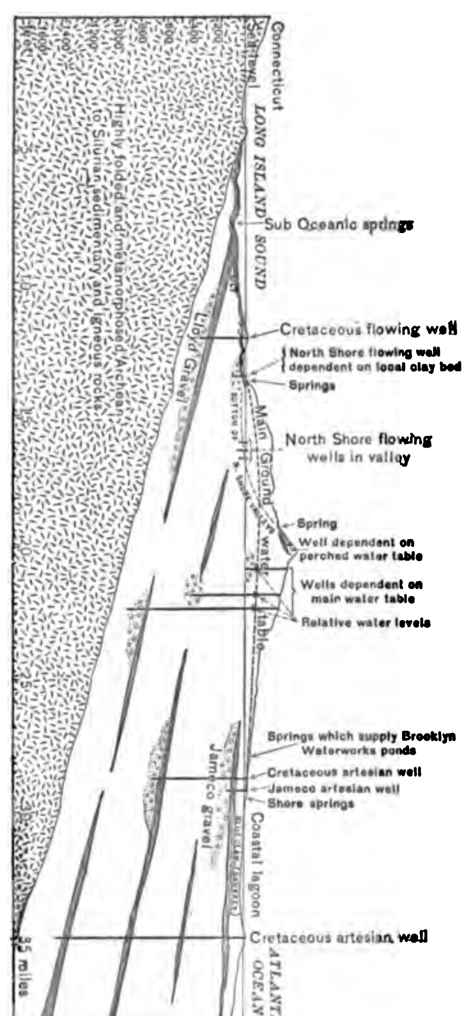


FIG. 1.—Diagrammatic cross section of Long Island, showing principal topographic and geologic factors influencing the underground-water conditions.

Topographically the western part of Long Island—the portion involved directly in this paper—may be said to consist of a single range of rolling hills, usually 150 to 250 feet high, though in one place attaining an elevation of over 400 feet. This hill range descends somewhat abruptly to the north shore, where it is cut by several reentrant bays occupying old valleys. On the south side is a very flat gravel plain, sloping gently to the ocean, along which a series of barrier beaches inclosing long marshes has been developed. To the east the hill range divides and produces two hilly peninsulas, each with a single ridge on the northern side.

Geologically the island may be regarded as a series of relatively porous gravel and sand beds, containing irregular and discontinuous clay masses, the whole limited below by the peneplained surface of a mass of highly disturbed and metamorphosed Paleozoic and pre-Paleozoic rocks, which have little water value except as a more or less complete barrier to downward percolation (fig. 1). While these unconsolidated beds represent, in the geologic time scale, several of the divisions of the Upper Cretaceous and as many as five Pleistocene or glacial stages, and as a whole are stratified deposits dipping at very low angles south and southeastward, they are, under the island, essentially continuous from a water standpoint, and the rain falling on the surface is relatively free to pass to any part of the mass. The Pleistocene beds which form the surface are, however, as a rule, coarse and more porous than the underlying Cretaceous and tend to increase the absorbing power of the island. As a result, the percentage of rainfall which passes into the streams without first going through the ground is extremely small.[a] This percolating water has entirely saturated the porous strata above the bed rock, except a limited portion at the surface, and has driven out the salt water which filled these beds when they were first deposited and which reoccupied them, at least in part, during the several submergences to which this region has been subjected. The surface of this zone of complete saturation, or the main ground-water table, is coincident with the sea level at the shores and becomes more and more elevated in passing inland, though the rate of increase of elevation is less than that of the surface, of which it is but a subdued reflection (fig. 1).[b]

This slope of the ground-water table permits the development of artesian wells at many points on the coast, at elevations which are commonly less than 10 feet above high tide. The head is, in all cases, due to the greater height of the ground water in the adjacent hill mass. In order that such a differential head may be developed, it is merely necessary that the water-bearing bed in question be coarser than the overlying beds. A clay or other impervious cover is not essential and, indeed, is often absent.

OBSERVATIONS OF THE UNITED STATES GEOLOGICAL SURVEY.

Observations on the fluctuations of the water level in wells were made by the Geological Survey near Huntington, Oyster Bay, Valley Stream, Millburn, Long Beach, and Douglaston, all villages on Long Island west of longitude 73° W., and between latitudes 40° 35′ and 40° 55′ N. (Pl. I.)

OBSERVATIONS WITH DIRECT-READING GAGES.

OBSERVATIONS AT HUNTINGTON, N. Y.

The Huntington observations, from which the other Survey observations developed, were undertaken to test the common report that the discharge of most of the artesian wells along the northern shore of Long Island fluctuated with the tide; in some cases the flow ranging from 0 at low tide to over 100 gallons per minute at high tide. Nearly all of these wells were being pumped, or were utilized to run rams, but permission was obtained to gage a newly completed well belonging to the Huntington Light and Power Company, at Huntington Harbor, until it should be connected with the pumps—a period of three or four days.

A direct-reading float gage of simple type was quickly constructed by Baker & Fox, Brooklyn, N. Y. This consisted of a 2-inch cylinder of brass carrying a ¼-inch aluminum rod 6 feet long and graduated to hundredths of a foot, with the zero point just

[a] Spear (Rept. New York City Commission on Additional Water Supply, 1904, p. 829) has estimated that 43 per cent of the total stream flow (or 14 per cent of the rainfall) can be considered as flood flow or as not having passed through the ground. He bases this judgment on the relative heights of the stream and ground-water levels near the south shore, where, as explained on page 51, a correct judgment can not be formed. The average flood flow is believed to be much less than 5 per cent of the precipitation.

[b] For details of the slope of the ground-water table see Prof. Paper U. S. Geol. Survey No. 44, 1906, Pls. XI, XII.

above the cylinder. For convenience in carrying, as well as to avoid the use of so long a rod except where absolutely necessary, the rod was divided into three parts and jointed. The cylinder was so constructed that it would just carry the total length of 6 feet, and when used with only 2 or 4 feet of rod, weights, balancing the effect of the part removed, were added to the bottom of the cylinder.

Some trouble was experienced by the float tending to approach the side of the well and develop a thin capillary film between it and the pipe, which decreased the sensitiveness of the gage. It is suggested that when direct-reading floats are used in wells of small diameter they be kept away from the walls of the well by means of

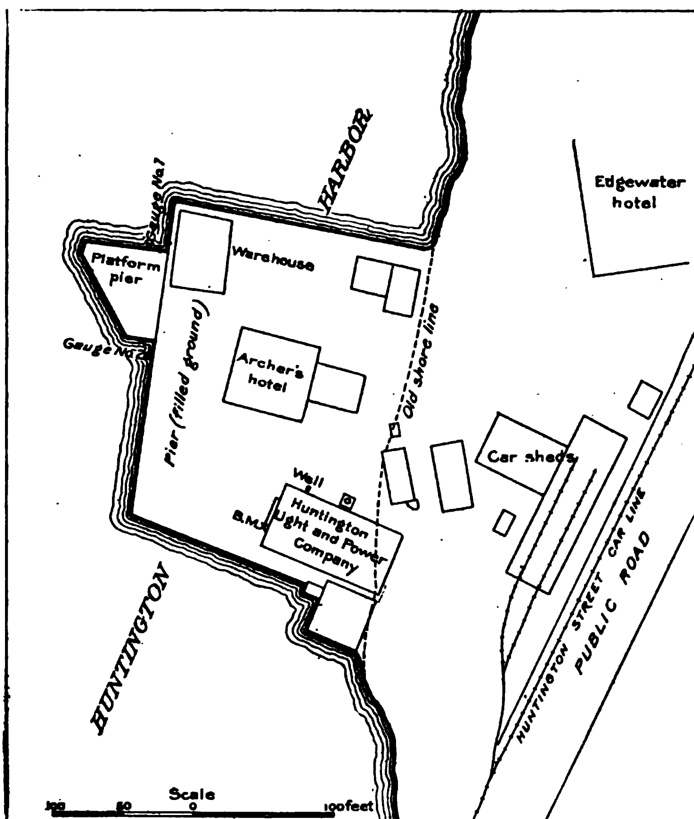


FIG. 2.—Sketch map showing location of well of Huntington Light and Power Company at Huntington Harbor, N. Y.

slightly arched wires, as in the float devised by Professor King for the self-recording gages used in the Madison experiments and later on Long Island.

The well of the Huntington Light and Power Company is situated on a dock at Huntington Harbor, near Halesite post-office (Pl. I, fig. 2.) The natural level of the surface at the point where the well is sunk is between high- and low-tide mark, but the ground has been built up by filling about 5 feet higher. The well is 75 feet deep and 4 inches in diameter, and the water rises in the pipe from 1 to 3 feet above the surface of the made ground. The well was piped above the limit of flow, so that all the fluctuations could be measured directly, rather than inferred from variations in the rate of discharge.

from below the blue-clay layer (fig. 6). This blue clay thins rapidly southward and entirely disappears half a mile south of the wells (fig. 7). It extends under Oyster Bay Harbor and is exposed in the clay pits on the south end of Center Island.[a]

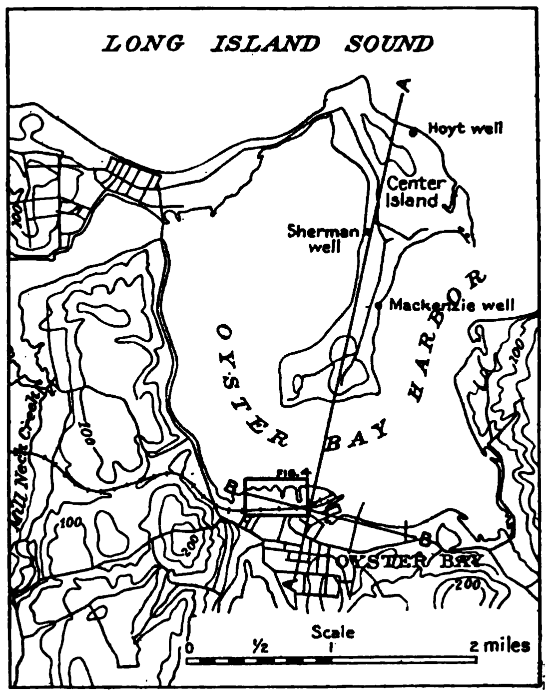


FIG. 5.—Sketch map showing topographic surroundings of wells shown in fig. 4 and location of sections shown in figs. 6 and 7.

All these wells were flowing, and in each case, before observations were commenced, lengths of pipe were added until the wells no longer flowed, even at high tide. Float gages similar to those used at Huntington were then inserted and the wells covered

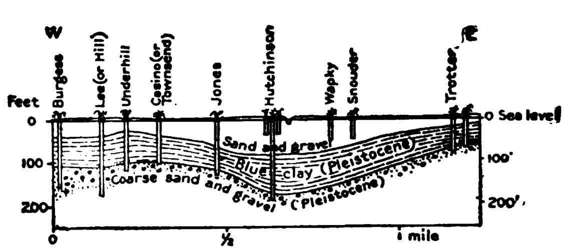


FIG. 6.—Section at Oyster Bay, N. Y., along line B–B, fig. 5, showing geologic relations of wells observed.

with flat-topped caps, each containing a smooth beveled hole through which the gage rod extended.[b]

[a] The folding of the beds here shown is due to ice shove. See Prof. Paper U. S. Geol. Survey No. 44, 1906, pp. 39–43.

[b] The general conditions of observation are well shown in Prof. Paper U. S. Geol. Survey No. 44, 1906, Pl. XIII, *A*. This view indicates, in a very graphic manner, the relation of the wells to the water of the bay and the considerable head developed by these fresh-water artesian wells on the seashore.

In order to obtain more refined results than were possible with the board gage at Huntington, a 3-inch pipe, perforated at a point several feet above the bottom, was driven in the harbor at the end of a row of piles and at a distance of about 200 feet

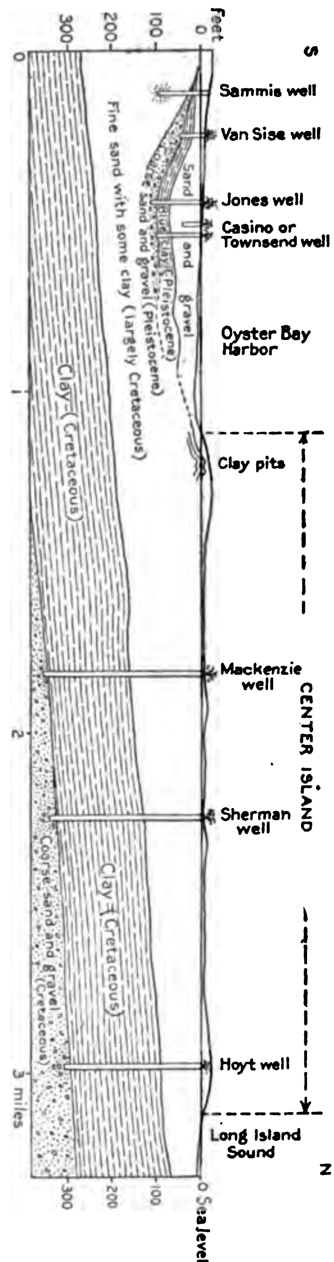


FIG. 7.—Section at Oyster Bay, N. Y., along line A–A, fig. 5, showing geologic relations of the artesian wells at Oyster Bay and on Center Island.

from the shore (fig. 4). This still box or tide well was fitted with a direct-reading float gage like those used in the artesian wells. This arrangement is not to be recommended during stormy weather, but fortunately during the whole time of observation at this place no trouble was experienced from that cause.

Observations were commenced on the Casino, Underhill, and Burgess wells on the evening of May 30, by a party in charge of Mr. Isaiah Bowman, and continued, with interruptions on the nights of May 30 and 31 and June 1, to 10 p. m. on June 4.

On June 10 and 11 observations were made on the Lee (or Hill) well, covering two high and two low tides, and for the purpose of comparison the Casino and tide wells were also observed. Observations were generally made every minute for thirty minutes preceding and following the times of high and low water, and from these values the curves shown in fig. 8 were drawn. Times of high and low water were found by plotting the observations near high- and low-tide marks on a much larger scale, in the manner shown in fig. 3. The values so obtained are indicated on fig. 8, and are given in the following table:

Difference in time between high- and low-water stages in four artesian wells at Oyster Bay, N. Y., and the tide in Oyster Bay Harbor.

[Time expressed in hours and minutes of 24-hour clock.]

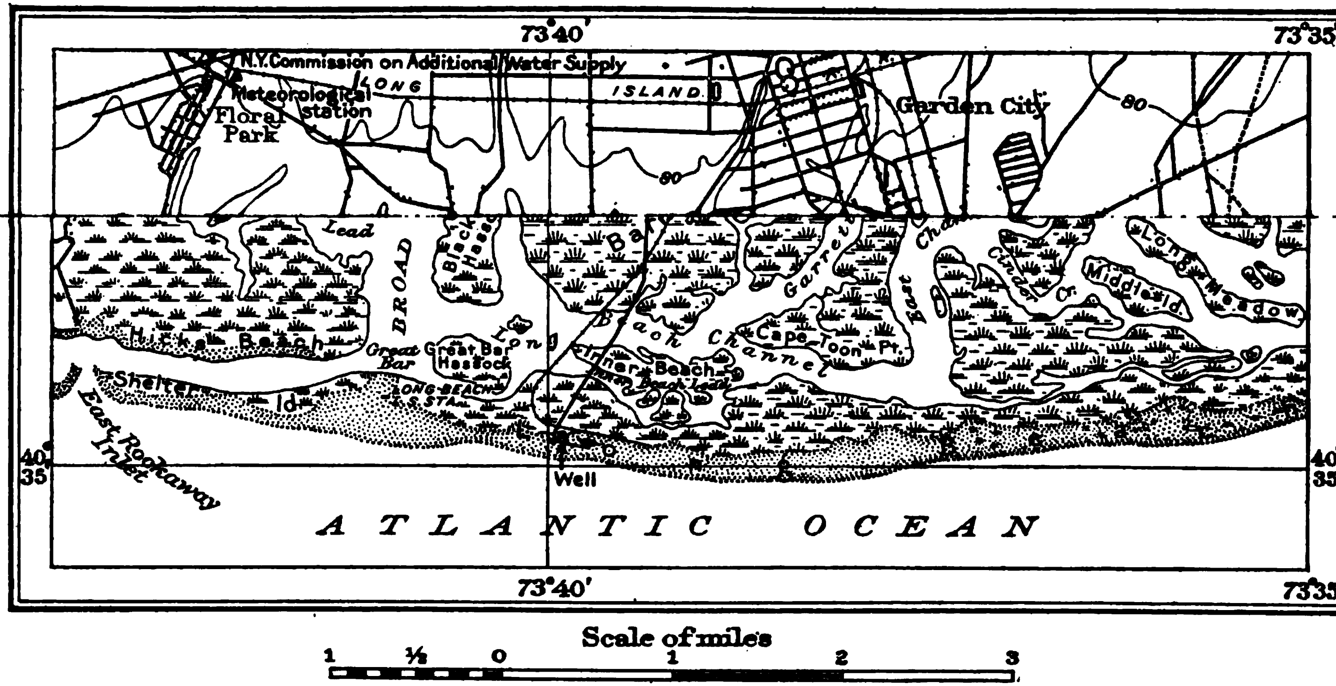
HIGH TIDES.

1903.	May 30.	May 31.		June 1.	June 2.		June 3.		June 4.	June 10.	June 11.	Average lag.
												Minutes.
Casino well	......	......	14.52	......	......	17.05	5.19	18.11	6.38	......	12.22	
Tide	......	......	14.43	......	......	16.54	5.12	18.02	6.29	......	12.20	
Difference (lag)	......	......	.09	......	......	.11	.07	.09	.09	......	.02	0.8
Burgess well	......	......	15.13	16.05	......	17.15	5.38	18.25	6.56	......	......	
Tide	......	......	14.43	15.44	......	16.54	5.12	18.02	6.29	......	......	
Difference (lag)	......	......	.30	.21	......	.21	.26	.23	.27	......	......	24.7
Lee well	......	......	......	......	......	......	......	......	......	a0.20	13.12	
Tide	......	......	......	......	......	......	......	......	......	23.48	12.20	
Difference (lag)	......	......	......	......	......	......	......	......	......	.32	.52	42
Underhill well	......	......	16.02	17.02	......	18.00	6.20	19.10	7.41	......	......	
Tide	......	......	14.43	15.44	......	16.54	5.12	18.02	6.29	......	......	
Difference (lag)	......	......	1.19	1.18	......	1.06	1.08	1.08	1.12	......	......	71.8

LOW TIDES.

1903.	May 30.	May 31.		June 1.	June 2.		June 3.		June 4.	June 10.	June 11.	Average lag.
Casino well	20.28	9.21	......	......	......	23.46	12.08	......	0.47	17.42	6.32	
Tide	20.10	9.04	......	......	......	23.36	11.55	......	.38	17.33	6.20	
Difference (lag)	.18	.17	......	......	......	.10	.13	......	.09	.09	.12	12.6
Burgess well	......	......	......	10.41	11.26	a0.10	......	12.28	1.04	......	......	
Tide	......	......	......	10.00	10.53	23.36	......	11.55	.38	......	......	
Difference (lag)	......	......	......	.41	.33	.34	......	.33	.26	......	......	33.4
Lee well	......	......	......	......	......	......	......	......	......	18.23	7.26	
Tide	......	......	......	......	......	......	......	......	......	17.33	6.20	
Difference (lag)	......	......	......	......	......	......	......	......	......	.50	1.06	58
Underhill well	21.25	10.17	......	11.22	12.06	a0.55	......	13.11	1.49	......	......	
Tide	20.10	9.04	......	10.00	10.53	23.36	......	11.55	.38	......	......	
Difference (lag)	1.15	1.13	......	1.22	1.13	1.19	......	1.16	1.11	......	......	75.6

a Morning of following day.



MAP OF PORTION OF SOUTHERN LONG ISLAND, SHOWING LOCATION OF HEWLETT, LONG BEACH, MILLBURN, AND LYNBROOK WELLS.

OBSERVATIONS WITH SELF-RECORDING GAGES.

INSTRUMENTS USED.

The continuation of the observations by means of self-recording gages was due to the timely interest of Mr. F. H. Newell and Prof. Charles S. Slichter. Mr. Newell

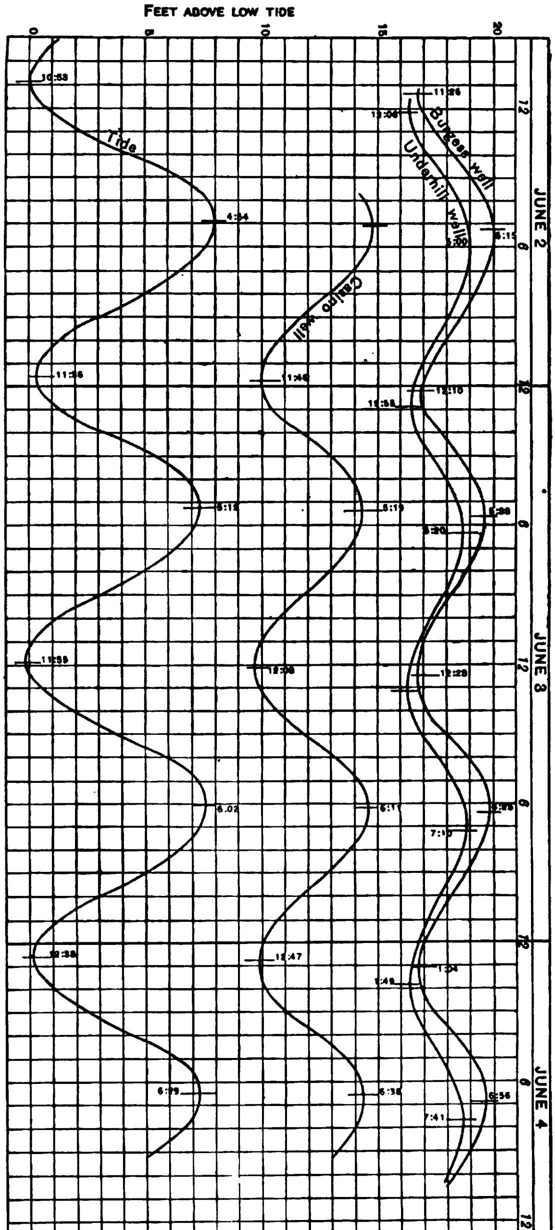


FIG. 8.—Well and tide curves at Oyster Bay, N. Y., showing tidal fluctuations due to plastic deformation. From observations with direct-reading floats by party in charge of Isaiah Bowman, under direction of A. C. Veatch.

visited the island when the observations at Oyster Bay were in progress, and at once directed that three Friez water-stage registers be purchased. These were supplemented by a gage constructed at Purdue University from the designs of Mr. Elwood Mead.

Shortly after Mr. Newell's visit, and before the Friez gages had been received, Prof. Charles S. Slichter arrived to take charge of the measurement of the rate of underflow. He kindly obtained the loan of five of the gages used by King in his experiments at Madison, Wis.;[a] of these, four were week gages and one a one-day gage. The King gages were constructed by H. Green, Brooklyn, from barograph stands; they consist of an ordinary barograph cylinder, driven by a double-spring marine clock, the recording device being a simple lever on a cone bearing with a pen on one end and a place for attaching the float on the other. At the point where the clock motion is transmitted to the drum there was a slight amount of play which King found would introduce into the records an error of one to two hours. A friction brake was, however, subsequently added to overcome this defect. The gages as received on Long Island were adjusted to magnify the fluctuations two or more times; and as this scale was entirely too great for the wells observed, the arm was extended until the ratio was 1:2 and a reduction of one-half thereby obtained. These gages were found to be more sensitive and reliable than any others used. By means of the simple lever with its cone bearing, the friction in this instrument is reduced to a minimum; the pens respond to the slightest movement of the water, and for the faithful reproduction of small fluctuations this simple type of gage is to be highly recommended.

In the Mead gage the recording drum is vertical and the pen is carried by a carriage working between two upright guides. The wire supporting the carriage winds about a wheel connected with the wheel around which a wire from the float passes, and is lifted and lowered as the float descends and rises. The float and the wheel to which the pen is attached are so related in diameter that the curve traced is 10/42 of the true scale. There is with this gage, as with most gages where the recording cylinder is driven by the clock, some lost motion at the point of connection. This is particularly bad in this instrument. On the Long Beach records great care was used in setting the gage and the trouble was avoided, but some of the curves from well No. 8, at Douglaston, are clearly in error two to three hours.

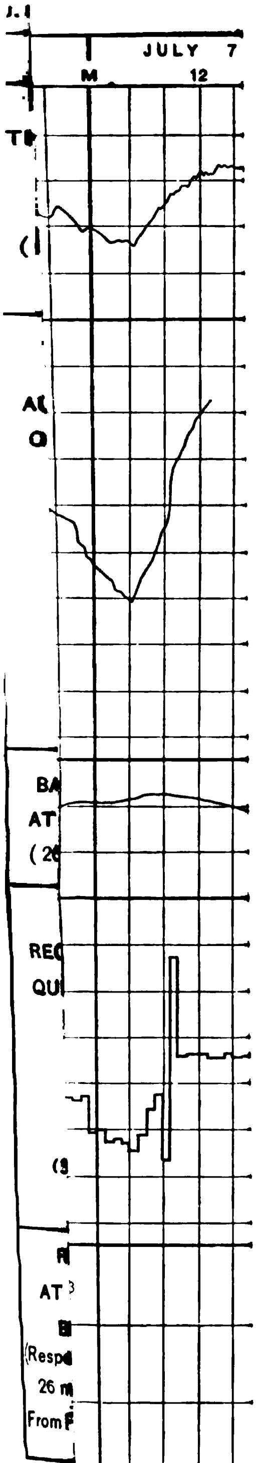
In the Friez gage[b] the recording drum is horizontal and is moved by the float, while the pen is moved by the clockwork. It was found that with the size of float that must be used in wells of small diameter the inertia of the drum in this instrument was such that it would not move until considerable head was developed and that small fluctuations were often not recorded. There was also a considerable amount of lost motion in the cogs used in the reducing device; and while an eccentric was provided for engaging the cogs closer, this could not be done without so increasing the friction that the instrument was useless. As a whole, this gage is not sufficiently sensitive for this kind of work, and the time element is entirely too small.

A water-stage register manufactured by a western house was also used, but the results obtained were not satisfactory because of the poor mechanical construction of the gage.

OBSERVATIONS ON WELL OF QUEENS COUNTY WATER COMPANY, 1 MILE WEST OF HEWLETT, N. Y.

Through the kindness of the chief engineer of the Queens County Water Company, Mr. Charles R. Bettes, an artesian well 181 feet deep and 3,300 feet south of the company's pumping station (Pl. II) was covered with a shelter for the protection of the gages and placed at the disposal of the Survey. This well, as is common with the wells of about the same depth sunk near the pumping station, passes through a layer of surface sand and gravel, then through beds of clay and other fine material

[a] Bull. U. S. Weather Bureau No. 5, 1892.
[b] Manufactured by Julian P. Friez, Baltimore, Md.



EWLETT, N. Y.

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A water-stage register manufactured by a western house was also used, but the results obtained were not satisfactory because of the poor mechanical construction of the gage.

OBSERVATIONS ON WELL OF QUEENS COUNTY WATER COMPANY, 1 MILE WEST OF HEWLETT, N. Y.

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[a] Bull. U. S. Weather Bureau No. 5, 1892.
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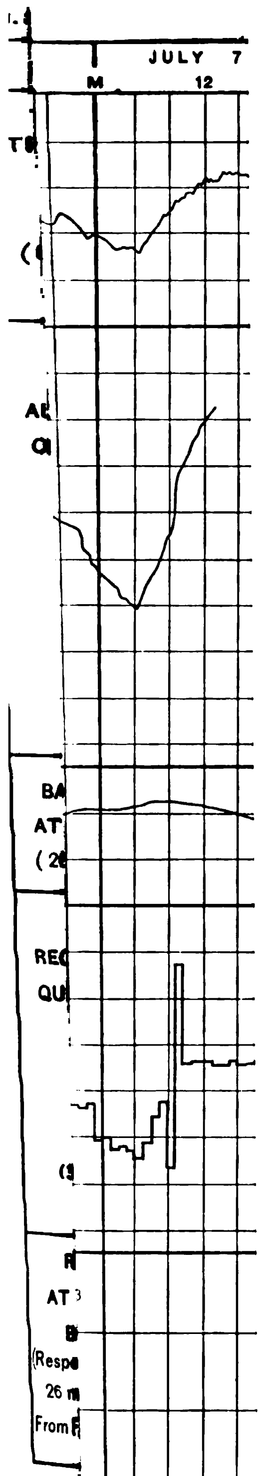

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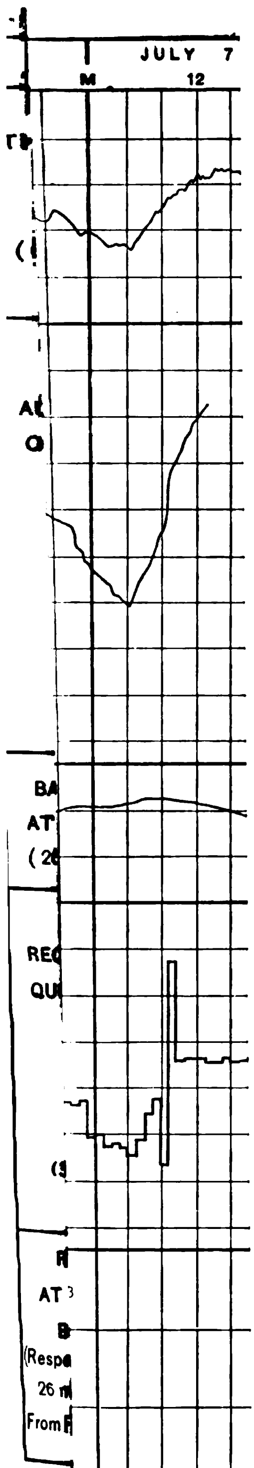

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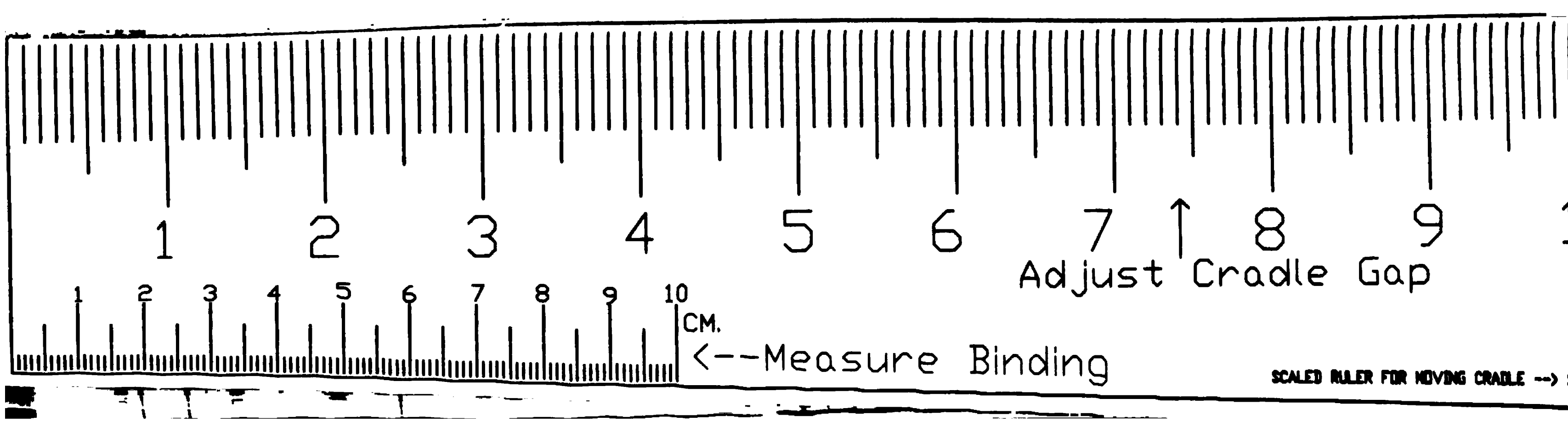
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into a rather coarse gravel, which yields an abundant supply of flowing water.[a] The whole section is of Pleistocene age. There are in the immediate vicinity of the pumping station thirty-two 5-inch wells, 33 feet deep, and nineteen 6-inch wells, 150 to 190 feet deep. These are arranged along two lines, one extending northwest and the other southwest from the pumping station. The extreme end of the south line is about 1,000 feet from the pumping station, and the well observed is therefore over 2,000 feet from the nearest pumped well and but slightly to one side of the probable direction of flow. (Pl. II.) It was the opinion of Mr. Bettes that this well was not affected by pumping, and through a misinterpretation of the first records it was believed that this surmise was correct. Considerable discussion was therefore caused when it was found that the well fluctuations simulated the thermograph curve in a remarkable manner, that these fluctuations were inversely related to the temperature (a rise in temperature causing a fall in water), and that the changes manifested themselves with a lag of but one or two hours behind apparently similar temperature fluctuations. (Pl. III.)

The hourly pumpage was kindly furnished by Chief Engineer Bettes, and this record, when plotted with the well curves, conclusively demonstrated that the rhythmical fluctuations were due more to pumpage than to temperature (Pl. III). Fluctuations of a somewhat similar character are produced by temperature changes (see p. 54), and this element is doubtless present in this curve. On Pl. III the effect of pumpage is clearly shown in the double cusps of the well curves on the night of July 5–6. The temperature curve shows no such variations. Similarly, the records on June 21, 25, and 26 show important differences between the well and temperature curves, which are largely due to pumping. These results are important because of the rapid rate of transmission. The effect of this pumping is felt at a distance of 2,000 feet or more, with a time lag of but one or two hours. This contrasts sharply with the very slow transmission noted in the pressure changes due to tidal loading and to the inflow and outflow along rivers (see pp. 60, 65). It conclusively proves that the supply here is large; that the beds are quite porous, and that the normal water flow is rapid.

In the records from the day gage, which was maintained here for the first ten days, the larger time scale brought out very clearly a series of regular minor fluctuations which were not clearly defined with the smaller scale of the week records. The most pronounced of this series recurs day after day and has a period of very nearly twenty minutes and a range of 0.06 to 0.08 inch.

Besides these vibrations, with a period of twenty minutes, there are several fluctuations of smaller amplitude and period. One series has a period of about five or six minutes, but it is so involved that little can be definitely stated regarding it. An instrument with a large time scale, 1 or 2 inches to the hour, and a vertical scale of once or twice the normal would record, at this place, a very complicated series of small recurrent vibrations.

OBSERVATIONS AT LONG BEACH, N. Y.

The deep flowing well of the Long Beach Association at Long Beach, N. Y. (Pl. II), offered a most excellent opportunity for the observations of tidal fluctuations. It is situated on a narrow, sandy barrier beach, separated from the main island by a sea marsh 2 to 3 miles wide, cut by narrow tidal channels, and is entirely removed from the influence of any pumping station.

This well is 3 inches in diameter and 386 feet deep. The water is obtained in sands of Cretaceous age and rises 2 to 4 feet above the surface of the ground or 10 to 12 feet above sea level. The general geologic relations may be inferred from the diagrammatic cross section given in fig. 1 (p. 9).

[a] For detailed record of strata in near-by wells see Prof. Paper U. S. Geol. Survey No. 44, 1906, p. 225. fig. 66.

The section reported by the driller, Mr. W. C. Jaegle, is as follows:

Section of well of Long Beach Association, at Long Beach, N. Y.

		Feet.
1.	White sand	0– 36
2.	Dark sand and creek mud	36– 40
3.	White gravel, containing salt water	40– 51
4.	White sand	51– 55
5.	Dark sand	55– 65
6.	White sand	65– 70
7.	White gravel	70– 73
8.	Yellow sand	73– 76
9.	Blue clay	76– 82
10.	Yellow gravel	82– 90
11.	Creek mud	90– 99
12.	Dark fine sand, containing lignite	99–101
13.	White sand	101–111
14.	Dark sand	111–119
15.	White sand, with lignite	119–121
16.	Blue clay	121–135
17.	Fine white sand	135–143
18.	Gravel, with salt water	143–145
19.	Dark sand	145–156
20.	Gravel, with salt water	156–158
21.	Clay	158–174
22.	White sand, containing at 190 feet a log of lignitized wood	174–192
23.	White gravel and salt water	192–196
24.	Clay	196–200
25.	Fine sand	200–220
26.	Solid blue clay	220–270
27.	White sand and wood, containing fresh water, sweet and chalybeate	270–276
28.	Clay	276–282
29.	White sand and wood	282–297
30.	Blue clay	297–305
31.	White sand, wood, and water	305–308
32.	Blue clay	308–317
33.	White sand, containing wood and artesian water	317–325
34.	Blue clay	325–340
35.	White sand and mineral water; has considerable CO_2, sparkling and effervescent	340–356
36.	Blue clay	356–360
37.	White sand and pure water	360–378
38.	Blue clay	378–380
39.	White sand	380–381
40.	White clay	381–383
41.	Fine sand, with artesian water	383–386

Mr. F. D. Rathbun was placed in charge of these observations and by a careful readjustment of the Mead gage obtained very excellent curves (Pl. IV). Indeed, for this character of work the results from the Mead gage, as set up by Mr. Rathbun, are better than from the Friez gage.

It was impossible to make tide observations at this point, and the values plotted on the curve are taken from those predicted by the Coast and Geodetic Survey[a] for East Rockaway Inlet, which is 2.8 miles west of the well. The difference in time

[a] U. S. Coast and Geodetic Survey Tide Tables for 1903, p. 346.

U. S. GEOLOGICAL SURVEY

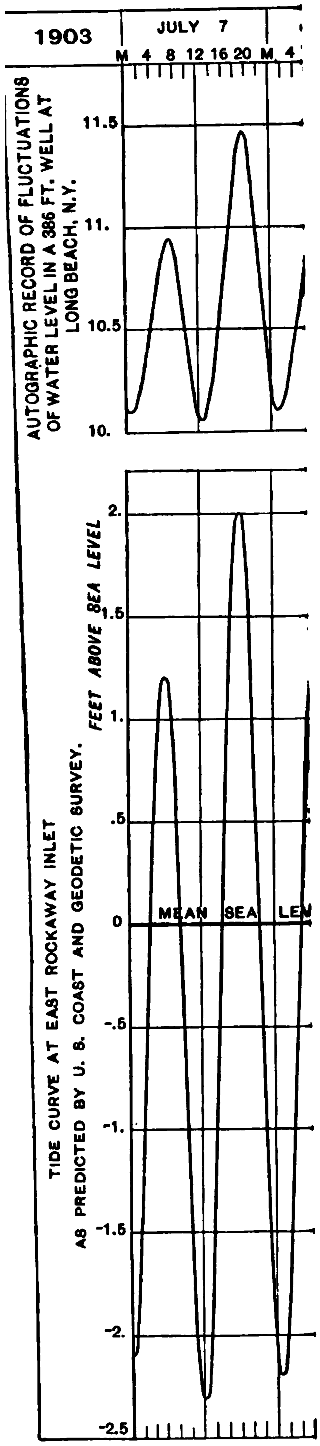

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Table showing difference in time between high and low water in a 386-foot well at L. Beach, N. Y., and tide at East Rockaway Inlet—Continued.

Date.	High water.			Low water.		
	Well.	Tide.	Difference.	Well.	Tide.	Difference.
1903.	*Time.*	*Time.*	*Hours and minutes.*	*Time.*	*Time.*	*Hours and minutes.*
July 13	11.50	9.44	2.06	4.50	3.44	[illegible]
	23.00	21.54	1.06	16.55	15.44	[illegible]
July 14	11.40	10.21	1.19	5.10	4.19	[illegible]
	23.45	22.28	1.17	17.10	16.21	[illegible]
July 15	....	....	....	5.55	4.55	[illegible]
	12.20	11.03	1.07	18.00	17.04	[illegible]
July 16	0.30	23.09	1.21	6.20	5.34	[illegible]
	....	....	....	19.20	17.55	[illegible]
July 17	1.10	23.56	1.14	7.20	6.27	[illegible]
Average	....	....	1.19	....	....	[illegible]

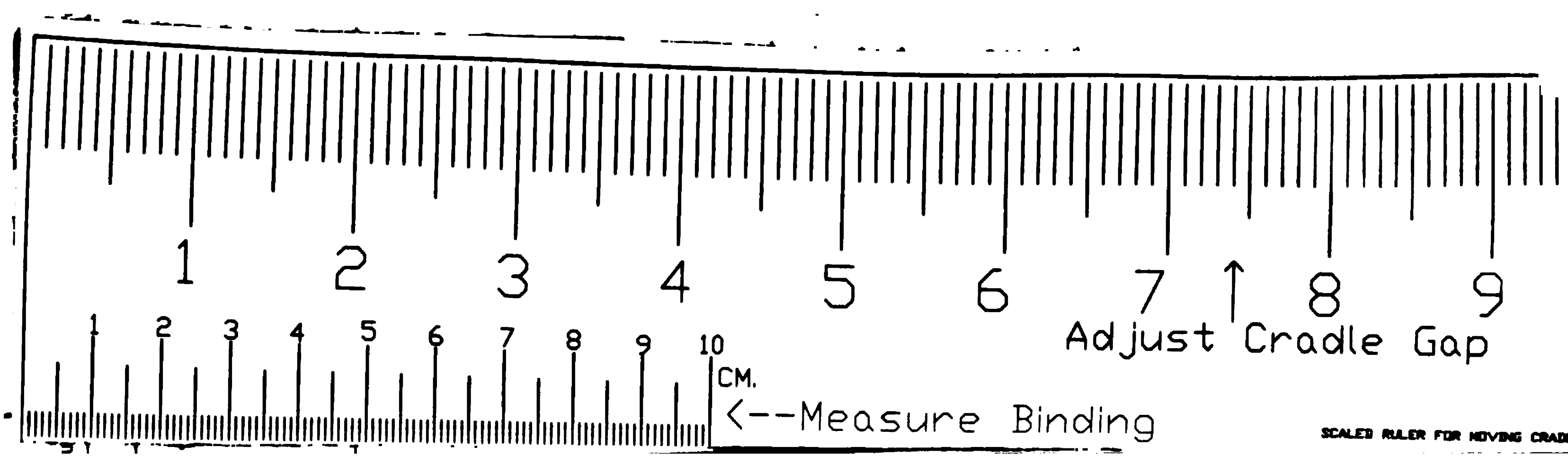
It will be noted that the lag at high water is greater than at low, which is just the reverse of what occurs in tidal rivers, where the water rises much more rapidly than it falls and the low-water lag is long and the high-water lag short. This is an important result bearing on the relation between the fluctuations of the water in wells and the oceanic tide and clearly refutes the doctrine that the tidal fluctuations here are due to leakage and that the fluctuations are analogous to those of tidal rivers. (See pp. 63–67.)

OBSERVATIONS ON THE SCHREIBER WELL NEAR MILLBURN, N. Y.

This well is located on the very edge of the sea marsh, about 2 miles south of Baldwin station on the Long Island Railroad (Pl. II). It is 8 inches in diameter and the total depth determined by sounding is 288.6 feet. The water will, when piped up, rise about a foot above the surface of the ground. After an unsuccessful attempt to record the fluctuations here with a Friez gage, which gave no results because of the small amplitude of the fluctuations, the King gage used on the Queens County Water Company well near Hewlett was set up and the record obtained from July 17 to August 5. This record shows the most erratic fluctuations obtained on Long Island (Pl. V).

In all the other records, while there are always many factors present, certain fluctuations can be definitely ascribed to temperature, atmospheric pressure, rainfall, pumping, or transmitted tides, but here either the curves produced by several of these factors have been so superposed that the character of each is thoroughly masked or new factors have been introduced. The most evident characteristic of these curves is the greater rapidity and abruptness in the fall of the water than in its rise. Abrupt drops of this character are known to be produced by changes in barometric pressure and by pumping. It will be noted in this case that these fluctuations are not represented in the barograph curve, and a comparison with the record from the 504-foot well at Lynbrook (Pl. VI), in which the geologic conditions are very similar, shows no correspondence, although the Lynbrook well is clearly greatly affected by barometric changes.

The nearest pumping stations are at Rockville Center and Freeport, and these are small village plants. At Rockville Center there were at this time four 8-inch wells, about 50 feet deep, and at Freeport 4 wells, about 35 feet deep. At Rockville Center about 150,000 gallons per day were pumped, and at Freeport about 100,000. These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stations are, respectively, 1.9 and 2.4 miles from the Schreiber well (Pl. II). The Rockville Center pumping station is, moreover, nearer the deep Lynbrook well than the Schreiber well, and the Lynbrook well shows no such fluctuations. It is therefore believed that these fluctuations are not due to pumping.

A third hypothesis is that the fluctuations are largely tidal, and that they represent the complicated stresses resulting from the culmination of the tides at different times in the neighboring network of creeks and channels. The conditions near this well are regarded as quite favorable for complex tides, but the resultant of these would be represented by a smooth curve, as is shown by the Long Beach well. The normal tidal curve in such narrow channels would, moreover, show a rapid rise and a gradual fall, while the well curve is just the reverse.

The fluctuations in this well are, so far as known, unique. The geologic conditions are believed to correspond in a general way with those in the 368-foot well at Long Beach and the 504-foot well at Lynbrook, both in the same region, but the characteristics of the curves are entirely different and apparently not related to either.

OBSERVATIONS AT LYNBROOK, N. Y.

A station was established one-half mile west of Lynbrook (Pl. II). At this point there were two test wells, one 504 feet deep and the other 72 feet, belonging to the Queens County Water Company. Through the kindness of Mr. Franklin B. Lord, president, and Mr. Charles R. Bettes, chief engineer, these wells were covered with a shelter, and a third well, 14 feet deep, driven about 6 feet from the other two. This gave a shallow surface well, a "deep well" (comparable to many of those used at the Brooklyn waterworks pumping stations west of this point) which flowed at the surface for about half the time, and a very deep artesian well, all within a few feet of each other and away from the zone of influence of any pumping station.

About 15 feet from the wells there is a small ground-water fed brook, the bottom of which has an elevation of about 10 feet above sea level; the ground at the wells is 11.3 feet above sea level, and the crest of the low swell, 1,000 feet to the west, about 20 feet (Pl. II, p. 16). The surface material is yellow loam, ranging from a few inches to 3 feet thick, then rather coarse sand and gravel. No record was preserved of the strata penetrated in the 72- and 504-foot wells, but a new well sunk during the summer of 1904, about 300 feet west of this group of wells, gave the following section:

Section of well of Queens County Water Company, one-half mile west of Lynbrook, N. Y.

	Feet.
Tisbury:	
1. Coarse yellow quartz sand; no erratic material	0–29
2. Light-gray sand	29–31
3. Same as No. 1	31–73
Cretaceous?:	
4. Light-gray silty clay	73–89
5. Light-yellow medium sand; no erratic material	89–150
Cretaceous:	
6. Fine to medium gray lignitic sand	150–158
7. Very fine black micaceous lignitiferous silt	158–200
8–9. Very fine dark-colored lignitiferous sand	200–228
10. Medium light-gray sand, with small amount of lignite	228–340
11. Dark-colored lignitiferous silty clay	340–363
12. Medium dirty-yellow sand, lignitic	363–403
13. Medium to coarse gray sand	403–536

The water in the 504-foot well, during the time of observation, stood from 0.8 foot to 2.2 feet above the surface; in the 72-foot well from 0.6 foot below to 0.5 foot above the surface, and in the 14-foot well from 0.6 foot below to 0.2 foot above the surface. The water in the 14-foot well rose above the surface three or four times for periods of a few hours. The elevation of the water table under the low swell to the west was probably about 13 feet.

Mr. Francis L. Whitney was placed in charge of the observations at this point, and King gages were installed on supports clamped to the well casings. The gages were maintained from June 25 to September 15, and some of the results are given in Pl. VI. These give a large amount of important material bearing not only on purely scientific problems, but on some of the live economic problems of the region. It will be noticed that as a whole the curves of these wells are parallel. This bears very directly on the old question of the source of the water in the deep wells on Long Island. It has long been a favorite hypothesis that in some mysterious way large quantities of water were introduced by great underground streams from the New England States, and this 504-foot well was one of the wells which were supposed to strike one of these streams. It has already been shown[a] that the source of the water in the deep wells on Long Island is from the rain that falls on the surface (see p. 10), and the really remarkable agreement of the general shape of these curves furnishes additional confirmation, pointing as it does to close interrelation and a common source.

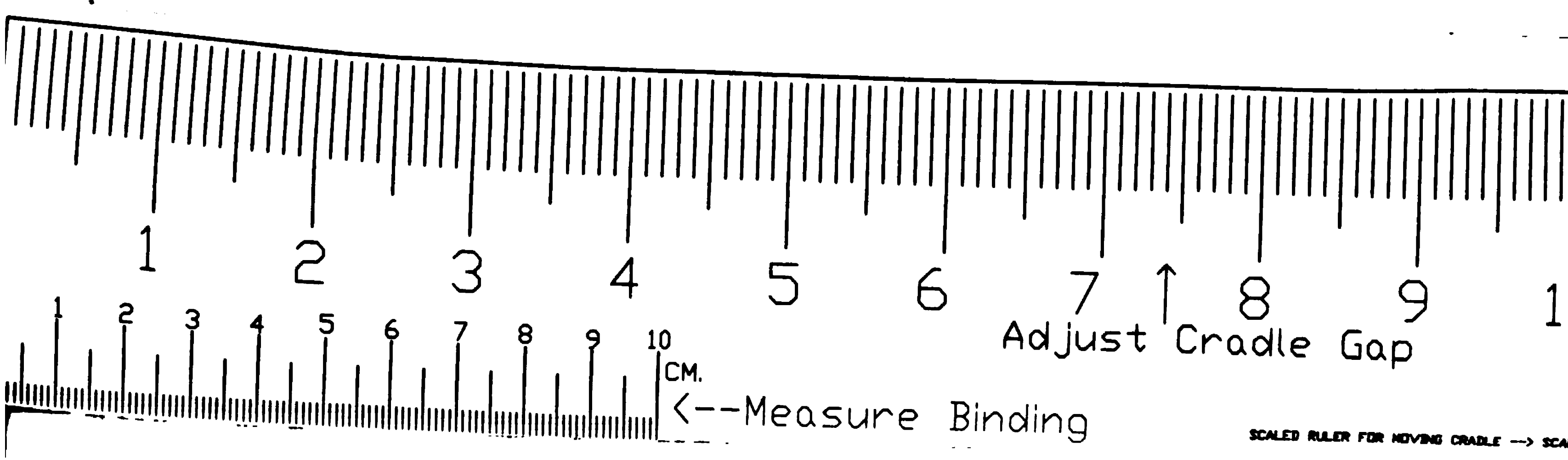
The behavior of these wells during rain storms shows that rain may affect the water level in wells in two ways, (1) without the water reaching the ground-water table, and (2) by actual infiltration and addition to the ground water. *In both* cases the effect is greatest in the shallow well. In the first case all wells commence to rise as soon as the rain begins, and rise abruptly, sometimes several inches. That this can not be due to actual infiltration is shown by its instantaneous character and by the fact that the water in the shallow 14-foot well, driven entirely in sand, rose above the surface of the ground four times under such circumstances. Such rises, moreover, produce no permanent deflection of the well curve. (See record for Aug. 18–22, Pl. VI.)

This sudden rise is due to a number of factors. In the first place, when the soil above the water table is filled with air the addition of water to the surface practically seals the outlets for the air and the weight of the rain is transmitted by this confined air to the water table. The effect of such a transmission is to hasten the discharge of the water at the ground-water outlet, and so produce an immediate rise in the streams. In this manner rains which never reach the ground-water table and which do not contribute directly to stream flow may immediately produce a greatly increased stream discharge. It should be noted in this connection that the well always rose before the adjacent brook, although the brook might later reach a higher elevation.

In the second case, when the water in the wells is elevated by the actual percolation of water, the water rises gradually and reaches its highest point several days or weeks after the rain, rather than in several minutes. In the case of the heavy rains which occurred on August 28, the 14-foot well reached its highest point before noon on the 29th, the 72-foot well at about 6 o'clock on the 29th, and the 504-foot well at noon on the 30th. There are three factors concerned in this last rise: (1) The instantaneous transmission of pressure due to weight of rain on the surface in the vicinity of the well; (2) actual percolation in the vicinity of the well, and (3) progressive deformations resulting from the weight of the rain at more distant points. The rise in the deeper wells is wholly due to the first and third causes. The curve in this case is actually displaced and returns to its former position only gradually, instead of at once, as in the case described above.

Barometric changes affect the 504-foot well most, but are occasionally perceptible in the 72-foot well. Temperature changes produce rhythmical daily fluctuations in the 14- and 72-foot wells; in the first the changes are very pronounced, amounting

[a] Prof. Paper U. S. Geol. Survey No. 44, 1906, pp. 67–69.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to as much as 1½ inches a day. The 504-foot well showed a regular fluctuation with two high and two low waters a day. The fluctuation, however, is not progressive, and so is not tidal.

The curve from the 504-foot well shows a great number of minor periodic oscillations, but the time scale is not sufficiently great to study them satisfactorily. The most pronounced of the series has a period of about forty minutes. The 72-foot well occasionally shows well-marked secondary oscillations, with a period of approximately eighty minutes. For a careful study of these, however, a much larger gage with a large time scale is demanded.

OBSERVATIONS AT DOUGLASTON, N. Y.

In the winter of 1902 and 1903 a number of shallow wells were sunk along the base of hills, east of Alley Creek, and near "The Alley," an old settlement just south of Douglaston, N. Y. (Pl. VII), for the Citizens' Water Supply Company. Six of these are flowing wells, and in the other two the water comes very near the surface. Through the kindness of Mr. Cord Meyer and his son, Mr. J. Edward Meyer, president and superintendent, respectively, of the Citizens' Water Supply Company, the flowing wells were piped up beyond the limit of flow and thus prepared for gaging.

The relative elevation, depth, and head in these wells are shown in the following table:

Elevations in wells of Citizens' Water Supply Company, at Douglaston, N. Y.

	Elevation of surface above sea level.	Depth of bottom of pipe below sea level.	Average height above sea level to which water will rise if piped up.
	Feet.	*Feet.*	*Feet.*
Alley Pond	17.2	....	17.2
Well No. 1	20.2	20.5	19
Well No. 2	10.2	25	9
Well No. 3	5	28	8.5
Well No. 4	5	25	18
Well No. 5	10.8	39	18
Well No. 6	10.1	35	19
Well No. 7	9.8	30	17
Well No. 8	10.5	17	15

The strata encountered vary considerably; some of the wells penetrate nothing but sand and gravel, and in others clay beds of greater or less thickness are found. The water is derived from the adjacent hill mass, the height of the ground water in which determines the head in these wells.

The tidal marsh to the west is a mass of soft black mud largely covered with a mat of growing vegetable matter, which is sufficiently firm to walk on, but which gives at every step. This surface mat of roots is often sufficiently tenacious to hold up when undermined by the small streams formed by the many springs that occur at the base of the hills, and these streams often flow through underground passages beneath the turf. The underlying mud or black ooze is over 10 feet thick in the upper end of the mud flat, and Mr. D. L. Van Nostrand states that, in driving piles for a dock at the bridge, the depth to "solid ground" was found to be 65 or 70 feet. The artesian pressure beneath this mud has caused the ground to rise in several places, with the resultant production of many small rapids (Pl. VII). At a number of points near the upper end of the basin, where the mud is thin, the water has broken through and produced low mud cones or mud volcanoes.[a] The drilling of the ar- sian wells and the fact that they were allowed to flow freely have perhaps in p relieved the pressure here, and during the three months the cones were obser they did not change materially, although on several occasions mud was seen risi from the craters and flowing down the sides. Three hundred feet north of the m flat and on the east bank of Alley Creek there is a lesser area of mud which is n covered at low tide. In this there is a marked mud flow, which is likewise proba connected with the artesian waters under discussion.

Mr. Francis L. Whitney was placed in charge of the work here, and he prepar wooden shelter boxes covered with tarred paper. These were securely clamped the top of the well pipes, which were steadied by means of guy lines.[b] A tide ga was established on the end of the crib of the drawbridge on the main turnpi (Pl. VII.). The crib furnished a very good still box, and the locality is as near th wells as it was possible to get, for to the south the creek bed is uncovered at low ti

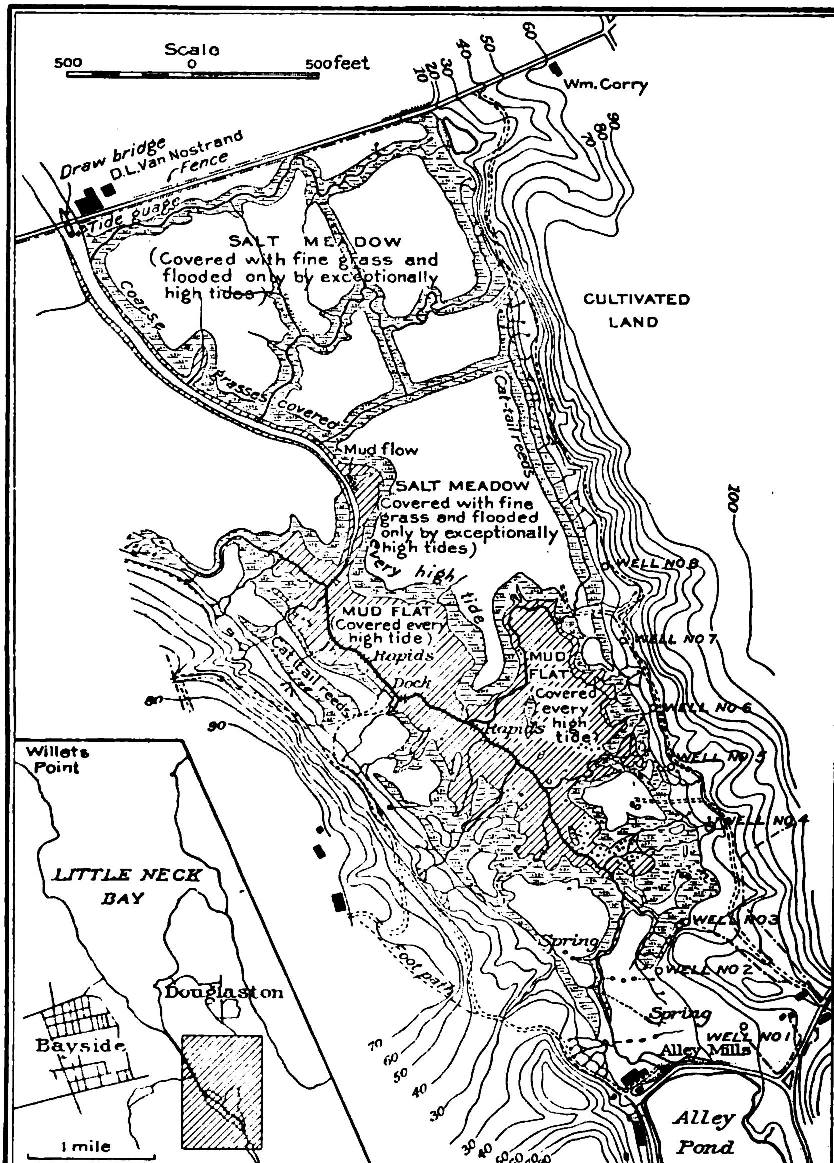
The equipment consisted of 3 Friez gages and 1 Mead gage. The Mead ga was placed on well No. 8 and furnished the only record running through th whole of the time of observation. One of the Friez gages was placed at the dra bridge during the whole period, but from one cause and another no record w obtained before August 6, and after that time the record was not complete By shifting the remaining gages records were obtained for a time from all th wells but No. 3. Some of the curves obtained from these observations are shown i Pl. VIII.

All these wells are clearly tidal, but when the question of the rate of propagati of tidal effect is considered many difficulties are encountered and the extreme co plexity of the problem at once becomes evident. The curves, while broadly resem bling each other, show many minor points of difference, which must be attribute to the varying shape of the tidal wave in the mud flat near the wells and the conse quent complexity and variation of the stresses involved. Thus the records fro wells 2 and 8 show that, while the relative amplitudes of the high tides agre perfectly and both show a tendency toward a double cusp at high tide, in well No. the second cusp is characteristically greater, while in well No. 8 the first cusp is often the greater; compare curves from July 27 to 29. The low-tide curves also show marked differences; thus, in well No. 2 there is a continued fall until the tide turns, which it does sharply; in No. 8 there is a long period of stagnation and the curve is rounded when the rise begins. Evidently these curves are not readily comparable with the tide gage at the bridge nor with each other, for each represents the resultant of a different set of forces. The conditions for the production of such differences are very favorable. The semiliquid marsh mud yields readily to all pressure changes, however slight; the liquidity of the mud varies greatly from point to point, and while the artesian water does not commonly escape through the mud covering it may, as shown by the mud cones, do so at any time, and such a point of relief would affect adjacent wells differently.

Another factor making exact time comparisons difficult is the small scale of the records and the great amount of lost motion in the Mead gage. The Mead records show unquestionable time errors of one to two hours, and for this reason the end values are more important than the initial ones of each record. Where evident errors occur in the record of the Mead gage for well No. 8 they have been corrected as far as possible, and an attempt has been made to indicate on the diagram the various details affecting the time values so far as known. For this purpose the end of each of the original record sheets has been indicated on Pl. VIII.

[a] See Prof. Paper U. S. Geol. Survey No. 46, 1906, Pl. XXVII, *C*.

[b] An illustration of the gage box on well No. 4 will be found in Prof. Paper U. S. Geol. Survey No. 44, 1906, Pl. XIV.



SKETCH MAP SHOWING LOCATION AND TOPOGRAPHIC SURROUNDINGS OF WELLS OF CITIZENS' WATER SUPPLY COMPANY NEAR DOUGLASTON, N. Y.

Black dots indicate location of mud volcanoes.
By A. C. Veatch, 1903.

OBSERVATIONS OF THE NEW YORK CITY COMMISSION ON ADDITIONAL WATER SUPPLY.[a]

The Long Island division of the commission on additional water supply under the direction of Mr. W. E. Spear, division engineer, during the period from the middle of April to the last of October, 1903, made many observations on the water level in wells on Long Island. In all about 1,200 wells were observed at intervals of from one to three days by means of steel tapes fitted with cup sounders. From these observations Mr. Spear endeavored to obtain the velocity of the downward capillary flow of the water on Long Island.

Meteorological stations, equipped with self-recording instruments, were established at Brentwood and Floral Park (Pl. I, p. 9). It was from these records that the thermograph, barograph, and rainfall curves shown on Pls. III, V, and VI were obtained.

Mr. Spear likewise obtained from the records of the Brooklyn waterworks data regarding the fluctuations of the water in shallow wells on the south shore and the effect of pumping at Merrick and Agawam.

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[a] Spear, Walter E., Long Island sources: Rept. Commission on Additional Water Supply for the City of New York, Nov. 30, 1903, New York, 1904, appendix 7, pp. 617-806

PART II.

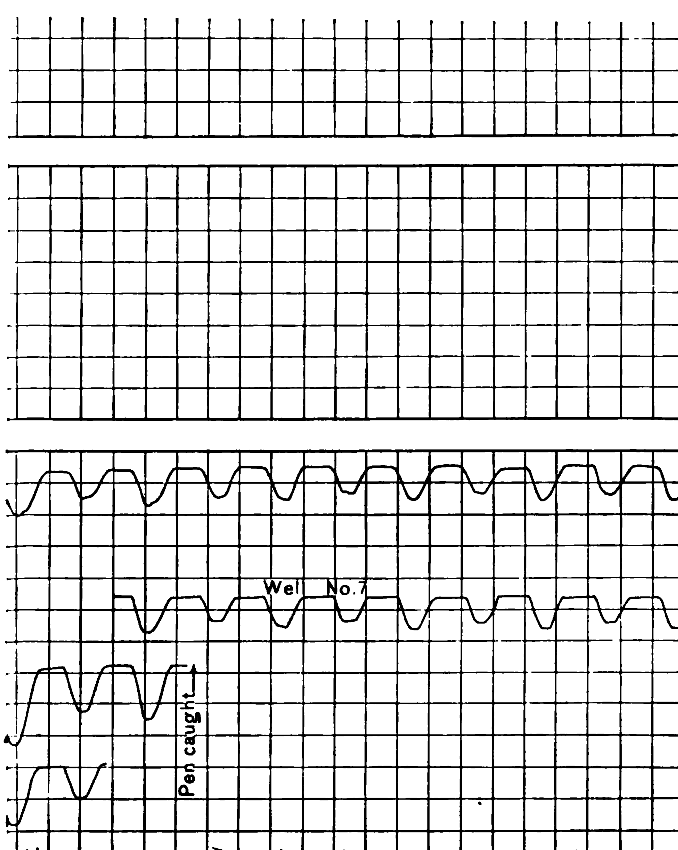
GENERAL DISCUSSION OF THE FLUCTUATIONS OF WATER LEVEL IN WELLS.

CLASSIFICATION OF CAUSES.

The vertical fluctuations of the ground-water table or the changes in the level of the water in wells may be grouped as follows:

A. Fluctuations due to natural causes.
  I. Rainfall and evaporation.
    1. Fluctuations not depending on single showers.
      a. Regular annual fluctuations.
      b. Irregular secular changes.
    2. Fluctuations produced by single showers.
      a. By transmission of pressure without any actual addition to the ground water.
      b. By the actual addition of rain to the ground water.
  II. Barometric changes.
  III. Thermometric changes.
    1. Fluctuation directly related to temperature.
    2. Fluctuation inversely related to temperature.
      a. At the surface of the ground-water table, directly through temperature changes.
      b. In deeper zones, by pressure changes produced by fluctuations of the preceding class.
  IV. Fluctuations produced by adjacent bodies of surface water: Rivers, lakes, the ocean.
    1. By changes in rate of ground-water discharge.
    2. By seepage.
    3. By plastic deformation due to varying loads.
  V. Fluctuations due to geologic changes.
B. Fluctuations due to human agencies.
  1. Settlement, deforestation, cultivation, drainage.
  2. Irrigation.
  3. Dams.
  4. Underground water-supply developments.
  5. Unequal loading.
C. Fluctuations due to indeterminate causes.

The relation between the fluctuations due to natural causes may be stated in this way: On the broad and irregular curves produced by the secular climatic and geologic changes are superposed the regular annual fluctuations, which are perhaps the most characteristic and important of the ground-water fluctuations due to natural causes; and on these, in turn, are superposed the simple rainfall, barometric, thermometric, tidal, and flood fluctuations. This complex curve, made up of many regular and irregular elements, is further modified by human agencies. The cumulative effect of these human agencies is irregular and the result is to modify—indeed, often to largely alter—the character of the broad irregular curves produced by secular climatic and geologic changes. Yet some of these human modifications have a periodic value which, in the case of cultivation, for example, may greatly change the amplitude of the annual fluctuations, or, in the case of pumping or the change of water level behind a milldam, may give rise to rather regular daily fluctuations.



F TIDE IN ADJOINING CREEK.

ges. At point marked * on well curve No. 8 lost motion is evident
original record sheets.

FLUCTUATIONS PRODUCED BY NATURAL CAUSES.

RAINFALL AND EVAPORATION.

REGULAR ANNUAL FLUCTUATIONS.

GENERAL CHARACTER AND CAUSE.

Woldřich, from a study of nine years' observations on a 19-foot well at Salzburg, concluded that the rise and fall of the ground water stands in no relation whatever to the

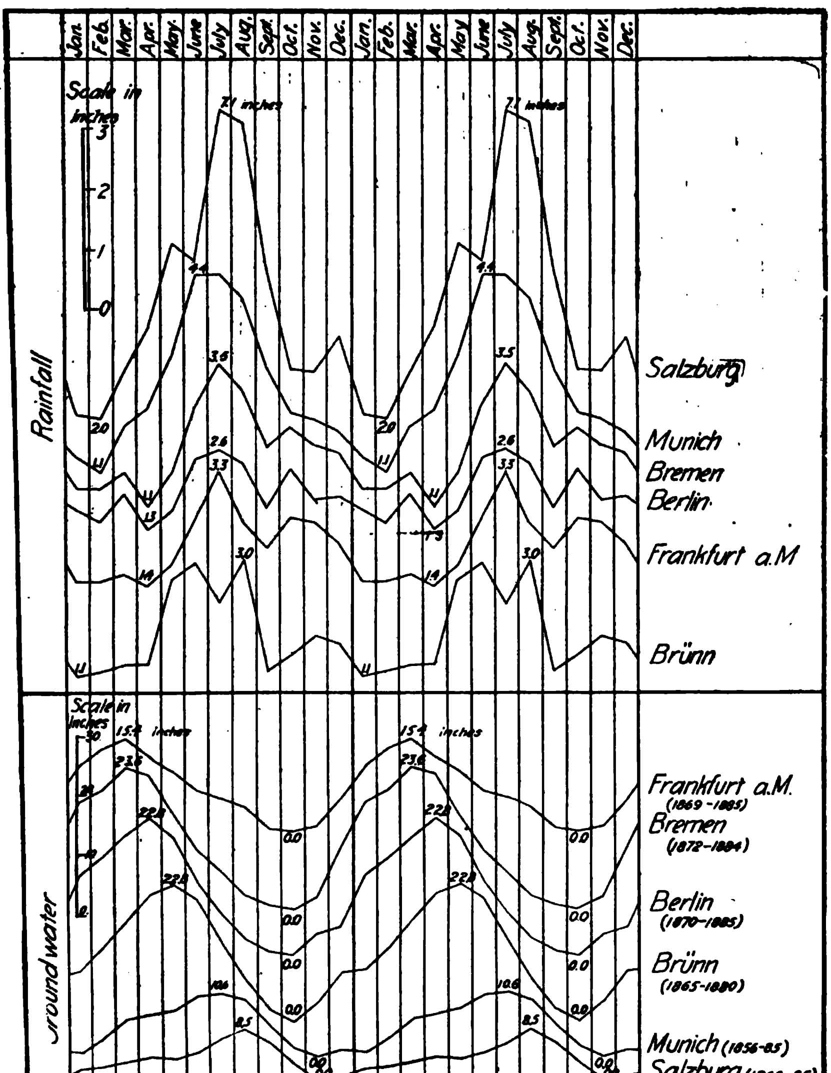


FIG. 9.—Yearly rainfall and water-level curves in shallow wells in middle Europe (after Soyka). The curves are duplicated for a second year to facilitate comparisons.

amount of rain, since with the same quantity of precipitation it at times rises, and again falls, and even with considerably increasing quantities of rain it often falls constantly.[a]

[a] Woldřich, Johann Nepomuk, Mitt. d. Techn. Klubs zu Salzburg, 1869, Heft 1, Zeitschrift d. Österreichischen Gesellschaft für Meteorologie, Bd. 4, 1869, pp. 273-279 Penck's Geographische Abhandlungen, Bd. 2, Heft 3, Wien, 1888, p. 23.

While so broad a statement is not entirely true for all localities in the North T-perate Zone, yet it properly emphasizes the fact that the relation between t ground-water fluctuations and the rainfall is not the simple one which might inferred from the statement that the rainfall is the source of the ground wa. Observations at many points in the North Temperate Zone have shown that

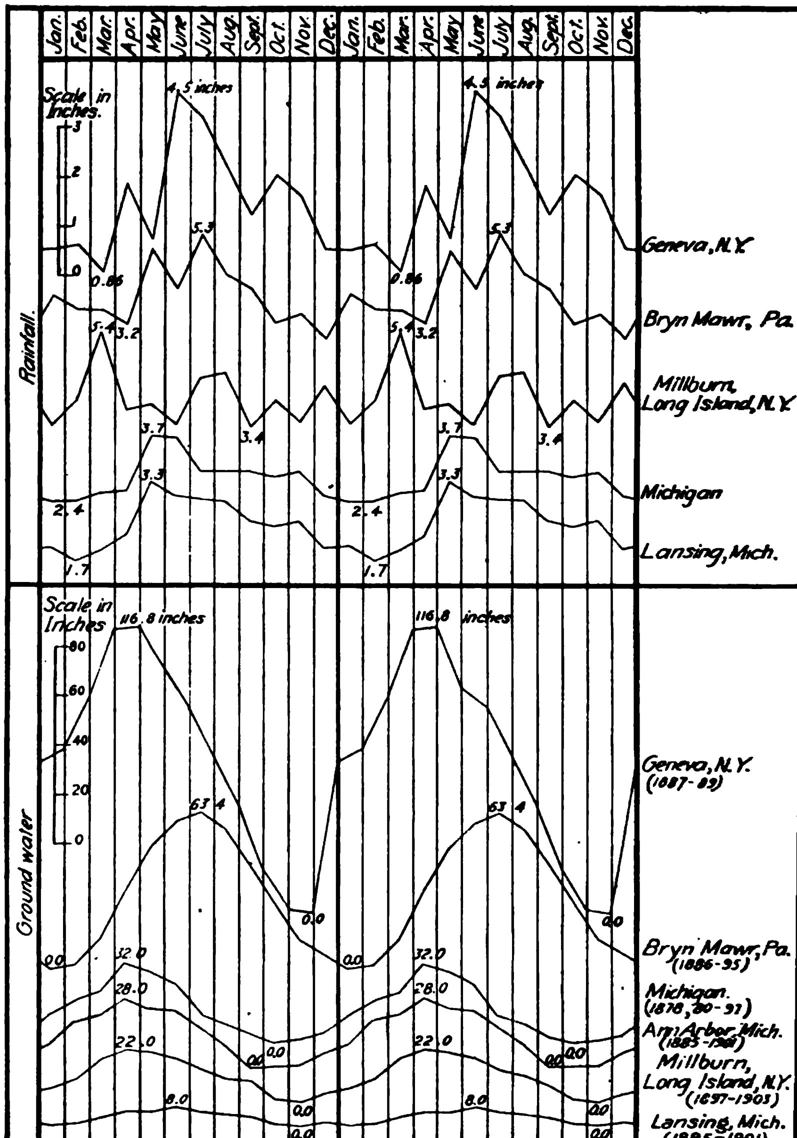


FIG. 10.—Yearly rainfall and water-level curves in shallow wells in the United States. The curves are duplicated for a second year to facilitate comparisons.

ground water fluctuates in a yearly period with a single maximum and minimum, and that this curve generally does not correspond with the rainfall curve (figs. 9, 10). Indeed, at Frankfurt, Bremen, Berlin, and Brünn, the highest point of the ground water is in the spring months at the time of least rainfall (fig. 9). The yearly curves of the ground water are much more regular than the rainfall curve, and on the whole in general shape they most resemble the annual temperature curve (fig. 11). The reason for this difference is the simple one that the fluctuations of the ground water depend not only on the absolute amount of the rainfall, but on the quantity that reaches the zone of complete saturation, or the ground-water table, and the time consumed in so doing. The quantity is affected by many factors, among which are the evaporation from the surface of the ground, the evaporation or transpiration from plants, the quantity retained in the soil above the zone of saturation, and the amount that runs directly off the surface without ever penetrating the ground. The time element is affected by the porosity and moisture content of the soil, the character of the covering, and to a greater or less extent by the height of the soil column. The general result is that the water is delivered gradually to the zone of complete saturation, and as the effects of single rains are thus generally

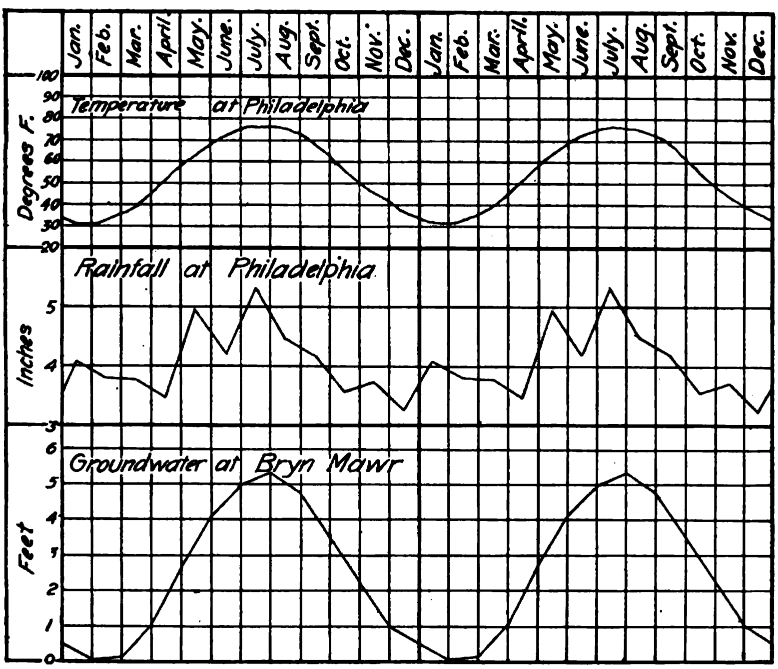


FIG. 11.—Annual curves based upon mean monthly averages of ground-water level at Bryn Mawr, Pa.,[a] and temperature and rainfall at Philadelphia, Pa.,[b] for the period 1886–1895, showing the general resemblance of the ground-water and temperature curves. The exact agreement of the maximum ground-water level and the maximum temperature is unusual.

minified and often entirely blotted out, a relatively smooth curve results (Pl. IX). This yearly period of the ground water is largely due to the periodic character of the evaporation, including plant transpiration. This depends on the temperature, and the net result for the year is a simple curve of the same shape as the mean temperature curve, although inversely related to it, hence the general resemblance of the yearly well curve to the temperature curve. Were the rain uniform throughout the year, and were there no lag due to transmission or unmelted snow, the maximum ground-water level would occur at the time of the minimum temperature and saturation deficit of the atmosphere, or, in the North Temperate Zone, in January. The effect of the irregular distribution of the rainfall is to change the time of the occurrence of this maximum. A moderate excess of rain in the summer, such as occurs

[a] Observations of W. S. Auchincloss; Waters within the Earth and Laws of Rainflow, 1897.
[b] Observations of U. S. Weather Bureau; Annual Summary Pennsylvania Climate and Crop Service.

at Frankfurt-am-Main, causes the maximum to advance to March, while at Bremen, Berlin, and Brünn, where the difference between the summer and winter rainfall is progressively greater, in the order named, the maxima occur respectively in March, April, and May (fig. 9). The extremely great summer precipitation at Munich and Salzburg, together with the low rainfall in January, causes the maximum at these places to advance to July and August.

In this connection the observations of (1) Dickinson and Evans, (2) Greaves, and (3) Lawes and Gilbert, near London, are of great interest. All of these observers endeavored to determine the amount of rain actually contributed to the ground

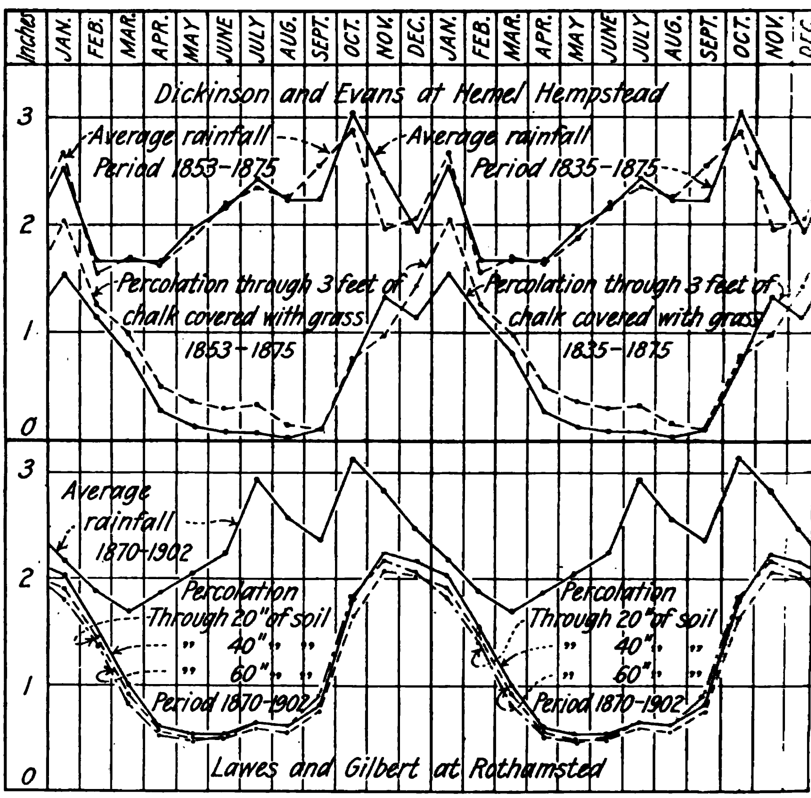
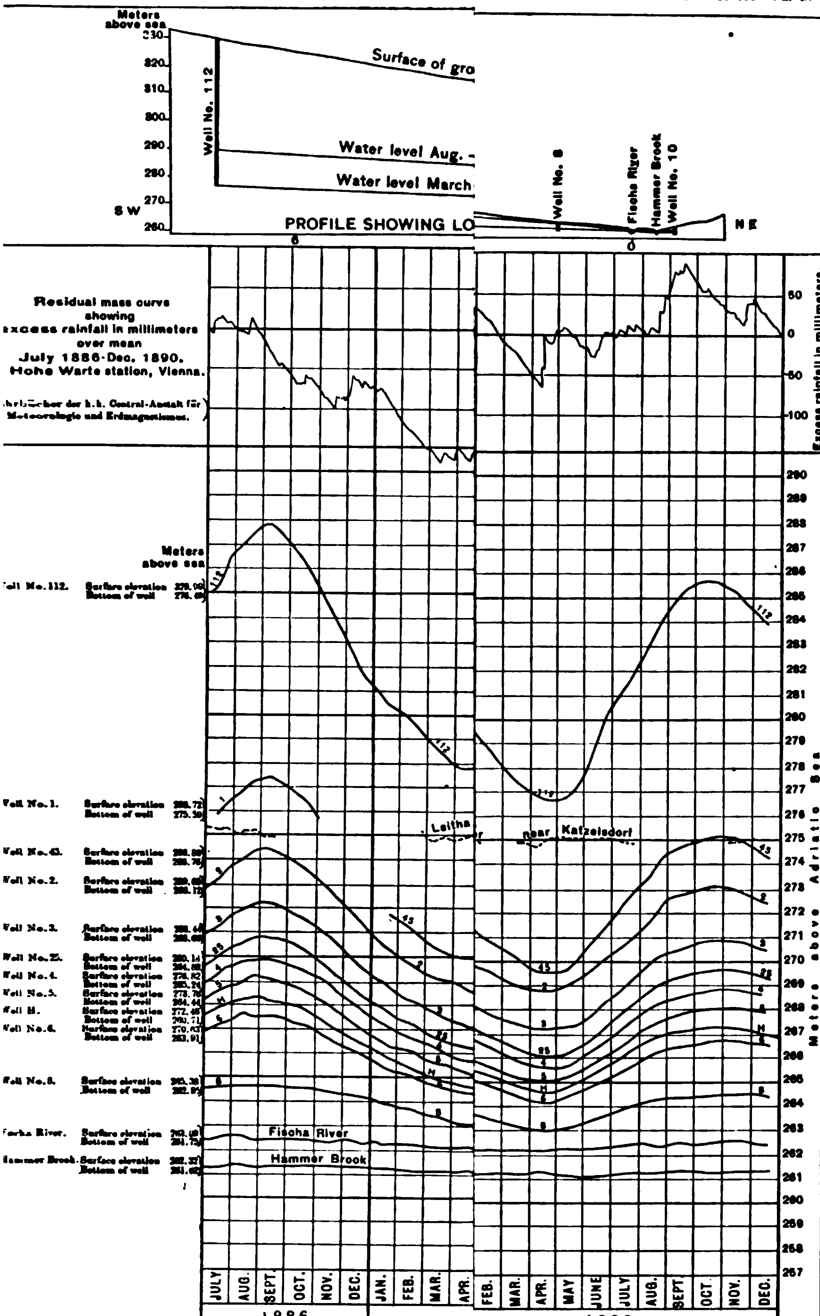


FIG. 12.—Results of English percolation experiments. In the Dickinson and Evans experiments the gages were buried in the ground; one was filled with ordinary Hertfordshire soil (a sandy, gravelly loam) and covered with sod; the other was filled with chalk and covered with a thin layer of soil and sod. In the Lawes and Gilbert observations columns of a rather heavy loam with clay subsoil in their natural state of consolidation were built in with brick and cement; no vegetation was allowed to grow on the gages, which were surrounded by meadow land. Curves are based on the monthly averages from September 1, 1870, to August 31, 1902.

water. Each used vessels with impervious sides and pervious bottoms, sunk level with the surface of the ground. The water percolating through the soil columns was collected and compared with the yield of the adjacent rain gages. In the case of the Dickinson and Evans and the Greaves experiments the boxes were filled with material supposed to represent the average soil of the region, in both cases a sandy loam. In the Lawes and Gilbert experiments actual blocks of soil were undermined and the results represent the amount of rainfall passing through a heavy loam with a clay subsoil in its natural condition of consolidation, but not covered with vegetation. The average results obtained are given in the following table and are partially shown in a graphic manner in fig. 12.



This illustrates regular annual fluctuations, irregular secular ... ws that the well fluctuations are generally independent of single rains. It also shows concentric ... ime of occurrence of the yearly maximum and minimum. (From Bericht des Ausschusses für di...

English experiments showing average amount of rain passing through soil during the different months of the year.

	Dickinson and Evans.[a]						Greaves.[b]					Lawes and Gilbert.[c]						
	1836–1875.			1853–1875.									Percolation through column of heavy loam with clay subsoil.			Percentage of rain percolating through—		
	Rainfall (average).	Percolation through 3 feet sandy loam covered with grass.	Percentage of percolation to rain.	Rainfall (average).	Percolation through 3 feet chalk covered with grass.	Percentage of percolation to rain.	Rainfall.	Percolation through 3 feet sandy loam covered with grass.	Percolation through 3 feet bare sand.	Percentage of rain percolating through soil.	Percentage of rain percolating through sand.	Rainfall (average 1870–1902).	20 inches deep.	40 inches deep.	60 inches deep.	20 inches.	40 inches.	60 inches.
	Inches.	*Inches.*	*Per ct.*	*Inches.*	*Inches.*	*Per ct.*	*Inches.*	*Inches.*	*Inches.*	*Per ct.*	*Per ct.*	*Inches.*	*Inches.*	*Inches.*	*Inches.*	*Per ct.*	*Per ct.*	*Per ct.*
January	2.54	1.55	61.0	2.68	2.05	76.5	2.000	1.250	1.875	62.5	93.8	2.17	1.80	2.02	1.90	82.9	93.0	87.6
February	1.67	1.14	68.3	1.55	1.26	81.3	1.375	.750	1.250	52.7	90.9	1.87	1.40	1.55	1.45	74.9	82.9	77.5
March	1.68	.79	47.0	1.70	.99	58.2	1.750	.750	1.625	42.8	92.8	1.69	.81	1.00	.92	47.9	59.1	54.4
April	1.66	.24	14.4	1.63	.49	30.0	1.500	.250	1.250	16.7	83.3	1.86	.52	.60	.56	27.9	32.2	30.1
May	1.97	.11	5.6	1.88	.34	18.0	2.250	.125	1.875	5.5	83.3	2.05	.48	.54	.49	23.4	26.3	23.9
June	2.15	.07	3.3	2.18	.29	13.3	2.125	.125	1.625	5.8	76.5	2.23	.51	.54	.50	22.9	24.4	22.4
July	2.43	.06	2.5	2.34	.31	13.2	2.125	.125	1.125	5.9	52.9	2.98	.65	.65	.60	22.2	22.2	20.5
August	2.23	.01	0.4	2.24	.13	5.8	2.625	.125	2.000	4.8	76.0	2.56	.61	.61	.56	23.8	23.8	21.9
September	2.22	.09	4.0	2.57	.09	3.5	2.125	.250	1.625	11.8	76.5	2.35	.90	.81	.77	38.3	34.5	32.8
October	3.05	.73	23.9	2.88	.74	25.7	3.000	.750	2.375	25.0	79.1	3.12	1.81	1.79	1.61	58.0	57.4	51.6
November	2.46	1.33	54.0	1.95	.98	50.2	2.125	1.250	2.000	58.8	94.0	2.81	2.17	2.24	2.07	77.2	79.7	73.7
December	1.92	1.13	58.0	2.06	1.43	69.4	2.000	1.250	1.375	62.5	68.8	2.46	2.05	2.17	2.03	83.3	88.2	82.5
Year	25.98	7.25	27.9	25.66	9.10	35.4	25.000	7.000	20.000	28.0	80.0	28.10	13.71	14.52	13.46	48.8	51.7	47.9

[a] Min. Proc. Inst. Civil Eng., vol. 2, 1843, pp. 160–161; vol. 9, 1850, p. 158; vol. 20, 1861, pp. 219–222; vol. 45, 1876, pp. 208–216.
[b] Artificial values based on twenty-two years' observation on soil and fourteen years on sand: Min. Proc. Inst. Civil Eng., vol. 45, session 1875–76, pt. 3, 1876, p. 28.
[c] Min. Proc. Inst. Civil Eng., vol. 45, 1876, pp. 59–60; vol. 105, 1891, pp. 35–46; The Rothamsted Experiments: Results and Summary Tables, 1903, p. 1.

Of these the Lawes and Gilbert results are perhaps of the greatest value, because they more nearly represent the normal conditions, and they extend through a sufficiently long period to obliterate temporary variations. While the quantitative values obtained from these experiments differ, the qualitative results, as shown by fig. 12, are essentially the same. All except the bare sand give curves of the same character as those obtained from the actual observation of ground-water fluctuations. The low percentage of water which passes through the soil in summer is emphasized by all, as is also the greater contribution during the winter months. Even in the bare sand, where the water sinks at once and so loses little by evaporation, a downward tendency of the curve during the summer months is evident. The effect of the heavy precipitation in the fall is particularly evident in the Lawes and Gilbert results (fig. 12), where it clearly hastened the time of occurrence of the maximum ground-water percolation by about two months.

EFFECT OF DEPTH OF SOIL ABOVE ZONE OF COMPLETE SATURATION ON TIME OF OCCURRENCE OF YEARLY MAXIMUM AND MINIMUM OF GROUND-WATER LEVEL.

It has been suggested that the soil above the ground-water table tends to destroy the effect of single rains by causing the water to be delivered gradually to the zone of complete saturation, whose upper surface is the water table. In the case of the English experiments at Rothamsted (Lawes and Gilbert) and Hemel Hempstead (Dickinson and Evans) the maximum percolation occurs from November to January, yet the water in the wells in that region, while commencing to rise about December, did not reach its maximum elevation until March,[a] a delay of about three months. Yet in any attempt to calculate the rate of percolation from these data two difficulties are encountered. In the first place the yearly maximum occurred at about the same time in this region in wells of all depths, and, furthermore, the Rothamsted results (fig. 12) show no very important difference in the time in which the water is discharged through 20, 40, and 60 inches of soil so far as it relates to the time of occurrence of the yearly maximum. In the second place the underground water is in motion, a certain amount is discharged at all times, and the amount increases with the head. The case is not, therefore, the simple one where water is caught in a measuring tube, as in the percolation experiments above described, but the water must reach the ground-water table at a rate greater than the rate of the outflow, else no rise will take place.

At Wiener Neustadt, Austria, a similar relation has been demonstrated by the observations made between 1883 and 1895, in connection with the Wiener Neustadt deep-well project for the supply of Vienna.[b] These wells are in a valley filling of fluvio-glacial material, somewhat irregular in character, which Suess describes as a series of old deltas.[c] The wells extend from Fisha River, a spring-fed stream, southwesterly along the Southern Railway. The land and the ground-water table beneath both rise gradually in this direction, but while the water table is at the surface of the ground at Fisha River, 6½ miles south, at St. Agyden, it is from 140 to 170 feet from the surface, the exact depth depending on the time of year. (See section at top of Pl. IX.) The curves obtained from this series of wells, extending roughly at right angles to the slope of the water plane, are entirely concentric, and the maximum and minimum occur at the same time, irrespective of depth of soil above the ground-water table. It would seem to follow from these data that no very satisfactory determination of the rate of downward percolation can be made from the relation of the time of greatest precipitation, or percolation, to the time of maximum

[a] Clutterbuck, James, Min. Proc. Inst. Civil Eng., vol. 2, 1842, p. 156.

[b] Bericht des Ausschusses für die Wasserversorgung Wiens: Österreichischer Ingenieur- und Architekten-Verein, 1895.

[c] Ibid., p. 32; see also Bericht über die Erfolge der Wiener Wasserleitungs-Commission, 1864; Karrer, F., Geologie der Franz Josephs-Quellenleitung; Abhandlungen der K.-k. geologischen Reichsanstalt, 1877.

ground-water level. The rise of the water is not determined by the simple delivery of water to the zone of complete saturation, but by the relation of the water so delivered to the rate of outflow. If the water is lowering, a certain amount is consumed in checking that tendency, and only the excess over the outflow is available for raising the ground-water level. Moreover, in several carefully observed instances, the depth of the soil above the ground water has been shown to have no effect on the time of occurrence of the yearly maximum.

The short-period observations on Long Island, New York, during the summer of 1903, however, gave quite different results from those obtained at Wiener Neustadt. The conditions do not appear to be essentially different; the glacial sands and gravels of the south plain of Long Island slope gradually to the ocean and in a similar way the valley glacial gravels of Wiener Neustadt slope to Fisha River, and there is apparently no great difference in the irregularity and complexity of the bedding. The Wiener Neustadt or Steinfeld Valley, it is true, is traversed by a large river, the Leitha, whose stages depend on the conditions affecting its headwaters in the mountains, but observations have shown that this stream, because of the silted character of its bed, affects only a few wells in its immediate vicinity, and is not to be regarded as a disturbing factor. (Compare the river stages with well curves on Pl. IX.) On Long Island the measurements in charge of Mr. Walter E. Spear, department engineer of the commission on additional water supply,[a] showed that, during the summer of 1903, the highest stage of the ground water occurred, as a rule, earlier in the shallow than in the deeper wells (fig. 13). Where the water level was less than 20 feet from the surface the highest stage of the ground water occurred in April, while where the water level was 60 to 75 feet below the surface it did not occur until August. The increase of the retardation was not always uniform. Thus the highest water in a 35-foot well near Jamaica (No. 551) occurred in May, while in a well of the same depth near Deer Park (No. 388) it did not occur until August, although in a well near Hicksville (No. 237), of about the same depth, the maximum occurred in May. Whether this irregularity is typical or is only a result of the rather peculiar season in which the measurements were made could be determined only by observations covering a period of years, instead of months. It should, however, be stated in this connection that along the south shore, where the Brooklyn water department has observed shallow wells for several years, the curve for 1903 is not greatly different from that of preceding years, indicating, so far as the shallow wells are concerned, that the year is not to be regarded as an abnormal one (fig. 15, p. 39). On the other hand, the results are so at variance with the thirteen years' observations at Wiener Neustadt, which apparently cover similar conditions, that further confirmation of these Long Island results, by additional observations, is needed before any conclusions can be drawn. Certainly the Wiener Neustadt data indicate that the depth of the soil above the ground-water table is of no importance in determining the time of occurrence of the maximum ground-water level. On the other hand, the Long Island observations suggest that a difference in thickness of 60 feet may delay the time of the occurrence of the maximum level four months.

The curves showing the result of the Long Island work indicate further that, in the soil in question, single showers frequently produce very definite effects in shallow wells, and that such effects become less as the depth of unsaturated material above the water table increases. Indeed, in the wells where the water is 30 or 40 feet below the ground, the curves are relatively smooth or the variations bear no evident relation to the rainfall.[b] Spear has attempted to trace the time of rise, due to given showers, from the shallow through the deeper wells, and so determine the rate of

[a] Long Island sources: Rept. New York City Commission on Additional Water Supply, 1904, appendix 7, Pl. IV, incorrectly numbered Pl. VI, p. 792.
[b] Many of the wells observed by the commission were open, dug wells, which were in use, and the minor fluctuations are partially due to this cause, as well as to barometric and thermometric changes,

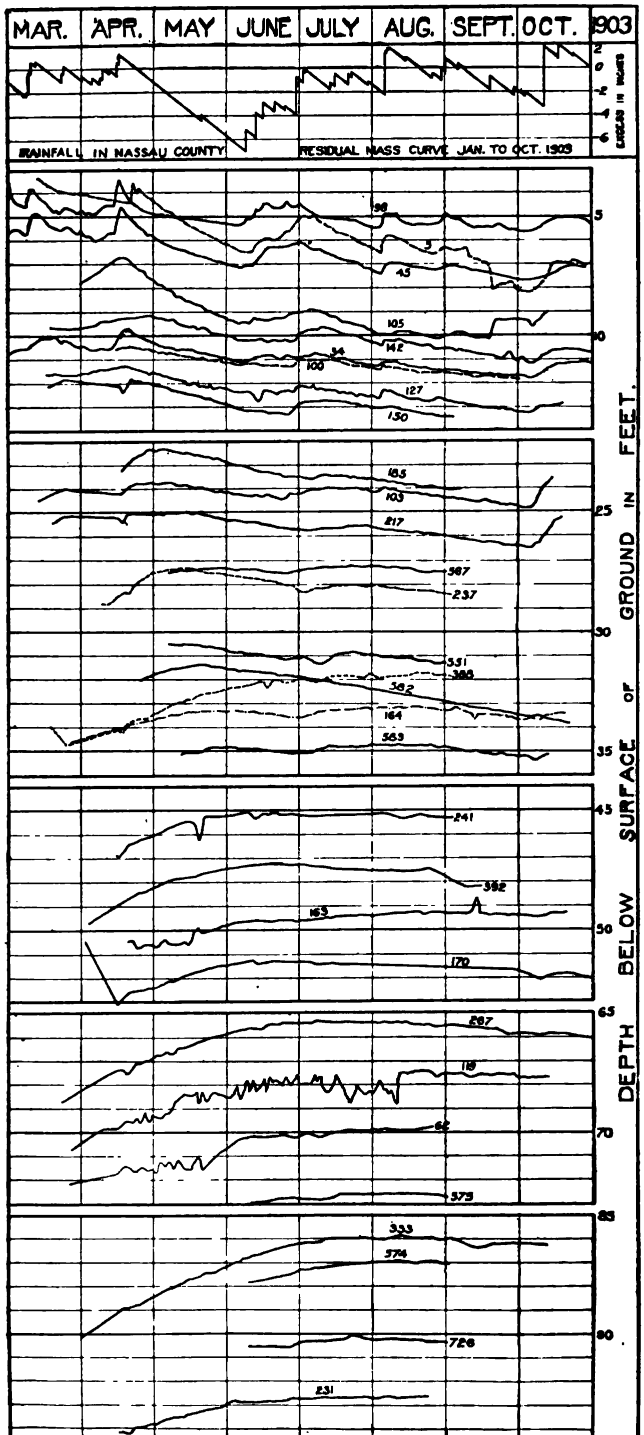


FIG. 13.—Fluctuations of water level in wells on Long Island, New York, based on observations at intervals of one to three days by the Long Island division of the New York City commission on additional water supply. (After Spear, 1904.) The location of the wells is given in Pl. I. The numbers are the same as those used in the commission's report.

downward percolation or downward capillary flow. There are certain difficulties in the way of determining the value sought in this manner. In the first place, the fluctuations in the shallow wells can not be satisfactorily correlated with those in the deep ones, and the only line which can be followed through the diagram prepared by Spear is the time of maximum ground water, which, as indicated above, in this region during the time of observation, in general lags proportionally to the depth. This gives a fairly regular curve and the remaining curves have been inferred on either side of this one. The yearly maximum, however, can scarcely be attributed to a single rain, but represents rather the culmination of a whole series of events, and hence can not be used as a basis of such a calculation.

In the case of regions like Wiener Neustadt it is clear that the results from calculations of this character would have no meaning, and, indeed, what do the values really represent on Long Island?

IRREGULAR SECULAR FLUCTUATIONS.

The observations of Dickinson and Evans and of Lawes and Gilbert with percolation gages developed the fact that, as a rule, not only did more water percolate in wet than in dry years, but that the percentage of rain water which passed through the soil columns was usually much greater in wet years than in dry ones. Thus while the average yearly percolation of 1870–1902 at Rothamsted (Lawes and Gilbert) was 49 per cent of the yearly rainfall of 28 inches, in the year 1878–79, when 40.2 inches of rain fell, 61 per cent of the rain water passed through a soil column 60 inches high, and in the year 1877–78, with a rainfall of 18.2 inches, the percolation was but 36 per cent (see p. 47.) The general tendency—although there are exceptional cases, such as recorded at Hemel Hempstead (Dickinson and Evans) in 1858–59, when, with a rainfall of 28 inches, 2 inches more than the annual average, the percolation was but one-third of 1 per cent instead of the usual 27 per cent—is for the small differences in the annual rainfall to have a rather magnified value in the ground-water fluctuations.

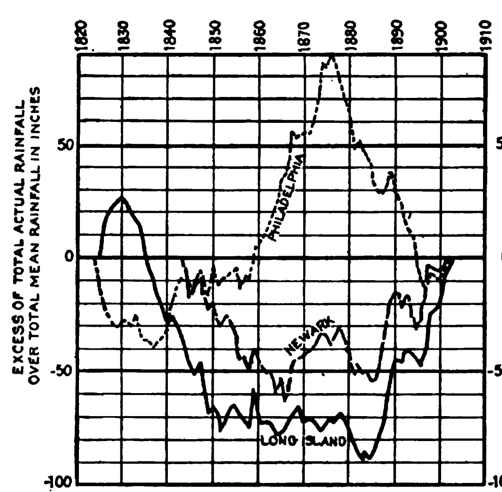


FIG. 14.—Residual-mass curves of rainfall for Long Island, N.Y., Newark, N. J., and Philadelphia, Pa. (After Spear, 1904.) These curves show the cumulative excess or deficiency of the total actual rainfall over the total mean rainfall for the periods under consideration, and as these excess or deficiency values are those which determine the long-period rise and fall of the ground water they indicate the general character of the secular fluctuation of the ground water occurring at these points.

The yearly variations of the rainfall are generally progressive over rather long periods (fig. 14), and corresponding broad, irregular variations of the ground-water level are produced. On Long Island the shallow wells observed by the Brooklyn waterworks show, besides the annual fluctuations, secular variations corresponding in general with those of the rainfall (fig. 15). Thus the lowest point in both curves is in 1901 and the highest in 1899. Many differences are, however, to be noted between the two curves. The annual curve, though it may be slightly modified, persistently recurs, whatever the rainfall. Note in this connection the regular downward course of the ground water in the latter part of

1898 and 1903, when the rainfall curve is rising, and the appearance in 1900 of a typical yearly curve when the rainfall curve falls rather regularly from the spring of 1899 to 1901.

Similarly, in the Wiener Neustadt observations (Pl. IX, p. 62) the secular variations of the ground-water level broadly follow the variations of the rainfall.

The observations of Auchincloss[a] on a well at Bryn Mawr, Pa., which have been plotted by Spear[b] in connection with the yearly rainfall and temperature curves, likewise show pronounced annual and secular fluctuations. Here, however, the secular fluctuations of the ground water, while broadly resembling the rainfall variations, show some points of difference. Thus, the minimum of the secular curve of the ground water is in 1885–86, while the minimum rainfall is in 1887, and the general shape of the two curves for 1886–87 and 1888 is by no means parallel. The positions of the maxima, however, agree closely, and there is a general falling off in both curves from 1889 to 1893–94. Though the extreme rains of the latter part of 1889 temporarily obliterated the annual curve, it quickly reasserted itself.

In general it may be said that irregularities in the yearly curve are due to irregularities in the rainfall occurring in the same year.

AMOUNT OF ANNUAL AND SECULAR FLUCTUATION.

The size of the annual fluctuations depends principally upon (1) the percentage of rainfall reaching the ground water; (2) the amount of free pore space of the strata in the zone affected by the fluctuations, and (3) the relation of the ground-water table to the topography of the region involved.

It is relatively self-evident that, where a single well is considered, the range of the yearly fluctuations will vary with the first factor, and that in general the same amount of infiltration will produce a greater fluctuation where the pore space is small than where it is large. It does not, however, follow that in a given region, in beds of the same porosity, the same annual rainfall under the same climatic conditions will produce the same results. Observations have shown that whatever the rainfall or porosity, provided the latter be reasonably constant in the area under consideration, the annual fluctuations approach zero at the point of discharge and tend to regularly increase in magnitude from that point to the ground-water divide.[c] Thus at Wiener Neustadt (Pl. IX, p. 62), near the ground-water discharge into Fisha River, where the depth to the ground-water table is about 5 feet, the yearly fluctuation is 3 to 4 feet, while at St. Agyden station, where the water plane is about 150 feet from the surface, the fluctuation is 25 to 30 feet, and the fluctuations in the intervening wells are proportional to their position between these two points. On Long Island the annual fluctuation 2 miles from the shore, at Millburn (figs. 13, 15), is 22 inches, while at the ground-water divide, 8 to 9 miles from the south shore, the fluctuation is about 10 feet. A few observations regarding the amount of the yearly fluctuation at different points have been collected in the table following. Many of these points of observation are located near the points of discharge, and the values as a whole are to be regarded as low.

In records for but a few years it is evidently impossible to separate the annual from the secular fluctuations. When, however, the observations cover a considerable period, it is possible to obtain a value for the secular fluctuation. This equals the total range less the average yearly fluctuation. A few such values are given in the table on page 40.

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[a] Auchincloss, W. S., Waters within the Earth and Laws of Rainflow, Philadelphia, 1897, p. 9.

[b] Spear, Walter E., Rept. Commission on Additional Water Supply for the City of New York, 1904, appendix 7, fig. 45, p. 822.

[c] The cross section from Watford to the Chiltern Hills midway between Colne and Gade rivers, which accompanies Clutterbuck's discussion of the "Periodic Alternations of the Chalk Water Level under London" (Min. Proc. Inst. Civil Eng., vol. 9, 1850, Pl. VI), is a most excellent diagrammatic illustration of this principle.

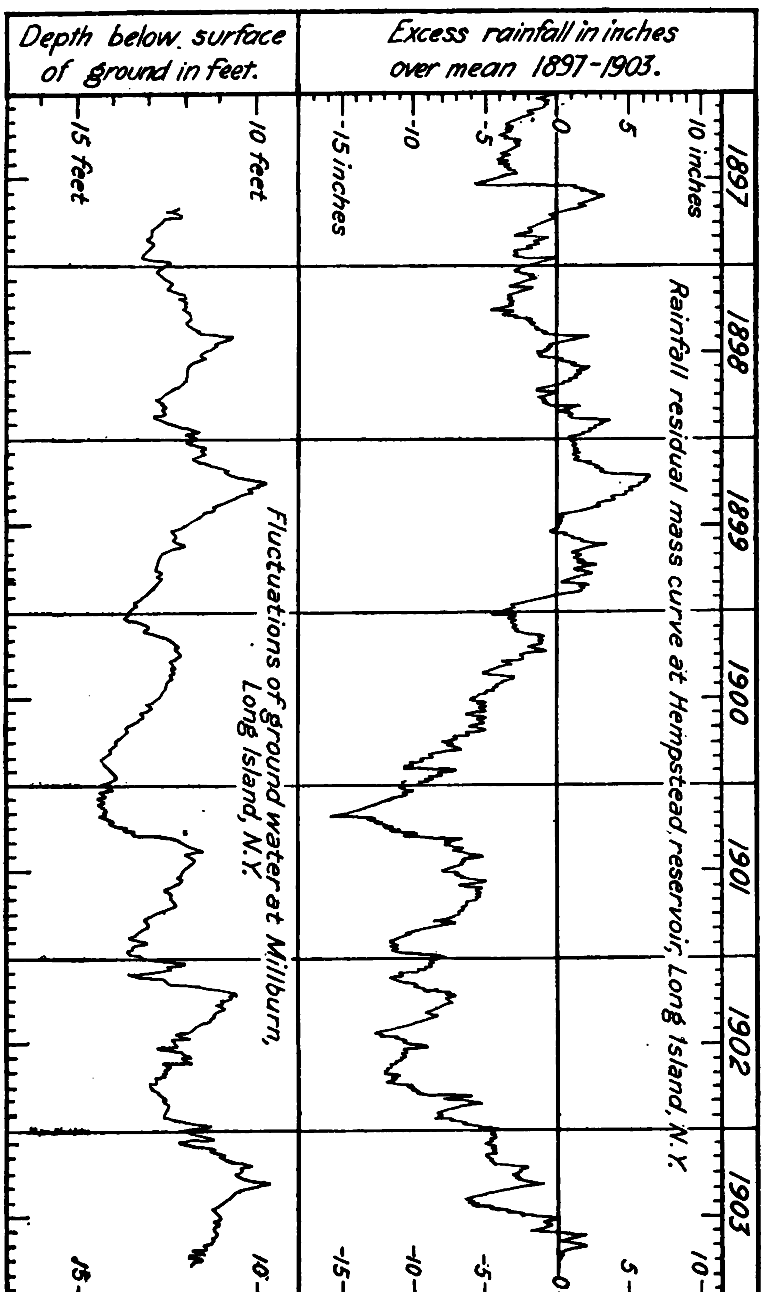

FIG. 15.—Annual and secular changes of the ground-water level and fluctuations due to single showers in a shallow well at Millburn, N. Y., from the observations of the Brooklyn water department. (After Spear, 1904.)

Table giving amount of annual and secular fluctuations of water in wells.

Location.	Average annual rainfall.	Character of water-bearing strata.	Distance from point of ground-water discharge.	Total depth of well.	Distance of mean high water from surface.	Average annual fluctuation.	Secular variation for period given.[a]	Period of observation.	Observer.
UNITED STATES.	*Inches.*		*Miles.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*		
Millburn, Long Island, N. Y.	44	Sand and gravel	0.02	25	10	1.8	....	1897–1908	Brooklyn waterworks.[b]
Hicksville, Long Island, N. Y.	44	....do	7.0	100	80	10.0	....	1908	Estimated from observations of New York commission on additional water supply.[c]
Ann Arbor, Mich.	....	Glacial drift	....	....	....	2.3	6.7	1885–1901	Michigan State board of health.[d]
Michigan wells (average of 18)	35	....do	....	8–55	....	2.6	....	1878–1897	Do.
Lansing, Mich.	30	....do	....	27	25	.6	3.7	1885–1901	Do.
Bryn Mawr, Pa.	49	Decomposed mica schist.	.25	65	43	5.3	13.7	1886–1895	Auchincloss.[e]
Geneva, N. Y.	28	Glacial drift	....	40	5	9.7	....	1887–1889	Emery.[f]
MIDDLE EUROPE.									
Munich (average of 2 wells)	32	Gravel	....	18	18	.9	4.5	1856–1885	V. Pettenkofer.[g]
Berlin (average of 37 wells)	23	Alluvium and glacial deposits (sand).	....	Shallow	....	1.8	....	1870–1885	[Soyka.][h]
Salzburg	46	....	....	19	....	.7	2.7	1860–1885	Spangler.[i]
Do	46	....	....	26	....	4.0	....	1876–1880	Liznar.[k]
Graz	35	....	....	19	....	3.0	....	1876–1880	Do.
Innsbruck	33	Sand and gravel	....	32	....	7.5	....	1876–1880	Do.
Trieste	51	....do	....	18	....	7.3	....	1876–1880	Do.
Klagenfurt	42	....	....	....	....	3.4	....	1877–1886	Do.
Do	42	....	....	30	....	2.6	....	1877–1886	Do.
Frankfurt a. M. (average of 6 wells).	25	Diluvium	....	25–50	16–30	1.2	....	1869–1885	[Soyka.][l]
Bremen	26	....	....	....	....	1.9	....	1872–1884	Liznar.[l]
Brünn	22	....	....	....	....	1.9	....	1865–1880	[Soyka.][l]
Prag	19	....	....	26	17	1.4	....	1876–1880	Do.
Krakau	25	....	....	20	11	4.7	....	1876–1880	Do.
Josephstadt	24	....	....	73	33	8.2	....	1870–1880	Do.

Baden				17	13	.8		1876-1880	Do.
Agram	35			32	16	8.4		1876-1880	Do.
Debreczin	25			21	11	1.8		1876-1880	Do.
Szegedin	21			19	8	1.9		1876-1880	Do.
Czernowitz	26			209	154	2.0		1876-1880	Do.
Wiener Neustadt	25	Sand, gravel, and conglomerate.	.8	8	3	2.6	3.6	1886-1890	Bericht des Ausschusses für die Wasserversorgung Wiens.
Do.	25	do	2.7	71	44	14.9	9.6	1886-1890	Do.
Do.	25	do	6.3	177	144	24.4	21.5	1886-1890	Do.
ENGLAND.									
Sussex, Alfriston	30	Chalk	.2	16	7	2.0		1885-1899	Whitley.[m]
Sussex, Jevington	30	do	2.5	198	120	75.0		1885-1899	Do.
Sussex, Eastdean	27	do	1.5	163	154	10.0		1885-1899	Do.
Hertfordshire	30	do				70.0			Evans.[n]
Hertfordshire (average)	30	do				20.0-30.0			Clutterbuck.
Southern England	30	do				2.0-135.0		1841-1851	Lucas.[o]
London		do				4.6		1841-1848	Clutterbuck.[p]
Chelgrove		do				[q]135.0		1841-1851	Lucas.[r]
Caterham		do				[q]100.0			Lucas.[s]
Cheshire, Anderton		New red sandstone?		225	75	93.0		1855	Northcote.[t]

[a] Maximum range for period given less the average yearly fluctuation.
[b] Rept. New York City Commission on Additional Water Supply, 1904, appendix 7, Pl. IX.
[c] Ibid., Pl. VI (should be IV), pp. 819-820.
[d] Ann. Repts. Michigan State Board of Health, 1879, 1881-1902.
[e] Auchincloss, W. S., Waters within the Earth and Laws of Rainflow, 1897, p. 9.
[f] Emery, Frank E., Sixth, Seventh, and Eighth Ann. Repts. New York Expt. Station, 1888-1890.
[g] Soyka, Schwankungen des Grundwassers: Penck's Geographische Abhandlungen, 1888, pp. 31-32.
[h] Ibid., p. 34.
[i] Ibid., pp. 37-38.
[k] Ibid., pp. 41-42.
[l] Ibid., pp. 48-51.
[m] Whitley, Henry Mitchell, Min. Proc. Inst. Civil Eng., vol. 142, 1900. pp. 342-351.
[n] Evans, Sir John, Min. Proc. Inst. Civil Eng., vol. 45, 1877, p. 124.
[o] Lucas, Joseph, Min. Proc. Inst. Civil Eng., vol. 47, 1877, pp. 70-167.
[p] Clutterbuck, Rev. James, Min. Proc. Inst. Civil Eng., vol. 9, 1850, p. 153.
[q] Extreme range for period given; it therefore includes secular as well as annual variations.
[r] Min. Proc. Inst. Civil Eng., vol. 47, 1876-77, pt. 1, 1877, p. 87.
[s] Ibid., p. 118.
[t] Northcote, A. B., Phil. Mag., 4th ser., vol. 14, 1857, p. 471.

FLUCTUATIONS DUE TO SINGLE SHOWERS.

In the foregoing consideration of the relation of the rainfall to secular and annual fluctuations, the most important factor was clearly the amount of water actually contributed to the zone of complete saturation, or the ground water.[a] In these cases the water table rises or falls because the amount of water received is greater or less than the outflow.

In the consideration of single showers, however, it is found that another factor is of as great or even greater importance. Single showers may affect the water level in a well in two ways: (1) By transmission of pressure without any actual addition to the ground water—indeed, in many cases the elevation of the water in the well is accompanied by an actual depression of the ground-water table; (2) by an actual contribution to the ground water whereby the level of the water table is raised.

FLUCTUATIONS PRODUCED BY SHOWERS BY TRANSMITTED PRESSURE WITHOUT ANY INCREASE IN THE GROUND WATER.

King observed at Madison, Wis.,[b] that the water often rose in wells very suddenly and sharply during summer rains, when an investigation of the soil showed that it was dry beneath the surface covering wet by the showers. Similar occurrences were recorded at Lynbrook, N. Y., where in a 2-inch well 14 feet deep every rain—during the period of observation, July 17 to September 10—was recorded and its duration and complexity indicated. Many of these fluctuations produce no permanent deflection of the ground-water curve and the evidence that no water from several of these rains reached the ground water is regarded as conclusive (Pl. VI; also note particularly the behavior of the shallow well August 19–21 and the fluctuations produced by rains of July 20, 22, 30, and August 6; the two successive showers of August 6–7 are particularly noteworthy).

In the case of wells which are not separated from the main water table by impervious layers and in which the water is not under artesian pressure, this is due largely to the hydrostatic transmission of pressure by means of the soil air. When the rain strikes the surface it closes the superficial soil pores or interstices and thus confines and compresses the air in the soil between the surface and the ground-water table. The weight of the rain is thus transmitted to the ground-water table, and the extra head so developed raises the water in wells and increases the discharge at the ground-water outlets. The effect on the stream flow is very analogous to the increase produced by a lowering of the barometric pressure. It is thus possible for a rain to produce instantly a change in the water level in wells and an increase in the ground-water outflow without contributing a drop to the ground water. This has an important bearing on the calculation of "flood flows" from ground-water-fed streams, for it is evident that in this manner a decided rise in the stream may be produced by a rain from which there is no direct run-off and which does not reach the ground-water table.

Two other factors may be involved in the production of the change in level during rains: (1) An actual elastic compression or plastic deformation of the soil, and (2) a change in the capillary conditions. King observed in a shallow well near a railroad track that the passage of a freight train caused a quick rise and fall of the water in the well (p. 75). Apparently the weight of the train compressed the earth and by decreasing the pore space caused the water to rise. The weight of the rain might have a similar effect. Under this hypothesis the water would rise on the addition of the rain and gradually fall on its removal by evaporation.

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[a] Ground water as here used does not include the hygroscopic and capillary water above the water table, or zone of complete saturation. For the purpose of this discussion, water is not considered "ground water" until it reaches the water table.

[b] Bull. U. S. Weather Bureau No. 5, 1892, pp. 20, 72–73.

Regarding the second hypothesis it is well known that changes in the surface conditions greatly affect the capillary action of the soil. At Purdue University it was found that the addition of a thin layer of soil to the surface of a lysimeter caused an immediate discharge of water.[a] This was attributed to a change in the capillary conditions, and it has been suggested that the wetting of the ground surface would produce a similar effect. King, however, observed [b] that a moderate wetting of the surface tended to increase the upward percolation, and the effect of wetting the surface at Lynbrook would therefore be to diminish rather than increase the rise due to rains which do not contribute to the ground water.

At the Colorado experiment station Headden has observed [c] that light rains during dry periods produce a comparatively great increase in the height of the water plane, while in intervals of abundant moisture, when the soil is wet, rains of this character do not produce such an increase; moderate rains are here sometimes accompanied by temporary depressions of the water plane. These observations may be explained on the basis that the soil air is the principal factor and that when the soil is very moist there is so little soil air that no effect is possible with slight showers. The cause of the temporary depression after moderate rains is not evident unless the conditions are unfavorable for the transmission of pressure by the soil air, but are such that the increased upward capillary action resulting from the moistening of the surface is sufficient to perceptibly decrease the ground water.

Where there is an impervious layer between the water-bearing strata and the local ground-water surface, and where there is artesian pressure, the added weight due to the rainfall in all cases acts directly. In case the rain is uniform over a considerable area this pressure may be regarded as acting on an elastic body and the same character of results is to be expected in both deep and shallow wells. Thus in the Lynbrook wells on July 22 and August 25 all wells show a sharp vertical rise (Pl. VI). On July 22 the rise started in the 14-foot well at 10 p. m., in the 72-foot well at 10.25, and in the 504-foot well at 10.34; on August 25 a somewhat similar lag is noted, the 14-foot well rising at 4.10 p. m., the 72-foot well at 4.20, and the 504-foot well at 4.24. The cause of this lag is not fully apparent. With a direct transmission of pressure such as the curve indicates no lag is to be expected. It may be that in this case the soil is to be regarded as having some of the plastic characters shown in other cases.

On the other hand, when the rainfall is unequally distributed in time and amount a plastic deformation may result, due to unequal loading, which will give rise to different results in wells of different depths. In the shallow well the zone of influence is relatively limited and the condition in this area may be regarded as fairly uniform. The result is therefore immediate and abrupt, as in the first case. In the deeper wells, however, the increasing zones of influence bring in more factors, which, arriving progressively from different sources, tend to produce a more and more gradual change. Thus, in the Lynbrook wells there were on August 6–7, 11, and 20 abrupt changes in the 14-foot well and a more gradual one in the 72- and 504-foot wells.

This plastic deformation in the surficial beds, produced by varying load and the response of the water to it, throws some light on the extreme complexity of the fluctuation recorded in the wells, for it suggests that variation in load, from whatever cause, will produce corresponding fluctuations. The water level in deep wells where an artesian head is developed may thus be very sensitive to local conditions, the effect of local rainfall and of the yearly fluctuations of the local ground-water level being felt to a greater or less degree by transmitted pressure in the deeper zones.

[a] Second Ann. Rept. Indiana Expt. Station, 1889–90, pp. 32–33.
[b] Seventh Ann. Rept. Wisconsin Agric. Expt. Station, 1890, p. 135.
[c] Headden, William P., A soil study, pt. 4, The ground water: Bull. Colorado Agric. Expt. Station No. 72, 1902.

FLUCTUATION OF THE GROUND-WATER LEVEL RESULTING FROM SINGLE SHOWERS, BY ACTUAL PERCOLATION.

The fluctuations produced by direct percolation are of a much less abrupt character than those just described; indeed, it is usually the case that the water is delivered so gradually to the water table that no change is noticed. Only in the shallow wells in coarse material can these fluctuations be identified, except in the case of extraordinary rains, when the result is an irregularity of greater or less importance on the regular annual ground-water curve.

On Long Island the shallow wells near the south shore are affected by most of the important rains, although part of the fluctuations recorded are of the character just described. (See figs. 13, 15.) This is due to the coarseness of the surficial material and to the nearness of the water table to the surface. In the wells in which the ground water is farther from the surface the effect of any rain can not be positively identified. In the Wiener Neustadt records the effect of single rains is entirely obliterated (Pl. IX), and in long observations of the chalk waters of England the general rule, to which, of course, there are exceptions where large underground caverns are concerned, is that the water is delivered very gradually to the ground-water table.

PERCENTAGE OF RAINFALL CONTRIBUTED TO THE GROUND WATER.

METHODS OF ESTIMATION.

In connection with this discussion of the fluctuation of the water level it may not be inappropriate to take up the allied question, to which reference has been made at several points, of the percentage of rain contributed to the ground water.

Estimates of this character have been made by three methods—(1) by means of the lysimeter, (2) by stream discharge, and (3) by changes in the level of the ground water.

ESTIMATION OF PERCOLATION BY MEANS OF LYSIMETERS.

By the lysimeter method the rain water passing through a column of soil in field conditions is measured directly. The gage used for this purpose consists of a vessel with impervious sides and a pervious bottom, filled with the soil to be tested, and buried so that the surface of the soil in the gage is at the same level as the surrounding ground. The discharge through the pervious bottom of the vessel is collected by a cone and conducted by a small tube to the measuring gages. In the early forms of the apparatus used by Dalton, 1796, and Dickinson, 1835, surface outlets were provided to discharge the excess rainfall, but these were abandoned when it was found that on the level surface of the gage there was no surface run-off.

Many observations have been made along this line, and while the results for long periods clearly have a greater value than those for short periods, some of these short-period values have been included in the table on the following page for the purpose of comparison.

Lysimeter results have been subjected to considerable criticism, and very differing views expressed regarding their value. It has been suggested (1) that the material in the gage is not in the natural condition of consolidation, and that, therefore, the results are too high; (2) that the underdrainage necessary to collect and carry the water from the base of the soil column to the measuring tube introduces an unnatural condition whereby the results are too low; (3) that the surface run-off factor is surpressed and the results are too high.

Results of lysimeter experiments to determine the amount of rainfall reaching the ground water.

Observer.	Locality.	Conditions of experiments.	Period.	Length in years.	Rainfall.	Percolation.	Per cent of rainfall.
					Inches in depth.	*Inches in depth.*	
Dalton [a]	Manchester, England	Cylinder 10 inches in diameter, 3 feet deep, filled with average Lancashire soil [b] and sunk level with ground; after first year covered with grass.	1796–1798	3	33.6	[c] 8.4	[c] 25.0
Maurice [d]	Geneva, Switzerland	Cylindrical iron vessel filled with soil	1796–1797	2	26.0	10.2	39.0
Gasparin [e]	Orange, southern France	do	1821–1822	2	28.0	7.0	20.0
Dickinson and Evans [f]	Nash (or Apsley Mills), Hemel Hempstead, Herts, England.	Gage sunk flush with ground, filled with soil 10 inches, sandy gravel 15 inches, clean coarse gravel 11 inches, [g] representing ordinary surface conditions of that region. From 1835 to 1858 gage consisted of a wooden cylinder 12 inches in diameter and 3 feet deep, with lead funnel and draw pipe at bottom. After 1853 a cast-iron cylinder, 18 inches in diameter and 3 feet high, was used. Surface in all cases covered with grass.	[h] Oct., 1835–Oct., 1884	[h] 49	28.0	7.5	26.8
Do	do	Wooden cylinder 12 inches in diameter, 3 feet deep, filled as above.	Oct., 1835–Oct., 1853	18	26.6	9.2	34.6
Do	do	Cast-iron cylinder 18 inches in diameter, 3 feet deep, filled as above.	Oct., 1853–Oct., 1875	22	25.7	5.6	21.7
Do	do	Cast-iron cylinder. Minimum yearly percolation of period 1835–1875.	Oct., 1858–Oct., 1859	1	27.95	.9	.3
Do	do	Cast-iron cylinder. Maximum yearly percolation of period 1835–1875.	Oct., 1859–Oct., 1860	1	36.9	12.4	33.6
Do	do	Cast-iron cylinder. Minimum yearly rainfall of period 1835–1875.	Oct., 1863–Oct., 1864	1	17.66	3.53	20.0
Do	do	Cast-iron cylinder	Oct., 1873–Oct., 1874	1	19.62	1.91	9.7

[a] Mem. Lit. Phil. Soc. Manchester, vol. 5, 1st ser., p. 360; Jour. Royal Agric. Soc., vol. 10, 1844.
[b] Described by Prestwich (On Water-bearing Strata about London, 1899, p. 111) as common soil of the new red sandstone area which contains a good deal of clay.
[c] Field has suggested (Min. Proc. Inst. Civil Eng., vol. 40, 1876, p. 88) that the underdrainage of Dalton's gage was very imperfect, and that this result is perhaps too low.
[d] Bibl. Universelle de Genève, Sciences et Arts, vol. 1; Gilbert, J. H., Min. Proc. Inst. Civil Eng., vol. 45, 1876, p. 56.
[e] Cours d'Agriculture, vol. 2, p. 116.
[f] Dickinson, John, and Evans, John, Min. Proc. Inst. Civil Eng. [vol. 2], 1843, pp. 160–161; vol. 9, 1850, p. 158; vol. 20, 1861, pp. 219–222; vol. 45, 1876, pp. 208–216.
[g] Min. Proc. Inst. Civil Eng., vol. 14, 1855, pp. 86–87.
[h] Obtained by combination of values given in Min. Proc. Inst. Civil Eng., vol. 105, 1891, p. 50, and vol. 20, 1861, p. 220.

Results of lysimeter experiments to determine the amount of rainfall reaching the ground water—Continued.

Observer.	Locality.	Conditions of experiments.	Period.	Length in years.	Rainfall.	Percolation.	Per cent of rainfall.
					Inches in depth.	*Inches in depth.*	
Dickinson and Evans	Nash (or Apsley Mills) Hemel Hempstead, Herts, England.	Cast-iron cylinder 18 inches in diameter, 3 feet deep, filled with broken chalk covered with a thin layer of soil and grass.	Oct., 1853–Oct., 1875	22	25.7	9.1	35.4
Charnock[a]	Ferrybridge, Yorkshire, England.	"Into a leaden vessel 1 foot square and 3 feet deep with drain at bottom was put 2 feet of gravel and the remainder filled with the average quality of magnesium-limestone soil." The soil was kept free from weeds and well stirred.	1842–1846	5	24.6	4.8	19.6
Greaves[b]	Lee Bridge, near London, England.	Slate box 3 feet square, sunk in ground and filled with loam, gravel, and sand, all well mixed beforehand, trodden in and turfed. Material intended to represent common Hertfordshire land.	1852–1873	22	25.84	6.87	26.4
Do	do	Slate box, filled as above. Year of minimum percolation, period 1853–1873.	1854	1	17.5	1.25	7.1
Do	do	Slate box, filled as above. Year of minimum rainfall, period 1853–1873.	1864	1	15.9	3.8	24.0
Do	do	Slate box, filled as above. Year of maximum percolation, period 1853–1873.	1866	1	31.7	12.6	39.7
Do	do	Slate box, filled as above. Year of maximum rainfall, period 1853–1873.	1872	1	37.2	12.0	32.2
Do	do	Slate box, filled with fine sand, kept free from vegetation.	1860–1873	14	25.7	21.4	83.3
Von Möllendorff[c]	Görlitz, Silesia, Germany.	Cylinder 1.25 meters deep, filled with clay, not covered with vegetation.	1853–1856	4	25.6	7.2	28.1
Do	do	Cylinder 1.25 meters deep, filled with loam, not covered with vegetation.	1853–1856	4	25.6	10.5	41.0
Do	do	Cylinder 1.25 meters deep, filled with sandy loam, not covered with vegetation.	1853–1856	4	25.6	10.4	40.5
Do	Tharoud, Saxony, Germany.	Calculated on discharge drain placed at a depth of 1.25 meters in a field covered with vegetation; clay soil.	1853–1856	4	28.8	11.7	40.8
Do	do	Calculated on discharge drain placed at a depth of 1.25 meters in a field covered with vegetation; loamy soil.	1853–1856	4	28.8	16.9	58.7
Risler[d]	Calèves, near Nyon, Switzerland.	By gaging drains 1.2 meters deep; compact impervious subsoil; land cropped as usual during experiment	1867–1868	2	41.0	[illegible]	30.0

Wollny[e]	Munich, Bavaria	Gage 1.20 meters deep, filled with sand, not covered with vegetation.	1878–1880	3	42.8	28.4	67.2
Do	do	Gage 1.20 meters deep, filled with loam, not covered with vegetation.	1878–1880	3	42.3	11.2	26.5
Do	do	Gage 1.20 meters deep, filled with peat, not covered with vegetation.	1878–1880	3	42.3	19.0	44.9
Lawes and Gilbert[f]	Rothamsted, Hertfordshire, England.	Gage one-thousandth of an acre area, soil rather heavy loam with clay subsoil built in, in natural state of consolidation with brick and cement. No vegetation allowed to grow on gages, which are surrounded by meadow land; soil column 20 inches deep.	Sept., 1870–Sept., 1902	32	28.1	13.7	48.8
Do	do	Same as above; soil column 40 inches deep.	Sept., 1870–Sept., 1902	32	28.1	14.5	51.6
Do	do	Same as above; soil column 60 inches deep.	Sept., 1870–Sept., 1902	32	28.1	13.5	47.9
Do	do	Same as above; soil column 60 inches deep. Year of maximum rainfall, period 1870–1902.	Sept., 1878–Sept., 1879	1	40.2	24.4	60.7
Do	do	Same as above; soil column 60 inches deep. Year of minimum rainfall, period 1870–1902.	Sept., 1897–Sept., 1898	1	18.2	6.5	35.7

[a] Charnock, Chas. [Results of percolation experiments near Ferrybridge, York, 1842–1846]: Jour. Royal Agric. Soc., vol. 10, 1849, p. 516.
[b] Greaves, Chas., Min. Proc. Inst. Civil Eng., vol. 45, 1876, pp. 19–48.
[c] Von Möllendorff, G., Die Regenverhältnisse Deutschlands. Görlitz, 1862; Gerhardt, I. P., Der Wasserbau, Leipzig, 1892, p. 37.
[d] Archives des Sciences, Bibliothèque Universelle, Sept., 1869, quoted by Gilbert, Min. Proc. Inst. Civil Eng., vol. 45, 1876, p. 56, and Gerhardt, Der Wasserbau.
[e] Wollny, E., Forschungen a. d. Gebiete d. Agrikulturphysik; Gerhardt, Der Wasserbau, Leipzig, 1892, p. 37.
[f] Lawes, Sir John, and Gilbert, Sir J. H., Min. Proc. Inst. Civil Eng., vol. 45, 1876, pp. 59–60; id., vol. 105, 1891, pp. 35–46; The Rothamsted Experiments, Plans, and Summary Tables, 1903, p. 1.

The objection regarding consolidation is well taken, though it clearly does not apply to the results at Rothamsted, where natural soil columns were used, and where very high results were obtained, or to other gages after the first few years, during which the soil has settled. In the Hemel Hempstead experiments the percolation between 1835 and 1843 was 42.5 per cent of the rainfall, while in the period 18[illegible]-1853 it was but 35 per cent, a decrease which is perhaps due in part to the gradual compacting of the soil, though it is also affected by the varying rate of percolation, for it must be remembered that the amount of percolation depends more on the time at which the rain falls and the manner in which it is distributed than on the actual amount.

Regarding the unnatural condition introduced by the method of drainage, it has been suggested on the one hand that the lower part of the soil column is thus exposed to evaporation, and that not only is there a loss in this manner not accounted for by the measuring gage, but that the dry condition of the basal layer would retard the percolation to a measureable extent and increase the loss by evaporation from the surface of the ground. Ebermeyer has shown, however, that with small lysimeters the lower portion of the soil column is damper than normal,[a] and he proposed to remedy this by constructing larger gages. These defects are clearly ones of construction which it is possible to remedy. At Rothamsted essentially the same results were obtained from soil columns 20, 40, and 60 inches high (fig. 12), showing conclusively that in this case the natural conditions were not essentially disturbed.

Indeed, it is believed that carefully conducted lysimeter observations, extending over long periods, such as are represented by the Dickinson and Evans and the Lawes and Gilbert experiments, give very important values bearing on this question, the Lawes and Gilbert results being particularly important and trustworthy. They indicate that in the climatic condition of middle England, with 28 inches of rainfall, half of the rainfall is contributed to the ground water through a rather heavy soil not covered with vegetation, and half of it evaporated. If the rainfall is greater, the percentage increases, if less, it decreases. Had the ground been covered with leaves, straw, or similar matter the percolation would have been greater; if covered with growing vegetation, less. Lawes and Gilbert estimate, from their observations on plant transpiration, that in this region 2 inches per year would represent the plant transpiration in the area of downs and waste land, where there was very little vegetation, while with a heavy grass or mangel crop it would amount to 7 inches or more. The average for the whole region was estimated at 3 to 4 inches. This would make the percolation for soil of this character, in the case of downs and waste land, 43 per cent; for the average mid-England district, 39 per cent, and for land covered with heavy grass or mangel crop, 25 per cent or less.

It should be noted in this connection that while the most of these observations, including those at Rothamsted, were made in connection with agricultural investigations, the Hemel Hempstead and Lee Bridge (Greaves) experiments were made for engineering purposes. The Hemel Hempstead observations were undertaken by a paper manufacturer, dependent on the water power of a spring-fed stream. He argued that stream flow depended on the amount of water which percolated through the soil; that measurements of this quantity would indicate the stream flow to be expected during the following summer. It is stated that he found that the indications of the gage during the winter enabled him to calculate the supply of water from the stream during the ensuing season, that he had always found the indication perfectly reliable,[b] and that he was accustomed to regulate the volume of the orders accepted for the summer season by the indication of the gage for the preceding winter.[c] Clutterbuck adds, though the relation is clearly more of a qualitative than

[a] Reported by R. H. Scott, Jour. Royal Agric. Soc., 2d ser., vol. 17, 1881, pp. 66-67.
[b] Min. Proc. Inst. Civil Eng., vol. 2, 1842, p. 158.
[c] Ibid., p. 157.

a quantitative one, that the rise in level of the water in the wells in that region is found to coincide with the readings of the Dickinson gages.[a]

The lysimeter certainly furnishes a very direct and exact means of determining the amount of water contributed to the ground water at any given point. The principal objection to it is that the block of soil tested is not necessarily representative of the whole area under investigation.

Somewhat similar experiments, having for their object the determination of the evaporation from plants, have been conducted by many agriculturists in this country, notably by King,[b] by means of tanks which can be lifted from the ground and weighed. This method is not so applicable to the amount of water contributed to the ground water as is the lysimeter type used above, for the results are less direct, the percolation being inferred from the evaporation generally under more artificial conditions than with the lysimeter.

ESTIMATION OF PERCOLATION FROM THE STREAM DISCHARGE.

The favorite method of estimating the evaporation of a given drainage basin is to subtract the stream discharge expressed in inches of rainfall over the drainage basin from the average rainfall. Thus it is assumed that—

Rainfall — Stream discharge = Evaporation.

The stream discharge is composed of (1) the rain water which flows into the drainage channels without penetrating the soil—this is, strictly speaking, the run-off, but as this word is now by common consent used for the whole quantity of water discharged by the river, this contribution may be somewhat arbitrarily referred to as the flood flow; and (2) the water which after a greater or less journey through the earth returns to the surface—this may be called the spring or ground-water flow of the river. It has been assumed that the ground-water flow of a river is its low-water flow and that any excess of this quantity can be regarded as flood flow. This is far from being a general fact. As the height of the ground-water table increases the stream discharge also increases, and it is possible to have high and low waters dependent entirely on the fluctuation of the ground-water discharge. In streams which cut the ground-water table and are clearly ground-water-fed streams, such as those on Long Island, a rise in the ground-water table changes the position of the head of the stream, and by thus increasing both the head and the area of discharge greatly increases the stream flow. The total range of the ground-water table near the coast is much less than near the ground-water divide, and the discharge during periods of high ground-water level may therefore be disproportionate to the changes in level recorded by the wells near the coast. Because of these great changes in the area of the discharge and the relatively free flow of the surface water, it often happens that the fluctuations of the stream height in the lower part of the stream are greater than the changes in the level in wells in the same region.

Heavy rains, with no surface run-off, may likewise produce sudden and considerable rises by increasing the spring flow by transmitted pressure in the manner described above (p. 42). In the 14-foot well at Lynbrook (p. 23), besides the sudden rises recorded for every rain, the water four times during the period of observation rose above the surface. In the first instance the water, much to the amazement of the observer, gushed over the top of the pipe a few minutes after the shower began, and, while after the pipe was raised this did not occur again, the records show that on several occasions the water rose higher than the ground surface. There appear, then, to be great and almost insurmountable difficulties in the way of the satisfactory separation of the stream discharge into spring or ground-water flow and flood flow.

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[a] Min. Proc. Inst. Civil Eng., vol. 9, pp. 153, 156.
[b] Ann. Repts. Wisconsin Agric. Expt. Station, 1892, pp. 94–100; 1893, pp. 152–159, 1894, pp. 240–280; 1897, pp. 228–231.

It is the belief of the writer that in the eastern United States the portion of the total stream flow attributed to ground-water contributions is commonly greatly under-estimated.

On Long Island Spear, from a comparison of the hydrographs of several of the streams near the south shore with the fluctuation in neighboring wells, has concluded that of the total stream discharge but 57 per cent is spring or ground-water flow.[a] This is an extremely low value, and from a consideration of the various factors involved it is believed that 90 per cent is much nearer the true value. On this basis the flood flow is but 3 or 4 per cent of the yearly rainfall.

In the simple equation, Rainfall—Stream flow=Evaporation, no account is taken of the underflow, it being assumed that all the ground water is returned to the stream above the point at which the measurement is made, an assumption which is far from correct. The result of this is to give to the evaporation a value just as much in excess of its true value as there is loss by underflow. Thus on Long Island, where the percolation is perhaps 60 per cent of the rainfall, the estimate of Spear[b] gives the total normal stream discharge as 33 per cent of the rainfall, and the estimates of the Brooklyn water department are still lower. This, according to the above formula, would give a loss by evaporation of 67 per cent, when it is actually about 40 per cent. It may be assumed, however, except in regions deeply covered with loose superficial material, such as Long Island, that the loss by underflow is less than the excess by flood flow, and that the total stream flow represents a quantity slightly larger than the percolation. With this in mind, some idea of the amount of percolation can be obtained from the following values:

Rainfall and run-off of drainage basins in the United States.

Drainage basin.	Years of record.	Average yearly rainfall.	Average yearly stream flow.	Percentage: Stream flow of rainfall.	Authority.
		Inches of depth.	*Inches of depth.*		
Watershed of southern Long Island	....	42.56	14.0	33.0	Spear.
Muskingum River, Ohio	1888–1895	39.7	13.1	33.0	Rafter.
Genesee River, N. Y.	1890–1898	40.3	14.2	35.0	Do.
Lake Cochituate, Mass	1863–1900	47.1	20.3	43.0	Do.
Mystic Lake, Mass.	1878–1895	44.1	20.0	45.3	Do.
Croton River, N. Y	1868–1899	48.07	22.33	46.5	Freeman.
Neshaminy Creek, Pa	1884–1899	47.6	23.1	48.5	Rafter.
Sudbury River, Mass	1875–1900	46.1	22.6	49.0	Do.
Sudbury River, Mass	1875–1902	46.38	22.75	49.0	Metropolitan water-works.
Perkiomen Creek, Pa.	1884–1899	48.0	23.6	49.0	Rafter.
Connecticut River, Conn.	1872–1885	43.0	22.0	51.0	Do.
Hudson River, N. Y.	1888–1901	44.2	23.3	52.7	Do.
Nashua River, South Branch, Mass.	1897–1902	51.32	27.56	53.7	Metropolitan water-works.
Pequannock River, Conn	1891–1899	44.2	26.8	60.6	Rafter.

In this table the large loss by underflow in the Muskingum and Genesee drainage basins is evident.

ESTIMATION OF PERCOLATION FROM CHANGES IN LEVEL OF THE GROUND-WATER TABLE.

The broader and more important fluctuations of the ground-water table are clearly due to the infiltration of water, and attempts have been made repeatedly to estimate

[a] Rept. New York City Commission on Additional Water Supply, 1904, p. [illegible].
[b] Ibid., p. 795.

the amount of infiltration by the rise in the ground water. After having determined the available pore space, which is by no means a simple matter, it appears very easy to calculate the amount of water which will cause the water level to rise a few inches or a few feet. The method is an attractive one, with an appearance of exactness and simplicity, and has often been tried. The results have very little meaning, however, for the very important reason that the same rainfall under the same climatic conditions will produce very different rises in material of the same porosity; for, as pointed out previously (p. 38), the amplitude of the fluctuation increases with the distance from the ground-water discharge. Thus, with material of the same porosity, an annual rainfall of 25 inches produces at Wiener Neustadt a fluctuation in one well of 1 foot, and in another a fluctuation of 25 feet (Pl. IX). It is evident that a calculation of infiltration based on the rise of water produced in well No. 1, will give very different values from that of No. 2, and yet it may be confidently asserted that the same amount of percolation is received in both. Similarly, in the chalk region of England, the fluctuation in the same region ranges from a few inches to 50 or 100 feet. The impossibility of accurately calculating the amount of percolation from the rise of the ground-water table is evident.

REFERENCES RELATING TO WELL FLUCTUATIONS DUE TO RAINFALL AND EVAPORATION.

The bibliography relating to fluctuations of water in wells due to rainfall is naturally very extensive, and an attempt has been made to collect a few only of these titles, important either for their general bearing or their special reference to the United States:

AUCHINCLOSS, W. S. Waters within the Earth and Laws of Rainflow, Philadelphia, 1897.

Gives record of fluctuations in well at Bryn Mawr, Pa., 1886–1895, showing annual and secular fluctuations.

BARBOUR, ERWIN HINCKLEY. Water-Sup. and Irr. Paper No. 29, U. S. Geol. Survey, 1899, p. 28; Nebraska Geol. Survey, Rept. of State Geologist, vol. 1, 1903, p. 106.

States that wells in Nebraska show an annual fluctuation independent of the rainfall, with the maximum occurring in February.

CLUTTERBUCK, JAMES. Observations on the periodic drainage and replenishment of the subterraneous reservoir in the chalk basin of London: Min. Proc. Inst. Civil Eng. [vol. 2], 1842, pp. 155–165; 1843, pp. 156–159.

——— On the periodic alternations and progressive permanent depression of the chalk-water level under London: Min. Proc. Inst. Civil Eng., vol. 9, 1850, pp. 151–180, Pl. VI.

EMERY, FRANK E. Notes on fluctuations in height of water in an unused well: Eighth Ann. Rept. New York Agric. Expt. Station for 1889, 1890, pp. 374–375, fig.

Records monthly observations from December, 1886, to December, 1889, on a 40-foot well at Geneva, N. Y., which shows a single yearly period independent of the rainfall.

FORTIER, SAMUEL. A preliminary report on seepage water and the underflow of rivers: Bull. Utah Expt. Station No 38, 1895.

On p. 30, under heading "Effect of subsurface temperature on rate of flow," are given discharge, temperature, and rainfall at Denver Water Company's plant at Cherry Creek, from 1888 to 1891. This is an infiltration gallery in fine sand 15 feet below the surface, and the discharge shows a rather regular yearly fluctuation with a minimum in February-March and a maximum, normally, in August-November. This fluctuation is ascribed by Fortier to changes in soil temperature. It should be pointed out, however, that, while the annual changes in soil temperature do affect the rate of flow (see p. 59), the yearly maximum is independent of this fluctuation and the agreement here is merely a coincidence.

FREUND, ADOLPH (secretary). Bericht des Ausschusses für die Wasserversorgung Wiens, Österreichischen Ingenieur- und Architekten-Verein, 1895.

Pl. V, Graphische Darstellung der Wasserstände im Steinfelde, 1883–1888.

GERHARDT, P. I. Handbuch der Ingenieurwissenschaften, vol. 3, Der Wasserbau, pt. 1, 1892, pp. 46–51.

Under heading "Schwankungen des Grundwassers," gives a summary based largely on the reports of Soyka.

HEADDEN, WILHELM P. A soil study, pt. 4, The ground water: Bull. Colorado Agric. Expt. Station No. 72, 1902.

Gives data regarding effect of single showers.

KING, FRANKLIN H. Fluctuations in level and rate of movement of ground water: Bull. U. S. Weather Bureau No. 5, 1892, pp. 72–74.

Discusses instantaneous percolation after rains.

KING, FRANKLIN H. Principles and conditions of the movement of ground water: Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, pp. 100–106.

Discusses "Elevation of ground-water surface due to precipitation and percolation," largely from standpoint of porosity.

LIZNAR, JOSEF. Ueber die periodische Änderung des Grundwasserstandes, ein Beitrag zur Quellentheorie: Gæa, vol. 17, 1881, p. 330; Meteorol. Zeitschr., Wien, vol. 17, 1882, pp. 368–371.

LUEGER, OTTO. Die Schwankungen des Grundwassers: Gæa, vol. 24, 1888, p. 630.

MICHIGAN STATE BOARD OF HEALTH. Annual reports, 1878–1903.

Contain monthly observations of water level at many points in Michigan.

SOYKA, ISIDOR. Experimentelles zur Theorie des Grundwasserschwankungen: Vierteljahrsschr. für öff. Gesundheitspflege, vol. 4, 1885, p. 592.

—— Der Boden (Handbuch des Hygiene und der Gewerbekrankheiten, vol. 2, pt. 3). Leipzig, 1887, pp. 251–351.

Contains much of the material incorporated in the following report.

—— Die Schwankungen des Grundwassers mit besonderer Berücksichtigung der mitteleuropäischen Verhältnisse: Penck's Geographische Abhandlungen, vol. 2, pt. 3, Wien, 1888.

SPEAR, WALTER E. Long Island sources: Rept. New York City Commission on Additional Water Supply, appendix 7, 1904, pp. 816–826.

Discusses fluctuation in elevation of ground-water surface.

TODD, JAMES E. Water-Sup. and Irr. Paper No. 34, U. S. Geol. Survey, 1900, p. 29.

The normal yearly maximum is here tentatively referred to the melting of snows or floods.

TRIBUS, L. L. Trans. Am. Soc. Civil Eng., vol. 31, 1894, pp. 170, 391–395.

Reports (p. 170) that in driven wells in New Jersey, 50 feet deep, the effect of rain was not felt in less than thirty hours; gives curve (pp. 391–395) showing fluctuation of water level in a well at Plainfield, N. J., waterworks, 1891–1894. This shows normal annual curves slightly affected by pumping.

WOLDŘICH, JOHANN NEPOMUK. Ueber den Einfluss der atmosphärischen Niederschläge auf das Grundwasser: Zeitschr. Meteorol., Wien, vol. 4, 1869, pp. 273–279.

—— Ueber die Beziehungen der atmosphärischen Niederschläge zum Fluss- und Grundwasserstand: Mitt. d. Techn. Klubs zu Salzburg, pt. 1, 1869.

FLUCTUATIONS DUE TO BAROMETRIC CHANGES.

CHARACTER AND CAUSE.

Changes in air pressure have been observed to affect wells in two ways; in some there is an inflow and outflow of air, and in others a rising and lowering of the water level.[a] The rise or outflow occurs with a falling barometer, and the depression or inflow with a rising barometer. In the case of flowing wells, when the external air pressure decreases, the air within the earth expands, and, as the escape through the soil is greatly retarded by friction and as the well offers a free escape, it finds relief through the well tubing. The power of this blast evidently depends on the area tributary to the well, the loss by friction, and the rate of lowering of the outside pressure. On the other hand, with a rising barometer the external air flows into the pipe to supply the volume lost by the compression of the soil air or earth air. If water is interposed between this included air and the well, the well becomes a rough differential-pressure gage in which the maximum change possible is about 12 inches of water for each mercurial inch of variation in the barometric pressure.

Where there is no soil air suitably confined to produce the result just described the air and other gases in the water so increase its compressibility that a small, though measurable result may be produced by the direct compression and expansion of the water. In the latter case the depth of water involved is an important factor, and it is probably for this reason that on Long Island, where the earth air occurs only in the porous surficial soil, from which it is relatively free to escape, in the wells at Lynbrook fluctuations due to barometric changes are clearly noticeable only in the 504-foot well, and these are relatively small. An examination of Pl. VI shows that there are no indications of barometric influence on the curves from the 72-foot well, but that in the 504-foot well there is a striking resemblance. The similarity is, however, in some places due in part to other causes. Thus the elevations of the water on July 18–19, August 5, and September 16, while closely following the barometric

[a] See in this connection Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, figs. 3, 4, 5; Water Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, fig. 39, p. 73.

curve, are partly due to rainfall. This is indicated by the fact that somewhat similar abrupt barometric depressions on July 26, August 20, and September 4–5, when there were no important rains, did not produce similar elevations of the water in the well. There is a further point of resemblance and dissimilarity between the curve for the 504-foot well and the barometric curve. The well curve shows a very marked semidiurnal fluctuation, the two parts of which are generally of about the same value, although they sometimes merge into a pronounced diurnal wave, as on August 1, 2, and 3. In the barometric curve, although there is a tendency toward a semidiurnal well wave, it is nowhere well marked. The semidiurnal well wave is clearly not tidal. Its resemblance to the barometer curve is, however, sufficiently close to lead to the belief that it is largely barometric, but modified by some other element, perhaps the diurnal temperature wave which shows in the 14- and 72-foot wells (pp. 24–25).

REFERENCES RELATING TO WELL FLUCTUATIONS DUE TO BAROMETRIC CHANGES.

BLOWING WELLS.

BARBOUR, ERWIN HINCKLEY. [Blowing wells in Nebraska]: Water-Sup. and Irr. Paper No. 29, U. S. Geol. Survey, 1889, pp. 78–82; Nebraska Geol. Survey, Rept. of State Geologist, vol. 1, 1903, pp. 93–97.

FARRLEY, T. On the blowing wells near North Allerton: Proc. Yorkshire Geol. and Polyt. Soc., n. s., vol. 7, pt. 3, 1880, pp. 409–421, Pl. VII.

HARRIS, GILBERT DENNISON. [Blowing wells in Rapides Parish, La.]: Water-Sup. and Irr. Paper No. 101, U. S. Geol. Survey, 1904, pp. 60–61: Louisiana Geol. Survey, Bull. No. 1, 1905, pp. 59–60.

LANE, ALFRED C. Water-Sup. and Irr. Paper No. 30, U. S. Geol. Survey, 1899, pp. 55–56.
Records shallow blowing well in Michigan.

VEATCH, A. C. Prof. Paper U. S. Geol. Survey No. 44, 1906, p. 74.
Records reported occurrence of blowing wells on Long Island, New York.

CHANGES IN WATER LEVEL.

ATWELL, JOSEPH. Conjectures on the nature of intermitting and reciprocating springs: Trans. Phil. Soc. London, No. 424, vol. 37, 1732, p. 301; Trans. Phil. Soc. London from 1665–1800 (abridged), vol. 7, 1809, pp. 544–550.
Describes irregular fluctuations of short interval in springs at Brixam, near Torbay, in Devonshire called "Lay well." These, he supposes, are produced by the action of natural siphons. Perhaps they are barometric fluctuations.

DENIZET. Sources sujettées à des variations qui paraissent liées à l'état du baromètre: Comptes rendus, vol. 7, 1839, p. 799.
Reports that springs at Voize are affected by changes in barometric pressure, and that the discharge is directly related to the pressure, instead of inversely, as has been proved by more recent work.

GOUGH, JOHN. Observations on the ebbing and flowing well at Giggleswick, in the West Riding of Yorkshire, with a theory of reciprocating fountains: Jour. Nat. Phil. Chem. Arts, ser. 2, vol. 35, 1813, pp. 178–193; Mem. Phil. Soc. Manchester, n. s., vol. 2, 1813, pp. 354–363.
The water level in this well fluctuates irregularly at short intervals, and Mr. Gough suggests that the fluctuations are produced by the obstruction resulting from the natural accumulation of air bubbles in the outlet and the relief resulting from their irregular escape. He cities the irregular fluctuations of the Weeding well in Derbyshire and the Lay well near Torbay, as proving that the prevailing idea of the production of these phenomena by natural siphons is erroneous. He concludes, very correctly, that too little is known of the fountain of Jupiter in Dodona and Pliny's well in Como to judge of the true cause of the fluctuations mentioned by Pliny.

KING, FRANKLIN H. [Influence of barometric changes on discharge and water level in wells and springs at Madison and Whitewater, Wis.]: Bull. U. S. Weather Bureau No. 5, 1892, pp. 44–53; Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, pp. 73–77.

LATHAM, BALDWIN. On the influence of barometric pressure on the discharge of water from springs: Brit. Assoc. Rept. for 1881, 1882, p. 614; Brit. Assoc. Rept. for 1883, 1884, pp. 495–496.
The fluctuation of the Croydon Bourne, due to barometric pressure, on one occasion exceeded half a million gallons per day. Observations in deep wells are also recorded, showing that fluctuations are inversely related to the pressure. The fluctuations are attributed to the expansion and contraction of the air and gases in the water.

KNIGHTLEY, T. E. Ebbing and flowing wells [in Derbyshire, England]: Geol. Mag., n. s., decade 4, vol. 5, 1898, pp. 333–334.
Suggests that irregularities in flow are due to cavernous limestone, which, by means of natural siphons, gives rise to "intermittent spring phenomena." The possibility of these fluctuations being due to barometric changes is not considered.

LUEGER, OTTO. Einfluss der Atmosphärendruckes auf die Ergiebigkeit von Brunnen und Qu[illegible]. Centralblatt d. Bauverwaltung, 1882, p. 8.

MILNE, JOHN. Seismology. London, 1898, p. 243.

Reports two sinkings and two risings of about 5 millimeters in a shallow well near Toki[illegible]. The sinkings took place between 2 and 6 p. m. and 2 and 5 a. m. Note: These fluctuations sugg[illegible] diurnal barometric wave, but they can be referred to it only if the time given is taken [illegible] the time at which the water commenced to sink and not the time of the low-water stage.

OLIVER, Dr. WILLIAM. Of a well that ebbs and flows . . . : Trans. Phil. Soc. London, No. 2[illegible] 17, 1693, p. 908; Trans. Phil. Soc. London from 1665–1800 (abridged), vol. 3, 1809, pp. 585–586.

Describes well near Torbay called "Lay well," that ebbs and flows from 16 to 20 times per [illegible]. See Atwell, above, and discussion of minor periodic fluctuations on p. 75.

PLINY THE YOUNGER (Caius Plinius Cæcilius Secundus). Epistolæ, lib. 4, epist. ult.

In his letter to Licinius Pliny describes the fluctuations in the discharge of a spring near [illegible] villa in Como, Italy, which he states ebbs and flows thrice a day. He suggests that the fluc[illegible]tions may be due to "the obstruction of air," tidal action, or some secret and unknown [illegible]ance in the nature of a valve. Pliny the elder, in his Historia Naturalis, lib. 2, cap. 106, re[illegible] the same spring, but states that it ebbs and flows every hour—a statement which is [illegible] Catanæus, the learned commentator on the Epistles. From the meager and contradictory [illegible] given it is unsafe to venture a decided opinion on the cause of these fluctuations, but they [illegible] be tentatively regarded as barometric.

ROBERTS, ISAAC. On . . . the variation in atmospheric pressure . . . causing oscillation [illegible] the underground water in porous strata: Rept. Brit. Assoc. for 1883, p. 405.

States that autographic records from a well at Maghill, near Liverpool, show such fluctuat[illegible].

SLICHTER, CHARLES S. Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, pp. 71–72.

Refers to reports of Latham, King, and Lueger.

TODD, JAMES E. Bull. Geol. Survey, South Dakota, No. 2, 1898, p. 116; Water-Sup. and Irr. Paper No. 34, U. S. Geol. Survey, 1900, p. 29.

Reports that well discharge varies inversely with barometic pressure.

FLUCTUATIONS DUE TO TEMPERATURE CHANGES.

OBSERVATIONS AT MADISON, WIS.—FLUCTUATIONS VARYING DIRECTLY WITH THE TEMPERATURE.

In 1888 King observed in certain shallow wells at Madison, Wis., that the wat[illegible] regularly for a portion of the summer months stood higher in the morning than at night.[a] Further observations during the period 1888 to 1892 showed that there was in many wells a diurnal wave, distinctly marked during the summer and dying out in winter, which was clearly not barometric in character, and was not produced by the unequal plant transpiration during the day and night. Suspecting that these changes were intimately related to temperature, King tried the following experiment:

A galvanized-iron cylinder, 6 feet deep and 30 inches in diameter, provided with a bottom, a[illegible] water-tight, was filled with soil, standing its full height above the ground in the open field. In the center of this cylinder and extending to the bottom a column of 5-inch drain tile was placed and the soil filled in about it and packed as thoroughly as practicable. Water was poured into the [illegible] formed by the tile until it was full, and allowed to percolate into the soil so as to saturate it and lea[illegible] the water standing nearly a foot deep in the well. When the water in this artificial well had be[illegible] nearly stationary one of the self-registering instruments was placed upon it.[b] In order to avoid a[illegible] complications due to percolation, the apparatus was provided with a cover which could be put on [illegible] times of rain and removed again during fair weather. The first records showed a small diurnal os[illegible]lation, and as the season advanced these increased in amplitude until finally the water rose in t[illegible] well during the day of July 8, 1.8 inches and fell during the following night 1.84 inches. After the diurnal oscillations had become so pronounced and so constant, a series of thermometers were introduced into the side of the cylinder, extending to different distances from the surface, and a rec[illegible] kept of the changes in the soil temperature; and the result of these observations was to show tha[illegible] turning points in the water curve fell exactly upon the turning points of the temperature of the s[illegible] in the cylinder. When this fact was ascertained, to show whether the correspondence in the tim[illegible] the two curves was due to a diurnal cause, other than temperature, which had its turning poin[illegible] related to those of the temperature as to cause the two to accidentally fall together, cold water w[illegible]

[a] King, Franklin H., Observations and experiments on the fluctuations in the level and rat[illegible] movement of ground water on the Wisconsin Agricultural Experiment Station Farm and at W[illegible] water, Wis.: Bull. U. S. Weather Bureau No. 5, 1892; also Ann. Repts. Wisconsin Agric. Expt. Stat[illegible] 1889–1893.

[b] For a figure of this apparatus see Bull. U. S. Weather Bureau No. 5.

brought from the well and, with a spray pump, applied to the surface of the cylinder all around. The water was applied on a hot sunny day, just after dinner, when the water was rising in the well, and the result was an immediate change in the curve, the water beginning to fall in the well. The water was then turned off, and the result of this change was to stop the fall of the water in the well, as shown by a change in the direction of the curve.

This led to the conclusion that there was a very positive connection between the changes in the soil temperature and changes in the level of the water in the wells, and that the fluctuation varied directly with the temperature; that is, the water in the wells rose with increasing temperature and fell when the temperature lowered. A specially constructed self-recording soil thermometer showed that at a depth of 18 inches the minimum temperature occurred at noon, and the the maximum a little after midnight. It was therefore argued that at a depth of 3 feet, or the level at which the wells were fluctuating, the maximum and minimum temperature would occur still later, and that the high water which occurred in the wells at 8 o'clock in the morning was due to the maximum temperature falling at that time at that depth. The autographic records, moreover, show that the well curves have the same characteristic as the temperature curve—there is in both a comparatively sudden rise and a long fall. King at first believed that these fluctuations were produced by a variation in the capillary action of the soil resulting from the change in temperature,[a] but afterwards concluded that the fluctuations were due not so much to "a change in the viscosity of the ground water as to variations in pressure due to the expansion and contraction of the gas confined in the soil within and above the water."[b]

Changes in capillary attraction and surface tension, due to temperature changes, are quite competent to produce fluctuations which are related to the temperature in the way observed. A rise in temperature, by decreasing the capillary attraction, causes some of the capillary water above the water table to be added to the zone of complete saturation, and so increases the level of the water in wells. Conversely, a decrease of temperature, by increasing the surface tension and capillary attraction, causes water to be transferred from the ground water to the partially saturated zone above it, and so lowers the water in wells. There is, then, a continual interchange, a flux and reflux, between the ground water and the water in the partially saturated zone above it. The amount of water involved in this change is probably small, but, because of the very small amount of unoccupied pore space existing immediately above the zone of saturation, a very slight shifting of water can produce a fluctuation of several inches in the surface of the zone of saturation or the water level in a well. This effect is marked only when the bottom of the well (supposing the well tube to be impervious) is very near the top of the ground-water table; it is not shown in deep wells because, while the position of the ground-water table is constantly changing in this way, there is, so far as deeper points are concerned, no important change in pressure. The total pressure at a given point below the ground-water table is essentially the same whether the water involved is in the saturated zone or 2 or 3 inches above it in an almost saturated layer. To this, more than to the fact that the variations in soil temperature at a given depth are less in winter than in summer, is due the fact that these fluctuations at Madison, Wis., were not shown by the twice-daily observations, made morning and evening between 1888 and 1892, until past or near the middle of July (when the water was nearing its yearly minimum), and from that time increased toward a maximum, occurring sometime in August (probably corresponding with the ground-water minimum), and then died away until the middle of October, when they became inconspicuous. It likewise explains the fact that not all the wells observed, though they were in a limited area, show these fluctuations, and why they begin at different times in adjacent wells of different depths.

[a] Bull. U. S. Weather Bureau No. 5, 1892, pp. 63, 67.
[b] Bull. U. S. Weather Bureau No. 5, 1892, p. 72; Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, pp. 76, 77.

To this relation are due the apparently anomalous phenomena observed in "well No. 5," which King records as follows:

> The ground-water level had fallen until well 5 was likely to become dry. In order not to lose records it was deepened by boring a hole in the center and curbing it with sections of 5-inch drain tile in the manner represented in the figure,[a] which shows the two water surfaces whose fluctuations are recorded in fig. 16. The original well, having an inside diameter of 1 foot and a depth of 5.5 feet, was bricked up to within 2 feet of the surface and then finished with a section of sewer pipe as shown in the cut,[a] where the character and arrangement of the soil through which the well penetrated may also be seen.[b]
>
> The facts are, strange as it does appear, that under these conditons and in such close juxtaposition oscillations so unlike in their character as the two under consideration were produced simultaneously. The level of the water in the outer well oscillated so as to stand in the morning from 0.1 to 0.5 inch above the level of the water in the inner one, and at night from 0.5 inch to 1.2 inches below its surface, and these differences were maintained with only the unglazed section of drain tile separating them. The large oscillations in this well became very pronounced and constant only a short time before it became dry, and the inner well did not take up the marked changes in level after the water fell below the bottom of the original well. No other well of this series, although constructed in the same manner, showed such marked oscillations.

The second suggestion, that the fluctuations are produced by the expansion and contraction of the air due to temperature changes, is not supported by the observed facts. The daily temperature fluctuation at the depths observed amounts to but a fraction of a degree, and the change in volume or vapor tension of the air resulting from this is quite incompetent to produce the fluctuations observed. Moreover, this involves an elevation in the well due to pressure of much the same character as that producing the fluctuations due to barometric changes. The effects of such pressure changes are felt not only at the surface of the zone of complete saturation, or the water table, but are transmitted for many feet below it, and in the case of well No. 5, described above, the fluctuations under such conditions would be shown in both wells, though in the deep one the amplitude would be slightly less.

On the whole there seems to be no other alternative than to regard these Madison fluctuations as the result of variations in the capillary attraction and surface tension of the water above the zone of complete saturation, produced by variations in temperature.

In fluctuations of this character a limiting factor is clearly the range of temperature, which decreases very rapidly with depth, so that at a relatively shallow depth, much less than the limit of no annual change, the fluctuations become imperceptible. The amplitude will, moreover, be affected by the size of the pore spaces, being greater in fine than in coarse material.

OBSERVATIONS AT LYNBROOK, N. Y.—FLUCTUATIONS INVERSELY RELATED TO TEMPERATURE.

Two of the wells at Lynbrook show pronounced fluctuations which are clearly due to temperature changes. These fluctuations, while they resemble those observed at Madison in the fact that the water is higher in the morning than at night, differ from them in two important respects. There is no connection between their occurrence and the relation of the ground-water table to the bottom of the well; they are best developed in a 2-inch well, 14 feet deep, whose bottom is about 13.75 feet below the ground-water level; they are distinctly developed in a well 72 feet deep, and are believed to form one of the elements in a compound curve obtained from the 504-foot well (Pl. VI, p. 24). There is also the further difference that, while these are clearly temperature fluctuations, they are inversely related to the temperature—that is, the water is high when the temperature is low. Avoiding all discussion of the

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[a] Omitted in this report.

[b] The stratification, as shown in the original cut, is as follows:

Section of well at Madison, Wis., which furnished fluctuations shown in fig. 16.

	Feet
1. Loam	[illegible]
2. Clay	[illegible]
3. Sand	[illegible]
4. Clay	[illegible]
5. Sand	2[illegible]

question of lag, this relation is conclusively shown by the shape of the curves. The characteristic of the air temperature curve is a quick rise and slower fall; well fluctuations directly related to the temperature, as those at Madison, therefore must show a quick rise and a slower fall, but in the Lynbrook wells there is a quick fall and slower rise (see Pl. VI, Aug. 1-3). These evidently belong in quite a different class from the temperature fluctuations observed at Madison, and involve quite a different relation between the soil temperature and the ground water.

The soil is a very poor transmitter of heat, and there is not only a very rapid diminution of the temperature range with depth, but a very considerable time lag. Swezey's observations [a] at Lincoln, Nebr., for a period of fourteen years show that while in winter the maximum temperature occurs in the air at about the middle of the afternoon, at a depth of 3 to 6 inches it occurs in the evening, and at 1 foot it is delayed until the following morning; below 1 foot it is scarcely appreciable. In summer the daily range is considerably greater at all depths, the changes are appreciable to a depth of at least 2 feet, and are retarded to about the same extent as in winter.

At Bronx Park, New York City, the record obtained by MacDougal [b] with a Hallock thermograph showed that at 1 foot below the surface the maximum daily temperature occurred between 8 and 11 p. m., and the minimum between 8

[a] Swezey, G. D., Soil temperature at Lincoln, Nebr., 1888 to 1902: Sixteenth Ann. Rept. Nebraska Agric. Expt. Station for 1902, 1903, pp. 95-129; Expt. Station Record, vol. 15, 1904, pp. 460-461.

[b] MacDougal, Daniel Trembly, Soil temperatures and vegetation: Monthly Weather Review, vol. 31, August, 1903, pp. 375-379.

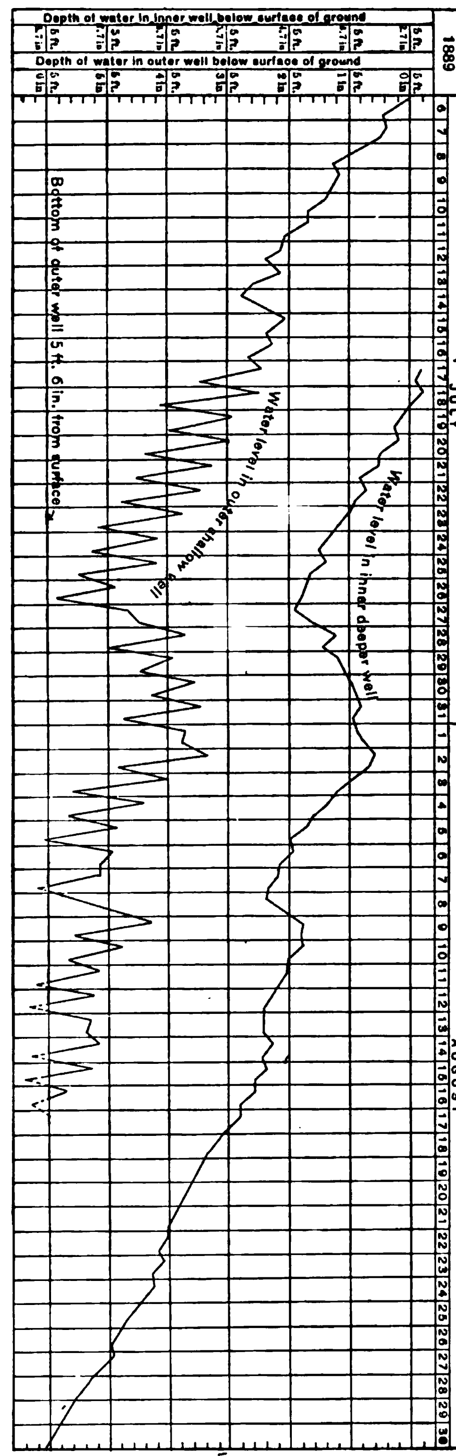

FIG. 16.—Record showing nontransmission of diurnal fluctuations produced by changes in capillary attraction due to temperature changes. Note sudden appearance of diurnal oscillations in large well a short time before it became dry and the absence of such fluctuations in a small well sunk inside of the large one. (Based on the twice-daily observations of King at Madison, Wis.)

and 10 a. m.; and the maximum daily range was but 2° C. (3.6° F.), which was reached on but two occasions. The total annual variation from June 9, 1902, to May 31, 1903, was 16.3° C. (29.4° F.). Similar results have been obtained by Callender at Montreal.[a]

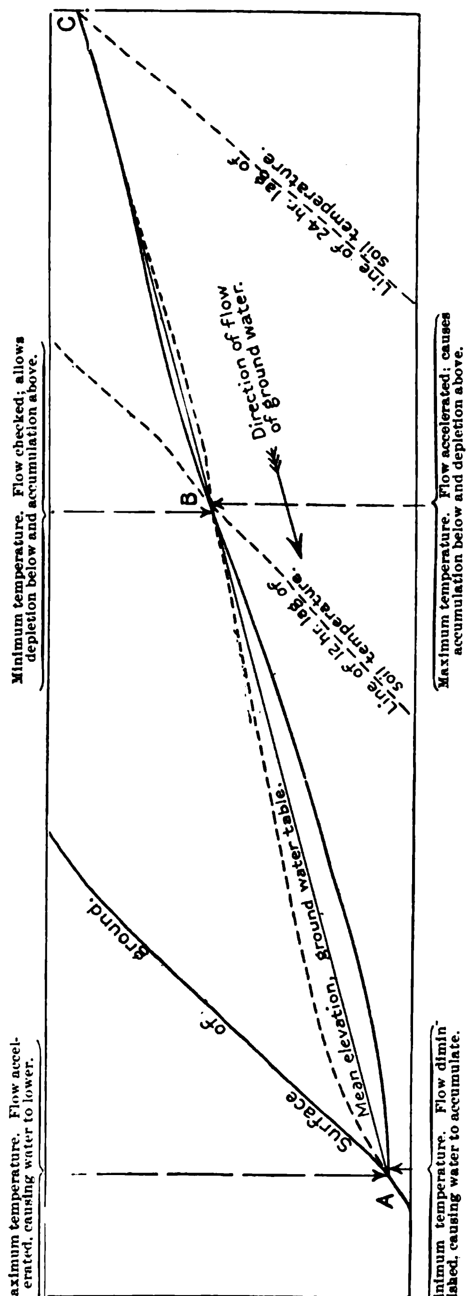

Fig. 17.—Diagram showing production of fluctuations of ground-water level by temperature changes affecting rate of flow. At the time of the maximum temperature at *A*, a minimum occurs at some point *B*, a distance of 1 or 2 feet from the surface. The effect of the increase in temperature is to increase the rate of flow, and of the lowering to decrease the flow. The flow is therefore checked at *B* and accelerated at *A*, and a partial depletion occurs between *A* and *B*, causing low water. When the minimum temperature occurs at *A* and the maximum at *B*, the reverse occurs, causing an elevation of the water. The maximum effect is much nearer *A* than *B*, because of the greater range in temperature, and the fluctuations, therefore, appear to be inversely related to the temperature.

As a result of this slow transmission of temperature, the temperature at the ground-water outlet may be at its maximum while a short distance away, where the water table is but a foot or two from the surface, the ground temperature may be at its minimum. Now, the rate of flow of water is greatly affected by temperature. Poiseuille found that water at a temperature of 45° C. flowed 2.5 times as fast under otherwise like conditions as water at 5° C.[b] This gives rise to the phenomena shown in fig. 17.

There is thus produced a distinct and periodic fluctuation of the ground water, which is great near the ground-water outlet and decreases rapidly in amplitude as the distance from the outcrop increases. The fluctuation is produced by an actual shifting of the water whereby the pressure conditions are constantly changed, and in this respect it differs from the Madison fluctuations (p. 54). It is this change in pressure that causes these fluctuations to show in the other wells, even to a depth of 500 feet. The same phenomenon of response to loading and relief from load is exhibited in some of the rainfall fluctuations described above (p. 42) and in the sympathetic tidal fluctuations (p. 65).

[a] Callender, Hugh L., Proc. and Trans. Royal Soc. Canada for 1895, 2d ser., vol. 1, sec. 3, p. 79, fig. 4.

[b] Quoted by King, Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, p. 82; see also Carpenter, L. C., Seepage and return waters from irrigation: Bull. Colorado Agric. Expt. Station No. 33, 1896, pp. 42–44.

These data suggest that the annual changes of the soil temperature may produce a somewhat analogous effect, the warm summer temperature assisting in the depletion or lowering of the water near the ground-water outlets, and the cold winter temperature, by rendering the water more viscous, retarding the outflow. In one respect the result would be in the same direction as the annual ground-water fluctuations, and this is perhaps to be considered one of the minor factors. It would also tend to make the time of occurrence of the maximum and minimum of the yearly fluctuation earlier near the ground-water outlets than on the divides—or just the condition observed on Long Island (p. 35).

OBSERVATIONS AT SHERLOCK, KANS.

Diurnal fluctuations due to temperature changes were observed by Mr. Henry C. Wolff, at Sherlock, Kans., in 1904, while working under the direction of Prof. C. S. Slichter.[a] Mr. Wolff reports that the wells are low in the evening and high in the morning, and that there is no important time lag between wells where the water line is 6 inches below the surface and those where it is 3 feet below. In a few wells where the water level was about 3 feet from the surface, which were observed long enough to show the shape of the curve, the characteristics of the Madison curve are shown—that is, there is a long fall and sudden rise.

DIURNAL FLUCTUATIONS OF CACHE LA POUDRE RIVER, COLORADO.

Carpenter has observed very regular diurnal changes in the height of Cache la Poudre River near Fort Collins, Colo.[b] Here the high water occurs at from 4 to 6 a. m., the low water at 8 p. m., and the extreme range of the daily change in river level noted was about 1 foot. The curves show the same characteristics as those observed by Wolff at Sherlock, Kans.; there is a long fall and a sudden rise, and they are therefore directly related to the temperature. Carpenter concludes that these fluctuations are due to differences in daily melting in the snow fields, and that the occurrence of the high water in the morning is due to the distance from the snow fields. While this is a very possible explanation, and in the writer's opinion it is probably the correct one, it is desirable to have the matter checked by other observations. If the fluctuations are purely due to daily waves moving down the river, due to melting snow, gages at other points should show the maximum and minimum at different times; if the fluctuations are due to variation in rate of ground-water discharge and are analogous to those described above, the time will be the same at different points.

REFERENCES RELATING TO FLUCTUATIONS PRODUCED BY TEMPERATURE CHANGES.

Besides the references given above to the discussions of King, Slichter, and Wolff, it is desirable to add here the early reference of Pliny to fluctuating wells belonging to this class:

PLINY THE ELDER (Caius Plinius Secundus). Historia Naturalis, lib. 2, cap. 106 [Pliny's Natural History, Bostock and Riley's translation, Bohn's Libraries, vol. 1, 1887, pp. 133–134].

"In the island of Tenedos there is a spring which, after the summer solstice, is full of water from the third hour of the night to the sixth." "The fountain of Jupiter in Dodona . . . always becomes dry at noon, from which circumstance it is called 'The Loiterer.' It then increases and becomes full at midnight, after which it again visibly decreases." Hardouin notes that there is a similar kind of fountain in Provence called "Collis Martiensis." These fluctuations clearly belong to the class produced by temperature changes.

FLUCTUATIONS PRODUCED BY RIVERS.

Rivers may produce fluctuations of the water level in wells in three ways: (1) By changing the height of the ground-water discharge; (2) by seepage or actual contributions to the ground water, and (3) by transmitted pressure or plastic deformation.

[a] The underflow of the Arkansas Valley in western Kansas: Water-Sup. and Irr. Paper No. 153, U. S. Geol. Survey.

[b] Carpenter, L. G., Bull. Colorado Agric. Expt. Station No. 55, 1901, figs. 2–6.

FLUCTUATIONS PRODUCED BY CHANGES IN RATE OF GROUND-WATER DISCHARGE.

In regions where the sides of the channels are pervious and the ground water contributes to the stream flow, the water table, after a period of long drought, slopes regularly down to the water surface. If the stream rises through causes not associated with local ground-water conditions, the ground-water table is found likewise to rise to a greater or less extent. This is accomplished in part by an outflow from the river and in part by the accumulation of the ground-water flow, which can not so readily escape under the new conditions as under the old. If this new level were permanently maintained a complete readjustment would take place, and a line roughly parallel to the initial position of the ground-water table would be developed; and if there were no new outlets developed by this elevation of the ground water, wells at all distances would be similarly affected. Actually, however, the river is constantly changing; a series of waves of unequal height and duration, representing the high and low waters, are constantly traveling down every stream, and no stage lasts long enough for the establishment of a perfectly graded ground-water table, even were there no other factors involved. The result of this unceasing change is that the fluctuations are greatest near the river and become imperceptible at very short distances. This is due not only to the rapidity of the fluctuations in the river, but to the slow rate of outflow and accumulation.

Observations made by Hess[a] along Aller River, near Celle, Germany, in 1866, give the values expressed in the table below:

Table showing lag of high- and low-water stages in wells along Aller River, behind high- and low-water stages in the river.

Test well No.—	Distance from river.	High water, Feb.–Mar., 1866.		Low water, Mar. 7, 1866.		High water, Apr. 1, 1866.	
		High water in well behind high water in river.	Rate per day.	Low water in well behind low water in river.	Rate per day.	High water in well behind high water in river.	Rate per day.
	Meters.	*Days.*	*Meters.*	*Days.*	*Meters.*	*Days.*	*Meters.*
1	47	5	10	2	23.5	4	12.5
2	140	5	28	3	47.0	5	28.0
3	351	17	21	4	88.0	10	35.0
4	468	19	24	7	67.0	......	......
5	584	21	28	......	......	......	......

Observations made by Slichter in very porous gravels in western Kansas, at Garden, Sherlock, and Deerfield, while showing a more rapid transmission than those just indicated, give very marked time lags. The conditions here are different in the respect that the ground water does not materially add to the stream flow, and the rise of the water in the wells is due wholly to seepage. The slope of the water plane is not toward the river, but downstream, at a rate very nearly the same as that of the stream. At Sherlock the water plane on July 27, 1904, sloped gently to the river from test well No. 5, but the water in test wells Nos. 2, 3, and 6 was lower than the river. Between 11 and 4 o'clock the river rose 1.6 feet, and then fell gradually. The beginning of the rise was felt in well No. 3, 400 feet north of the river, in less than two hours; in wells No. 2, 900 feet north of the river, and No. 5, 550 feet south of the river, in between three and four hours; in well No. 4, 1,000 feet south of the

[a] Hess, Beobachtungen über das Grundwassers der norddeutschen Ebene. Zeitschr. des Architekten und Ingenieurvereins in Hannover, vol. 16, 1870, quoted by Soyka, Der Boden, Leipzig, 1887, pp. 262-267, figs. 23-25; and Gerhardt, Der Wasserbau, Leipzig, 1892, pp. 49-50, Pl. I, figs. 7, 8. The diagrams given by Soyka and Gerhardt do not check with the values given in the text, and as the figures are clearly carelessly drawn the text values are reproduced here.

river, in four hours. Well No. 6, 2,500 feet from the river, fell during the whole period of observation. The difference in time in the occurrence of the maximum is expressed in the following table:

Difference in time between high water in Arkansas River and wells on its banks, near Sherlock, Kans., July, 1904.

No. of well.	Distance from river.	Lag.
	Feet.	*Hours.*
3	[a]400	3-6
2	[a]900	12
1	[a]500	36
5	[b]550	108+(?)
4	[b]1,000	108+
6	[b]2,500	(c)

[a] North.
[b] South.
[c] No rise in five days.

It will be noted that the most rapid transmission was toward the north, where the water plane sloped away from the river, while the slight rise of the water plane toward wells Nos. 4 and 5 produced a very marked retardation.

These observations seem to indicate that the rate of transmission is greater when the water plane slopes from the river than when it slopes toward it and help to explain the great retardation observed at Celle. In no instance in this Kansas work were the effects of floods observed in wells at distances of more than one-fourth of a mile from the river.

In case there is open connection between the river and the well, such as might be afforded by limestone caves, changes in level may be felt at considerable distances with but very little time lag. Very rapid fluctuations would, however, be obliterated here, for the well would act very much as the still box used in tidal work, which consists of a large well connected with the ocean by means of a relatively small passage opening at some distance below the water surface. The wave action is entirely obliterated, because the water does not have time to flow in and out of the well in the period between fluctuations. The gradual changes of the tide are, however, exactly recorded. But direct cavernous connections between wells and waterways are rare, and generally river changes act through the interstices of the soil in the way observed at Celle and Sherlock or by transmitted pressure as described below.

FLUCTUATIONS PRODUCED BY IRREGULAR INFILTRATION FROM RIVERS WITH NORMALLY IMPERVIOUS BEDS.

Besides rivers which have pervious sides and into which the ground water is always free to flow or out of which the water flows whenever the river level exceeds that of the ground water, there are many rivers which, under normal conditions, so plaster their beds with fine silt that water is unable to flow either in or out. Nearly all rivers carrying large amounts of fine silt [a] are normally in this condition, and it is thus that the water in many delta regions is able to flow at heights above the surrounding land, and rivers in other regions flow at heights much above the normal ground-water level.

Thus the Leitha at Katzelsdorf, near Wiener Neustadt, flows at a height of 10 to 70 feet above the level of the ground water at that place, the height depending on

[a] For an example of the silting power of a clear stream see Freeman, John R., Percolation through Embankments and the Natural Closing of Leaks, Boston Soc. Civil Eng., June 20, 1888.

the stage of the ground water, although there is at all times considerable seepage. (Pl. IX.)

The Rio Grande in a similar way flows from central New Mexico to the sea, always above the ground-water level.

When the rivers silt up their beds, where the water plane slopes down to the river and there is a tendency to discharge into the river, the silting develops an artesian head which causes the water to rise above the level of the river in wells sunk within the channel. Such occurrences have been reported by Salbach in the Elbe.[a] This Elbe occurrence may, however, be due to a sheet of clay not connected with the river-silt deposits of the present régime. During floods such rivers frequently scour out their beds and establish a connection between the surface and the underground waters. When the river is above the ground-water table, this leakage will raise the water level; when the reverse is the case, it may, by permitting a discharge of the artesian water, cause the water level in a near-by well to lower.

Slichter has found at Mesilla Park, N. Mex., that the greater part of the underflow in the valley is derived in this way from the flood waters of the Rio Grande.[b] Preceding the flood of October 5, 1904, the ground-water level was several feet below the river channel, although the river contained considerable water. The effect of the flood was well marked at well No. 7, three-fourths of a mile from the river, but did not affect well No. 6, 1.3 miles from the river, in seventeen days. The rate of transmission is evidently quite similar in this case to those given above.

Quite different from these slow rates of change in the ground-water level due to river changes are the changes in the London wells ascribed by Clutterbuck and Buckland[c] to floods in Colne River at Watford, Hertfordshire. According to these observers a rise in the Colne produces a rise in the London wells, 15 miles distant, in a few hours. These floods are due to heavy rains and it seems much more probable that the observed rise is due to the weight of the rain on the local London area than to the transmitted pressure from a flood 15 miles distant. Fluctuations of this character due to rainfall have been observed at Lynbrook, N. Y. (p. 42), and no such rates of lateral transmission have been observed either from the river flood or tides, with the possible exception of the tidal wells at Lille, France (p. 64.) Certainly there is no evidence of such great underground caverns between Lille and the sea as this rate of transmission would require, though its very occurrence, if conclusively proven, would indicate some such connection.

FLUCTUATIONS DUE TO A PLASTIC DEFORMATION PRODUCED BY VARYING VOLUMES OF WATER CARRIED BY RIVERS.

The alternations of load due to the irregular waves, whose crests are the high- and low-water stages, which are constantly passing down every river, produce fluctuations anagalous to those produced by tides, though lacking their periodic character. They resemble also the sympathetic fluctuations produced in the 72- and 504-foot wells at Lynbrook due to variations in load produced by rainfall and thermometric changes (pp. 43, 58). The zone in which these fluctuations will be distinctly recognizable will be limited to a mile or two in the immediate vicinity of the river.

REFERENCES RELATING TO FLUCTUATIONS OF THE WATER IN WELLS PRODUCED BY RIVERS.

BUCKLAND, DR. Min. Proc. Inst. Civil Eng. [vol. 2], 1842, p. 159.

Reports that wells in London rise in a few hours after floods at Watford, 15 miles distant.

CLUTTERBUCK, JAMES. Observations on the periodical drainage and replenishment of the subterraneous reservoir in the chalk basin of London: Min. Proc. Inst. Civil Eng. [vol. 2], 1842, pp. 157, 158; 1843, p. 162.

Same as under Buckland.

[a] Salbach (Baurath at Dresden, Saxony), Experiences had during the last twenty-five years with waterworks having an underground source of supply. Trans. Am. Soc. Civil Eng., vol. 30, 1893, p. 314.

[b] Slichter, Charles S., Observations on the ground waters of Rio Grande Valley: Water-Sup. and Irr. Paper No. 141, U. S. Geol. Survey, 1905, p. 27.

[c] Min. Proc. Inst. Civil Eng., 1842, pp. 158, 159, 1843, p. 162.

FULLER, MYRON L. Notes on the hydrology of Cuba: Water-Sup. and Irr. Paper No. 110, U.S. Geol. Survey, 1905.

Records, on p. 189, fluctuations of spring level at Vento, Cuba, due to changes in level of Almendares River, evidently acting through a free connection of large size in the limestone.

HARRIS, GILBERT D. Underground waters of south Louisiana: Water-Sup. and Irr. Paper No. 101, U.S. Geol. Survey, 1904, p. 14; Bull. Louisiana Geol. Survey No. 1, 1905, p. 4.

Discusses effect of Mississippi River on level of wells along its banks.

GERHARDT, P. Der Wasserbau, vol. 1, pt. 1, 3d ed., 1892, pp. 48–49.

Gives Hess's observations on Aller River.

SLICHTER, CHARLES S. Observations on the ground waters of Rio Grand Valley: Water-Sup. and Irr. Paper No. 141, U.S. Geol. Survey, 1905, pp. 18, 25–28, 30.

Gives observations on effect of outflow from river on ground-water level at El Paso, Mesilla Park, and Berino.

——— The underflow of the Arkansas Valley in western Kansas: Water-Sup. and Irr. Paper No. 153, U.S. Geol. Survey.

Gives results of observations on the influence of floods in Arkansas River on the water level in wells at Garden, Sherlock, and Deerfield, Kans., in the summer of 1901.

SOYKA, ISIDOR. Die Schwankungen des Grundwassers: Penck's Geographische Abhandlungen, vol. 2, pt. 3, 1888.

Chapter 3, "Die Beziehungen des Grundwassers zu den oberirdischen Wasserläufen," contains an excellent discussion of middle European conditions.

THOMASSAY, RAYMOND. Géologie pratique de la Louisiane, 1860.

Contains an entirely fanciful discussion of the seepage of the water from Mississippi River.

TODD, JAMES E. Geology and water resources of a portion of southeastern South Dakota: Water-Sup. and Irr. Paper No. 34, U.S. Geol. Survey, 1900, p. 29; Bull. South Dakota Geol. Survey No. 2, 1898, p. 116.

Records that many deep wells have a greater discharge when Missouri River is high, and suggests that the increased hydrostatic pressure checks the leakage.

VEATCH, A. C. Geology and underground water resources of northern Louisiana and southern Arkansas: Prof. Paper U. S. Geol. Survey No. 46.

Records fluctuations of water level in artesian wells at Fulton, Ark., agreeing with stages of Red River. These are ascribed to pressure of river water acting at the outcrop of the water-bearing sands in river bed several miles from the town. This probably is a case of transmitted pressure, the blue clay over the water-bearing layer acting as a diaphragm and producing fluctuations in wells near the river.

FLUCTUATIONS PRODUCED BY CHANGES IN LAKE LEVEL.

Variations in lake levels of whatever cause produce fluctuations of the level in wells along their shores (1) by checking the rate of outflow when the ground water is draining freely into the lake and (2) by transmitted pressure and deformation as in ocean tides described below. The pressure of deep-seated waters might also be slowly affected by the weight of the sediment deposited in the lake beds. This would tend to equalize itself by back flow, and is perhaps a factor of no great importance, except when considered for very long periods. King has, however, obtained a flow artificially by ordinary sedimentation in a tank.[a]

FLUCTUATIONS PRODUCED BY VARIATIONS IN THE OCEAN LEVEL—TIDAL WELLS.

As partially indicated in the references on page 67, wells and springs which fluctuate with the tide have been observed on nearly all coasts and under many different geologic conditions. These fluctuations are produced in three ways: (1) By transmission of pressure through open cavities or passageways affording a free communication between the wells and the ocean; (2) by a checking of the rate of discharge of the normal ground-water flow through porous beds freely connecting with the ocean; and (3) by a deformation of the strata due to the alternate loading and unloading of the tides. In this last case, instead of leakage being an important factor, as it is in the first two, the fluctuations are greater the more nearly complete the separation of the oceanic and ground waters.

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[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, pp. 79–80.

FLUCTUATIONS PRODUCED BY CHANGES IN RATE OF OUTFLOW.

The first two classes differ more in the rate at which the change takes place and the character of the zone of influence than in the manner in which it is produced. If the ocean level is raised the first effect is to check the velocity of outflow; but before any change occurs in the level of near-by wells it is necessary that water accumulate at the point of observation by actually flowing in. This change, then, is clearly dependent on the same factors which influence the rate of flow, and in underground caverns where the velocity is a question of miles per day this accumulation will be rapid, there will be a relatively short lag, and the distance from the shore to which the rise can be propagated before the water begins to fall will be comparatively great. Its influence will, however, be restricted to wells along limited lines, following the course of the underground passageway. On the other hand, when the water is flowing through the interstices of porous strata, where the motion is one of feet per day instead of miles, the accumulation will be slow, the lag proportionately greater, and the zone of influence, while not extending so far from the ocean, will perhaps occupy a larger area, because of its uniform distribution along the coast. When the ocean level falls the reverse will occur.

Where there is considerable velocity, as in a cavernous opening, the velocity of outflow retards the effect of the rising tide and hastens that of the falling tide and there is then, as in tidal rivers, a greater lag at low than at high water. When the outflow is slow, as from porous beds, the velocity is not sufficient to exert a very great retarding influence.

The fluctuations in porous materials along the seashore are clearly the same in character and cause as those occurring under similar conditions along river courses (see p. 60), and the same great time lag is to be expected. The difference is only in the very regular periodic nature of the oceanic fluctuations. Nearly all the shallow tidal wells noticed along the seacoasts belong to this class. Such are clearly the tidal wells[a] reported near Bombay and along the Malabar coast; at Barren Island, in the Andaman Sea; at Perim Island, in the Red Sea; at South Foreland light-house, Kent; the shallow wells at Seagirt, N. J.; and perhaps the wells at Newton Nottage, Glamorganshire, Wales, and Chepstow, Monmouth, England. At Perim Island, in shallow pits at a distance of 60 to 300 feet from the shore, the lag is such that the high water in the wells appears to agree with the low water in the ocean. A similar lag is reported in a well 14 feet deep and 400 feet from the river at Chepstow. At Newton Nottage, where a shallow well 500 feet from the ocean was observed by Mandan, the lag is but three hours.

Experience has shown that wells which are sunk entirely in porous beds near the seacoast should have their bottoms about midway between high and low tide; if they are deeper there is generally an infiltration of salt water. Such wells are commonly dry at low tide, but frequently furnish good supplies of fresh water at high tide, at which time it is necessary to obtain all the water used for domestic and other purposes.

Wells dependent on underground cavernous openings, such as required by the first case, are quite rare. The fluctuations of the Iceland springs, reported by Hallan de Roucroy, are perhaps connected with such open fissures, though any opinion from the data presented is but a hazard. Perhaps the most remarkable occurrence of tidal fluctuation is that reported at Lille in an artesian well at the citadel. These fluctuations, which amount to 0.415 meter, are referred by Bailly[b] to the tides of the English Channel, 30 miles away, with a time lag of but eight hours. The only basis on which these fluctuations can now be explained is a supposition of a relatively open connection between the well and the ocean, which, it must be admitted, is a very unsatisfactory hypothesis.

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[a] See references, pp. 67-69. [b] Comptes rendus, vol. 14, 1842, pp. 310-314.

TIDAL FLUCTUATIONS IN WELLS PRODUCED BY PLASTIC DEFORMATION.

Besides the shallow wells, depending on ordinary porous surficial beds, there are, along and near the seacoast, many deep artesian wells which show tidal fluctuations. In many of these wells there are clearly no underground caverns involved, the water-bearing beds being ordinary porous strata in which the water flows through the small interstices at a rate to be expressed in feet rather than miles per day, and in which accumulation or depletion by simple flowage will be correspondingly slow. There is, moreover, every reason to believe, in some cases where there are thick clay beds above the water-bearing strata, which are known to be continuous for many miles, that there are no near suboceanic outlets of importance. In case there is some distant outlet it is evident from the slow rate of change shown in the examples given above, where there was a sudden increase or decrease in the volume of river water (pp. 60, 61), that the fluctuation produced by a simple checking or hastening of the rate of outflow could be propagated but a short distance, and that a long period of time would be necessary for even that.

There is, however, in the case of waters under artesian head a new factor introduced which is of very great importance. The pressure of the artesian water exerted against the retaining cover, which may be assumed at present to be clay, tends to elevate it, thus placing the clay under an upward stress. The addition of any weight on the surface tends to disturb the equilibrium. If there is no outlet and the weight is applied uniformly, the additional weight can not change the position of any portion of the mass, except to the very slight degree of the elastic compressibility of the water and the soil. If, however, there is any escape for the confined water, such as would be afforded by a well tube, the mass will yield and the water be forced up in the tube. Were the clay layer perfectly elastic, or in the condition of a stretched elastic membrane above a perfectly mobile body, there would be no time lag, and the water in the well would exactly follow the fluctuations of the ocean waters; but the clay is not to be regarded as an elastic diaphragm, and the water-bearing sand is not a perfectly mobile body; moreover, for such a deformation to be felt in a well water must be transferred from one point to another, and this involves a time element. The deformation is essentially a plastic one; the clay yields to the superposed weight and the water is lifted in the well, but if there were no pressure from below the clay could not return to its original position. In the case of tides along the coast only the portion of the clay layer under the ocean is loaded, and that loading is a progressive one from a distant point toward the shore. The effect is a deformation in which the clay layer is depressed under the ocean and elevated under the land. When the weight is removed the artesian pressure tends to reestablish the old conditions of equilibrum, and the clay layer is lifted under the ocean and sinks under the land.

If the artesian pressure is high, compared with the tide when the ocean water commences to fall after high tide, this pressure lifts the clay quickly and thus tends to shorten the high-tide lag in a near-by well; as the tide falls the high pressure enables the clay to follow the tide closely; at low tide the artesian pressure is clearly in the ascendancy and the clay still rising in the ocean area. As the tide begins to rise it must overcome this artesian pressure before any deformation occurs, and the rising curve in the well therefore lags more behind the tide than the falling curve. Under such conditions the high-tide lag is less than the low-tide lag. Conversely, when the artesian pressure is low compared with the tide, at high tide the feeble artesian pressure but slowly lifts the clay weight and the lag is long; at low tide, when the clay diaphragm is high, the greater tidal weight quickly overcomes the feeble resistance of the artesian water and the lag is short; it may then happen that the low-tide lag will be less than the high-tide lag.[a] It is evident that between the two extremes thus

[a] This plastic deformation should not be confused with the elastic deformation of the earth which Darwin has considered in his calculations of the effect of tides on seacoasts. He assumes that the

indicated there are all possible variations, and that the thickness and plasticity of the beds above the water-bearing layers are important modifying factors. The fluctuation in a well in such cases does not furnish an exact measure of the amount of deformation; it furnishes only a fair indication of the variation in pressure at the particular point at which the well is sunk.

The maximum effect is felt at the seacoast near low-tide mark and gradually decreases inward, disappearing in a few miles. It is less if there is leakage from sub-ocean springs, for in such cases the escape of the water decreases that available for the elevation of the water in the tube.

On Long Island the tidal fluctuations observed in the wells at Huntington, Oyster Bay, Long Beach, and Douglaston are clearly of this character, all depending more on the deformation of the overlying layer through tidal load than on changes of discharge in leakage. At Huntington (p. 10), Oyster Bay (p. 13), and Douglaston (p. 25) the lag is greater at low than at high tide, as would be expected from the great head and shallow depths, while at Long Beach, where the head is low, the water-bearing sand fine, and the thickness of overlying strata great, the reverse is true (p. 19). The Oyster Bay observations give the following values:

Summary of observations on tidal wells at Oyster Bay, N. Y.

Well.	Depth.	Distance from ordinary high tide.	Low-water lag.	High-water lag.
	Feet.	*Feet.*	*Minutes.*	*Minutes.*
Casino	93	In water.	12.6	[illegible]
Burgess	155	50	33.4	24.7
Lee	188	100	58.0	42.0
Underhill	114	500	75.6	[illegible]

It will be noticed that the lag here increased with the distance from the shore, and that the low-water lag increased more rapidly with depth than the high-water lag. The cause of the very small difference between the high- and low-water lags in the Underhill well, the one farthest from the shore, is not clear, but it is apparently related to the lessening of the tidal influence with the increasing distance.

The Long Beach well is affected both by the tide in the ocean and in the channels behind it, the curve being, as would be expected, a simple resultant of the two stresses. Because of the shallow bars at the openings the irregularity in the height of the inner low tide is less than in the ocean, and the effect of this difference is shown in the greater regularity of the low-tide heights in the well than in the ocean (Pl. IV, p. 20). Here the high-tide lag is one hour and nineteen minutes and the low-tide lag forty minutes, when compared with the ocean, while compared with the tide behind the bar the high-tide lag is practically nothing and the low-tide lag is nearly two hours.

To this same class belong nearly all the deep artesian wells along the seacoast which fluctuate with the tide. The phenomenon observed is the result of actual deformation, and the occurrence of tidal fluctuations in deep wells does not, as has been commonly supposed, prove a connection between the water-bearing strata and the ocean. Examples of this type are afforded by the deep wells along the New Jer-

earth has an elasticity equal to twice that of the stiffest glass, and the elastic compression produced by loading a sphere of such material of the same size as the earth with a tide of 5 feet is calculated on the supposition that the ocean is in the shape of a narrow canal. According to this the tides of the Atlantic coast may cause the land to rise and fall as much as 5 inches. See Darwin, G. H., on variations in the vertical due to elasticity of the earth's surface: Brit. Assoc. Rept., 1882, p. 388. Phil. Mag., 5th ser., vol. 14, 1882, pp. 409–427. The Tides, Boston and New York, 1898, pp. 139–143, quoted by Milne, J., Nature, vol. 38, 1883, p. 367, Seismology, London, 1898, pp. 236–237.

sey coast, the wells at Pensacola, Fla., the deep wells at Greenwich Hospital, London, and the wells along the Lincolnshire and Yorkshire coasts.

Shelford[a] has presented a very clear diagram of the conditions on the Lincolnshire coast. This shows overflow springs occurring at the top of a porous layer and the base of an impervious one, and the relation between ordinary overflowing and tidal wells, all depending on the same strata. Here, as is almost universally the case, the tidal wells occur only on the shore, and the wells 2 and 3 miles inland are not affected. In explaining the phenomenon Shelford supposed that the water found an outlet in Silver Pit, a deep hole in the ocean bed about 18 miles from the coast, which he has represented in his drawing as the ground-water outlet, and that changes in level in the discharge produced the tidal fluctuations. Such a simple change in the rate of discharge could affect the wells 18 miles distant only if there were a large open cavernous connection. That there is no such cavern is shown by the fact that the effect diminishes very rapidly in passing inland, entirely dying out in 2 miles. There is no reason why the effect should be propagated 18 miles to the coast and then suddenly cease, when tidal wells of the same character penetrating a thick clay bed and obtaining water in the upper porous layer of the chalk occur along the whole Lincolnshire and Yorkshire coast. In many cases springs near the coast, deriving their supply from the water beneath the clay, are likewise tidal. The cause of this phenomenon in the Bridlington Quay wells, Yorkshire, was correctly given by Inglis in 1817. He recognized in the clay layer a moving diaphragm affected by the tidal pressure from above and the artesian pressure from below.

REFERENCES RELATING TO TIDAL FLUCTUATIONS IN WELLS AND SPRINGS.

ANONYMOUS. London Athenæum, August, 1860; Jour. Franklin Inst., vol. 72, 1861, pp. 309–310.

States that tides in wells near the sea are universal, and records their occurrence about Bombay and along the Malabar coast wherever the material dug through is porous. Wells dug in trap rock are not tidal.

BAILLY. Rapport sur les variations observées dans la dépense du puits artésien de l'hôpital militaire de Lille et dans les hauteurs de la colonne d'eau quand on a interrompu l'écoulement: Comptes rendus, vol. 14, 1842, pp. 310–314.

Gives observations showing that fluctuations, having a range of 0.415 meter, are clearly tidal and occur eight hours behind the tide on the adjacent coast between Dunkerque and Calais. Reports that tidal wells also occur at Noyelle-sur-Mer, Département de la Somme, and at Fulham, London, England.

BRAITHWAITE, FREDRICK. Min. Proc. Inst. Civil Eng., vol. 9, 1850, p. 168; vol. 14, 1855, pp. 507–522.

"At Greenwich Hospital, London, the land springs ebb and flow 2 feet 6 inches, the sand springs, 3 feet, and the chalk springs, 4 feet 6 inches every tide." The total depth of the chalk well referred to is 149 feet.

CHRISTIE, JAMES. Jour. Franklin Institute, vol. 101, 1901, p. 193.

Reports fresh-water well near shore which fluctuates with the tide.

CLUTTERBUCK, JAMES. Min. Proc. Inst. Civil Eng., vol. 9, 1850, p. 170.

Explains tidal fluctuation in wells on basis of leakage between high- and low-tide marks.

——— Min. Proc. Inst. Civil Eng., vol. 14, 1855, pp. 510–511.

Wells at Ramsgate, England, are sunk to half-tide level. These begin to fall at half tide, are dry at low tide, and begin to rise at half tide on the flood.

——— Min. Proc. Inst. Civil Eng., vol. 19, 1860, p. 33.

Reports that wells at Portsmouth, England, are tidal, and concludes that this proves a free connection with the sea.

DESAGULIERS, Rev. J. T. An attempt to account for the rising and falling of the water of some ponds near the sea, etc. Trans. Phil. Soc. London, No. 384, vol. 33, 1724, p. 132; Trans. Phil. Soc. London from 1665–1800 (abridged), vol. 7, 1809, pp. 39–41.

Reports well at Greenhithe in Kent, between London and Gravesend, which appears to fluctuate inversely with the tide. This he explains by imagining a natural siphon.

DOUGLAS, JAMES NICHOLS. Min. Proc. Inst. Civil Eng., vol. 47, 1879, p. 88.

Chalk well at South Foreland light-house, Kent, England, 283 feet from face of cliff, 280 feet deep, with bottom level with half tide, has a peculiarity common to many wells of this region in that it is dry at low tide and filled with pure spring water at high tide.

[a] Shelford, W., Min. Proc. Inst. Civil Eng., vol. 90, 1887, p. 69.

FRAZER, PERSIFOR. Notes on fresh-water wells of the Atlantic beach: Jour. Franklin Inst., 1890, vol. 130, p. 231.

Reports well at Sea Girt, N. J., 20 feet deep, which rises and falls with tide in ocean 150 feet distant.

HALLAN DE ROUCROY. Comptes rendus, vol. 12, 1841, pp. 1000–1001.

States that well at Lille, France, shows tidal fluctuations.

HUTTON, Capt. F. W. Trans. and Proc. New Zealand Inst., 1895, vol. 28, 1896, p. 655.

States that artesian wells at New Brighton are affected by the tide.

INGLIS, GAVIN. On the cause of ebbing and flowing springs [at Bridlington, Yorkshire]: Phil. Mag., vol. 50, 1817, pp. 81–83.

"When the recess of the ocean lessens the pressure upon the upper surface, the hydraulic pressure on the under stratum must raise the whole mass in proportion as the force is superior to the resistance. The return of the tide brings with it the weight and altitude of its mass of water and acts on the flexibility of the clay as a pressure would on a hydraulic blowpipe."

KING, FRANKLIN H. Fluctuations in the level and rate of movement of ground water: Bull. U. S. Weather Bureau No. 5, 1892, pp. 52–53.

Suggests that tidal fluctuations may be produced in wells by coastal deformation.

MANDAN, H. G. Note on ebbing and flowing well at Newton Nottage [Glamorganshire, Wales]: Abs. Proc. Geol. Soc. London, 1898, pp. 85–86; Nature, vol. 58, 1898, pp. 45–46.

Shallow well 500 yards from shore ebbs and flows with the tide; lag about three hours.

MALLET, F. R. Ebbing and flowing wells: Nature, vol. 58, 1898, p. 104.

Shallow wells in volcanic ash on Barren Island, Andaman Sea, show tidal fluctuations clearly due to retardation of leakage.

McCALLIE, S. W. A preliminary report on the artesian-well systems of Georgia: Bull. Geol. Survey Georgia No. 7, 1898, p. 112.

Reports three artesian wells at Tybee Island, near Savannah, Ga., 240 feet deep, one of which is affected by the tide.

MOORE, H. C. A well intermitting inversely with the ebb and flow of the tide: Trans. Woolhope Naturalists Field Club, 1892, pp. 23–24; Jour. Manchester Geol. Soc., vol. 10, 1894, pp. 223–224.

Well at Chepstow, Monmouth, fluctuates inversely with the tide. Shallow pits on Perim Island, Straits of Bab el Mandeb, Red Sea, 20 to 100 yards from shore, are full of fresh water at low tide, empty at high tide; explained on basis of time required for filtration.

PEARSON, Rev. W. Observations on some remarkable wells near the seacoast at Brighthelmstone and other places contiguous. Jour. Nat. Phil. Chem. Arts, vol. 3, 1802, pp. 65–69.

States that shallow wells at Brighton fluctuate with the tide, but with a lag of two hours. He ascribes the fluctuation to retardation of leakage.

PLINY THE ELDER. (Caius Plinius Secundus.) Historia Naturalis, lib. 2, cap. 106: (Pliny's Natural History, Bostock and Riley's translation, vol. 1, 1887, pp. 134–135).

"There is a small island in the sea opposite the river Timavus, containing warm springs which increase and decrease at the same time with the tide of the sea."

RIVIÈRE. Comptes rendus, vol. 9, 1839, p. 553.

Spring at Givre, canton Montiers-les-Maux, fluctuates with tide.

ROBERT, E. Comptes rendus, vol. 14, 1842, pp. 417–418.

Reports that springs near Buder, Olafsen, and Paulsen, Iceland, ebb and flow with the tide.

ROBERTS, ISAAC. On the attractive influence of the sun and moon causing tides . . . in the underground water in porous strata: Rept. Brit. Assoc., 1883, p. 405; see also Proc. Liverpool Geol. Soc., vol. 4, pt. 3, 1881, pp. 233–236.

Reports that in a well sunk in Triassic sandstone in which the water rose 60 feet above sea level, autographic records showed solar and lunar tides. (See p. 69.)

SHELFORD, W. Min. Proc. Inst. Civil Eng., vol. 90, 1887, p. 68.

Describes wells 200 feet deep, on the North Sea, near Louth, Lincolnshire, which fluctuate 3 feet with spring tides.

SINCLAIR, W. F. Ebbing and flowing wells: Nature, vol. 58, 1898, p. 52.

Describes well at Alibag, near Bombay, in sand dunes about 25 yards from high-tide mark, which fluctuated with the tide after heavy rains when the ground water level was high. Tide in well occurred later than that in ocean.

STORER, Dr. JOHN. On an ebbing and flowing stream discovered by boring in the harbor of Bridlington [Yorkshire]: Phil. Trans., vol. 105, pt. 1, 1815, pp. 54–59; Phil. Mag., vol. 45, 1815, pp. 432–436.

S., W. On ebbing and flowing springs: Phil. Mag., vol. 50, 1817, p. 267.

States that wells near Hull under conditions similar to those at Bridlington are not tidal.

TRAUTWINE, J. C., jr. Jour. Franklin Inst., vol. 51, 1901, pp. 193–194.

Explains tidal wells on basis of free discharge in ocean, as from an open tube; changes in pressure at discharge change water level in wells as if they were piezometers along a conduit.

TRIBUS, L. L. Trans. Am. Soc. Civil Eng., vol, 30, 1893, p. 695.

Mentions tidal fluctuations in wells at Pensacola, Fla., 1½ miles from the shore front, 4 to 6 inches in diameter, and from 60 to 280 feet deep. Water rises 16 to 17 feet above sea level and fluctuates 6 to 10 inches daily with the tide. He supposes, therefore, that they tap subterranean rivers which have free connection with the ocean. Note: The tides at Pensacola are rather irregular, with a small semidiurnal and large diurnal value, and it is quite possible that a portion of the fluctuation observed is due to barometric and thermometric changes.

VERMEULE, C. C. Water supply for wells: Ann. Rept. New Jersey Geol. Survey for 1898, 1899, p. 163.
States that many wells along the coast of New Jersey show tides corresponding in period, but not in time of occurrence, with the tides of the ocean, and with a smaller range.

WOOD, JAMES G. Jour. Manchester Geog. Soc., vol. 10, 1894, pp. 237-239; Abs. Proc. Geol. Soc. London, 1896, p. 86.
Reports well near that described by H. C. Moore (see above), and suggests that well is fed by water coming along fault, which passes under the river; that at high tide this fault is closed, cutting off supply, and at low tide opens again, allowing an influx; and that therefore well fluctuates inversely with tide. (Note: A simple leakage would, on account of slow propagation of change, explain the phenomena quite as well, and more naturally.)

WOOLMAN, LEWIS. Artesian wells in New Jersey: Ann. Rept. New Jersey Geol. Survey for 1898, 1899, pp. 76, 78, 79.
Records that the height of water in many artesian wells along the New Jersey coast fluctuates with the tide. At Ventor fluctuations were noted in a well 813 feet deep, which had a range of 7¾ to 14¼ inches, and a lag of approximately forty-five minutes. Similarly at Avalon, in a well 925 feet deep, the fluctuation observed had a range of from 10¼ to 15¾ inches.

YOUNG, Rev. G. and J. BIRD. A Geological Survey of the Yorkshire Coast. 4°. Whitby, 1822; 2d ed., 1828.
Ebbing and flowing springs, Bridlington, pp. 22-24; intermittent springs, pp. 27-28.

TIDES IN THE GROUND WATER PRODUCED BY DIRECT SOLAR AND LUNAR ATTRACTION.

The ground water has not an extended level surface like the ocean, where the tides range from nothing to 50 feet, or even the Great Lakes, where the tidal fluctuation is but a few inches. The ground-water table is comparatively level only over areas which are but a fraction of the size of the Great Lakes, and direct ground-water tides would be of extremely small size. It seems quite unlikely, therefore, that the fluctuations in the Maghull (Liverpool) well are due to direct solar and lunar attraction, as Roberts [a] suggests, but, as King [b] has already pointed out, are rather to be ascribed to the action of the ocean tides on the near-by coast.

FLUCTUATIONS DUE TO GEOLOGIC CAUSES.

In regions of abundant rainfall the ground-water table is but a subdued reflection of the surface topography, and any changes in the topography will therefore change the position of the ground-water table. If a stream valley is filled by sedimentation, the ground water is raised over the whole tributary area up to the ground-water divide; if the stream valley is eroded, the water level is in like manner lowered. Similarly, if a lake is produced by a landslip or destroyed by the erosion of its outlet, or the ocean level is changed by orographic movements, the ground-water table likewise is changed. To these broad generalizations certain exceptions are to be mentioned. If a river is intrenched in an impervious layer overlain by porous strata, it is evident that the position of the impervious bed in the bank, when the water level in the river is below it, is the factor which determines the position of the ground-water table. A stream may thus lower its bed without affecting the adjacent ground water. Examples of this kind are found in the Isar at Munich and the Salzach at Salzburg, both of which have deepened their beds in recent times, due to regulating works, without lowering the adjacent ground water, because the deepening was entirely in impervious material. [c]

Solution and deposition by percolating waters may cause a gradual depression or elevation of the water level; solution, by increasing the porosity and consequent rate of flow, will enable quite a quantity of water to escape along certain lines and so lower the water level; deposition, in an opposite way, will raise it.

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[a] Roberts, Isaac, Rept. Brit. Assoc. 1883, 1884, p. 405.
[b] King, Franklin H., Bull. U. S. Weather Bureau No. 5, 1892, p. 54.
[c] Soyka, Penck's Geographische Abhandlungen, vol. 2, pt. 3, Wien, 1888, pp. 60, 63.

In regions where the rainfall is low and the ground-water table is below the level of the rivers changes in topography naturally have little effect, except where the erosion is sufficient to cut the ground-water table. Generally in such regions the rivers contribute to the ground water by seepage, and the amount so contributed becomes small when the conditions are favorable for the deposition of silt with which the rivers plaster up their beds. During flood periods the rivers scour out the silt and again allow the percolation of water.

Earthquakes may produce fluctuations due to several causes: Small fluctuations may result directly from the earth's tremors; a deformation without faulting may produce changes in pressure; and faulting may make new ground-water outlets which will cause the water in neighboring wells to rise or fall according to their relation to the faulting.

Geyser phenomena may produce both periodic and irregular fluctuations of the water level, and Slichter has suggested that the peculiar periodic fluctuations at Urisino Station, New South Wales (p. 76), may be due to such a cause.

In this connection it may be well to refer to the hypothetical siphon, or Tantalus-cup arrangement, which the old philosophers gave as an explanation of intermittent springs,[a] a theory which has survived in Houston's Physical Geography, a work still used in the high schools in some parts of this country.[b] From a geologic standpoint the existence of such a siphon arrangement as this theory postulates may be regarded as almost impossible because of the difficulty of finding an air-tight passage. The fluctuations are now known to be due in many cases to causes not understood at the time this hypothesis was advanced, and in the light of our present knowledge an intermittent spring depending on a natural siphon for its action would be regarded as a most exceptional phenomenon. It would be necessary to do more than prove that a spring or well ebbs and flows to establish the existence of such a siphon.

FLUCTUATIONS PRODUCED BY HUMAN AGENCIES.

EFFECT OF SETTLEMENT, DEFORESTATION, AND CULTIVATION ON THE LEVEL OF WATER IN WELLS.

It is well known that many hillside springs throughout the entire eastern United States which furnished water when the country was first settled are now dry, that large areas of former marsh land are now in cultivation, and that streams on which boats plied in the early days are no longer navigable. The rainfall records do not indicate that there have been any radical climatic changes, and the changes are clearly the result of human occupation.[c]

Part of this is due to the fact that large areas have been artificially drained by tiles, ditches, or absorption pits; beaver dams and other stream obstructions, such as the Great Red River Raft, have been removed, with the consequent drainage of greater or less areas.[d] Some of the hillside springs have merely been buried as the soil washed in from the surrounding lands, while others have been affected by the drainage of the lower lands.

Different kinds of vegetation use different amounts of water[e] and affect the surface

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[a] See Regnault, Père, Phil. Conversations, vol. 2, conversation 6; Dechales, Tract. 17 de Fontibus Naturalibus, etc.; Desaguliers, Rev. J. T., Trans. Phil. Soc. London, No. 384, vol. 33, 1724 (abridged edition Trans. 1665–1800, vol. 7, pp. 39–41); Atwell, Joseph, Trans. Phil. Soc. London, No. 424, vol. 37, 1732 (abridged edition Trans. 1665–1800, vol. 7, pp. 544–555).

[b] For a more recent suggestion of the same theory, see Knightly, T. E., Geol. Mag., n. s., decade 4, vol. 5, 1898, pp. 333–334.

[c] See, in this connection, King, Franklin H., Bull. U. S. Weather Bureau No. 5, 1892, p. 42; Lane, Alfred C., Water-Sup. and Irr. Paper No. 30, U. S. Geol. Survey, 1899, pp. 54–55.

[d] Prof. Paper U. S. Geol. Survey No. 46, 1906.

[e] The literature on the amount of water transpired from plants and evaporated from the earth under different conditions is very extensive, but the results are neither readily comparable nor readily applicable to natural conditions, because of the differing and in many cases unnatural conditions under which these experiments have been tried. For a review of the literature, see Harrington, M. W., Review of forest-meteorology observations, and Fernow, B. E., Relation of forest to water supply; Bull. Bureau of Forestry, U. S. Dept. Agric. No. 7, 1873; Lueger, Otto, Die Wasserversorgung der Städte: Der städtische Tiefbau, Bd. II, pp. 176–177, 196–205, bibliography, pp. 143–161; Wollny, Ewald, Expt. Station Record, vol. 4, 1893, pp. 531–533; King, F. H., The Soil, New York, 1895, Drainage and Irrigation, New York, 1899.

evaporation in different ways, and a change in the plant covering or crop over large areas may clearly result in a broad elevation or lowering of the water level. Similarly, certain methods of cultivation conserve more moisture than would find its way into the ground under certain natural conditions, while others allow large quantities to flow off the surface. Fertilizers and manures affect the rate of percolation in different ways; some greatly hasten and others retard the percolation of the soil water. The relation of cultivation to the position of the ground water is therefore very complex, and it is clearly possible to have different results on adjacent fields. In regard to the effect of forested areas on percolation it should be pointed out, on the one hand, that (1) a portion of the rain water, varying from 8.5 to 59 per cent[a] of the yearly rainfall, is caught in the crowns of the trees and is evaporated without reaching the ground; (2) the absorption capacity of the forest litter and moss is great, and water can be contributed to the ground water only after this is saturated; while the evaporation from this surface is slow, it is to be considered evaporation from a saturated surface, and the net result may be greater than from a region where the water sinks rapidly into the ground; (3) the old litter or humus is, according to the experiments by Riegler, Ebermeyer, and Wollny, practically impervious, and, while the fresh litter may absorb large quantities of water, the impervious humus or rotted litter prevents the water from reaching the ground water; (4) the roots of the trees in some cases draw from ground water that is entirely out of the reach of ordinary field plants. Moreover, the direct observations of Ototzky[b] and Henry and Tolsky[c] yield the positive result that in Russia and France the level of the ground water is decidedly lower under forests than under cleared land. The results of Ototzky's observations are summarized in the Experiment Station Record in the following words:

> This is a translation from the Russian giving the results of a hydrological survey in the steppes region. The conclusion is reached that, physico-geographical conditions being the same, the level of ground water is lower in forests than in adjacent steppes or in general in neighboring open spaces. The level falls as forests are approached, the fall sometimes being very sudden, and it is more marked in case of old forests than new.

On the other hand, it should be pointed out that the stream flow from forested regions is more constant than from unforested ones,[d] and as this is to be considered as due to ground-water phenomena it indicates a greater percolation.

On the plains, groves and hedgerows by acting as wind breaks tend to elevate the water level by decreasing the surface evaporation.

It is well known to agriculturists that it is possible to cultivate the soil so that the evaporation will be greater than under natural conditions or so that the moisture will be conserved. It is thus possible to either increase or decrease the ground water by cultivation. In the semiarid region of the Middle West, where the rainfall is light, the secret of the so-called "dry or arid farming" is to so prepare the soil as to insure the percolation of all the rain water or of a very large percentage of it, and to prevent its escape by evaporation. To accomplish this various methods of subsoiling, subsurface rolling, and surface mulching, either by pulverizing the soil or by the addition of straw or manure, have been employed, in some cases with marked success. I am informed by Prof. Charles S. Slichter that in western Kansas, where the Campbell system is employed, the ground water has in places been raised several feet by the increased percolation.

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[a] See Bull. Division of Forestry, U. S. Dept. Agric. No. 7, 1893, pp. 100–101, 130–131, and references therein given; also Lueger, Wasserversorgung der Städte, p. 197.

[b] Ototzky, P., Ann. Sci. Agron. 1897, vol. 2, No. 3, pp. 455–477, pls. 2; review, Expt. Station Record, vol. 9, 1898, p. 1041.

[c] Henry, E., and Tolsky, A., Ann. Soc. Agron. 1902, vol. 3, No. 3, pp. 396–422; review, Expt. Station Record, vol. 15, 1903, p. 125.

[d] See Bull. Division of Forestry, U. S. Dept. Agric., No. 7, 1892, pp. 158–170; Manson, Marsden, Comparison of low-water discharge from a timbered with that from a comparatively timberless area: Water-Sup. and Irr. Paper No. 46, U. S. Geol. Survey, 1901, pp. 46–47.

EFFECT OF IRRIGATION.

Irrigation, almost without exception, raises the ground-water level, and in regions where there is no natural ground-water outlet so placed that it furnishes a sufficient natural escape for the underflow, elaborate systems of tiling and pumping are necessary to keep the water level from reaching the surface in the low places and converting them into marshes or alkali flats. Carpenter reports that in the Cache la Poudre Valley, Colorado, the water level has been raised 20 to 40 feet.[a] The effects of irrigation in the King River Valley, California, are shown in Water-Supply and Irrigation Paper No. 58, Pl. XXVI.

On Long Island only limited areas have so far been irrigated, but these bid fair to rapidly increase. On account of the very porous character of the soil and the fact that all the water used must be obtained from the ground water of the region involved, there is no danger of serious raising of the ground-water level; indeed, the net result here of extreme irrigation, which would have to be done by pumping, would be a lowering of the ground-water level to the extent of the added loss by evaporation and plant transpiration. When the water for irrigation is supplied wholly from springs, as it is at one or two points near Flushing, or where supplied from the city waterworks, as at Elmhurst,[b] the result is a local raising of the ground-water level

EFFECT OF DAMS.

In regions where the ground water is tributary to the stream channels the effect of the ponding of streams, except where the material of the bed of the reservoir is entirely impervious, is to raise the ground-water level. As the pond or reservoir is relatively permanent, the ground water generally has time to adjust itself to the new conditions, and an elevation is produced which is persistent as long as the reservoir lasts. Thus on Long Island, where dams were built in all the little streams at an early day, the effect has been to abnormally raise the ground-water level over considerable areas.

In mill ponds of this character the use of the water during the day and the accumulation during the night give rise to a periodic fluctuation of the water in the wells along their banks which tends to accentuate the temperature effect.

In regions where the ground-water table is below the stream, ponding will increase the leakage, though this may naturally check itself in time by the deposition of silt and colloidal material.

EFFECT OF UNDERGROUND WATER-SUPPLY DEVELOPMENTS.

Underground water is developed for water supply in one of four general ways: (1) By subsurface dams, (2) by infiltration galleries, (3) by pumping from single wells or groups of wells, and (4) by flowing wells.

EFFECT OF SUBSURFACE DAMS.

In regions where there are valleys with impervious sides filled with porous material a dam across the valley will pond this underflow and force it to the surface. This has been employed in many regions of the West where dry stream beds with considerable underflow abound. The effect of such a structure on the ground-water level is shown in Water-Supply and Irrigation Paper No. 67, Pls. V, VII.

EFFECT OF INFILTRATION GALLERIES.

Infiltration galleries may either raise or lower the ground-water level. When constructed along the line of contact of a pervious and impervious bed they may act in

[a] Carpenter, L. G., Seepage or return waters from irrigation: Bull. Colorado Expt. Station, No. 33, 1896, p. 4.
[b] Bull. Office Expt. Stations, U. S. Dept. Agric., No. 148, 1904.

a way analogous to a subsurface dam. Where deep in pervious layers they offer a new outlet at a lower level than the natural one, and so depress the water plane. This effect is the same as that in a pumped well, except that here the cone of depression is greatly lengthened in one direction.

EFFECT OF PUMPING.

The first effect of pumping is to develop a more or less symmetrical cone of depression, of which the well is the center. The steepness and slope of the cone depend on such factors as the porosity, rate of flow, rate of pumping, and uniformity of soil. The effect of such a depression in the porous material on Long Island is to lower the water in adjacent wells and drain the near-by ponds and marsh areas.[a]

The effect of this lowering of the water table has a marked effect on the stream flow on Long Island, as is shown by the following table, compiled by L. B. Ward:

Effect of ground-water pumping in diminishing stream flow, from 1873 to 1899, in the old watershed of the Brooklyn waterworks, comparing five-year periods.[a]

Period.	Average annual rainfall.	Average annual rainfall collected, referred to watershed as a whole.		Area of watershed.	Driven-well supply.	
					Expressed as rainfall.	Daily per square mile.
	Inches.	*Per cent.*	*Inches.*	*Square miles.*	*Inches.*	*Gallons.*
1873–1877	43.33	25.07	10.86	52.30	([b])	([b])
1878–1882	41.58	29.60	12.31	55.14	([b])	([b])
1883–1887	43.30	31.60	13.68	64.42	2.95	140,392
1889–1893	45.05	38.43	17.31	65.54	5.85	278,883
1895–1899	43.14	36.32	15.67	66.44	7.76	369,581

Period.	Other pumped sources of supply.		Daily total per square mile derived from all sources in the watershed.	Water collected as stream flow, referred to 50 square miles of watershed.		
	Expressed as rainfall.	Daily per square mile.		Daily per square mile.	Expressed as rainfall.	
					Amount.	Proportion of total.
	Inches.	*Gallons.*	*Gallons.*	*Gallons.*	*Inches.*	*Per cent.*
1873–1877	0.18	8,659	517,206	532,034	11.17	25.79
1878–1882	.99	47,063	585,978	594,310	12.48	30.02
1883–1887	2.30	109,041	651,506	518,071	10.88	25.13
1889–1893	4.17	198,605	824,195	455,153	9.56	21.22
1895–1899	2.74	130,224	745,983	327,122	6.89	15.96

[a] Merchants' Association of New York, The Water Supply of the City of New York, 1900, p. 186.
[b] Began in 1883.

While all pumped wells cause a cone of depression, in regions where the ground water moves rapidly and where the demand does not exceed the supply the recovery is very rapid, as is shown by the figures prepared by W. E. Spear from the records of the Brooklyn water department.[b]

Several points are noteworthy about these diagrams. The water-bearing strata, in the deep and shallow wells, in each case are separated by rather fine material which may usually be called a clay. There is a distinct flow below the clay layer with a velocity, as shown by Slichter, of 6 feet per day, while that immediately above

[a] See discussion in Prof. Paper U. S. Geol. Survey No. 44, 1906, pp. 78–79.
[b] Rept. New York City Commission on Additional Water Supply, 1904, appendix 7, Pls. XI, XII.

the clay layer is but 18 inches. Near the surface the velocity is 3 to 15 feet per day. It is therefore interesting to notice at Agawam, which is essentially a deep-well station, the sympathetic effect of pumping in the lower layer on the water in the upper. There is also an important difference in the depression produced in well No. 11 and well No. 16. This indicates local irregularities in porosity. At Merrick the wells connected to the suction are all shallow but one; the effect of pumping is therefore more marked in the shallow than the deep wells. It is here quite normally greatest in the center well. The recovery after pumping is very rapid in both cases, indicating that the supply is a free one and that the plants have not overdrawn it.

The records for a 181-foot well at the Queens County Water Company pumping station, near Hewlett, N. Y., show regular fluctuations due to pumping (Pl. III, p. 18). This well is 3,000 feet from the pumping station and 2,000 feet from the nearest pumped well, and the records showed fluctuations of such a regular rhythmical character that they were at first thought to represent fluctuations due entirely to temperature changes. Further consideration in connection with the record of pumping from the station shows that the fluctuations are almost wholly due to pumping, although there is perhaps a slight temperature effect involved.

The response of the water level in the deeper wells to changes of pressure at the surface, due to rainfall, tidal, barometric, and thermometric fluctuations, suggests that the removal by pumping of the surface ground water over an artesian stratum will, by relief from load, produce sympathetic fluctuations in deep wells where there is absolutely no connection between the water-bearing strata.

EFFECT OF ARTESIAN-WELL DEVELOPMENTS.

The universal experience in artesian basins has been that after a time the head decreases. This is due to many causes. When the whole basin is affected, it indicates that the outflow or pumping from the wells exceeds the inflow from the porous strata, and a gradual decrease is to be expected until these factors are balanced. A well or group of wells may be influenced by interference from a single well favorably situated. Thus the drilling of a well in which the outflow is many feet below that of nearby wells quickly affects the head of the higher wells. Very often where but a few wells here and there are affected the decrease in head is to be regarded as wholly due to leakage, either on the outside of the casing or by the failure of the casing through corrosion.

All artesian wells are sooner or later pumped, and the effect of the pumping is often to lower the head over wide areas. The diminution of the head in the chalk wells in London during the early part of the nineteenth century is well shown by Clutterbuck.[a]

EFFECT OF LARGE CITIES ON THE WATER LEVEL.

Aside from the general lowering of the water level in cities due to pumpage, another factor tending in the same direction is a decrease of the inflow of rain waters. The mass of buildings, paved streets, and drainage systems absolutely prevent the infiltration of rain water over wide areas. This loss is, however, in part replaced by leakage from the water and sewer systems.

The loading resulting from the placing of large and heavy buildings on small areas will have the same effect as loading due to any natural causes, except that the former is so gradual that in most cases the water has time to escape laterally. It is conceivable, however, that the loading may exceed the rate of outflow and a temporary measurable increase in the artesian head be produced, but this is of such slight value as to be of theoretical rather than practical importance. The effect is practically nothing when the building rests on bed rock, as at New York, and reaches its maxi-

[a] Min. Proc. Inst. Civil Eng., vol. 9, 1850, pl. vi, p. 180.

mum at points like New Orleans, Galveston, and other coast towns underlain by unconsolidated Tertiary and Quaternary beds. A much more readily measurable effect would be produced by large fires, which in a short time would remove a large weight from a limited area. These pressure effects would be noticeable only in wells in which the water is already under artesian head and when the overlying beds have considerable plasticity. They would clearly be greatest in unconsolidated materials and would decrease with the thickness of the strata above the water-bearing layer.

EFFECT PRODUCED BY LOADED FREIGHT TRAINS.

The sensitiveness of the water in wells to any change in load at the surface is strikingly illustrated by the oscillation produced by slowly moving freight trains at Madison, Wis. This is described by King as follows: [a]

> While the self-registering instrument was upon well No. 48, it was observed that there were frequent records of sharp short-period curves shown upon the sheet, which at first were supposed to be the result of accidental jars which the instrument sustained; but the frequency of their occurrence and the fact that they always indicated elevations of the water led to a closer scrutiny and their final association with the movement of trains past the well. On the eight-day instrument these fluctuations are shown as single dashes. but with the one-day form the curve was open. The well in which these disturbances occur is situated about 140 feet from the railroad track and has a depth of 40 feet. It is tubed up with 6-inch iron pipe to the sandstone, 37 feet below the surface, and the water has a mean depth of about 20 feet in it.
>
> The strongest rises in the level of the water are produced by the heavily loaded trains, which move rather slowly. A single engine has never been observed to leave a record, and the rapidly moving passenger tains produce only a slight movement, or none at all, which is recorded by the instrument. The figure shows the curve to be produced by a rapid but gradual rise of the water, which is followed by only a slightly less rapid fall to the normal level, there being nothing oscillatory in character indicated by any of the tracings nor observable to the eye when watching the pen while in motion. The downward movement of the pen usually begins when the engine has passed the well by four or five lengths, and when the pen is watched, it may be seen to start and to descend quite gradually, occupying some seconds in the descent.

This is very similar to the various pressure effects noted above, due to tidal, barometric, and rainfall loading, and to transmitted fluctuations due to variations in local load produced by temperature changes, except that the time of lateral transmission is rather shorter, and it is not clear that the water is under artesian head.

FLUCTUATIONS DUE TO INDETERMINATE CAUSES.

SMALL FLUCTUATIONS.

The extreme susceptibility of the water level in wells to pressure changes would lead one to expect many minute fluctuations; and, indeed, all the well curves show a great number of such fluctuations superposed on the larger fluctuations produced by the dominant element at that point. Many are clearly compound waves of very complex character and represent the resultant of many forces. They emphasize the continued state of unrest of the earth's surface. These fluctuations can be properly studied only with instruments having both a large vertical and time scale, and their elucidation would necessitate corresponding meteorologic instruments of great delicacy.

On the day gages at Hewlett (p. 18) there is a distinct series of minor fluctuations with a well-defined period of about twenty minutes. These greatly resemble the minor oscillations in the tidal curves at many points. [b]

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[a] Bull. U. S. Weather Bureau No. 5, 1892, pp. 67-68.

[b] See Airy, Sir G. B., On the seiches or nontidal undulations of short period at Malta: Phil. Trans. Royal Soc., 1878, pp. 123-138 Dawson, W. Bell, Notes on secondary undulations recorded by self-registering tide gages, Trans. Royal Soc. Canada, sec. 3, 1895, pp. 25-26; Illustrations of remarkable secondary tidal undulations in Nova Scotia, Trans. Royal Soc. Canada, sec. 3, 1899, pp. 23-26. Duff, A. W., Secondary undulations shown by recording tide gages; Trans. Nat. Hist. Soc. New Brunswick, 1897; Am. Jour. Sci., 4th ser., vol. 3, 1897, pp. 406-412; Am. Jour. Sci., 4th ser., vol. 12, 1901, pp. 123-139. Denison, F. Napier, The Great Lakes as a sensitive barometer: Canadian Eng., Oct.-Nov., 1897: Secondary undulations of tide gages, Proc. Can. Inst., n. s., vol. 1, 1897, pp. 28-31; The Great Lakes as a sensitive barometer: Proc. Can. Inst., n. s., vol. 1, 1897, pp. 55-63; The origin of ocean tidal secondary undulations: Proc. Can. Inst., n. s., vol. 1, 1897, pp. 134-135.

The secondary oscillations in the tide curve at Swansea, England, have a time interval of fifteen to twenty minutes; at Malta, twenty-one minutes; and at Sydney, twenty-six minutes; while Denison has observed on Lake Huron oscillations with periods of fourteen, eighteen, twenty-two, and forty-five minutes. As no such secondary tidal oscillations have been observed near Long Island, and as the Hewlett well is at such a distance from the coast that it is not affected by tides 4 feet high, these oscillations are clearly not of transmitted ocean origin. Denison's observations led him to the conclusion that many of the secondary oscillations are due to barometric fluctuations, and the occurrence of these fluctuations in wells must be regarded as strong confirmatory evidence of his conclusion.

Besides these fluctuations with a period of twenty minutes, there are several other minor vibrations with smaller amplitudes and periods; one series seems to have a period of five or six minutes, but is not very sharply defined.

In the wells at Lynbrook (p. 23) minor fluctuations with periods of forty and eighty minutes have been clearly recognized in a mass of still smaller fluctuations.

FLUCTUATIONS AT MILLBURN, N. Y.

Extremely irregular fluctuations with a range of as much as 1 inch were obtained from a well at Millburn, N. Y. (Pl. V, p. 22). These are quite different from any of the other curves obtained and no cause can be assigned for these irregularities. Not the least strange part of the curve is that its general character changes sharply on July 29. (See discussion, pp. 22–23.)

FLUCTUATIONS AT URISINO STATION, NEW SOUTH WALES.

The fluctuations reported by Professor David[a] at Urisino Station, between Wanaaring and Milparinka, in the northwest corner of New South Wales, 200 miles from the ocean, are unique. Two subartesian wells, one 1,680 and the other 2,000 feet deep, in which the water rises to within 15 or 20 feet of the surface, show regular rhythmical pulsations with a range of 4 to 5 feet every two hours. That is, there are here six almost equal "tides" of large size every twenty-four hours. Prof. Charles S. Slichter has suggested the very probable explanation that the fluctuations are due to a sort of periodic geyser phenomena. This is quite competent to produce the fluctuations observed and the high temperature of the water in this basin lends considerable color to the suggestion.

[a] David, T. W. E., Notes on artesian water in New South Wales and Queensland: Jour. and Proc. Royal Soc. New South Wales for 1893, vol. 27, pp. 429–430.

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CLASSIFICATION OF THE PUBLICATIONS OF THE UNITED STATES GEOLOGICAL SURVEY.

[Water-Supply Paper No. 155.]

The serial publications of the United States Geological Survey consist of (1) Annual Reports, (2) Monographs, (3) Professional Papers, (4) Bulletins, (5) Mineral Resources, (6) Water-Supply and Irrigation Papers, (7) Topographic Atlas of United States—folios and separate sheets thereof, (8) Geologic Atlas of the United States—folios thereof. The classes numbered 2, 7, and 8 are sold at cost of publication; the others are distributed free. A circular giving complete lists may be had on application.

Most of the above publications may be obtained or consulted in the following ways:

1. A limited number are delivered to the Director of the Survey, from whom they may be obtained, free of charge (except classes 2, 7, and 8), on application.

2. A certain number are delivered to Senators and Representatives in Congress for distribution.

3. Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be had at prices slightly above cost.

4. Copies of all Government publications are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted by those interested.

The Professional Papers, Bulletins, and Water-Supply Papers treat of a variety of subjects, and the total number issued is large. They have therefore been classified into the following series: A, Economic geology; B, Descriptive geology; C, Systematic geology and paleontology; D, Petrography and mineralogy; E, Chemistry and physics; F, Geography; G, Miscellaneous; H, Forestry; I, Irrigation; J, Water storage; K, Pumping water; L, Quality of water; M, General hydrographic investigations; N, Water power; O, Underground waters; P, Hydrographic progress reports. This paper is the fifty-second in Series O, the complete list of which follows (PP=Professional Paper; B=Bulletin; WS=Water-Supply Paper):

SERIES O, UNDERGROUND WATERS.

WS 4. A reconnaissance in southeastern Washington, by I. C. Russell. 1897. 96 pp., 7 pls. (Out of stock.)

WS 6. Underground waters of southwestern Kansas, by Erasmus Haworth. 1897. 65 pp., 12 pls. (Out of stock.)

WS 7. Seepage waters of northern Utah, by Samuel Fortier. 1897. 50 pp., 3 pls. (Out of stock.)

WS 12. Underground waters of southeastern Nebraska, by N. H. Darton. 1898. 56 pp., 21 pls. (Out of stock.)

WS 21. Wells of northern Indiana, by Frank Leverett. 1899. 82 pp., 2 pls. (Out of stock.)

WS 26. Wells of southern Indiana (continuation of No. 21), by Frank Leverett. 1899. 64 pp. (Out of stock.)

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WS 34. Geology and water resources of a portion of southeastern South Dakota, by J. E. Todd. 1900. 34 pp., 19 pls.

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WS 54. Geology and water resources of Nez Perces County, Idaho, Pt. II, by I. C. Russell. 1901. 87-141 pp. (Out of stock.)

Water-Supply and Irrigation Paper No. 156 Series N, Water Power, 11

DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

WATER POWERS OF NORTHERN WISCONSIN

BY

LEONARD S. SMITH

WASHINGTON
GOVERNMENT PRINTING OFFICE
1906

CONTENTS.

ILLUSTRATIONS.

WATER POWERS OF NORTHERN WISCONSIN.

By L. S. SMITH.

INTRODUCTION.

Significance and extent of water-power resources.—Unlike other great natural resources of the State, such as the forest and mineral wealth, the utilization of which means the final destruction of the source of supply, the water-power resources are as certain and eternal as the sunshine. The importance of water powers to a State so remote from coal mines as is Wisconsin is not likely to be overestimated. Unquestionably these powers are destined to exercise a wide influence on the development of the State. So far as known, not a single important river in the State has as yet been made to fully produce its available power. The lower Fox may be said to come the nearest to this, with a total of 31,898 actual horsepower,[a] all produced in the 35 miles between Lake Winnebago and Green Bay. This large water power has caused the district to take high rank as a paper and pulp manufacturing center. Wisconsin, Chippewa, and St. Croix rivers can each be made to produce power equaling and even exceeding that of the lower Fox. Growth in the development of Wisconsin water powers has been very rapid. During the ten years ending in 1900 the gain was 75 per cent. The following figures show the growth during the last thirty years:

Wisconsin water powers developed.

	Horsepower.
1870	33,700
1880	45,300
1890	56,700
1900	99,000

The annual saving represented by this power over the cost of an equivalent amount of steam power, computed at $20 per horsepower, reaches the sum of nearly $2,000,000.

Sources of information.—Judging from the scant literature descriptive of Wisconsin water powers, but little attention has been directed in the past to this great natural resource of the State. The longest and most accurate description is contained in the Tenth Census of the United States. In Geology of Wisconsin, volume 3, 1880, will be found good detailed descriptions of the Lake Superior rivers from the standpoint of a geologist. Very reliable information regarding the upper headwaters of the larger rivers is given in the reports of the Chief of Engineers, U. S. Army, for the years 1879–1883, inclusive, to which frequent reference is herein made. This work of surveying reservoir sites involved the running of many hundred miles of levels, thus securing numerous water levels on lakes and rivers. The maps of these surveys were never published, but copies of the originals have been obtained, and no pains or expense has been spared to preserve and present these data. A fourth source of information, and a most welcome one, both because of its intrinsic value and because it marks the beginning of a rational and systematic study of Wisconsin water powers, is the detailed survey of part of Chippewa River and the daily discharge records of

[a] Rept. Chief Eng. U. S. Army, 1897, p. 2737.

many important water-power rivers carried on by the United States Geological Survey during the years 1903, 1904, and 1905.[a]

Finally, the data here presented would have lacked much of whatever value and completeness they may have had it not been for the generous support of hydraulic engineers and mill owners.

After exhausting all possible sources of information by correspondence, however, it was found that many points of importance could be cleared up only by a personal visit to the field. In this manner visits were made to St. Croix River at Taylors Falls; to Apple and Willow rivers; to Eau Claire and Chippewa Falls, on Chippewa River; to Black River Falls and Neillsville, on Black River; to Grand Rapids, Stevens Point, Tomahawk, and Rhinelander, on Wisconsin River, and to Oshkosh, Appleton, Menasha, Kaukauna, and Depere, on Fox River.

The importance of these water-power resources to the development of the State would certainly justify it in cooperating financially with the United States Geological Survey in extending the investigation to include hypsometric surveys of the important rivers, especially in the region now undeveloped.

PHYSICAL GEOGRAPHY OF NORTHERN WISCONSIN.

GEOLOGY.[b]

The rock formations of northern Wisconsin readily fall into three classes—the pre-Cambrian crystalline rocks, the Paleozoic rocks, and the Glacial drift. The pre-Cambrian and Paleozoic formations are adjacent to one another, but the loose Glacial drift is distributed irregularly over all the hard-rock formations of the region.

PRE-CAMBRIAN ROCKS.

The pre-Cambrian crystalline rocks consist of various kinds of igneous rocks, such as greenstone or trap rocks, granite, diorite, rhyolite, schists, and gneisses, and varieties of metamorphosed sedimentary rocks, such as quartzite, slate, limestone, conglomerate ferruginous rocks, slate, and schists. The rocks here classed as pre-Cambrian include all those often referred to as the Laurentian (Archean), Huronian, and Keweenawan series. The various kinds of crystalline rocks generally stand on edge, trend in various directions, and form irregular belts and areas throughout the region.

The area of crystalline rocks covers the principal part of northern Wisconsin. Its northern boundary is approximately parallel to and very near the adjacent shore of Lake Superior on the west it projects irregularly into Minnesota; on the south it extends to the central part of the State, and on the east it reaches within 25 to 40 miles of Green Bay.

The pre-Cambrian region is the highest portion of the State, and in these crystalline highlands the large rivers have their source and flow outward in all directions. The crystalline rocks are generally hard. They do not everywhere have this character, however, and the lack of uniformity causes much irregularity in the surface features. High, rounded knobs of hard granite and quartzite dot the surface of the region, and the abrupt variations in the character of the rock along the river valleys have caused the formation of numerous rapids and waterfalls. The slope of the pre-Cambrian region is relatively steep on the Lake Superior side and comparatively gentle toward the east, south, and west.

PALEOZOIC ROCKS.

The Paleozoic rocks consist of alternating formations of comparatively incoherent, friable sandstone and hard, compact limestone lying unconformably upon the upturned edges of the crystalline rocks and dipping slightly toward the north, east, south, and

[a] The Wisconsin legislature of 1905 appropriated $2,500 for the purpose of surveying the water powers of the State in cooperation with the United States Geological Survey, which has set aside an equal amount for this purpose. In the fall of 1905 Wisconsin, Black, and Flambeau rivers were surveyed. This work is in charge of Leonard S. Smith.

[b] Prepared by S. Weidman, State geologist of Wisconsin.

west—the dip thus being away from the broad central core of the pre-Cambrian region. The Paleozoic rocks of northern Wisconsin include the following formations, named from the base upward: (1) Cambrian ("Potsdam") sandstone, (2) "Lower Magnesian" limestone, (3) St. Peter sandstone, and (4) "Trenton" limestone.

The Cambrian sandstone is by far the most abundant Paleozoic rock of the region. Along the shore of Lake Superior, where it is generally called the Lake Superior sandstone, it forms a strip less than a mile in width at the Michigan boundary, increasing to 15 miles in width at the Minnesota boundary. For variable distances of 15 to 40 miles about the broad central area of the pre-Cambrian to the west, south, and east, the Cambrian is the principal surface rock. It is only adjacent to the shore of Green Bay on the east and in St. Croix and Pierce counties on the west that limestone and sandstone later than the Cambrian occur to any notable extent.

The surface features of the Cambrian sandstone district are mainly broad valley bottoms, dotted here and there with a few pinnacles of hard sand rock. In the region of the limestone, however, the valleys are generally sharp and narrow, and the uplands constitute the main portion of the landscape. The hills and sharp ravines in the limestone district are in sharp contrast with the broad, graded valley bottoms of the sandstone district.

GLACIAL DRIFT.

The Glacial drift consists of a loose, incoherent mass of bowlders, gravel, sand, and clay. In some places the coarse drift is abundant, while in other places clays and sand prevail. The drift has a very irregular thickness throughout the area. It was deposited upon the older crystalline and Paleozoic rocks during the several successive glaciations in Wisconsin and the adjacent region.

Drift in variable quantity occurs throughout northern Wisconsin, being very abundant in the northeastern, northern, and northwestern parts of the region, while in a very irregular but considerable area in the southwestern part the drift is very thin.

The surface of a large part of the drift-covered region is very irregular and uneven, and consists of hills and ridges alternating with basins, swamps, and lakes. In some places the drift covering completely obliterates the topographic features of the crystalline and Paleozoic rocks; in other places it only modifies the older topography. On the whole, however, the glaciation of the region exerted a considerable influence on the distribution of the drainage lines and in shaping the minor inequalities of the land surface. The drift region, from the topographic point of view, may be divided into two general districts—one covered by the older drift series and the other by the later drift. In the district of the older drift, the southwestern part of northern Wisconsin, there are no lakes or ponds, and swamps are very rare. Here the topography is mature and the land has good surface drainage. In the district of the later drift, however, which includes the main portion of northern Wisconsin, the glacial deposits are abundant; ridges and hills of bowldery material occur, and lakes, swamps, and sags are common. In this district, therefore, the surface drainage is often very poor and large amounts of water are held in swamps and ponds. Here, also, there are marked differences in the surface features prevailing over large parts of the district. Along its border is the terminal moraine, often called the "kettle moraine," having a width ranging from 3 or 4 to 20 miles and consisting of numerous drift hills and ridges closely associated with sags, lakes, and ponds. This terminal moraine extends across the entire continent. In crossing this portion of the State it turns north a few miles east of Grand Rapids, thence extends to Antigo, thence in a sinuous belt westward to Barron County, and thence southwest into Minnesota. Back of this terminal moraine—that is, to the east and north—are similar belts known as "recessional moraines," separated one from another by broad areas having the general features of the hard rocks beneath. Between the moraine belts are broad tracts of sandy land, called "barrens," which cover considerable portions of the northwestern part of the State. Along Lake Superior is a broad belt of nearly flat clay land which may be mentioned, though it has no influence on the distribution of the water powers of the region.

TOPOGRAPHY.

The abundant water-power resources of Wisconsin are the result of its unique topography. A wide and comparatively flat highland crosses the northern part of the State. This divide varies in elevation from 1,900 feet in the eastern part to 1,000 feet in the western part, and extends to within 30 miles of Lake Superior. From it the rivers descend radially in all directions except eastward. Owing to the fact that Lakes Superior and Michigan bound the State on the north and east, while Mississippi River forms the southwestern and the larger part of the western boundary, all the rivers must needs find a low trough into which to discharge, and that at a short distance from their source. This condition results in a rapid fall and large water powers.

About 9 per cent of the total area considered belongs to the abrupt Lake Superior watershed and the remainder to the broad southeast, south, and southwest slopes. The divides between the rivers which drain this southern slope are almost imperceptible, in some cases being entirely lost in labyrinths of lakes and swamps.

Hills over 300 feet in height are rare. A few "mounds," or isolated steep hills with extremely narrow bases, rise out of the sandy plains of Jackson and Clark counties, and a few larger, more massive hills, one 1,940 feet above the sea, occur in the valleys of the larger rivers, besides the low, broad hills which form the crests of the Penokee and Copper ranges. These hilly tracts do not cover over 5 per cent of the total area, while about 45 per cent is level upland and about 50 per cent is rolling country, of which a considerable portion is steeply rolling "kettle" or "pot-hole" land.[a]

The surface features are discussed elsewhere, under the head of "Geology," and also in connection with the drainage of each river.

HYDROGRAPHY.

St. Croix, Chippewa, Black, and Wisconsin rivers drain 70 per cent of the northern half of the State, an area nearly equal to that of the State of Maine. The Lake Superior rivers drain only 9.3 per cent and those flowing into Green Bay the remaining 20.7 per cent.

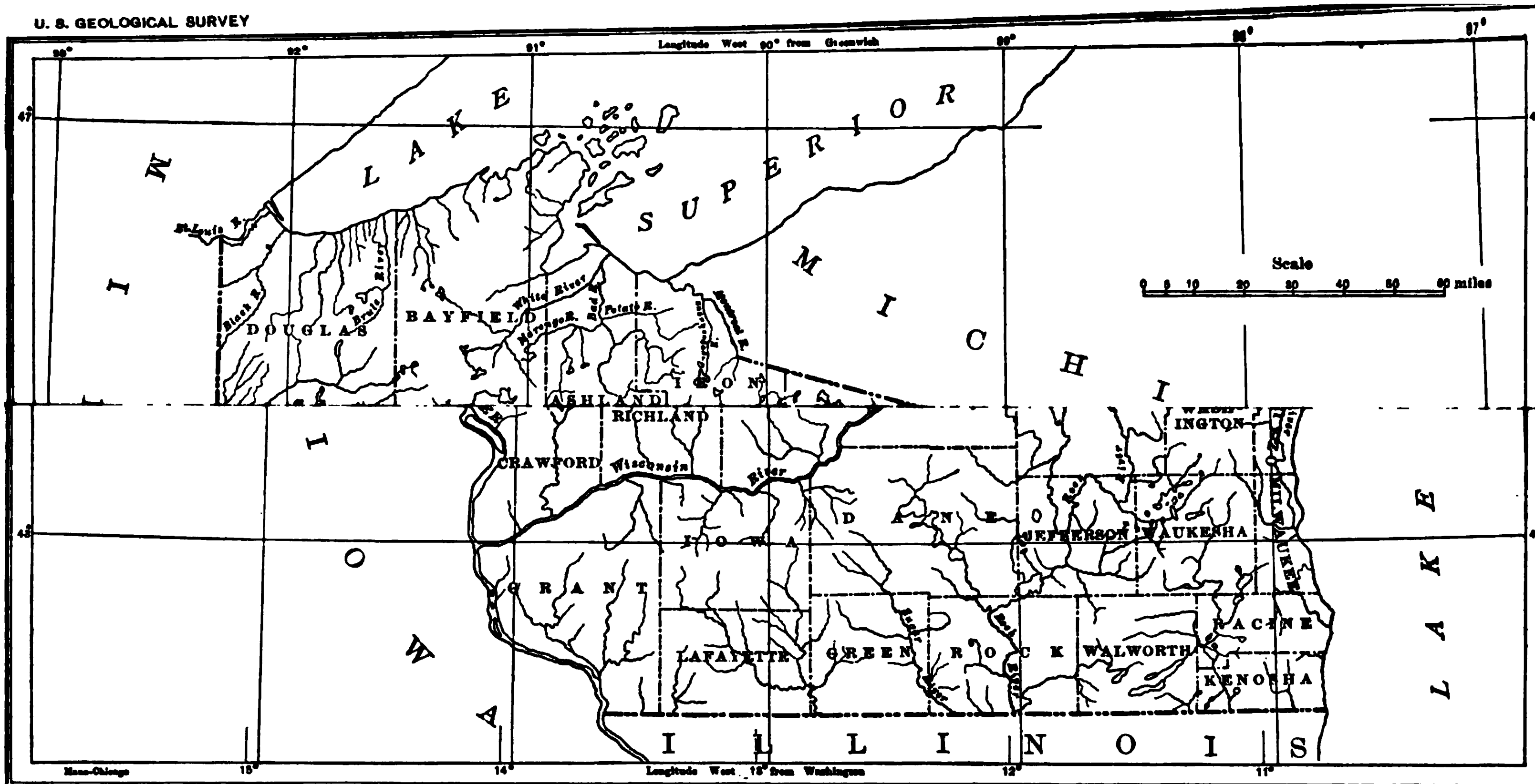
In general, each of the important rivers may be divided into three divisions, differing widely in physical characteristics. First, the headwaters, marked by sluggish streams with low divides, fed by numerous and extensive swamps and lakes, frequently so interlaced that it is impossible to trace out the river divides. Here many of the lakes have dam sites forming natural reservoirs for the river below. Bowlder rapids are here of frequent occurrence. Second, a stretch of maximum descent along the center reach of the river, abounding in numerous falls and long stretches of rapids. This part of the river is always in the region of the pre-Cambrian crystalline rocks, the southern border of which marks the lower limit of the rapids.[b] Third, the lower portion of the course, where for a distance of about 50 miles the river flows through sandstone and limestone, the descent being very slight. This region is, therefore, devoid of water power. In fact, the United States Government has improved the larger rivers along this reach for the purpose of navigation without the use of locks.

As compared with the upper Mississippi basin in Minnesota, the area under discussion may be said to have a steeper grade, the middle portion, containing the main water powers, having an average fall of 3 to 8 feet to the mile. Because of the storage effect of the lakes and swamps, the low-water run-off is as high as from 0.3 to 0.8 second-foot per square mile of drainage area. Probably about a third of the total rainfall finds its way into the streams.

The general use and control of these northern rivers for logging purposes in the past tended to decrease the value of the water powers by withholding the water at times when most needed. All logging on rivers is fast disappearing. Indeed, on many rivers, like the Wisconsin, it has practically given way entirely to railroad transportation. This leaves the rivers free for the permanent development of their water powers. The effect on

[a] Roth, Filibert, Forestry Conditions of Northern Wisconsin: Bull. Wis. Geol. and Nat. Hist. Survey, No. 1, 1898, pp. 2-3.

[b] The only important exception to this rule is on Wisconsin River at Kilbourn, where the river descends rapidly about 16 feet in the dalles of the Potsdam sandstone.



DRAINAGE MAP OF WISCONSIN.

the stage of water which these dams have had in the past suggests their enlargement, extension, and systematic operation for the sole purpose of increasing the low-water flow.

The United States engineers have surveyed 32 large reservoirs in Wisconsin and have constructed five such reservoirs in Minnesota. The total capacities of the proposed Wisconsin reservoirs are as follows:[a]

Storage capacity of proposed reservoirs in Wisconsin.

River.	Area of overflowed lands.	Storage capacity.
	Acres.	*Cubic feet.*
St. Croix	[b] 102,092	34,334,000,000
Chippewa	Not given.	25,239,000,000
Wisconsin	25,832	19,557,000,000
		79,130,000,000

The intelligent operation of even a part of these reservoirs would have a marked effect in steadying the river discharge. This point will be separately discussed in connection with the several rivers. It may be remarked here that nature, by providing numerous swamps and upward of 1,400 lakes for this region, has accomplished unaided a decided regulation of the water supply.

The availability of these water powers varies greatly on the different rivers, or even on parts of the same river. Those on Wisconsin River, for example, are all reached by the Chicago, Milwaukee and St. Paul Railway, which parallels the river for 100 miles, and by other railroads at certain points. The powers on the lower Chippewa are likewise available; but as yet, because of the small population, the railroads have not built generally into the upper part of the region. The rapid opening up of farms now in progress will soon bring a demand for better transportation.

The present bulletin discusses the water powers of the northern rivers, for the reason that these powers are the least known and least developed.

SOILS.[c]

The soils of northern Wisconsin may be grouped into seven readily recognized classes.

Sandy soils are found in regions known as flood plains, and owe their origin to the sorting action of flowing water as it issued from the melting ice. The two largest areas of this type are found in central Wisconsin east of Black River and in the northwestern part of the State. These soils are so coarse and open that nearly all the rain soaks into the ground, reappearing later at lower levels as springs.

Sandy loams cover a much broader area than the sandy soils, being roughly coincident in distribution with the Potsdam sandstone, from which they have in large part been derived.

Prairie loam is a light, open soil, more closely allied to those described above than to the following ones. It is usually underlain by from 3 to 5 feet of coarse, open gravel. In northern Wisconsin the largest area of this type is found in St. Croix County.

Clayey loam is finer and contains more clay than the soils already described. Such a soil has a great capacity for holding water. "The area of northern Wisconsin covered by this type of soil is larger than that occupied by any other variety."

Loamy clay is still heavier and more clayey than the last, with smaller particles. There are three considerable areas of the soil in this region.

Red clay soil is the most peculiar, the finest grained, and heaviest in the State. It is practically impervious to water. Its areas border Lakes Superior and Michigan.

[a] Rept. Chief. Eng. U. S. Army, 1880.
[b] Including 27,406 acres in Minnesota.
[c] Condensed from F. H. King's description in Northern Wisconsin Handbook.

Swamp soil includes all swamp and marsh land soils. While few very large single areas are covered by these soils, the aggregate amount is probably not less than 1,000,000 to 1,500,000 acres. Some of these lands are now covered by a growth of white cedar, others with tamarack and spruce, the latter being usually found on the borders of both tamarack and cedar swamps, while still others are simply sedge marshes, some of which are yearly cut for hay. In many other swamp areas fires have killed the trees, causing all the small anchoring roots to die and decay, so that the winds have overturned nearly every tree.

Many of the northern swamps are underlain by vast beds of peat, while all have a thick covering of moss and humus. Both these factors play an important part in delaying the water in its journey to the streams.

FOREST CONDITIONS.

"Northern Wisconsin in its primeval state was a vast forest of magnificent timber." This could be said to-day of large areas. The central portion of this region includes mixed forest in which, though the pine has nearly all been cut, there still remain over 5,000 feet of hard wood and hemlock per acre, besides other timber equally valuable. The total area covered by forests of this grade amounts to 8,000 square miles, about the same as that of the State of Massachusetts.

Mr. E. T. Sweet[a] enumerates 34 different kinds of trees which he found on the Lake Superior slope alone. Additional species found on the southern slope would increase this number considerably.

The lumberman's labors were first directed to getting out the pine, both because of its high value and because of the fact that he could float it downstream to market. This industry, including the manufacture into lumber, had an invested capital in 1900 of $100,168,000 and turned out a product valued at $81,983,000.[b] This easily places it as the most important industry of the State. Only two other States exceeded this in 1900. In the same year, according to the United States Census report, Wisconsin was the leading State of the Union in lumber and timber products, their total value being $58,000,000. The amount of pine timber is limited and already its production is waning. Its place is being taken, to a large extent, by hard-wood timber, by cedar posts and poles, and by hemlock lumber and bark. The changes wrought annually by the lumberman's ax and the succeeding forest fires are very considerable. The recent appointment of a State forestry commission promises much for the protection and fostering care of these great interests.

The once popular belief that this northern area was worthless after the loss of its timber has given way in the past ten years to a general confidence in its agricultural possibilities. This is amply evidenced by the rapidity with which these lands are being opened up by farmers and by their rapid appreciation in market value. In 1895 only 7 per cent of the 18,000,000 acres of the northern half of Wisconsin was cultivated. This region has furnished 85,000,000,000 feet B. M. of pine lumber alone in the past sixty years. The gradual clearing of the timber has doubtless had an effect on the run-off of the rivers. Under the changing conditions the rainfall will be less absorbed by the soil and will get to the streams in a shorter period. This is especially true of the swamps, where the fires have burned the thick humus and moss which formerly delayed the passage of the water to the lakes and rivers. It is only fair, however, to call attention to the fact that large areas of the original timber consumed by forest fires have been replaced by a second growth of both hard and soft timber, much of it in the form of dense thickets, which shade and protect the ground more effectually even than the original forest.

[a] Geol. Wisconsin, vol. 3, 1880, p. 328.

[b] U. S. Census, 1900, pt. 1, p. 293.

CLIMATIC CONDITIONS.

TEMPERATURE.

The climate of this region is characterized by a large amount of sunshine, with high temperatures in summer and extreme cold, deep snows, and clear skies in winter. The summer heat and winter cold are generally tempered by the influence of the bordering lakes. Lakes Superior and Michigan cover an area of over 54,000 square miles and never freeze over in winter. Although the prevailing wind is from some westerly quarter, this is so frequently broken up by the passing of storm centers from the lakes that both the temperature and the humidity of the air are affected by these great bodies of water. Wisconsin rivers are generally frozen over between December 1 and March 30. The following table gives the highest and lowest temperatures for each month of the year for the twelve years ending 1883 at places in or adjacent to this region:

Highest and lowest temperatures for each month of the year for the twelve years ending 1883.[a]

Locality.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept	Oct.	Nov.	Dec.
Duluth:												
Maximum	51	57	62	75	91	92	99	93	90	78	65	51
Minimum	−38	−34	−26	3	26	36	46	45	30	8	−29	−34
Marquette:												
Maximum	56	69	70	81	92	95	100	96	97	87	66	59
Minimum	−26	−27	−14	3	22	31	40.3	39.7	28	18	−9	−20
Escanaba:												
Maximum	45	52	57	65	83	88	92	89	84	75	61	48
Minimum	−26	−32	−20	2	20	34	42	38	26	17	9	−23
Alpena:												
Maximum	52	58	66	72	91	97	97	92	92	83	63	56
Minimum	−27	−27	−14	2	22	33.5	45	40	29.3	22	−4	−15
St. Paul:												
Maximum	49	59	68	82	94	94	100	98	94	87	72	56
Minimum	−31	−32	−22	7	24	39	46	43	30	15	−24	−30
La Crosse:												
Maximum	59	65	72	83	96	98	101	96	92	84	70	60
Minimum	−43	−34	23	10	29	40	52	44	31	18	−21	−37

[a] King, F. H., Northern Wisconsin Handbook, 1896.

In connection with the sudden lowering of the winter temperature, a most interesting phenomenon was observed on St. Croix River by United States engineers in the early winter of 1882:[a]

> This was the apparently close relation between the temperature and the mean velocity and discharge of the stream, the stand of the water being at the same time nearly constant. In the early winter it was found that each cold wave which increased the thickness of the ice about one-tenth of a foot at a time was accompanied by a great falling off of the discharge, to be followed by a partial recovery during the next few days, the same phenomenon recurring with great regularity at each cold wave. The recovery of discharge being in each case only partial, the gradual tendency was downward until the apparent minimum was reached, when there was no appreciable change for several weeks.

PRECIPITATION.

The average rainfall for twenty-five years over the entire State is close to 32.3 inches, distributed by seasons as follows: Winter, 4.7 inches; spring, 7.6 inches; summer, 11.7 inches; autumn, 8.3 inches. If the rainfall of the northern half alone be considered, these figures would probably need to be slightly increased. It is worthy of note that 60 per

[a] Rept. Chief Eng. U. S. Army, 1883, p. 1470.

cent of the rainfall comes in the summer and autumn months, while the least fall is during the winter months. December, January, and February are the months of minimum run-off, both because of smaller precipitation and because of low temperatures and resulting deep frosts.

In general, it may be said that the precipitation in Wisconsin exceeds that of Minnesota and Michigan and about equals that of Iowa.

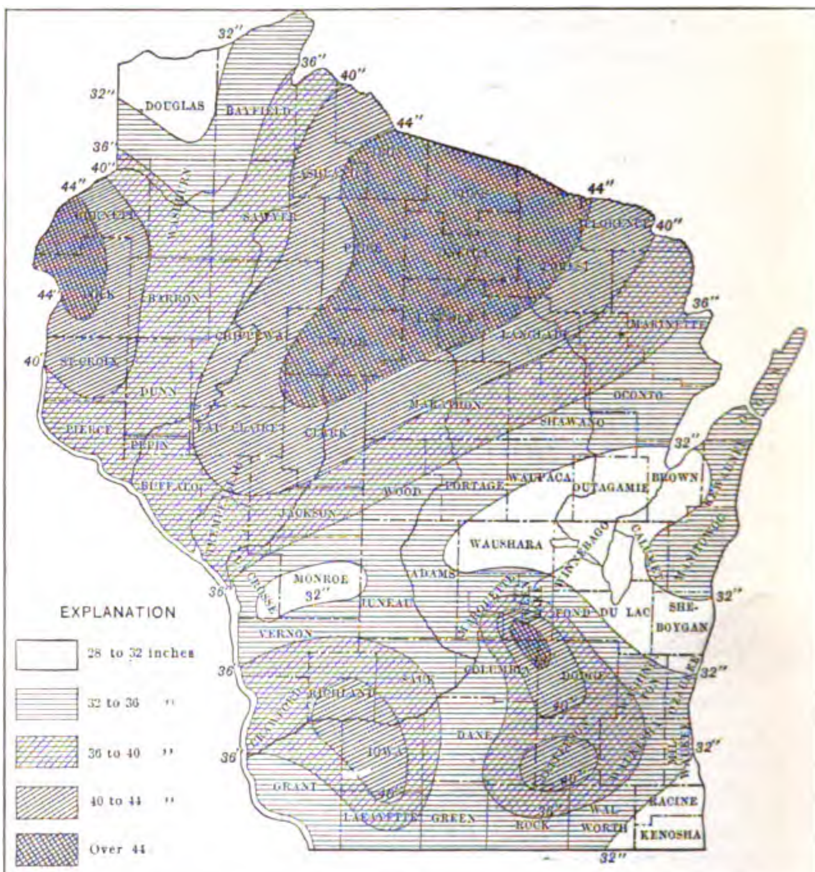


FIG. 1.—Rainfall map of Wisconsin.

Because of its bearing on the run-off of the various river systems, whose discharge measurements for 1903 and 1904 are herein given, the precipitation map shown in fig. 1 has been prepared. It will be noted that the heaviest rainfall occurred in the northern part of the State, averaging about 40 inches.

The following table shows some details of the distribution of rainfall by months:

Average precipitation at five stations in Wisconsin for twenty years.[a]

Detail.	Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
PERCENTAGES.													
Distribution	100	4.9	4.3	6.1	7.2	10.5	13.8	12.1	11.0	12.0	7.9	5.5	4.9
Classification of days:													
On which rain fell—													
Mean	41.2	43.0	41.9	40.4	40.6	43.7	48.1	41.1	38.2	41.7	36.1	39.3	43.5
Maximum	56.5	77.4	79.2	59.4	59.3	67.5	70.0	66.2	60.6	70.0	60.8	71.0	79.6
Minimum	27.0	18.7	11.3	19.3	21.9	20.7	30.6	17.4	21.3	15.9	12.8	14.9	19.3
Without rain	58.7	57.9	58.1	60.2	59.6	56.3	51.9	58.9	61.8	57.6	63.9	60.7	56.5
Trace to 0.25′	31.4	41.1	36.8	32.4	30.6	30.6	30.9	27.6	25.7	28.4	26.2	32.1	37.7
0.25′ to 0.50′	5.0	3.3	3.4	4.1	5.8	5.3	7.5	5.8	5.3	6.3	4.8	3.9	3.7
0.50′ to 1.00′	3.4	1.4	1.5	2.8	3.0	4.4	5.7	4.9	4.6	4.6	3.6	2.5	1.7
1.00′ to 2.00′	1.4	.3	.3	.4	1.2	1.8	3.2	2.0	1.9	2.6	1.6	.7	.4
2.00′ to 3.00′	.2	.2	....	.1	.3	.4	.7	.7	.6	.3	....	....	....
3.00′ to 5.00′	.1	....	....	....	....	.1	....	.2	.1	.2	....	....	....
Over 5′	.2	....	....	....	....	....	....	....	....	.2	....	....	....
NUMBER OF DAYS.													
Greatest consecutive—													
With rain	26	12	16	12	13	13	11	13	15	13	19	14	14
Without rain	14	9	9	8	8	9	13	8	8	8	8	9	13
INCHES OF RAIN.													
Heaviest in 1 day	7.23	2.6	1.8	2.1	2.9	3.1	2.9	4.5	3.9	5.6	7.23	1.8	1.5

[a] Moore, W. L., Rainfall of the United States: Bull. D, U. S. Dept. Agriculture.

The amount of precipitation is fairly constant for the winter and a portion of the fall and spring months, but varies considerably in the summer months.

Exceptionally dry periods occur about once in fifty years, when the average for three consecutive years is 22 inches and the least for one year is 13.5 to 20.5 inches. Dry periods occur once in twenty-five years, when the average for three consecutive years is 24.2 inches and the least for one year is 20.3 inches. Moderately dry periods occur once in ten years. The exceptionally dry periods are preceded by an exceptionally wet period, when the annual precipitation has been as high as 50 inches. This is followed by a period of moderately heavy rainfall, with a maximum of 45 inches. The last exceptionally dry period occurred in 1894 to 1896.[a]

The year 1903 had a moderately heavy rainfall. If the above cycle can be depended on, the next period of maximum rainfall may be expected about the year 1908.

Fig. 2 shows the progressive averages of the precipitation at Milwaukee for the past seventy years, computed by the formula—[b]

$$\frac{a+4b+6c+4d+e}{16}=c'$$

where c represents the rainfall of the year in question and b and a stand for the rainfall in the two years preceding, while d and e represent the rainfall of the following two years.

[a] Kirchoffer, W. G., master's thesis.
[b] After Blandford. See Bull. D, U. S. Weather Bureau.

This curve makes clearer the nature of the rainfall cycle.

In the following table are shown the long-term precipitation records of four typical

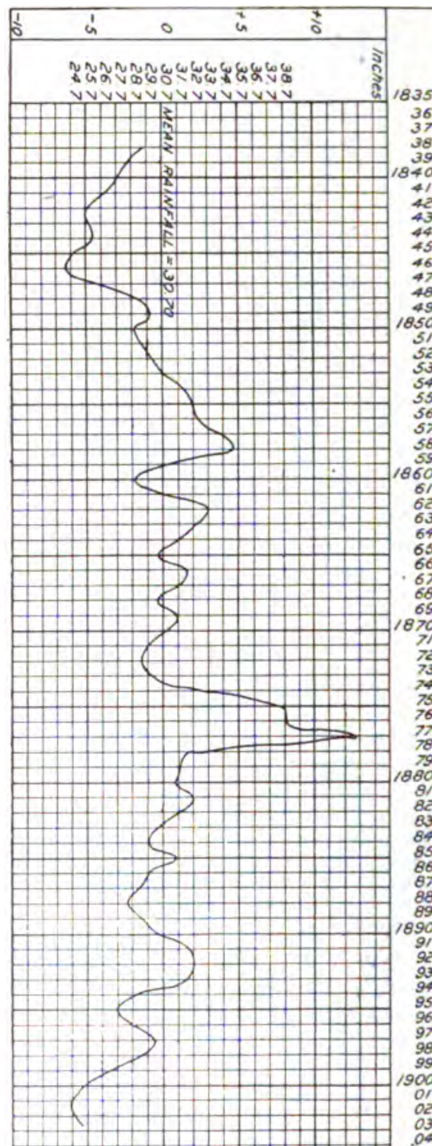


FIG. 2.—Chart showing rainfall at Milwaukee, 1837-1904.

stations in this general region. The rainfall at Milwaukee appears to be considerably less than the average of the State.

Precipitation at Milwaukee and Embarrass, Wis., and Duluth and St. Paul, Minn.

Year.	Milwaukee.	Embarrass.	Duluth.	St. Paul.	Year.	Milwaukee.	Embarrass.	Duluth.	St. Paul.
1845	20.5	....	....	....	1875	35.6	43.9	27.0	30.7
1846	25.3	....	....	....	1876	50.4	48.9	32.3	23.6
1847	22.4	....	....	....	1877	46.2	34.4	34.3	28.7
1848	33.5	....	....	....	1878	38.3	37.6	28.1	22.6
1849	31.1	....	....	....	1879	24.9	41.6	45.3	32.5
1850	26.4	....	....	....	1880	30.0	49.8	38.2	29.8
1851	30.4	....	....	....	1881	39.1	57.4	37.6	39.2
1852	29.3	....	....	....	1882	28.4	49.0	38.0	23.1
1853	30.0	....	....	....	1883	29.5	42.2	23.2	26.5
1854	31.7	....	....	....	1884	30.6	62.1	35.8	26.1
1855	36.0	....	....	....	1885	32.6	42.6	20.0	25.3
1856	29.0	....	....	....	1886	31.5	45.4	33.3	22.9
1857	30.9	....	....	....	1887	30.5	43.6	28.5	25.9
1858	44.9	....	....	....	1888	23.5	43.9	27.3	25.8
1859	28.9	....	....	29.1	1889	31.7	33.8	32.0	17.1
1860	24.0	....	....	34.2	1890	30.1	44.0	24.1	23.5
1861	31.9	....	27.7	30.5	1891	29.8	41.2	29.5	21.8
1862	38.3	....	35.4	34.5	1892	35.0	44.9	28.5	32.6
1863	31.8	....	22.8	15.7	1893	32.9	23.1	23.3	26.0
1864	27.8	28.9	17.8	14.9	1894	27.8	....	31.7	25.8
1865	30.1	36.3	20.9	38.1	1895	24.9	16.7	22.3	24.3
1866	34.0	34.5	30.6	27.9	1896	29.0	32.4	27.1	34.7
1867	24.6	29.0	25.7	33.6	1897	31.0	25.3	30.9	30.5
1868	29.4	38.8	....	30.7	1898	32.4	28.1	19.7	25.3
1869	37.8	39.3	....	32.2	1899	22.8	27.8	30.5	27.5
1870	26.6	41.9	....	32.1	1900	30.1	....	23.1	34.2
1871	32.0	37.7	31.2	30.7	1901	18.1	....	26.7	25.8
1872	26.2	28.5	30.1	29.6	1902	28.6	....	26.1	31.8
1873	30.6	35.0	38.8	34.6	1903	23.4	....	28.0	37.9
1874	30.8	31.0	36.5	35.5	1904	29.9	....	24.5	34.1

FOX RIVER SYSTEM.

DRAINAGE.

Lake Winnebago, the largest inland lake in Wisconsin, divides Fox River into two radically different sections, the upper and the lower Fox. The upper river approaches from the southeast to within about a mile of Wisconsin River at Portage, then turns to the northeast on its course to Lake Winnebago. It winds, with low banks, through broad savannas having only a gentle slope, passing a total distance of 25 miles through three long lakes before reaching Lake Winnebago.

Mud Lake, Buffalo Lake, and Lake Puckaway have been caused by the deposits of affluents which the main stream has not been able to wash away, plainly indicating that the present upper Fox did not erode its course, for it has not even the power to keep itself free, but instead is filling up. Lake Butte des Morts and Lake Winnebago are depressions which the present tendency is to fill up.[a]

Major Warren's hypotheses for these peculiar conditions have been widely accepted, and are so interesting that they are here given:

We have only to suppose that all the waters of Lake Winnebago basin (including that of the upper Fox) formerly drained to Wisconsin River; that a slow change of level in this region elevated the southwestern part and depressed the northeastern part till a large lake was formed, which finally overflowed, forming the course of the lower Fox. This explains the present doubling back in the course of the

[a] Warren, G. K., Rept. on Wisconsin and Fox rivers, 1876.

upper Fox and tributaries, and it accounts for the close relation and yet opposite courses of Fox and Wisconsin rivers. As the level changed the erosion at the outlet could not keep pace with it and so prevent a lake forming, because a granite ridge lies near the surface between the Wisconsin and Buffalo Lake. When the lower Fox outlet formed the loose material covering the rocks rapidly gave way and lowered the lake level down to the rock, which now (1875) keeps it to its present level. The period of this change was post-Glacial, because this alluvial terrace is free from Glacial drift, which it could not have been if formed before in a region like this, surrounded by Glacial drift deposit.

UPPER FOX.

Fox River descends only 40.4 feet in the 95.5 miles between Portage and Lake Winnebago—an average fall of less than 0.5 foot to the mile.

The following table shows the river profile in detail as given by United States engineers.

Profile of Fox River from Lake Winnebago (Oshkosh) to Portage lock (Fort Winnebago).

Station.	Distance— From Lake Winnebago.	Distance— Between points.	Elevation above sea level.	Descent between points. Total.	Descent between points. Per mile
	Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
Lake Winnebago	0	....	746.1	....	
Eureka lock, crest	24.6	24.6	748.8	2.7	
Berlin lock, crest	32.9	8.3	750.6	1.8	
White River lock, crest	33.9	10.0	755.7	5.1	
Princeton lock, crest (Lake Puckawa)	43.3	9.4	760.2	9.4	0.C
Grand River lock, crest	64.0	20.7	763.9	3.7	
Montello lock, crest (Lake Buffalo)	67.3	3.3	768.9	5.0	
Governor Bend lock, crest	91.4	24.1	774.7	5.8	
Fort Winnebago lock (Portage)	95.5	4.1	781.6	6.9	

Fox River has been improved for navigation by the Federal Government along this entire distance by the building of 10 locks, but the slight fall gives few opportunities for water power.

The first dam on the Fox is at Pardeeville, where a head of 14 feet is available. Wisconsin River is about 10 feet above Fox River at Portage, and this fall could be utilized by a dam near the Fort Winnebago lock. A considerable quantity of water could be discharged through the canal with safety.

At Montello, 28 miles below, a turbine is installed under a head of 3 feet, developing power for a gristmill. No developed power is in use on the river below this point.

The three principal tributaries of the upper Fox have a fall of about 250 feet—much greater than that of the main river; they are all found on the north side. These branches, Montello, Mecan, and White rivers, start as clear, steady springs, running from the sand ridges of the drift covering that portion of the basin. They are each about 20 miles long, and would be unimportant except for the fact that their fall, combined with their steadiness of flow, makes them of considerable value.

Montello River joins the upper Fox at Montello. A dam at this point has a head of 11 feet, furnishing power for a flouring mill and a woolen mill. This head could be easily increased to 16 feet.

The following table shows the principal developed powers on the tributaries of the upper Fox.

Developed water powers on tributaries of upper Fox River.

Location and stream.	Owner and use.	Head.	H. P.
Hatton, Little River	C. F. Stollyman, flour and feed	10	33
Lawrence, Duck Creek	C. E. Pierce, flour and feed	11	70
Manchester, Grand River	G. Pfeiffer, flour	12	45
Marblehead, De Nevue Creek	D. I. Williams, flour and feed	30	30
Markesan, Grand River	P. Wieski, flour and feed	11	30
Oxford, Neenah Creek	H. Larmer, flour and feed	9	70
Do	H. E. McNutt, flour and feed	14	190
Pine River, Pine Creek	Skinner & Johnson, flour and feed	14	60
Poysippe, Pine Creek	W. H. Paulsen, flour and feed	9	70
Princeton, ditch from Mecan River	Teske & Zierka, flour, feed, and electric light	19	180
Ripon, Silver Creek	Nohr Milling Co., flour and feed	12	120
Saxeville, Pine Creek	B. W. Heald, flour and feed	10	54
Waumander, Waumander Creek	A. G. Ochsner, flour	15	27
Wautoma, Mecan River	William Henke, flour and feed	8	36
Westfield, Montello River	Cochran & Nettinger, flour and feed	10	85

LOWER FOX.

GEOLOGY AND TOPOGRAPHY.

East of Wolf River Valley is the more prominent though similar valley of Green Bay and Lake Winnebago. In pre-Glacial time it must have been much smaller in size, having been excavated to its present great size by the glacier. Lake Winnebago alone covers about 200 square miles, while the area of the connecting valley below (lower Fox River) is 400 square miles.

The western slope of both valleys is gradual, but the eastern slope is precipitous, being cut out of the soft Cincinnati shales overlain by the hard "Niagara" limestone. The bed is the hard "Galena" limestone of the "Trenton" series. The eastern side of the lower Fox River drainage basin rises abruptly 100 to 200 feet above the water in Green Bay, and continues as a line of cliffs along the eastern shore of the present Lake Winnebago, and thence southward, though largely covered with drift in the southern part of the State. The glacial action sent down an immense ice sheet, cutting out the valley of Lake Michigan, while a branch tongue gouged out Green Bay Valley to its present size. On the peninsula between Green Bay and Lake Michigan was formed the prominent Kettle Range, a medial moraine.

The floor of Green Bay Valley has a rapid rise, Lake Winnebago being 166 feet above Green Bay. The portion of the old valley now occupied by the upper Fox was largely filled with drift, and it seems probable that to the action of the glacier in cutting down the intervening "Lower Magnesian" rampart and in partially filling the upper valley of Fox River is due the change in the flow of upper Fox and Wolf rivers through the newly enlarged Green Bay Valley to the lake. It is also likely that the change in flow is partly due to a depression toward the north, which occurred during or after the recession of the glacier, as suggested by Major Warren. This depression caused an advance of Lake Michigan, which rearranged the drift and deposited the red clays. By means of the latter this ancient shore of the lake can now be traced northward beyond Shawano, on Wolf River, westward up Fox River above Berlin, and southward to a few miles north of Fond du Lac. Lake Winnebago is a comparatively modern reservoir, formed in the valley by the deposition of glacial drift.

PROFILE.

The table below gives in detail the profile of the river to-day, after the extensive navigation improvements by the United States Government:

Profile of Fox River from Lake Winnebago (Menasha) to Green Bay.[a]

Station.	Distance.		Elevation above sea level.	Descent between points.	
	From Menasha.	Between points.		Total.	Per mile.
	Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
Menasha dam, crest	0.0	....	746.1	....	....
Appleton upper lock, crest	5.1	5.1	736.5	9.6	[illegible]
Appleton locks, foot	6.3	1.2	699.7	36.8	[illegible]
Cedars lock, crest	9.6	3.3	699.7	.0	[illegible]
Littlechute locks:					
Crest	10.6	1.0	690.0	9.7	[illegible]
Foot	11.6	1.0	653.8	36.2	[illegible]
Grand Kaukauna locks:					
Crest	13.3	1.7	653.8	.0	[illegible]
Foot	14.2	.9	603.3	50.5	[illegible]
Rapide Croche lock:					
Crest	17.9	3.7	603.3	.0	[illegible]
Foot	17.9	.25	593.9	9.4	[illegible]
Little Kaukauna lock:					
Crest	23.9	6.0	593.9	.0	[illegible]
Foot	23.9	.2	587.7	6.2	[illegible]
Depere lock:					
Crest	29.8	5.9	587.7	.0	[illegible]
Foot	29.8	.0	580.0	7.7	....
Green Bay	35.2	5.4	580.0	.0	....

[a] From United States engineer's profile of the river.

These improvements have changed the river into long stretches of slack water, with perhaps short rapids at the foot of a dam, except at Grand Kaukauna and Grand Chute, the site of the city of Appleton, where the rapids are passed by canals, while the river flows over its original steep bed.

RAINFALL AND RUN-OFF.

The United States engineers have maintained a gaging station at Rapide Croche dam ever since March, 1896. The assistant engineer in charge, L. M. Mann, states that the crest of the dam at this point is well suited for a weir. Care is taken to read the gage three times daily, the mean reading being used to calculate the daily discharge.

According to these records the mean low-water discharge for the past eight years was 1,409 second-feet and the average discharge 3,007 second-feet; 2,660 second-feet may be regarded as the ordinary flow of the river. Because of the steadying effect of Lake Winnebago and the lakes above, formed by the expansion of upper Fox and Wolf rivers, the discharge of the river is remarkably uniform. At Appleton the ordinary variation from low to high water is scarcely more than 2 or 3 feet throughout the year.

The following table gives the maximum, the minimum, and the average flow for each month for nearly nine years, ending December, 1904, as measured at Rapide Croche dam, and also the rainfall and run-off for the same period:

Estimated monthly discharge of lower Fox River at Rapide Croche dam.

[Drainage area, 6,200 square miles.]

Month.	Discharge in second-feet.			Run-off.		Rainfall.	
	Maximum.	Minimum.	Mean.	Second-feet per square mile.	Depth in inches.	Inches.	Per cent of rainfall.
1895.							
January	4,972	2,262	3,931	0.634	0.731	....	....
February	5,201	2,545	4,320	.697	.726	....	....
March	5,796	2,062	3,947	.637	.734	....	....
April	12,706	3,076	8,510	1.37	1.53	....	....
May	6,386	4,233	5,610	.905	1.04	....	....
June	15,416	6,628	12,760	2.06	2.30	....	....
July	11,982	3,451	7,612	1.23	1.42	....	....
August	5,173	3,047	4,424	.714	.823	....	....
September	5,072	2,242	3,988	.643	.717	....	....
October	4,185	2,071	3,417	.551	.635	....	....
1896.							
March	1,739	697	1,284	.207	.239	1.14	21.0
April	1,765	406	940	.152	.170	4.39	3.87
May	4,246	1,563	3,140	.506	.583	5.23	11.1
June	4,605	2,173	3,726	.601	.670	2.75	24.4
July	3,863	880	2,787	.450	.519	3.09	16.8
August	2,607	123	1,470	.237	.273	3.09	8.83
September	390	9	146	.024	.027	3.23	.84
October	1,888	145	1,065	.172	.198	2.55	7.76
November	2,882	985	2,007	.324	.362	3.06	11.8
December	3,558	838	2,367	.382	.440	1.04	42.3
The year	....	....	....	....	....	29.57	....
1897.							
January	3,795	1,512	2,762	.445	.513	1.37	37.5
February	3,522	1,297	2,765	.446	.464	1.17	39.6
March	5,344	1,160	2,711	.437	.504	2.19	23.1
April	8,728	3,296	6,132	.989	1.10	2.00	55.0
May	5,344	2,519	4,016	.648	.747	1.74	42.9
June	4,749	2,032	3,246	.524	.585	5.06	11.6
July	4,071	1,297	3,200	.516	.595	3.51	16.9
August	3,230	116	1,881	.303	.349	2.00	17.4
September	1,588	272	833	.134	.150	2.53	5.9
October	2,608	299	1,424	.230	.265	2.15	12.3
November	2,664	861	1,862	.300	.335	1.50	22.3
December	3,770	806	2,314	.373	.430	.86	50.0
The year	8,728	116	2,762	.445	6.04	26.08	23.2
1898.							
January	3,158	1,425	2,559	.413	.476	.71	67.1
February	3,196	1,494	2,359	.380	.397	1.21	32.8
March	3,872	1,782	2,968	.479	.552	2.18	25.3
April	5,692	2,568	4,079	.658	.734	2.02	36.4
May	6,852	2,204	4,743	.765	.882	2.75	32.1
June	4,969	1,604	3,216	.519	.579	3.84	15.1
July	2,553	438	1,571	.253	.292	3.09	9.45
August	2,805	866	1,817	.293	.338	3.00	11.3
September	1,795	442	1,088	.175	.195	2.36	8.25
October	2,368	383	1,201	.194	.224	3.15	7.10

Estimated monthly discharge of lower Fox River at Rapide Croche dam—**Continued.**

Month.	Discharge in second-feet. Maximum.	Discharge in second-feet. Minimum.	Discharge in second-feet. Mean.	Run-off. Second-feet per square mile.	Run-off. Depth in inches.	Rainfall. Inches.	Per cent of rainfall.
1898.							
November	2,725	1,234	2,213	0.357	0.398	1.49	26.7
December	2,805	994	2,175	.351	.405	.35	116.0
The year	6,852	383	2,499	.403	5.47	26.15	20.9
1899.							
January	2,417	771	1,905	.307	.354	1.12	31.6
February	2,810	1,014	2,075	.335	.349	.90	38.8
March	3,435	995	2,252	.363	.418	2.31	18.2
April	5,707	1,447	3,657	.590	.658	3.00	21.9
May	8,767	3,787	6,209	1.00	1.15	3.08	37.3
June	8,571	4,018	6,298	1.02	1.14	5.40	21.2
July	5,171	1,741	3,786	.611	.704	3.29	21.4
August	3,505	791	1,836	.296	.341	2.73	12.5
September	1,437	707	988	.159	.177	2.68	6.60
October	2,079	398	1,144	.185	.213	3.02	7.05
November	2,648	613	2,119	.342	.382	.74	51.6
December	2,572	105	2,042	.329	.379	1.47	25.4
The year	8,767	105	2,859	.461	6.26	29.74	21.1
1900.							
January	2,684	841	2,174	.351	.405	.74	54.7
February	3,024	1,044	2,247	.362	.377	1.56	24.2
March	3,677	1,110	2,556	.412	.475	1.09	43.6
April	4,355	1,107	3,414	.551	.615	2.82	21.8
May	4,054	1,383	2,976	.480	.553	1.61	34.5
June	2,208	258	873	.141	.157	2.68	5.86
July	2,413	131	958	.154	.178	6.45	2.76
August	2,646	1,057	1,831	.295	.340	4.30	7.91
September	3,518	1,107	2,021	.326	.364	6.17	5.90
October	8,036	1,734	5,230	.844	.973	7.08	13.7
November	9,597	4,948	8,062	1.30	1.45	1.57	92.4
December	8,222	1,668	4,353	.702	.809	.69	117.0
The year	9,597	131	3,058	.493	6.70	36.76	18.2
1901.							
January	4,349	1,939	3,526	.569	.656	.90	72.9
February	4,634	1,825	3,773	.609	.634	.46	138.0
March	6,431	1,742	3,839	.619	.714	3.04	23.5
April	12,033	2,469	8,960	1.45	1.62	.79	205.0
May	6,905	3,453	4,994	.805	.928	2.72	34.1
June	5,087	1,741	3,723	.600	.669	4.62	14.5
July	4,557	2,045	3,501	.565	.651	6.41	10.2
August	3,846	1,130	2,176	.351	.405	2.38	17.2
September	1,687	675	1,221	.197	.220	3.96	5.56
October	3,873	9,910	2,551	.411	.474	2.93	16.2
November	3,873	1,640	3,256	.525	.586	1.25	46.9
December	3,672	1,464	2,768	.446	.514	.81	63.5
The year	12,033	675	3,691	.596	8.07	30.27	26.7

Estimated monthly discharge of lower Fox River at Rapide Croche dam—Continued.

Month.	Discharge in second-feet.			Run-off.		Rainfall.	Per cent of rainfall.
	Maximum.	Minimum.	Mean.	Second-feet per square mile.	Depth in inches.	Inches.	
1902.							
January	3,136	765	2,263	0.365	0.421	0.69	61.1
February	3,480	696	2,142	.345	.359	1.53	23.5
March	4,019	1,135	2,892	.466	.537	1.50	35.8
April	3,252	947	2,335	.377	.421	2.42	17.4
May	12,317	1,471	4,935	.796	.918	4.02	22.8
June	11,868	3,491	6,930	1.12	1.25	3.89	32.1
July	5,703	1,647	4,304	.694	.800	5.47	14.6
August	4,086	1,311	2,896	.467	.538	1.40	38.4
September	1,865	515	1,266	.204	.228	2.81	8.11
October	3,024	435	1,818	.293	.338	1.94	17.4
November	3,184	756	2,394	.386	.431	2.90	14.9
December	3,100	892	2,274	.367	.423	1.93	21.9
The year	12,317	435	3,037	.490	6.66	30.50	21.8
1903.							
January	3,756	1,206	2,760	.445	.513	.47	109.5
February	3,652	1,675	2,949	.476	.496	.80	62.0
March	8,437	1,780	3,827	.617	.711	3.12	22.8
April	9,297	3,886	6,500	1.05	1.17	3.14	37.3
May	7,378	3,043	5,532	.892	1.03	5.87	17.5
June	6,791	2,656	5,061	.816	.910	2.14	42.5
July	5,571	1,856	4,124	.665	.767	5.47	14.0
August	4,449	1,438	3,446	.556	.641	6.23	10.3
September	5,519	1,829	4,321	.697	.778	5.91	13.2
October	5,826	2,590	4,686	.756	.872	2.75	31.7
November	5,077	1,733	3,686	.595	.664	1.14	58.3
December	3,702	1,319	2,885	.465	.536	.71	75.5
The year	9,297	1,206	4,148	.669	9.09	37.75	24.1
1904.							
January	3,869	1,185	3,074	.496	.571	.38	150.0
February	4,134	1,565	3,128	.505	.545	1.45	37.6
March	7,425	1,724	3,398	.548	.632	1.80	35.1
April	9,637	1,612	6,669	1.08	1.20	1.86	64.5
May	11,682	4,456	8,707	1.40	1.61	5.93	27.1
June	9,793	2,336	6,682	1.08	1.20	3.99	30.1
July	4,111	1,416	3,105	.501	.578	3.98	14.5
August	4,043	1,551	2,985	.481	.554	3.01	18.4
September	2,631	988	1,854	.299	.334	5.75	5.81
October	6,434	1,324	3,457	.558	.643	4.73	13.6
November	6,935	1,667	4,056	.654	.730	.30	243.5
December	4,594	1,812	3,618	.584	.673	2.13	31.6
The year	11,682	988	4,228	.682	9.270	35.31	26.2

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1895.												
1	3,012	4,672	4,697	4,798	4,373	6,628	11,982	4,884	4,608	2,071	......	......
2	2,262	4,328	4,438	3,185	5,852	7,339	10,639	4,963	5,072	2,132	......	......
3	3,350	4,766	4,285	3,076	5,880	6,935	10,433	4,964	3,387	3,546	......	......
4	3,781	5,164	4,260	5,064	5,910	7,225	10,585	4,936	3,048	3,584	......	......
5	4,447	3,765	2,846	5,595	5,949	8,277	10,664	5,173	3,788	3,306	......	......
6	4,269	2,689	2,611	5,998	6,135	15,416	10,639	3,411	4,102	3,554	......	......
7	4,285	4,785	3,982	5,804	4,493	14,244	9,415	3,385	4,487	3,932	......	......
8	3,024	4,973	3,998	7,052	4,233	14,178	9,206	4,382	4,395	3,057	......	......
9	2,967	5,063	4,021	8,803	5,546	15,132	7,865	4,847	4,430	2,504	......	......
10	4,244	5,009	4,419	9,422	6,386	14,286	7,416	4,898	3,969	3,899	......	......
11	4,422	5,173	4,214	11,435	6,155	14,122	8,803	4,242	3,106	3,971	......	......
12	4,594	4,061	2,704	12,354	6,037	14,060	9,124	4,847	4,372	4,022	......	......
13	4,559	2,892	2,264	12,706	6,066	14,559	9,066	3,577	4,144	3,884	......	......
14	4,102	4,830	2,062	12,187	4,642	14,585	8,769	3,383	4,327	3,764	......	......
15	3,112	5,201	3,892	12,033	4,399	14,219	8,521	4,849	4,447	2,774	......	......
16	2,497	4,785	3,685	11,063	6,179	14,365	7,165	4,937	4,243	2,227	......	......
17	3,916	4,723	3,876	11,110	6,165	14,588	6,699	4,937	2,856	3,955	......	......
18	4,244	4,637	4,313	11,637	6,274	13,981	8,860	5,027	2,826	4,052	......	......
19	4,602	3,369	2,529	12,277	5,891	14,277	7,873	4,785	4,236	4,086	......	......
20	4,654	2,545	2,098	11,879	6,115	14,279	6,597	3,047	4,405	3,939	......	......
21	4,619	4,839	4,228	10,336	4,803	14,191	6,577	3,526	4,447	3,956	......	......
22	3,027	4,910	4,490	10,639	4,386	13,419	5,804	4,794	4,294	2,088	......	......
23	2,963	4,847	5,796	9,406	5,804	13,674	4,583	4,892	4,337	2,173	......	......
24	4,742	4,777	5,757	9,032	6,047	13,513	3,451	5,036	2,974	3,917	......	......
25	4,829	4,356	5,626	9,208	5,880	12,524	5,797	5,019	2,242	4,021	......	......
26	4,576	3,055	4,508	7,661	5,880	11,892	6,179	4,804	4,030	4,069	......	......
27	4,972	2,575	5,546	5,862	5,841	14,021	5,498	3,593	4,421	4,081	......	......
28	4,653	4,186	4,353	6,164	4,566	12,265	5,545	3,518	4,162	4,160	......	......
29	3,413	......	4,268	6,144	5,129	12,509	5,192	4,145	[illegible]	2,710	......	......
30	3,137	......	4,395	3,362	6,597	12,251	3,562	4,086	4,127	2,263	......	......
31	4,576	......	4,185	......	6,312	......	3,480	[illegible]	......	4,185	......	......
Total	121,850	120,975	122,346	255,292	173,925	382,954	235,989	137,156	119,657	105,942	......	......
1896.												
1	......	......	888	1,272	2,406	2,789	3,694	912	259	145	985	[illegible]
2	......	......	697	1,027	2,088	4,246	3,728	1,048	298	608	1,048	[illegible]
3	......	......	1,386	922	1,563	4,246	3,761	1,512	179	838	1,512	[illegible]
4	......	......	1,226	760	1,639	4,141	2,032	2,314	78	608	1,537	[illegible]
5	......	......	1,395	1,048	3,493	4,282	880	2,461	36	683	1,872	[illegible]
6	......	......	1,338	780	3,166	3,967	1,820	2,578	36	1,440	1,872	[illegible]
7	......	......	1,430	761	3,312	3,897	3,592	2,607	179	880	1,792	[illegible]
8	......	......	1,113	922	2,993	3,135	3,394	2,490	328	1,070	1,563	[illegible]
9	......	......	919	859	3,230	4,461	3,863	1,250	390	880	1,093	[illegible]
10	......	......	1,271	964	2,032	4,282	3,728	1,392	122	1,093	2,201	[illegible]
11	......	......	1,479	859	1,575	4,605	3,761	1,818	192	608	2,229	[illegible]
12	......	......	1,407	406	3,361	4,389	2,201	1,818	136	779	2,638	[illegible]
13	......	......	1,407	644	3,427	4,106	1,952	1,765	48	1,093	2,720	[illegible]
14	......	......	1,271	859	3,476	3,296	3,592	1,723	48	964	2,729	[illegible]
15	......	......	981	922	3,558	2,913	3,525	2,006	205	922	1,115	[illegible]
16	......	......	837	985	3,396	4,037	3,525	965	145	1,093	1,160	[illegible]
17	......	......	1,437	1,048	2,686	3,796	3,694	1,352	192	1,138	2,460	[illegible]
18	......	......	1,248	985	2,460	3,897	3,296	2,006	134	608	2,729	[illegible]
19	......	......	1,486	644	3,591	3,897	1,872	2,032	78	722	2,668	[illegible]
20	......	......	1,366	608	3,897	3,897	1,672	2,061	195	1,362	2,729	[illegible]

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1896.												
21	....	....	1,414	943	3,897	2,431	2,913	2,006	17	1,647	2,882	1,613
22	....	....	1,024	943	3,897	2,461	2,607	1,899	145	1,440	1,639	2,519
23	....	....	1,024	901	4,071	3,967	2,314	2,454	134	1,563	1,440	2,460
24	....	....	1,739	985	2,729	4,211	2,607	838	36	1,613	2,008	2,431
25	....	....	1,461	1,273	2,387	3,932	2,607	327	112	543	2,789	838
26	....	....	1,224	556	3,932	4,141	1,563	123	122	556	2,821	1,160
27	....	....	1,179	780	4,246	3,897	1,563	259	27	1,440	2,669	838
28	....	....	1,605	1,027	4,071	2,608	2,913	375	9	1,272	1,818	1,792
29	....	....	1,337	1,765	4,161	2,173	2,669	406	374	1,845	1,205	2,913
30	....	....	1,522	1,765	4,071	3,693	2,490	453	134	1,888	1,672	2,607
31	....	....	1,707	....	2,519	....	2,578	312	....	1,672	....	2,229
Total	....	....	39,818	28,213	97,330	111,793	86,406	45,582	4,388	33,013	60,204	73,362
1897.												
1	2,201	1,563	1,472	3.728	5,344	3,263	3,761	2,314	468	1,160	1,160	1,820
2	2,173	3,040	3,263	4,037	4,071	3,659	3,863	1,765	621	1,070	1,983	1,832
3	1,512	3,361	3,263	4,966	4,002	3,727	4,037	3,103	343	702	2,209	2,128
4	2,088	3,103	3,135	3,296	4,886	3,727	2,490	3,103	328	556	2,615	2,226
5	2,789	3,072	3,395	4,037	5,079	3,761	1,297	3,230	328	1,138	2,642	1,638
6	3,104	3,263	2,608	5,624	4,713	2,201	2,173	3,072	390	1,115	2,664	1,196
7	3,394	1,440	1,183	6,329	4,605	2,402	4,002	3,040	272	1,160	1,973	2,401
8	3,008	1,563	1,393	6,329	4,354	3,361	4,071	1,926	539	1,183	1,390	2,474
9	2,759	3,008	2,431	6,533	3,198	3,230	3,932	1,416	702	985	1,861	2,497
10	2,607	3,361	2,490	6,533	2,759	3,329	3,932	2,760	556	741	1,964	2,748
11	2,286	3,040	2,461	5,231	4,354	3,459	2,402	2,608	664	556	1,912	2,568
12	3,072	3,198	2,578	4,789	4,497	3,659	2,402	2,490	390	1,345	1,964	1,905
13	3,198	3,198	2,461	6,614	4,461	2,229	3,459	2,490	390	1,512	2,183	1,102
14	3,072	2,189	1,512	6,410	4,141	2,229	3,558	2,608	819	1,512	1,481	2,512
15	3,040	1,297	1,160	6,533	4,425	3,394	3,694	1,440	1,416	1,897	1,092	3,409
16	2,788	3,103	2,490	6,329	3,329	4,749	3,897	1,345	1,588	1,952	1,833	3,439
17	1,644	3,072	2,314	6,779	2,913	4,037	3,694	3,431	1,440	1,048	1,912	3,770
18	1,872	3,230	2,619	7,072	4,106	3,727	2,229	2,638	1,205	722	1,964	1,880
19	3,230	3,198	3,460	5,419	4,282	3,727	2,006	2,431	761	1,897	2,432	2,213
20	3,795	3,361	5,344	7,114	4,425	2,286	3,394	1,765	702	2,061	2,412	1,559
21	3,459	1,639	2,821	7,582	4,318	2,314	3,558	1,093	943	1,897	1,387	2,424
22	3,198	1,723	2,461	7,326	4,461	3,525	3,761	1,048	1,205	1,897	861	2,665
23	3,394	3,135	3,694	8,728	3,072	3,525	3,694	1,352	1,393	299	2,045	2,642
24	2,759	3,522	3,626	8,549	2,759	3,592	3,525	1,273	1,369	1,115	1,207	2,732
25	1,899	3,378	3,328	6,946	4,141	3,263	1,926	1,273	1,160	1,239	2,476	1,878
26	2,945	3,458	3,394	6,329	4,246	3,528	2,117	901	644	2,117	2,107	806
27	2,945	3,135	3,135	7,539	4,246	2,343	3,008	1,160	838	2,490	2,148	1,657
28	3,103	1,765	1,512	6,329	4,071	2,032	3,198	343	1,138	2,373	1,281	2,900
29	3,361	....	1,962	5,459	4,071	3,932	3,394	374	1,183	2,608	1,097	2,808
30	3,103	....	3,394	5,459	2,638	3,727	3,394	116	1,183	2,286	1,602	2,923
31	1,818	....	3,694	....	2,519	....	3,329	403	....	1,512	....	2,969
Total	85,616	77,415	84,053	183,948	124,486	97,397	99,197	58,311	24,978	44,145	55,857	71,721
1898.												
1	3,063	2,793	2,676	4,056	3,776	4,969	2,553	876	1,652	602	2,609	2,805
2	2,099	2,762	2,698	4,176	3,799	4,522	2,496	1,546	1,795	554	2,351	2,766
3	1,425	2,931	2,753	2,846	5,508	4,579	1,771	1,649	1,639	517	2,237	2,714
4	2,535	3,196	2,908	2,568	5,016	4.397	438	1,670	1,317	857	2,134	1,789
5	3,000	3,062	2,752	3,890	6,668	3,578	1,868	1,585	726	595	2,187	1,647
6	3,039	2,038	1,699	4,072	6,852	2,388	1,865	1,667	1,454	771	1,457	2,479

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—**Continued.**

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1898.												
7	2,946	1,766	1,782	4,159	6,098	4,018	1,769	973	1,532	801	1,234	2,608
8	2,977	2,504	3,064	4,115	4,921	4,408	1,639	1,074	1,629	1,024	2,617	2,699
9	1,873	2,665	3,669	4,150	4,333	4,276	1,466	1,653	1,735	383	2,475	2,572
10	1,396	2,583	3,761	2,925	5,889	4,175	1,096	1,685	1,714	638	2,485	1,379
11	2,953	2,701	3,394	2,837	5,904	4,364	1,042	1,695	1,133	1,056	2,497	2,153
12	2,969	2,679	3,192	4,291	5,683	3,215	1,535	1,660	750	1,092	2,542	1,539
13	2,901	1,723	1,868	4,273	5,487	2,524	1,757	1,835	1,288	1,246	1,543	2,345
14	2,946	1,578	1,902	4,054	5,432	3,690	1,798	1,213	1,115	1,155	1,451	2,637
15	3,079	2,331	3,224	4,189	4,018	3,703	1,842	1,226	1,186	1,156	2,438	2,791
16	1,864	2,549	3,288	4,301	3,457	3,723	1,792	1,765	1,080	745	2,453	2,715
17	1,569	2,490	3,111	2,924	4,551	3,167	1,074	2,273	1,177	755	2,552	2,215
18	2,908	2,468	3,135	3,025	5,088	3,690	1,245	2,460	778	1,300	2,563	1,343
19	2,799	2,497	3,435	4,372	4,758	2,343	1,699	2,805	442	1,178	2,482	1,462
20	3,078	1,528	1,938	4,650	4,647	2,076	1,639	2,752	760	1,304	1,747	2,453
21	2,859	1,587	2,235	4,776	4,739	2,453	1,616	2,024	818	1,892	1,281	2,409
22	2,915	2,403	3,280	5,692	3,577	2,623	1,664	866	959	1,865	2,704	2,398
23	1,925	2,409	3,200	4,894	3,157	2,603	1,685	2,571	906	1,136	2,549	2,241
24	1,741	2,505	3,122	3,176	4,338	2,624	1,088	2,579	933	760	2,499	2,303
25	2,962	2,403	3,643	3,462	4,594	2,488	989	2,482	676	2,043	2,725	994
26	2,985	2,505	3,361	4,656	4,471	1,623	1,557	2,468	491	1,815	2,572	1,017
27	3,077	1,494	2,565	5,089	4,739	1,604	1,641	2,572	681	1,845	1,486	1,544
28	3,158	1,914	3,035	5,136	4,612	1,863	1,618	1,942	693	2,064	1,402	2,412
29	2,964	....	3,872	4,767	3,857	2,230	1,687	1,195	877	2,233	2,606	2,446
30	1,851	....	3,728	4,839	2,204	2,567	1,661	1,848	693	1,480	2,519	2,315
31	1,461	....	3,736	....	4,872	....	1,117	1,717	....	2,368	....	2,244
Total	79,317	66,064	92,026	122,360	147,045	96,483	48,707	56,326	32,629	37,239	66,397	67,435
1899.												
1	1,533	2,111	1,931	3,279	3,787	5,209	5,042	3,345	1,292	831	2,104	2,305
2	1,488	2,174	2,113	1,837	5,020	5,238	4,031	3,505	1,437	764	2,648	2,455
3	2,366	2,275	2,134	1,447	5,121	5,432	3,133	3,121	842	1,560	2,542	1,667
4	2,417	2,261	2,113	3,756	5,417	4,733	3,678	2,396	1,003	1,037	2,446	952
5	2,187	1,453	1,352	4,249	5,395	4,018	3,361	2,418	1,411	996	1,742	2,549
6	2,393	2,458	1,279	4,641	5,500	5,518	4,923	1,745	1,121	1,193	1,261	2,411
7	2,173	2,175	1,976	4,839	4,446	5,633	5,171	956	991	1,100	2,579	2,475
8	1,466	1,303	1,932	4,425	4,145	5,565	4,726	2,043	953	928	2,622	2,572
9	1,465	2,681	2,027	2,823	5,924	5,062	3,942	2,585	996	774	2,523	2,530
10	2,316	2,572	2,062	2,249	6,192	5,369	3,019	2,394	792	964	2,635	1,775
11	2,329	2,810	2,034	3,741	6,618	4,406	4,592	2,343	578	922	2,619	1,030
12	2,316	2,047	2,355	3,821	7,601	4,334	4,715	2,357	945	1,056	1,890	2,366
13	2,106	1,711	1,336	3,812	8,050	5,878	4,882	1,547	825	1,144	1,143	2,496
14	2,406	2,588	1,913	3,976	7,301	7,091	4,852	1,250	891	982	2,187	2,352
15	1,514	2,646	2,289	3,721	7,763	7,080	4,974	1,852	889	779	2,381	2,395
16	1,604	2,534	2,692	2,199	8,562	7,681	3,773	1,812	831	398	2,351	2,339
17	2,254	2,619	2,612	2,004	8,767	7,408	3,021	1,829	831	969	2,352	1,780
18	2,379	2,453	2,553	3,461	7,838	6,702	4,516	1,946	1,001	1,039	2,261	1,259
19	2,352	1,243	1,553	3,946	8,421	6,853	4,511	1,973	1,166	1,179	1,774	2,576
20	2,347	1,291	995	3,778	8,431	8,123	4,186	1,026	996	1,304	613	2,381
21	2,330	1,902	2,731	3,955	7,046	8,095	3,881	1,163	1,039	1,327	2,301	2,385
22	1,409	2,117	3,001	4,154	6,272	8,571	3,701	1,578	991	885	2,205	2,417
23	1,506	2,152	2,756	2,798	5,263	8,515	2,459	1,609	855	685	2,237	2,497
24	1,729	1,995	2,832	2,813	5,333	8,277	1,741	1,772	707	1,480	2,209	1,752
25	1,562	2,104	2,829	4,336	5,451	7,338	3,193	1,317	719	1,551	2,352	105
26	1,690	1,395	1,910	4,524	5,263	6,594	3,336	1,330	922	1,404	1,873	958

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1899.												
27	1,981	1,014	1,243	4,565	5,525	7,372	3,364	944	1,138	1,829	905	2,123
28	771	2,007	3,192	5,707	5,898	6,029	3,395	791	1,099	2,029	2,329	2,185
29	1,506	....	3,297	4,913	5,160	5,797	3,232	1,307	1,206	1,493	2,159	2,417
30	1,346	....	3,325	3,947	5,466	5,007	2,051	1,351	1,166	769	2,330	2,241
31	1,803	....	3,435	....	5,513	....	1,982	1,299	....	2,079	....	1,659
Total	59,044	58,091	69,802	109,716	192,489	188,928	117,383	56,913	29,653	35,450	63,573	63,299
1900.												
1	841	2,395	2,432	2,454	3,636	1,482	276	2,646	1,976	1,734	9,535	6,008
2	2,352	2,366	2,468	1,107	3,882	1,574	511	2,106	1,640	4,007	9,397	4,372
3	2,391	2,446	2,490	3,554	4,028	1,375	319	1,946	1,421	5,185	9,597	3,512
4	2,406	1,641	1,707	3,846	4,010	1,102	131	1,966	1,962	5,479	8,225	5,674
5	2,479	1,044	1,110	3,967	4,054	1,929	820	1,638	2,141	4,846	7,650	5,943
6	2,568	2,366	1,342	4,063	2,779	2,208	352	1,287	1,894	4,758	8,989	5,807
7	1,576	2,381	2,576	3,791	1,674	2,085	345	1,956	2,050	3,657	9,039	5,683
8	1,213	3,024	2,639	2,903	2,461	1,467	294	1,912	2,113	2,848	8,418	5,439
9	2,406	2,420	2,614	1,751	2,283	1,302	365	2,075	1,333	4,827	8,658	8,222
10	2,669	2,398	2,636	3,510	2,275	989	473	1,973	1,143	5,165	9,260	3,586
11	2,684	1,675	1,921	3,770	2,240	1,013	353	2,043	1,973	5,117	7,830	5,245
12	2,646	1,071	1,352	4,067	2,285	1,217	333	1,498	2,071	4,813	7,567	5,052
13	2,677	2,316	2,666	4,006	1,784	1,189	394	1,057	2,187	4,668	8,775	5,075
14	1,736	2,654	2,973	4,080	1,753	756	382	1,825	2,075	3,559	8,456	5,301
15	1,236	2,707	3,036	2,560	3,741	353	538	1,905	2,184	3,028	8,076	4,099
16	2,527	2,729	2,961	2,237	3,989	497	979	1,889	1,163	4,461	6,611	2,725
17	2,424	2,662	2,935	3,942	3,799	394	1,176	2,039	1,167	4,885	6,909	2,240
18	2,639	1,895	2,069	4,355	3,885	437	1,067	2,002	1,949	5,285	5,654	4,118
19	2,534	1,314	1,354	4,089	3,791	573	907	1,434	1,899	5,254	4,948	4,292
20	2,657	2,861	2,710	4,137	2,665	523	1,192	1,120	2,062	5,703	7,536	4,314
21	1,719	2,840	2,832	4,072	2,105	569	1,152	2,120	2,020	4,599	7,875	4,378
22	1,043	2,606	2,904	2,932	3,729	585	799	2,007	2,169	4,225	8,249	4,457
23	2,632	2,687	2,954	1,977	3,937	264	1,170	1,875	1,433	6,510	8,189	4,035
24	2,549	2,646	3,296	3,856	3,872	262	1,846	1,966	1,107	6,270	8,792	1,668
25	2,461	1,434	2,259	4,089	3,889	396	2,047	1,924	2,329	7,427	7,100	3,224
26	2,579	1,296	1,300	4,106	3,694	258	1,982	1,440	2,891	7,572	6,917	2,621
27	2,420	2,411	3,159	4,063	2,539	360	1,979	1,115	2,810	7,624	8,415	4,068
28	1,508	2,624	3,644	4,225	1,383	298	1,905	1,912	3,391	6,393	8,632	4,147
29	1,219	....	3,677	2,862	2,103	386	1,536	1,928	3,518	6,336	8,328	4,111
30	2,166	....	3,560	2,064	2,149	341	1,672	2,127	2,561	7,855	8,239	3,191
31	2,439	....	3,652	....	1,859	....	2,413	2,023	....	8,036	....	1,728
Total	67,396	62,909	79,228	102,435	92,273	26,184	29,708	56,754	60,632	162,126	241,866	134,935
1901.												
1	3,822	3,475	4,362	2,469	6,328	4,803	2,045	3,834	1,197	1,161	3,664	2,586
2	4,000	3,734	4,262	4,664	6,905	3,802	3,496	3,846	1,161	1,782	3,804	1,685
3	4,215	2,659	2,741	5,086	5,990	3,473	3,740	3,787	1,636	1,863	2,674	2,653
4	4,202	1,825	2,109	5,209	6,032	4,913	2,618	2,648	1,682	1,748	1,789	3,520
5	4,253	3,723	4,073	5,385	4,769	5,087	3,194	1,713	1,687	1,713	3,699	3,497
6	3,016	4,096	4,481	6,081	3,799	4,905	4,097	2,553	1,682	1,228	3,728	3,459
7	2,043	4,352	4,262	6,777	5,707	4,647	2,945	2,605	1,649	991	3,711	3,648
8	3,939	4,515	4,262	7,075	5,536	4,846	2,722	2,321	1,105	2,067	3,728	2,329
9	4,279	4,541	4,210	10,675	5,498	2,834	4,185	2,406	893	2,145	3,584	1,598
10	4,349	2,796	2,908	10,986	5,574	2,654	4,557	2,336	1,449	2,265	2,585	3,257
11	4,159	2,045	1,793	11,579	5,428	3,873	4,080	1,503	984	2,468	1,654	3,585
12	4,009	4,164	2,206	12,033	4,183	3,916	4,154	1,607	1,199	2,314	3,447	3,669

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1901.												
13	3,046	4,181	4,000	11,573	3,754	4,443	4,041	2,579	1,051	1,740	3,766	[illegible]
14	2,235	4,434	4,121	11,088	5,012	4,273	3,131	2,512	1,194	1,260	3,766	[illegible]
15	3,914	4,262	4,059	10,728	5,564	3,911	2,809	2,497	706	3,633	3,771	[illegible]
16	3,929	4,292	4,169	11,869	5,428	2,586	3,996	2,642	1,146	3,357	3,856	[illegible]
17	2,480	3,068	2,818	11,935	5,371	1,741	4,089	2,575	1,009	3,252	2,697	[illegible]
18	3,542	2,227	1,742	11,614	5,011	3,475	4,002	1,617	1,039	3,366	1,661	[illegible]
19	3,981	4,309	3,967	11,467	4,099	4,087	3,898	1,404	1,004	3,136	3,644	[illegible]
20	2,965	4,470	4,030	11,334	3,715	4,148	4,067	1,737	1,090	2,099	3,873	[illegible]
21	2,835	4,392	4,554	9,926	5,006	3,948	2,912	1,607	1,056	1,574	3,728	3,312
22	3,521	4,500	4,143	9,302	5,261	3,956	2,230	1,764	675	3,539	3,833	[illegible]
23	3,885	4,393	4,109	11,103	5,312	3,137	3,623	1,885	1,020	3,728	3,717	[illegible]
24	3,885	3,085	3,433	9,826	4,686	2,493	3,899	1,795	1,276	3,622	2,485	[illegible]
25	3,898	2,489	4,615	9,467	4,938	3,336	3,936	1,542	1,449	3,664	1,640	[illegible]
26	3,813	4,402	6,431	9,240	3,884	3,506	3,833	1,130	1,513	3,749	3,212	[illegible]
27	2,563	4,634	4,994	8,395	3,759	3,474	3,791	1,849	1,449	2,719	3,585	[illegible]
28	1,939	4,577	4,084	7,401	4,875	3,554	2,668	1,892	1,437	1,646	3,476	[illegible]
29	3,286	....	4,367	7,244	3,453	3,506	2,138	1,932	1,073	3,597	3,432	[illegible]
30	3,668	....	4,245	7,271	5,010	2,354	3,724	1,622	1,124	3,873	3,456	[illegible]
31	3,643	....	3,469	....	4,920	....	3,921	1,725	....	3,791	....	[illegible]
Total	109,314	105,640	119,019	268,802	154,807	111,681	108,541	67,465	36,635	79,090	97,665	[illegible]
1902.												
1	3,096	2,595	3,368	2,765	2,393	9,573	5,703	3,727	910	1,286	2,962	[illegible]
2	3,104	917	1,190	2,964	2,469	10,488	5,601	3,276	1,111	1,499	1,004	[illegible]
3	2,938	1,488	1,632	3,011	2,423	11,868	5,447	1,544	1,682	1,596	1,314	[illegible]
4	3,060	2,446	2,497	3,106	1,471	11,462	3,372	2,532	1,690	1,498	2,716	[illegible]
5	1,058	2,513	2,632	3,252	2,079	11,050	3,453	3,692	1,723	435	2,931	[illegible]
6	1,676	2,609	2,758	1,603	2,845	10,407	3,010	3,912	1,865	736	2,854	[illegible]
7	2,776	2,609	2,827	1,706	3,537	9,886	3,349	4,086	704	1,576	2,973	[illegible]
8	2,860	2,408	2,754	3,134	4,079	7,311	5,534	3,999	1,244	1,751	3,012	[illegible]
9	3,136	696	1,135	3,102	3,131	7,886	5,533	3,894	1,709	1,613	914	[illegible]
10	2,993	1,427	1,575	3,147	4,018	8,343	5,326	1,532	1,759	1,613	1,273	[illegible]
11	2,915	2,389	3,866	3,102	2,075	8,209	5,280	2,362	1,653	1,526	2,977	[illegible]
12	765	2,595	3,274	3,119	2,743	6,222	5,136	3,666	1,550	651	3,028	[illegible]
13	1,285	2,587	3,608	1,459	4,637	6,431	4,639	4,047	1,669	1,042	2,946	[illegible]
14	2,494	2,477	3,690	1,759	4,917	3,998	3,264	4,031	821	2,116	2,891	[illegible]
15	2,344	2,455	3,567	2,108	5,056	5,840	5,125	3,853	1,083	2,391	2,891	[illegible]
16	2,461	1,063	1,603	2,366	4,917	4,169	5,142	3,958	965	2,250	756	[illegible]
17	2,514	1,421	1,765	2,449	4,615	6,156	5,265	1,500	1,203	2,219	1,308	[illegible]
18	2,505	2,492	3,707	2,265	2,484	6,282	5,281	2,097	1,208	2,716	2,841	[illegible]
19	1,139	2,413	3,841	2,376	2,692	5,439	5,163	2,539	1,317	715	2,832	[illegible]
20	1,099	2,587	3,841	1,086	4,682	5,992	2,896	2,733	1,174	1,455	2,962	[illegible]
21	2,422	2,366	3,994	2,069	5,085	6,088	3,150	2,913	652	2,120	2,954	[illegible]
22	2,113	2,486	3,969	2,267	4,862	3,852	4,248	2,927	887	2,646	2,915	[illegible]
23	2,450	1,085	1,852	2,069	4,725	4,215	4,209	2,869	1,473	2,534	958	[illegible]
24	2,422	1,377	1,723	2,314	4,949	5,866	4,325	1,311	1,452	2,676	1,349	[illegible]
25	2,429	2,210	3,777	1,986	5,719	6,089	4,566	1,773	1,174	2,681	2,977	[illegible]
26	927	2,424	3,925	2,247	7,422	6,001	4,287	2,763	1,369	1,086	3,184	[illegible]
27	1,389	2,366	4,019	947	9,941	5,707	1,647	2,891	1,336	1,286	2,860	[illegible]
28	2,425	3,480	3,841	1,571	11,227	5,740	2,265	2,879	515	1,785	2,962	[illegible]
29	2,558	....	3,926	2,381	12,317	3,491	3,441	2,789	851	2,963	3,123	[illegible]
30	2,425	....	1,769	2,312	9,869	3,852	3,733	2,158	1,222	3,024	1,138	[illegible]
31	2,373	....	1,735	....	9,599	....	4,024	1,529	....	2,875	....	[illegible]
Total	70,151	59,981	89,660	70,042	152,978	207,913	133,414	89,782	37,971	56,360	71,805	70,479

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1903.												
1	2,587	1,675	1,829	6,192	6,009	5,677	5,571	4,393	4,195	5,312	2,664	2,819
2	2,631	2,102	1,780	6,597	6,306	6,791	5,419	2,827	4,281	5,293	2,102	3,385
3	2,587	3,008	3,191	6,806	6,145	6,069	5,278	3,011	4,298	2,805	4,674	1,915
4	1,206	3,053	3,197	7,649	4,275	5,429	3,509	3,845	4,325	2,618	4,772	3,669
5	1,479	3,097	3,453	9,297	6,118	5,386	2,055	4,136	4,324	5,321	4,769	3,628
6	2,662	3,394	3,318	7,729	5,989	5,989	2,721	3,756	1,829	4,993	4,965	1,653
7	2,818	3,387	3,665	7,661	6,237	3,747	3,943	3,707	1,936	5,826	4,929	1,973
8	2,767	1,825	2,301	6,379	6,078	4,089	3,746	3,787	3,905	5,595	1,975	3,507
9	2,832	1,837	2,070	6,342	5,911	5,812	4,093	2,206	4,708	5,339	2,098	3,621
10	2,409	3,390	3,560	6,783	3,629	5,969	3,796	2,875	4,809	5,339	4,230	3,702
11	1,559	3,581	3,981	8,283	3,964	5,564	4,009	3,671	5,077	2,845	4,957	3,595
12	1,522	3,593	4,199	7,807	5,698	6,009	1,856	4,021	5,086	2,577	5,077	3,644
13	2,954	3,652	4,064	7,402	5,467	5,735	3,370	4,134	2,734	5,340	4,731	1,623
14	3,187	3,501	3,962	8,394	5,771	3,501	4,992	4,030	2,731	5,509	4,846	1,827
15	3,610	2,005	2,550	8,517	5,467	3,818	4,684	4,134	5,199	5,482	2,226	2,964
16	3,655	2,042	2,207	6,236	5,841	5,631	5,157	1,438	5,312	5,510	1,859	3,516
17	3,493	3,368	4,178	6,216	3,421	5,668	5,149	2,003	5,293	5,434	4,219	3,694
18	1,295	3,269	4,650	6,339	3,869	5,716	5,139	4,064	5,387	2,760	4,466	3,702
19	1,953	3,285	8,437	4,188	6,088	5,354	2,700	4,099	4,442	2,957	4,888	3,677
20	3,503	3,431	6,454	4,518	6,127	5,232	3,174	4,047	3,751	5,599	4,504	2,000
21	3,476	3,468	3,887	6,429	6,127	3,326	5,047	4,021	2,489	5,134	4,282	1,916
22	3,664	2,379	1,815	6,246	5,933	3,645	4,945	1,874	5,106	5,368	1,733	3,328
23	3,436	1,749	2,340	6,657	6,157	5,429	5,102	3,869	5,369	5,293	2,307	3,424
24	3,543	3,232	5,086	5,784	3,043	5,476	5,176	1,965	5,481	5,283	4,422	1,908
25	1,293	3,107	5,055	4,796	3,824	5,411	4,918	3,794	5,415	2,590	4,484	3,353
26	1,935	3,068	4,838	3,886	6,216	5,335	2,538	4,291	5,519	2,666	3,874	1,584
27	3,289	3,457	4,839	4,376	7,378	5,204	2,189	4,449	3,481	5,133	3,310	1,319
28	3,435	3,627	4,967	6,207	6,421	2,656	4,442	4,244	2,991	5,302	3,378	2,162
29	3,527	....	3,979	6,011	6,138	3,089	4,555	4,099	5,013	5,264	1,977	3,134
30	3,514	....	3,337	5,288	6,657	5,084	4,254	1,781	5,059	5,217	1,873	3,619
31	3,756	....	5,454	....	5,199	....	4,321	2,257	....	5,264	....	3,587
Total	85,577	82,582	118,643	195,015	171,503	151,841	127,848	106,828	129,635	145,268	110,591	89,448
1904.												
1	3,497	2,371	3,408	4,127	6,742	9,539	3,428	2,745	2,631	2,124	4,245	3,869
2	3,595	3,586	3,457	3,878	7,316	9,411	3,282	3,489	2,399	1,355	4,767	4,405
3	1,898	3,545	3,545	1,612	5,794	9,283	2,682	3,617	2,231	1,324	4,785	4,354
4	2,221	3,538	3,522	4,091	5,477	9,793	2,243	3,636	1,327	2,499	4,750	3,005
5	3,189	3,306	3,505	4,317	5,804	8,253	2,799	3,546	1,622	3,385	4,829	2,527
6	3,507	3,505	1,724	4,507	5,784	8,404	3,578	3,676	1,949	3,480	4,423	4,077
7	3,869	1,664	1,769	4,410	5,813	8,799	1,416	2,575	2,000	3,678	2,611	4,379
8	3,861	2,073	3,392	4,879	4,456	8,248	3,578	2,833	2,013	3,628	4,346	4,413
9	3,752	3,530	3,147	5,334	5,417	6,968	3,483	3,833	1,863	2,412	4,742	4,209
10	1,762	3,595	3,545	2,564	7,548	8,179	2,603	3,754	2,096	3,563	4,794	4,277
11	2,027	3,636	3,668	3,887	10,052	8,483	2,769	3,987	1,312	3,797	4,847	2,881
12	3,719	3,710	3,465	5,737	10,960	8,139	3,459	3,924	1,545	3,354	5,137	2,452
13	3,628	3,663	1,818	6,108	11,682	8,027	3,452	4,043	1,907	3,592	3,280	3,069
14	3,465	1,565	1,988	7,168	11,183	8,527	3,658	2,698	2,103	3,609	2,663	3,638
15	3,572	1,904	3,289	7,495	9,810	8,125	3,475	3,087	2,162	3,717	4,525	3,699
16	3,587	3,504	3,644	8,015	10,168	8,315	3,428	3,226	2,124	2,345	4,637	3,757
17	1,481	3,400	3,424	7,647	11,022	8,281	2,466	3,098	2,091	1,894	4,576	3,721
18	2,018	3,662	3,481	8,233	10,960	7,356	2,832	3,217	1,568	3,453	4,481	2,871
19	3,166	3,628	3,130	9,637	10,604	5,999	3,459	3,197	1,361	3,379	4,469	2,299

Mean daily discharge, in second-feet, of lower Fox River at Rapide Croche dam—**Continued**

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
20	3,352	3,710	1,739	9,434	10,168	5,776	3,344	3,240	1,762	4,488	2,538	3,71[illegible]
21	3,329	1,689	2,036	9,399	9,574	6,167	3,206	2,214	1,825	4,470	2,286	4,4[illegible]
22	3,408	2,237	3,913	9,190	8,571	5,241	3,420	1,722	988	4,248	4,102	4,[illegible]
23	3,392	3,760	4,335	9,018	9,099	3,758	3,359	2,443	1,988	3,018	4,421	4,111
24	1,185	3,962	5,185	7,436	9,845	3,617	2,345	2,531	1,888	2,915	3,264	4,[illegible]
25	2,096	4,082	7,425	7,126	10,168	2,336	2,585	2,396	1,276	4,336	3,379	2,253
26	3,668	4,134	5,429	8,823	10,812	2,336	2,909	1,551	1,381	4,694	4,285	1,812
27	3,727	3,810	2,354	9,075	9,389	2,585	3,146	2,503	1,975	4,542	2,748	3,1[illegible]
28	3,869	1,843	2,504	9,028	9,527	3,474	3,498	2,595	1,925	4,575	1,667	3,700
29	3,752	2,091	3,359	9,017	8,720	3,538	3,584	2,115	2,267	6,434	3,127	3,6[illegible]
30	3,848	....	3,748	8,868	8,471	3,491	4,111	2,389	2,052	3,690	6,935	4,504
31	1,885	....	4,385	....	8,989	....	2,659	2,645	....	3,158	....	4,311
Total...	95,307	90,703	105,333	20,060	269,925	200,448	96,253	92,525	55,631	107,156	121,689	112,163

Unlike many other northern rivers the lower Fox is rarely troubled with ice gorges, because the ice on Lake Winnebago melts gradually. It is stated that trouble is sometimes experienced from anchor ice forming on the rapids in exceptionally cold weather, but this is largely prevented by the system of slack-water navigation.

The absence of great freshets prevents backwater and allows the construction of the mills out into the stream, as well as connecting sidetracks on short trestles only a few feet above the water, with perfect safety.

The bed of the river in nearly all cases is in hard limestone. Excellent quarries of fine building stone have been opened for use in both the Government and private improvements of the river.

WATER POWERS.

GENERAL STATEMENT.

No other river system in the State has so large a proportion of its total descent concentrated in its lower reaches as has the Fox. Between Lake Winnebago and Green Bay the river descends a total of 166 feet in a series of eight rapids. The total drainage area of the river is 6,449 square miles, of which area 6,046 square miles, or 94 per cent, are included above the outlet of Lake Winnebago. These two facts—the large concentration of fall in the lower river and the location of 94 per cent of its drainage area above this concentration—have the effect of producing extensive and valuable water powers.

Before any improvements had been made the river flowed between wooded clay bluffs from 10 to 70 feet or more in height, in some places rising abruptly from the river's edge on each side. Through this channel ran the clear, dashing river over its limestone bed from 300 to 1,000 feet wide. Great changes have since been made. [a]

[a] Tenth Census.

The following table gives the location and amount of fall at each of these rapids before improvement, according to surveys of Major Suter in 1866:

Rapids on lower Fox River in 1866 (before improvement). [a]

Name.	Descent.	Distance apart.
	Feet.	*Miles.*
Depere	8	....
Little Kaukauna	8	6.0
Rapide Croche	8	6.0
Grand Kaukauna	50	4.5
Littlechute	38	2.5
Cedar rapids	10	.75
Grand Chute	38	4.0
Winnebago rapids	10	4.25
Green Bay to Lake Winnebago	170	28.0

[a] Warren, G. K., Report, 1876, p. 29.

LEGAL STATUS.

In 1846 Congress passed an act granting a large amount of land to the State of Wisconsin for the purpose of making a navigable route from Lake Michigan along Fox River to Wisconsin River. In 1853 the State, after expending $400,000 upon the improvements, passed the whole matter, including the land, into the hands of the Fox and Wisconsin Improvement Company. This company issued bonds, completed the improvement, and in 1856 the first steamer passed through from Mississippi River to Green Bay. On the advent of railroads soon after the route fell into disuse, and the company was unable to pay interest on its bonds. Suit was brought by the holders of these bonds, and the franchises, property, and land grants of the company were sold to a corporation organized in 1866 as the Green Bay and Mississippi Canal Company. In 1870 the United States appraised the value of the locks and canals at $145,000, took possession of them on the payment of this sum, and has since exercised control in the interests of navigation.

The Green Bay and Mississippi Canal Company still exists and retains its land grants, water-power franchises, and other property. The company claims the right to all surplus water after the needs of navigation are supplied. This claim includes the right to tap the canals at any point and draw off the water, provided navigation is not interfered with, as well as the right to take all the surplus flow of the river at the head of each rapids and use it at that level. This claim has been confirmed by the United States Supreme Court. The company does not claim ownership of power which is developed at a level below the head of a rapids by persons owning the land and using water which has passed the tailraces of the company.

In some cases this company owns the power, while others own the land. These interests have in some instances been mutualized in a joint company; in others protracted lawsuits have resulted in preventing the development and use of the water power up to the present time. The water powers at Rapide Croche and Little Kaukauna dams have not been improved for this reason.

As the low-water flow of the river falls far short of being sufficient for the turbines now installed, frequent controversies and lawsuits concerning the ownership of the water have resulted. Finally a few years ago the Neenah and Menasha Water Power Company, composed of practically all the users of water for power purposes on the river, was formed to regulate the use of the surplus water not required for navigation. Under the rules of the Secretary of War water may not be drawn below the crest of the Menasha dam except by his special permit. Such permission is frequently given, however, to help out the great manufacturing interests concerned.

Fox River discharges from Lake Winnebago in two nearly parallel channels, distant about three-fourths of a mile from each other. These branches join in less than 2 miles in Lake Butte des Morts, an expansion of the river 3 miles long and extending at right angles to the general direction of the river.

Menasha and Neenah are located at the lower end of the two channels, Menasha on the north side of the northern channel and Neenah on the south side of the southern channel These cities are about 1 mile apart and have a total population of about 12,000.

The river banks are here only 10 feet or less high. There is a dam in each channel, with an average head of 8 feet, the two maintaining the level of Lake Winnebago. These dams would develop 2,400 theoretical horsepower.[a]

The riparian owners on the Neenah channel improved the water powers before the ship canal was begun, and thus obtained a prior right under a State charter. Most of the manufactories are located on the strip of land, averaging 125 feet wide, between the river and the race.

NEENAH.

The Kimberly Clark Paper Company is the most extensive user of water power at Neenah, having installed 20 turbines under a head of 7½ feet, rated at 1,560 horsepower. In addition, this firm has 550 steam horsepower, all used in the manufacture of sulphite and ground wood pulp. The Neenah Paper Company has installed 11 turbines under a head of 7 feet, rated at 838 horsepower, and reports an additional 750 steam horsepower, all used in the manufacture of paper. The Winnebago Paper Mills have installed turbines under a 9-foot head, rated at 854 horsepower, which is supplemented with 450 steam horsepower.

Other power users in Neenah are included in the following table:

Additional water powers at Neenah.

Owner and use.	Turbines.		Steam H. P.	Remarks.
	Head.	H. P.		
	Feet.			
Kreuger & Lachmann, flour	8.0	460	125	
Neenah Boot and Shoe Manufacturing Co	8.0	39	12	Use steam when water is cut off.
Neenah and Menasha Gas and Electric Light Co.	7.5	199	125	
Robert Jamison, machine shop	8.0	94	10	
Wulff, Clausen & Co., flour	8.0	123	60	Burned.

MENASHA.

The Government canal is located at Menasha. This canal has a total length of about 4,320 feet, its single lock being located at the lower end, near Lake Butte des Morts. This dam develops 2,487 theoretical horsepower at ordinary flow. The Federal Government entered into an agreement with certain persons under which they constructed the navigation improvements and received in return the ownership of the resulting water powers. As a consequence the Green Bay and Mississippi Canal Company has no interest in these water powers.

A dam 475 feet long at the head of the canal develops a head of 8.2 feet, though some of the turbines work under heads of 6 to 8 feet. The strip of land between the canal and river is used for the location of numerous manufacturing plants, all the power, except that of the Howard Paper Mill, being taken from the canal.

[a] This estimate is based on an ordinary discharge of 2,660 second-feet, equal to a run-off of about 0.43 second-feet per square mile.

The largest water-power user at Menasha is the George A. Whiting Company, which owns the right to "first-class water." Its 6 turbines work under an average head of 8 feet and are rated at 503 horsepower. The company, which is engaged in the manufacture of paper, has also installed 265 steam horsepower.

Another large concern is the Menasha Wooden Ware Company, whose turbines work under an average head of 5 feet and are rated at 414 horsepower. This is supplemented by 1,090 steam horsepower.

The other important water-power users in Menasha are included in the following table:

Additional water powers at Menasha. [a]

Owner and use.	Turbines.		Steam H. P.	Remarks.
	Head.	H. P.		
	Feet.			
Gilbert Paper Co	5	243	800-1,000	
Howard Paper Co	5	321	200	
John Strange Paper Co	5	156	250	Leased.
Banner Flouring Mills	5	90	50	
MacKinnon Excelsior Co	6	124	225	
MacKinnon Pulley Co	6	25	25	
John Schneider, planing mill	6	124	....	
Valley Knitting Co., hose, mittens, etc	4	38	....	
Menasha Woolen Mills	5	35	Small engines.	When water is low.

[a] Authority, L. M. Mann, United States assistant engineer.

For the entire distance of 5 miles between Menasha and the Appleton upper dam the river affords slack-water navigation; indeed, it has been claimed that later improvements on the Appleton dam have caused the water at Menasha to back up a foot or more above its original level. As Appleton is approached the clay banks rise to a height of 50 or 60 feet.

APPLETON.

Fall.—Because of their intrinsic value, as well as on account of their early development, the Appleton powers are not excelled on the lower Fox. According to the Government profile the river has a total fall of 36.7 feet in a distance of 1.2 miles. This head is developed by three dams, which divide the river into upper, middle, and lower levels, with estimated theoretical horsepower [a] at ordinary flow of 4,238, 2,225, and 2,558, respectively.

At Appleton the river by a gradual bend changes its course from northeast to southeast, again turning to the northeast just above the lower dam. On the left bank the clay bluffs rise steeply 50 to 70 feet, while on the opposite bank is a flat extending for 3,500 feet, and perhaps 1,300 feet wide, beyond which rise high bluffs, as on the left bank. For the purposes of navigation the Government has constructed two dams, dividing the descent into two levels. The second or middle dam was constructed by private enterprise and is used exclusively for water power.

Upper dam.—The upper dam is a substantial stone structure. It extends from the foot of State street on the left bank normal to the shore for 250 feet, thence diagonally downstream for 700 feet to a point 400 feet from the right bank. From this latter point a retaining wall or long pier extends downstream 800 feet to the right bank. The head varies from about 10 feet at the upper end of the dam to 18 feet at the lower end, the average, as given by the Government engineers, being 14 feet. Its available water power is taken from a race along the left bank, from the ship canal on the right bank, and from the adjacent retaining wall.

[a] Estimated by U. S. Asst. Engr. L. M. Mann, on flow of 170,000 minute-feet, at 4,508, 2,367, and 2,721.

The extreme variation of head is stated at 2 feet, but the ordinary variation is only half that amount. It is due to the manner of using water by the Neenah mills, and to the prevalence of strong winds blowing continuously on Lake Winnebago and changing its volume of discharge.

The race on the left bank is 600 feet long, several extensive paper, pulp, and flouring mills occupying the strip of land between it and the river. Here are located the Appleton Paper and Pulp Company, with installed turbines under 11-foot head, rated at 550 horsepower; the Kimberly & Clark Company; the Vulcan and Tioga mills, with about 710 and 770 turbine horsepower, respectively; and the Atlas paper mill, with 766 turbine horsepower. The Appleton Waterworks Company, 1,400 feet below, receives power from this canal through a flume which affords a head of 18 feet. The above powers by long-established usage are recognized as belonging to the respective companies, and not to the Green Bay and Mississippi Canal Company.

Of the power developed on the right bank, nearly all is taken from the long pier. The Green Bay and Mississippi Canal Company owns the land on this side of the river and leases power to users.

The head here varies from 12 feet near the upper end of the pier to 16 feet at the lower end. The water is taken through ten arched openings in the stone pier from the large bay above. This power is fully developed by the Wisconsin Traction, Heat, Light, and Power Company, with turbines under 16-foot head, rated at 2,250 porsepower (besides 2,000 steam horsepower).

Of the few unused power sites on this dam the greater number are located on the ship canal, and, as heretofore stated, are owned by the Green Bay and Mississippi Canal Company. The following table gives the developed powers:

Water powers on the United States canal at Appleton.

Owner and use.	Water power.			Steam H. P.
	Average head.	Rated H. P.	Entitled to—	
	Feet.			
Riverside Paper and Fiber Co	14.0	383	300	....
Appleton Chair Co., furniture	7.5	26	25	35
Union Toy and Furniture Co	8.0	50	25	30

Middle dam.—The middle dam also is independent of both the Government work and the Green Bay and Mississippi Canal Company. It was built by private capital for water-power purposes only. It is 2,400 feet below the upper dam and is about 450 feet long. The dam was constructed of timber in 1877 and has its foundation in limestone. A canal leads down the north (left) bank. The south end of the dam abuts on Grand Chute Island, West's hydraulic canal being supplied from the adjacent basin.

Previous to 1877 power had been developed by wing dams passing upstream from both banks for several hundred feet. The present dam is reported to have an average head of 7.3 feet, developing at ordinary flow (2,660 second-feet) 2,190 theoretical horsepower. The head at the various factories and mills varies from 7 to 14 feet, depending on their location, the variation being similar to that at the upper dam. The water level is remarkable for uniformity.

The north-shore race is 800 feet long, supplying a head varying from 9 feet at the upper end to 12 feet at the lower.

West's canal starts at the right abutment of the dam and extends down Grand Chute Island for about 1,700 feet, nearly parallel to the river. It has a width of about 130 feet, with earth and stone embankment about 3 feet above the water surface. The head averages 10 feet. Several fine power sites still unoccupied on this canal are especially desirable because of excellent transportation facilities.

The following table gives the important users of water power from the middle dam:

Water powers on the middle dam, Appleton.[a]

Owner and use.	Water power.			Steam H. P.
	Average head.	Rated H. P.	Entitled to—	
Fox River Paper Co.:[b]	*Feet.*			
Ravine mill; Lincoln mill; Fox River mill	11.0	2,126	¾ flow of Fox River less 25 H. P.	1,050
Patton Paper Co.; Patton Pulp Co.	8.0	814	1,250 H. P	500
Telulah paper mill, pulp	8.5	465	3,000 sq. in.	500
	14.0	903		
Appleton Machine Co	5.0	14	500 sq. in	25
Appleton woolen mill, paper, knitting, etc	5.0	47	90 H. P	50
Fourth Ward planing mill, lumber	8.0	28	30 H. P	
Marston & Beveridge, hubs and spokes	8.0	77	75 H. P	
Valley iron works	7.0	47	40 H. P	

[a] Authority, L. M. Mann, U. S. assistant engineer.
[b] Power used by Fox River Paper Co. (three mills) are located on West's canal; the other powers are on the left bank.

Lower dam.—The lower or Government dam is located about three-fourths of a mile below the middle dam and just below the lower bend of the river, at a point where the river is 485 feet wide. The dam extends downstream from the left bank 417 feet, at an angle of about 45° with the channel, to an embankment which extends 600 feet farther downstream. The lower-level ship canal is back of this embankment. The river runs close to the left bank, which is high and steep, while on the right bank a flat 200 to 300 feet wide intervenes between the shore and the bluffs. There are four methods of utilizing the power—viz, from the abutment of the dam, from the race on the left bank, from the ship canal, and from the Telulah Water Power Company's canal on the right shore. The average head of this dam is stated at 8.5 feet, which at ordinary flow gives 2,550 theoretical horsepower. The report of Capt. L. M. Mann, on whose authority the above statement is made, shows that about 850 horsepower remain to be installed. There is said to be a fall of 3 feet in the 1,500 feet below the dam. This water power is owned by the Green Bay and Mississippi Canal Company.

The left or west-shore race starts at a point 450 feet above the dam and extends nearly parallel to the channel a distance of 1,200 feet below the dam. The bluffs rise steeply from the water, so that mills must extend out over the river. It is claimed that this race is entitled to one-fourth of the stream flow.

The right or east canal, known as the Hyde & Harriman canal, has several good locations for mills. The land adjacent was owned by Mr. W. Hyde and Judge J. E. Harriman, while the power belonged to the Green Bay and Mississippi Canal Company. These interests were united and the canal completed in 1880. It starts at the head of the ship canal and skirts the bluffs for its entire length of 2,250 feet, leaving a wide strip of flat land between it and the river. An earth embankment forms the river side. The cross section of the canal at its upper end is 120 by 7 feet, but it gradually decreases. Its head varies but slightly and is said to average 10 feet. The most important mill on this canal is that of the Telulah Paper Company, with a total of 11 turbines, rated at 1,368 actual horsepower.

CEDARS DAM.

This dam backs up the water for the entire distance of 3.3 miles to the lower Appleton dam, affording slack-water navigation. Fox River in this stretch is hemmed in by high clay banks and has an average width of 600 feet. At a short distance below the dam however, a small creek enters from the north, causing the bluffs to recede from the river and follow up the creek, leaving a flat area of perhaps 35 acres. The dam is situated about 1,000 feet below the point where the bluffs leave the river. It crosses the river in a normal distance of 810 feet. It has an average head of 9.7 feet, which at an ordinary flow of 2,660 second-feet gives 2,910 theoretical horsepower. This power is owned by the Green Bay and Mississippi Canal Company, but the entire power is leased to the Kimberly & Clark Paper Company for a paper mill. This firm reports an installation of 33 turbines, under a head of 11 feet, rated at 4,217 actual horsepower.

LITTLECHUTE.

The next Government dam is located 4,000 feet below the Cedars dam at a small village called Littlechute. The river has extensive rapids at this point, there being a total descent, according to the Government profile, of 36.2 feet in the 2 miles between the foot of the Cedars lock and the backwater of the Kaukauna dam below. These rapids are passed by a canal 6,500 feet long on the left bank of the river. One lock of 16-foot lift is located about 1,000 feet from the head of the canal, and a composite lock of about 20-foot head is located at the lower end of the canal.

The river is about 840 feet wide at the dam site. On the left bank the bluffs retreat from the river slightly, leaving a narrow flat and some small islands. On the right bank there is a break of perhaps 1,500 feet in the bluffs. This power and the adjacent land belong to the Green Bay and Mississippi Canal Company. The dam has a head of 12 feet, but the total available head, because of the adjacent rapids in the 7,000 feet below the dam, is stated to be 34 feet. This descent, with a flow of 2,660 second-feet, gives 10,200 theoretical horsepower. It is certain that to develop more than half this amount would require a large expenditure of money. At the present time 20 feet of fall have been developed.

The Littlechute Pulp Company has installed 24 (mostly 54-inch) turbines under a head of 12 feet, rated at 3,000 actual horsepower. The power next in importance on this dam, and the only power not leased from the Green Bay and Mississippi Canal Company, is that of a flouring mill owned by Arnold Verstigen, run by 6 turbines rated at 100 horsepower.

COMBINED LOCKS DAM.

About a mile below the Littlechute dam is the Combined Locks dam, owned by the Combined Locks Paper Company. A view of this dam, together with part of the company's plant, is shown in Pl. II, *B*. The company has 49 turbines installed, rated at 4,438 practical horsepower, leased from the Green Bay and Mississippi Canal Company.

GRAND KAUKAUNA DAM.

A descent of 50.3 feet in a distance of less than a mile entitles the Grand Kaukauna rapids to first place in all the water powers of the lower Fox River. Both topographic and transportation conditions are very favorable for improvement. The Kaukauna dam is distant 2.5 miles from the Littlechute dam and produces slack water to the end of the Littlechute canal. The rapids are passed by a ship canal 7,400 feet long, extending from the dam and including 5 locks with an aggregate lift of 50.3 feet, all located on the left bank of the river. At its middle point this canal is distant 1,000 feet from the river. The river is about 700 feet wide at the dam, but a quarter of a mile below broadens out between several islands to a maximum width in the middle of the rapids of over 2,000 feet. The islands are low, but all have the limestone base. These islands, together with the flats on both sides of the river, give fine facilities for water-power development. The distance across the valley from bluff to bluff is about 3,500 feet.

The water powers are made available in three or more ways, viz, from the ship canal, from the Kaukauna Water Power canal, and from the Edwards & Mead canal. There is

A. DAM ON LOWER FOX RIVER AT DEPERE.

Looking east.

B. COMBINED LOCKS DAM ON LOWER FOX RIVER AT LITTLECHUTE.

Private dam; plant cost $1,250,000.

a frontage of 900 feet or more on the upper level of the ship canal suitable for power development and furnishing an average head of about 16 feet. The Kaukauna Water Power canal starts 400 feet above the dam, thence runs 400 feet at an angle from the shore of about 45°. At a point about 200 feet from the river it turns and runs parallel to the south channel of the river for 2,000 feet. Its greatest width, 150 feet, is at the bulkhead. Its minimum width is 86 feet and its depth is 11 feet. There is said to be a descent of 2 feet in the total length of 2,400 feet, and the average head furnished is 18 feet. Along the side and end of the canal there is a total frontage of 2,100 feet available for power sites and mills.

The Kaukauna Water Power Company's claims to one-half the flow of the river were denied by the Green Bay and Mississippi Canal Company at the time of the construction of these improvements, and the matter was taken into the courts for adjudication. After successive trials in the State courts the question was finally settled by the United States Supreme Court October, 1898, in favor of the Green Bay and Mississippi Canal Company, which thereupon purchased the entire plant and canal of the Kaukauna Water Power Company.

In this decision the Supreme Court held broadly that the use of the surplus waters created by the Government dam and canal at Kaukauna belonged to the Green Bay and Mississippi Canal Company, but that "after such waters had passed over the dam and through the sluices and had found their way into the unimproved bed of the stream, the rights and disputes of the riparian owners must be determined by the State court."

The Edwards & Mead canal was built under the direction of Capt. N. M. Edwards, engineer for the Green Bay and Mississippi Canal Company. Advantage was taken of a branch of the main north channel running between two large islands; this was formed into a pocket by damming the ends and sides. This channel starts 600 feet below the bridge, and the dam was placed 1,000 feet below its head. As the water is taken from below the first level of the rapids the Green Bay and Mississippi Canal Company could make no legal claim to it, but subsequent to its development bought the power. The sides of the channel are substantially built of earth on the south side and dry rubble masonry on the north side.

Recently very comprehensive plans have been prepared for the improvement of the lower level at Kaukauna, which will produce 6,500 theoretical horsepower. These plans include the blasting out of the tailrace so as to develop a 21-foot head at the present Government dam, and also the construction of a new masonsy dam below which will develop 27 feet additional. As this dam would render useless some of the present improvements below the Government dam, it will be necessary to purchase such property before the new dam can be constructed. These developments will be made as soon as a suitable tenant is found.

At the present time the Green Bay and Mississippi Canal Company offers for rent 3,000 theoretical horsepower already developed at the headrace of the Kaukauna WaterPower Company's canal, recently purchased. Large store buildings at this point, though partially destroyed by fire, could readily be converted into a large manufacturing plant.

The city of Kaukauna has 5,000 inhabitants and is on the main line of the Chicago and Northwestern Railway, being also reached by the Fox River Valley Electric Railway.

The following table gives a list of the power users at Kaukauna and the installed turbine power:

Water powers on Fox River at Kaukauna. [a]

Owner.	Water power.			Steam H. P.
	Average head.	Rated H. P.	Entitled to—	
	Feet.			
Badger Paper Co	16	1,230	....	450
Chicago and Northwestern Rwy. shops	7	47	75	110
Kaukauna Fiber Co	14	194	100	200
Kaukauna Machine Co	14	250	75	15
Kaukauna Electric Light Co	14	194	....	100
Thilmany Pulp and Paper Co	14	389	275	175
Western Paper Bag Co	15	1,400	400	310
Outagamie Paper Co	21	816	1,500	....
Lindauer Pulp Co	12	....	....	....
Reese Pulp Co	12	440	....	350
Thilmany Pulp and Paper Co	12	709	....	567

[a] Nos. 1-4 are owned jointly by the Green Bay and Mississippi Canal Company; Nos. 5-9 are leased from the same company; Nos. 10 and 11 are leased from same company and Edwards.

Below Kaukauna Rapids the river is from 1,200 to 2,200 feet wide for nearly 2 miles, but it gradually contracts to a width of about 500 feet for the lower half of its course between Kaukauna and Rapide Croche. Almost without exception the bluffs rise directly from the river for the entire distance. Navigation is also by slack water from the Grand Kaukauna Canal to the Rapide Croche dam.

RAPIDE CROCHE DAM.

The Rapide Croche dam is located 4.5 miles below the Grand Kaukauna dam and was built by the Government for navigation purposes. It is about 450 feet long and has an average head of 8.5 feet. The bluffs rise on either side close to the river, except on the left bank at the site of the ship canal. This canal starts just above the dam and extends downstream for a distance of 1,760 feet to the lock. This forms a strip of land well suited for power or mill sites, being 900 feet long and varying in width from 20 feet at the ends to 200 feet at the middle. This ground and 120 acres adjacent is owned by the Green Bay and Mississippi Canal Company.

The Rapide Croche dam develops 2,400 theoretical horsepower, which may be leased on extremely favorable terms. At the present time this power is not utilized. Its location, nearly midway between Green Bay and Appleton, is convenient for the development of electric power for railroad or other purposes. The Chicago and Northwestern Railway and the Fox River Valley Electric Railway are close at hand on the left bank.

LITTLE KAUKAUNA DAM.

Six miles below the Rapide Croche dam is located another Government dam which furnishes slack-water navigation in this stretch of the river. This dam is about 550 feet long and furnishes a head of 8 feet. The bluffs rise close to the right bank, but on the left bank recede for several hundred feet. Advantage is taken of this fact to locate the Government canal here. This canal is 950 feet long and has a single lock at its lower end.

The power here, like that at Rapide Croche, is owned by the Green Bay and Mississippi Canal Company, while the riparian rights are owned by other parties. This fact has led to a protracted legal struggle, which has resulted in preventing the utilization of the valuable water powers. It is stated on good authority that these suits have recently been settled and that improvements will soon be made.

A large number of water-power lots would be made available by the construction of a tailrace parallel to the canal about as shown in fig. 3. An 8-foot head with a flow of 2,660 second-feet, gives 2,400 theoretical horsepower.

DEPERE DAM.

This dam at Depere, a city of over 4,000 inhabitants, about 7 miles from the mouth of Fox River, is the last dam and lock on the river. A view of it is shown in Pl. II, *A*. The dam is of crib construction, about 2,000 feet long, and furnishes an average head of 7 feet, which, at an ordinary flow of 2,660 second-feet, gives 2,100 theoretical horsepower. A modern steel bridge is located just below the dam.

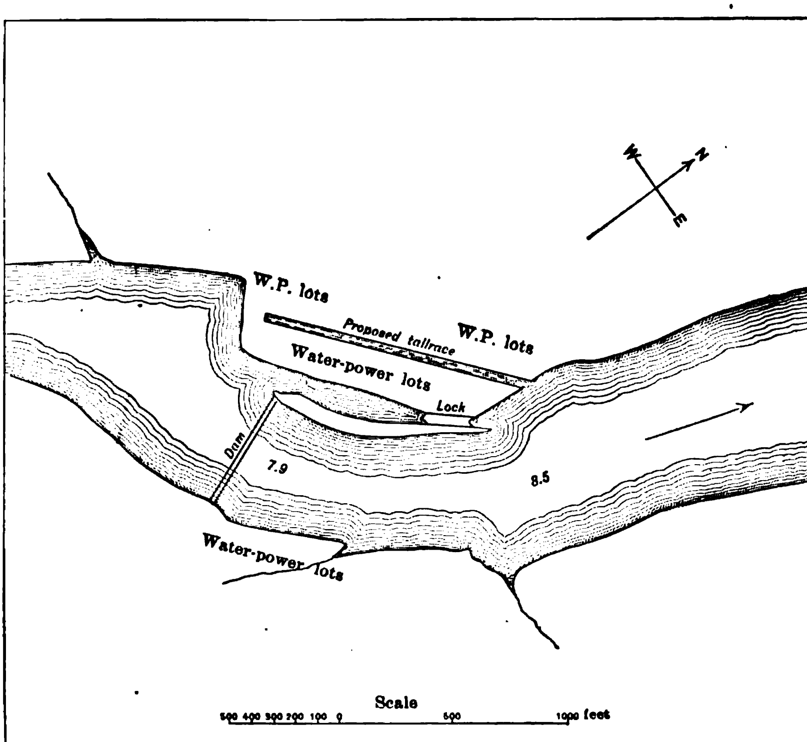


FIG. 3.- Plan of water-power development at Little Kaukauna, Wis.

This power does not belong to the Green Bay and Mississippi Canal Company, for it was built under a contract whereby the riparian owners were to have the use of the power in return for the maintenance of navigation improvements.

The American Writing Paper Company, which has one of the largest and most modern paper mills on the river, has installed 16 large turbines, with a rating of 1,565 practical horsepower. In addition the company uses 1,300 steam horsepower. It is entitled to the total power of the river less 290 horsepower. The value of its annual product is stated at $600,000.

On the right bank, taking water from the ship canal, are located the J. P. Dousman Company's flouring mill, with 175 actual turbine horsepower, and the Depere Electric Light and Power Company's plant, with 100 actual turbine horsepower. The flouring mill has a capacity of 300 barrels a day. These are the last powers on the river.

RAILROADS.

Attention has elsewhere been called to the fact that the freedom from freshets which lower Fox River enjoys allows the building of railroad side tracks over or across the river so as to reach any mill no matter how situated. The river thus enjoys excellent railroad facilities. The Chicago and Northwestern Railway closely follows the left bank of the river between Neenah and Green Bay, and a branch performs a similar service for all the mills between Menasha and Kaukauna on the right bank. The Chicago, Milwaukee and St. Paul Railway reaches Neenah, Menasha, and Appleton, while another branch parallels the river between Green Bay and Depere. The Wisconsin Central line reaches Neenah and Menasha. Besides the steam lines, the river's entire length is closely followed by an electric interurban railroad, which provides a train every hour at reduced rates.

The navigation improvements maintained by the Federal Government provide for a 6-foot channel between Oshkosh and Green Bay. While this channel is insufficient for the larger freight boats navigating the Great Lakes, the commerce on lower Fox River has been sufficient to reduce the railroad freight rates to an exceedingly reasonable basis. This gives the numerous factories on this river a very marked advantage in shipping both raw materials and finished products. This advantage, together with the extremely low rates at which water power may be rented ($5 to $10 per annum per horsepower), has already made this one of the largest manufacturing districts in the State.

MENOMINEE RIVER SYSTEM.

This river is formed by the junction of Michigamme and Brule rivers, and for its entire length of about 104 miles forms the boundary between Wisconsin and Michigan. It flows in a general southeasterly direction, entering Green Bay at Marinette.

DRAINAGE.

The Menominee drainage basin is narrow in its lower portion, but widens as the stream is ascended, the river receiving important branches near its source. Its total drainage area is about 4,000 square miles, of which 1,450 square miles is in Wisconsin.

Like Chippewa River, it has a main arm to the north, Michigamme River, which is nearly as long as the main river, its source, in fact, being within 12 miles of Lake Superior. This has an important bearing on the discharge of the Menominee, because it secures the large run-off due to the heavy precipitation of that region as well as the steadying effect of the enlarged drainage. The combined drainage area of Brule and Michigamme rivers amounts to 1,769 square miles [a]—nearly one-half that of the entire river system.

PROFILE.

From its head, at the junction of Brule and Michigamme rivers, to its mouth, a distance of about 104 miles, the river descends about 700 feet. In addition to this its Wisconsin tributaries descend about 300 feet, and those in Michigan 470 feet. The opportunities for water power are numerous, because of the frequent concentrations of descent in rapids along the entire course of the river. The following descriptions of the most important water powers are taken from data furnished by Messrs. O'Keef & Orbison, hydraulic engineers, of Appleton, Wis., who also loaned maps and profiles of the river, and from the very full descriptions by James L. Greenleaf, C. E., in the census report.[b]

Quoting from the latter:

> It will be evident from the following account that there is an immense amount of water power on the Menominee awaiting development, the concentrations of the descent in numerous rapids and falls supplying remarkably fine opportunities for improvements. Any works for the utilization of the power would have to be so constructed as not to interfere with the manufacturing company in the driving of logs; but dams, etc., could be built so as to be no hindrance to the passage of logs.

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[a] Tenth Census, vol. 17, p. 57.

[b] Water powers of the Northwest: Tenth Census, vol. 17, pp. 59–60.

In the table that follows will be found a statement in detail of the descent of Menominee River, together with other valuable data:

Profile of Menominee River from its mouth to head of upper rapids, Twin Falls.[a]

No.	Station.	Distance— From mouth.	Distance— Between points.	Elevation above sea level.	Descent between points. Total.	Descent between points. Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river	....	....	580.0	....	....
2	Dam No. 1, foot	2.0	2.0	580.0	....	....
3	Dam No. 2, foot	2.5	.5	587.0	7.0	14.0
4	Dam No. 3, foot	2.75	.25	594.0	7.0	28.0
5	Schappies rapids, foot	7.7	5.0	612.0	18.0	3.6
6	Schappies rapids, head	8.7	1.0	622.0±	10.0	10.0±
7	Grand rapids, foot (mouth of Little Cedar River)	22.0	13.3	649.0±	27.0	2.0±
8	Grand rapids, head (NW. ¼ sec. 32, T. 34 N., R. 23 E.)	24.5	2.5	669.0±	20.0	8.0
9	Railroad crossing, Ross	26.5	2.0	671.8	2.8	1.4
10	White rapids, foot (lot 1, sec. 30, T. 35 N., R. 21 E.)	50.7	24.2	683.4	11.6	48.0
11	White rapids, head (south line sec. 7, T. 35 N., R. 22. E.)	53.7	3.0	714.4	31.0	103.0
12	Pemena rapids, foot (mouth Pemena Creek)	61.5	7.8	748.3	30.3	3.9
13	Pemena rapids, head (south line sec. 5, T. 36 N., R. 22 E.)	63.0	1.5	767.1	18.8	12.5
14	Pemena dam, foot	67.0	4.0	773.1	6.0	15.0
15	Pemena dam, crest	67.5	.5	786.2	13.1	26.2
16	Sturgeon Falls, foot	77.0	9.5	803.9	17.7	1.9
17	Sturgeon Falls, head	77.5	.5	816.8	12.9	25.8
18	Sturgeon River, mouth	78.1	.6	818.0	1.2	2.0
19	Norway, Mich. (where public road joins river)	80.1	2.0	824.0	6.0	3.0
20	Iron Mountain, Mich. (500 feet above old ferry)	84.1	4.0	851.0	27.0	6.7
21	Little Quinnesec Falls, foot	85.4	1.3	878.0	27.0	20.7
22	Little Quinnesec Falls, head	85.65	.25	942.0	64.0	256.0
23	Big Quinnesec Falls, foot	89.9	4.25	966.0	24.0	5.6
24	Railroad bridge south of Iron Mountain	91.15	1.25	1,020.0	54.0	43.3
25	Highway bridge south of Iron Mountain	92.4	1.25	1,045.0	25.0	20.0
26	Railroad bridge, river siding	100.4	8.0	1,065.3	20.3	2.5
27	Twin Falls (500 feet below lower rapids)	101.4	1.0	1,072.5	7.2	7.2
28	Twin Falls (head of upper rapids)	102.1	.7	1,099.8	27.3	3.9

[a] Authority: No. 1, U. S. Lake Survey; Nos. 2-6, Menominee River Boom Company; Nos. 7, 8, and 10-18, T. W. Orbison; No. 9, Wisconsin and Michigan Railway; Nos. 19-27, U. S. Geol. Survey; No. 28, Chicago and Northwestern Railway.

GEOLOGY.

While the surface is largely covered, generally deeply, by glacial drift, the Menominee and all its tributaries flow over hard, pre-Cambrian crystalline rocks as far south as the mouth of Pike River, or fully two-thirds its length. In this region important iron mines are found. Below the mouth of Pike River the Menominee flows 10 miles across the Cambrian sandstone, then for 18 miles across the next higher layer, the "Lower Magnesian" limestone, and for the last 8 miles to its mouth across the "Trenton" group of limestones.[a]

The crossing of the Cambrian sandstone results in no rapids of importance, but two rapids occur in passing the "Lower Magnesian" and the "Trenton" limestones. Most of the rapids, of course, are in the harder crystalline rocks above the mouth of Pike River.

The topography of the country through which Menominee River flows can not be described as mountainous, but many high ridges give diversity to the surface. The Wisconsin branches, Pine and Brule rivers, rise side by side with the Flambeau and the Wisconsin in

[a] Geol. Wisconsin, p. —.

a high, flat plateau, abounding in lakes and swamps. In many cases the rivers head in lakes but a few rods apart, or even in the same swamp. These lakes and swamps have an elevation of nearly 1,600 feet above sea level, or 1,000 feet above Lake Michigan. The Michigan branches flow from a similar though even higher region, and it is certain that these swamps and lake reservoirs exert a marked influence in steadying the discharge of the river.

RAINFALL AND RUN-OFF.

Because of the paucity of data concerning the discharge of rivers in this region, it is exceedingly difficult to estimate the ordinary discharge. The discharge measurements in this district have been made since 1901, and most of them since 1903.

The rivers mentioned below are similarly situated with respect to Lake Superior, which is perhaps the governing factor in determining the rainfall. In 1903 Escanaba River yielded a minimum of 700 second-feet from 891 square miles. Measurements made by the I. Stevenson Company indicate a minimum flow of this river, in a dry year, of 400 second-feet. Measurements of Iron River, continuing from November, 1901, to April, 1904, show a minimum flow of 0.8 second-foot per square mile for two months in 1902, and the same for February, 1903. It seems reasonably certain that except in unusually dry years the ordinary low-water discharge of these rivers is not far from 0.6 second-foot per square mile. In 1904, a year of average rainfall, the minimum run-off occurred in the month of December, when it averaged 0.77 second-foot per square mile.

In the following tables will be found the maximum, minimum, and mean discharge in second-feet of Menominee River at Little Quinnesec Falls during twelve months of 1898 and 1899:

Estimated monthly discharge of Menominee River at Little Quinnesec Falls, Wis.[a], May, 1898, to August, 1899.

[Drainage area, 2,432 square miles.]

Date.	Discharge.			Run-off.	
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.
1898.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*
May	3,802	2,443	3,086	1.26	[illegible]
June	3,616	1,447	2,459	1.01	[illegible]
July	2,740	655	1,439	.59	[illegible]
August	4,968	498	2,282	.94	[illegible]
September	3,544	797	2,566	1.05	[illegible]
October	5,735	1,947	3,248	1.34	[illegible]
November	3,601	1,484	2,766	1.14	[illegible]
1899.					
April	4,642	3,083	4,011	1.65	[illegible]
May	4,485	3,744	4,112	1.69	[illegible]
June	4,624	2,017	3,476	1.43	[illegible]
July	2,521	804	1,819	.75	[illegible]
August	1,789	1,408	1,573	.65	[illegible]

[a] For the daily discharge for this time see Water-Supply Paper No. 83, pp. 256–257. Measurements were made by J. H. Wallace, C. E., and furnished by Kimberly & Clark, of Niagara, Wis.

It will be seen that the smallest monthly average during this time was 0.59 second-foot per square mile of drainage. Lumbering operations on Menominee River, though declining since 1892, are still active. The operation of the many logging dams must have a great effect on the regimen of the river. In a few years the lumber will be so nearly removed that it will be cheaper to carry logs by railroad. Then the dams can be used to augment the low-water flow. This will greatly enhance the value of the water powers.

The average annual rainfall of this region is estimated by the Tenth Census at 35 inches, or 10 per cent in excess of the average of the State.

The following table gives the annual precipitation in the valleys of Wolf, Oconto, Peshtigo, and Menominee rivers for the eleven years ending in 1904:

Annual precipitation, with averages, at seven stations in Wisconsin covering eleven years.

Station.	1894.	1895.	1896.	1897.	1898.	1899.	1900.	1901.	1902.	1903.	1904.	Average.
	In.	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*Inches.*
Amherst	......	......	35.1	30.2	28.7	30.2	37.6	......	32.1	......	30.2	34.7
Koepenick	23.8	24.9	32.2	25.5	28.1	31.3	46.6	33.0	27.7	42.9	43.0	32.6
Florence	27.6	27.2	29.2	25.7	27.5	34.3	37.9	32.7	......	43.3	......	31.7
Oconto	29.8	29.9	36.0	28.1	29.7	26.4	38.0	28.1	29.3	34.1	34.7	31.3
New London	......	......	......	27.4	29.0	......	35.6	28.1	34.3	28.8	31.1	30.8
Shawano	27.9	......	32.8	25.3	25.3	27.9	39.3	......	......	......	......	29.8
Waupaca	......	......	33.6	26.5	24.3	32.4	......	26.0	30.8	32.0	32.0	29.7
Average	27.3	27.3	36.3	27.0	27.5	30.8	39.1	29.6	30.8	36.2	34.2	31.5

The summary given above, embodying observations of the yearly rainfall from 1894 to 1904, inclusive, at seven near-by stations, shows the average rainfall of this section for the above period to be 31.5 inches. This is very conservative, for earlier observations for longer periods show larger averages, as will be seen from the following:

Record of precipitation at two stations in Wisconsin prior to 1894.

[From the Smithsonian tables.]

Station.	Period covered.	Precipitation.
	Years.	*Inches.*
Embarrass	35	38.3
Weyauwega	12	44.1

There is reason to believe that the rainfall at the headwaters of these rivers is in excess of that on the lower part of the drainage area, where most of the observation stations are located.

The following table compiled from Bulletin C, United States Weather Bureau, shows the result of observations of precipitation and temperature in the basins of Fox, Oconto, Menominee, and Wolf rivers for the years stated prior to 1876:

Record of precipitation and temperature at nine stations in Wisconsin prior to 1876.

Station.	Period of observation.	Precipitation.					Temperature.	
		Spring.	Summer.	Autumn.	Winter.	Year.	Summer.	Winter.
		Inches.	*Inches.*	*Inches.*	*Inches.*	*Inches.*	*° F.*	*° F.*
Wautoma	1871–1874	5.50	6.25	1.98	3.16	25.92	....	....
Portage	1836–1845	5.58	11.46	7.63	2.83	27.50	68.22	[illegible]
Weyauwega	1861–1873	6.74	17.85	14.23	5.31	44.13	68.20	[illegible]
Waupaca	1867–1874	5.50	14.50	6.92	3.93	25.92	70.17	[illegible]
Menasha	1857–1858	6.83	10.73	7.06	5.14	29.76	65.30	[illegible]
Appleton	1856–1871	7.65	10.24	6.92	3.70	28.51	67.46	[illegible]
Green Bay	1858–1865	6.18	9.35	10.43	4.46	32.42	68.10	[illegible]
Embarrass	1864–1874	8.14	12.49	8.21	5.73	34.57	66.82	[illegible]
Escanaba	1872–1876	8.52	13.72	10.57	3.28	36.09	....	....

It will be noted that the upper portion of this drainage area is scarcely represented in the above tables, the stations where rainfall observations were made being grouped in the lower portion of the river valleys. There is reason to believe that the average rainfall would be found to be sensibly larger for a series of stations more evenly distributed so as to include the northern portion.

The following discharge measurements, gage heights, and rating table are the result of observations by hydrographers of the United States Geological Survey on Menominee River, near Iron Mountain, Mich.:

Discharge measurements of Menominee River at Homestead bridge, near Iron Mountain, Mich., 1902 to 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
		Feet.	*Sq. feet.*	*Ft. pr. sec.*	*Feet.*	*Sec.-feet.*
1902.						
September 4	Horton and Gregory	....	....	....	1.90	[illegible]
November 4	W. V. Savicki	....	....	....	2.67	[illegible]
1903.						
April 9[a]	L. R. Stockman	202	1,532	2.22	5.40	[illegible]
April 19	do	....	2,000	2.78	7.32	[illegible]
July 22	do	208	1,455	2.17	4.20	[illegible]
August 25	do	208	1,342	1.76	3.60	[illegible]
September 16	do	212	2,875	[b]3.41	10.38	[illegible]
October 27	do	205	1,477	1.93	3.99	[illegible]
1904.						
May 18	E. Johnson, jr	210	2,312	2.68	7.95	[illegible]
June 1	do	210	2,522	3.01	8.97	[illegible]
August 10	do	205	1,101	1.42	2.06	[illegible]
September 5	do	210	1,571	2.02	4.34	[illegible]
October 11	F. W. Hanna	225	2,408	3.20	8.25	[illegible]
November 18	E. Johnson, jr	210	1,511	1.94	4.02	[illegible]
1905.						
April 12	S. K. Clapp	220	2,271	2.90	7.43	[illegible]
May 22	do	215	2,035	2.32	6.85	[illegible]
June 15	M. S. Brennon	208	1,421	1.78	3.67	[illegible]
July 13	do	225	2,100	2.50	6.58	[illegible]
August 13	do	207	1,346	1.83	3.24	[illegible]

[a] Stream full of logs; probably log jam.

[b] Mean velocity=85 per cent of surface velocity.

Mean daily gage height, in feet, of Menominee River near Iron Mountain, Mich., September 4, 1902, to December 31, 1905.

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1902.					1902.				
1	......	1.67	2.52	1.60	17	1.40	1.65	6.45	2.55
2	......	1.53	2.80	2.22	18	1.45	1.92	5.65	2.70
3	......	1.55	2.95	2.80	19	1.35	1.60	5.35	2.63
4	1.90	1.45	2.72	2.25	20	1.35	1.65	5.00	2.75
5	1.60	1.55	2.85	1.85	21	1.20	1.57	4.47	2.75
6	2.00	1.58	2.95	1.95	22	1.45	1.65	4.45	2.57
7	2.25	1.60	2.50	2.25	23	1.52	1.67	3.90	2.40
8	2.35	1.67	2.60	2.70	24	1.48	2.42	3.92	2.32
9	2.05	1.77	2.50	3.45	25	1.47	2.80	3.45	2.35
10	1.92	1.30	2.40	3.35	26	1.40	3.22	3.30	2.20
11	1.87	1.50	2.45	3.60	27	1.40	2.95	3.00	2.10
12	1.95	1.55	3.27	3.35	28	1.35	3.57	2.62	2.00
13	1.65	2.85	4.85	3.05	29	1.38	3.07	2.55	2.15
14	1.53	2.95	6.07	2.90	30	1.55	2.83	2.62	2.20
15	1.45	2.47	6.88	2.85	31	......	2.75	......	2.10
16	1.40	1.82	6.57	2.90					

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1903.												
1	2.55	2.35	2.38	4.68	7.95	9.30	3.70	6.25	4.45	4.30	3.80	3.60
2	2.52	2.30	2.38	5.15	7.85	8.05	4.80	6.20	4.05	4.40	3.80	3.65
3	2.42	2.20	2.42	5.65	9.55	6.30	6.50	4.55	3.70	5.20	3.75	3.55
4	2.48	2.15	2.40	6.35	8.80	7.60	3.85	5.60	3.50	6.60	4.10	3.55
5	2.50	2.15	2.35	2.25	9.45	7.05	3.50	6.60	3.65	7.70	4.00	3.70
6	2.30	2.28	2.48	4.75	9.48	6.50	7.40	8.00	4.00	7.50	3.80	3.60
7	2.25	2.25	2.55	8.30	9.72	3.40	6.30	9.00	4.65	7.65	3.50	3.40
8	2.40	2.22	2.68	5.50	9.60	4.85	5.95	8.90	5.70	7.55	3.65	3.35
9	2.35	2.20	2.72	5.50	9.32	4.85	5.45	7.20	6.65	7.10	3.50	3.35
10	2.30	2.28	2.72	6.05	7.90	4.45	5.25	6.70	6.90	6.85	3.40	3.05
11	2.35	2.40	2.75	6.25	8.98	3.90	5.20	6.75	5.80	6.70	4.10	2.90
12	2.30	2.28	3.00	7.25	8.10	3.50	6.00	6.60	5.50	6.50	4.95	2.60
13	2.20	2.22	3.32	7.15	9.90	4.40	4.65	6.50	7.60	6.25	3.50	2.50
14	2.10	2.25	3.55	6.70	9.45	6.75	4.60	5.40	8.10	5.75	3.20	2.40
15	2.18	2.20	3.50	7.45	9.18	4.50	4.00	5.60	9.00	5.50	3.25	2.45
16	2.22	2.25	3.48	7.52	8.65	4.45	3.40	5.60	10.50	5.40	3.55	2.40
17	2.25	2.22	3.55	7.55	7.60	4.55	3.05	3.70	11.20	4.85	3.25	2.40
18	2.25	2.18	4.25	7.80	6.22	4.80	2.75	4.05	10.40	4.90	2.85	2.35
19	2.32	2.20	6.25	8.30	7.55	3.75	2.85	4.25	9.45	4.90	2.65	2.65
20	2.25	2.18	8.38	7.45	8.72	4.80	2.40	4.05	8.70	4.90	2.85	2.35
21	2.10	2.10	8.85	7.58	9.05	2.30	3.10	3.75	8.00	5.20	2.50	2.30
22	2.20	2.00	7.50	7.05	7.40	3.10	4.20	3.70	6.85	5.30	3.25	2.25
23	2.22	1.95	6.20	6.90	9.40	2.70	3.15	3.75	6.95	4.35	3.00	2.35
24	2.15	2.20	5.95	7.65	6.80	3.10	3.10	3.75	6.35	4.20	3.10	2.30
25	2.12	2.25	5.80	7.50	7.80	2.80	3.25	3.35	5.90	4.05	2.90	2.35
26	2.25	2.32	6.15	7.75	7.15	2.00	4.80	4.05	4.45	3.80	3.00	2.65
27	2.35	2.45	5.42	7.00	8.45	3.80	5.30	4.65	5.50	3.90	2.85	3.15
28	2.25	2.38	5.50	6.45	10.40	2.70	5.70	3.85	4.80	3.85	3.00	3.25
29	2.35	....	5.15	6.82	11.85	2.45	6.85	3.90	4.40	3.80	3.10	3.10
30	2.35	....	4.65	7.45	10.10	2.00	8.20	4.00	4.00	3.75	3.60	3.05
31	2.20	....	4.30	....	10.75	....	7.10	4.55	....	3.70	....	3.15

Mean daily gage height, in feet, of Menominee River near Iron Mountain, Mich., September 4, 1902, to December 31, 1905—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
1	......	4.30	4.75	4.65	7.90	8.45	4.60	2.35	1.80	3.05	3.10	[illegible]
2	......	4.25	4.70	4.55	7.95	6.95	2.35	3.30	3.15	2.75	3.65	[illegible]
3	3.20	4.30	4.55	4.35	8.70	5.65	2.20	2.90	3.70	2.65	4.10	[illegible]
4	2.90	4.50	4.10	4.60	9.70	6.30	2.90	3.00	4.95	2.45	3.75	[illegible]
5	2.70	4.30	3.90	5.10	8.15	9.60	2.80	1.30	4.30	2.75	3.25	[illegible]
6	2.75	4.25	4.50	4.85	9.70	7.30	3.70	2.70	4.00	2.85	3.50	[illegible]
7	2.75	4.20	4.90	4.70	8.50	8.80	3.60	1.30	4.10	2.55	3.80	[illegible]
8	3.20	4.20	4.80	4.65	10.40	7.50	3.60	1.20	4.10	2.90	4.85	[illegible]
9	3.60	4.25	4.70	6.05	11.95	8.55	4.20	1.45	3.70	3.80	4.10	[illegible]
10	3.50	4.00	3.95	6.20	(a)	8.50	5.30	3.20	3.15	6.70	3.10	[illegible]
11	3.45	3.90	3.50	4.10	11.80	7.50	4.10	3.80	2.90	7.60	3.20	[illegible]
12	3.20	3.95	3.00	3.80	11.10	6.70	4.20	4.40	3.10	8.25	3.20	[illegible]
13	3.10	3.90	3.15	3.65	10.70	6.70	1.80	4.00	2.55	7.80	3.25	[illegible]
14	3.40	3.85	3.30	3.70	10.55	5.90	4.30	1.90	2.85	7.35	3.05	[illegible]
15	4.25	3.95	3.25	3.75	9.65	5.40	3.75	2.05	3.50	6.40	2.95	[illegible]
16	4.30	3.80	3.00	4.10	9.05	4.95	4.30	3.70	3.40	5.90	3.05	[illegible]
17	4.30	3.75	3.05	3.95	8.10	4.60	1.65	3.60	2.80	5.55	2.80	[illegible]
18	4.35	3.75	6.35	4.25	8.15	4.50	3.25	4.00	3.15	5.30	3.85	[illegible]
19	4.35	3.70	3.45	4.40	8.15	3.70	3.05	3.10	2.60	5.20	3.15	[illegible]
20	4.40	3.90	3.15	4.30	6.25	3.50	2.50	3.15	2.70	5.00	2.92	[illegible]
21	4.15	3.75	3.15	4.35	4.90	3.50	4.30	2.40	2.95	5.00	2.90	[illegible]
22	4.10	3.60	3.40	4.40	6.15	4.40	2.70	3.65	2.30	5.05	2.67	[illegible]
23	4.25	3.60	3.45	5.45	6.10	3.70	2.30	3.45	2.55	5.05	2.77	[illegible]
24	4.25	3.55	3.75	6.05	7.05	3.70	1.30	3.90	3.80	5.15	2.85	[illegible]
25	4.20	3.65	4.55	7.35	8.25	5.45	2.75	3.90	3.90	5.05	2.75	......
26	4.20	3.55	4.60	7.25	9.60	7.20	1.40	3.40	3.95	5.15	2.42	[illegible]
27	4.25	3.75	4.35	7.45	(a)	6.75	2.70	2.70	4.15	5.05	1.92	[illegible]
28	4.20	4.55	4.40	8.35	10.90	5.60	1.50	2.30	4.00	4.85	1.75	[illegible]
29	4.50	4.90	4.30	7.75	10.00	6.70	3.10	1.90	3.50	4.35	2.00	[illegible]
30	4.45	......	4.55	7.40	8.70	6.70	1.45	2.05	3.37	4.30	2.07	......
31	4.35	......	4.60	......	8.00	......	1.30	1.95	......	3.90	......	[illegible]
1905.												
1	2.60	......	2.35	7.40	8.60	3.60	5.50	4.20	2.80	2.82	3.30	[illegible]
2	2.58	......	2.38	6.80	8.40	5.60	8.60	3.70	3.90	2.80	3.30	[illegible]
3	......	......	2.45	6.70	8.50	2.40	6.40	3.40	7.20	2.50	3.20	[illegible]
4	......	......	......	7.00	9.10	2.40	7.10	3.50	8.00	2.42	3.00	[illegible]
5	2.35	......	......	7.90	9.20	3.50	8.00	3.40	7.80	2.42	3.00	[illegible]
6	2.60	2.70	......	8.00	9.20	6.30	8.00	3.45	7.00	2.35	3.00	[illegible]
7	2.60	......	......	7.45	9.30	6.90	7.60	3.30	6.30	2.40	2.95	[illegible]
8	2.80	......	2.60	6.80	9.20	7.30	7.60	3.25	5.30	2.42	2.88	[illegible]
9	2.82	......	2.62	6.80	9.80	6.30	5.80	3.40	4.60	2.40	2.92	[illegible]
10	......	......	2.60	6.60	9.60	7.10	4.20	3.30	4.40	2.35	3.00	[illegible]
11	......	......	......	7.00	9.10	5.20	5.20	3.40	4.10	2.40	3.00	[illegible]
12	......	......	......	7.30	9.00	5.80	4.90	3.20	3.70	2.45	3.00	[illegible]
13	......	2.40	......	7.40	9.10	5.70	5.40	3.20	3.35	2.50	2.90	[illegible]
14	......	......	......	7.40	8.30	5.70	5.20	3.30	3.00	2.55	2.98	[illegible]
15	......	......	2.38	6.70	7.40	5.80	5.60	3.30	2.95	2.80	3.10	[illegible]
16	3.20	......	2.30	6.40	8.60	6.40	4.30	2.80	3.60	2.90	3.15	[illegible]
17	......	......	2.22	6.10	9.70	7.30	4.40	2.50	4.30	2.82	3.05	[illegible]
18	2.90	2.55	2.25	6.20	10.10	7.20	6.10	2.65	4.40	2.98	2.90	[illegible]
19	2.92	......	2.35	5.80	10.20	10.20	3.90	2.50	4.80	3.10	2.88	[illegible]

a Gage under water. b River frozen.

Mean daily gage height, in feet, of Menominee River near Iron Mountain, Mich., September 4, 1902, to December 31, 1905—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1905												
20	2.95	....	2.50	5.80	9.30	9.30	3.40	2.70	4.90	3.30	2.90	2.95
21	2.78	2.35	2.40	5.70	10.20	6.70	2.20	2.72	4.80	3.50	3.05	2.85
22	2.75	....	2.45	6.00	7.40	8.40	2.05	2.25	4.40	3.50	3.05	2.70
23	2.50	....	2.50	6.20	7.40	6.50	2.95	2.00	3.80	3.50	3.10	2.60
24	....	....	2.65	6.00	7.40	6.50	3.90	2.22	3.50	3.50	3.05	2.60
25	....	2.35	2.98	5.80	6.00	5.80	3.70	2.28	3.45	3.55	3.15	2.62
26	....	2.40	3.49	6.40	5.20	5.60	3.70	2.20	3.15	3.50	3.20	2.65
27	....	2.40	3.80	6.80	7.60	7.80	3.40	2.28	2.95	3.40	3.05	2.70
28	....	2.45	4.60	7.60	5.60	7.60	3.95	2.10	3.05	3.20	2.90	2.55
29	3.00	....	6.00	8.00	4.80	8.00	4.60	2.12	2.85	3.20	2.50	2.40
30	....	....	7.60	8.60	5.80	8.90	4.70	2.07	2.80	3.20	1.80	2.40
31	....	....	7.40	....	3.40	....	4.60	2.00	....	3.30	....	2.48

Rating table for Menominee River near Iron Mountain, Mich., September 4, 1902, to December 31, 1905.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
1.2	1,032	2.8	2,080	4.4	3,242	6.8	5,230
1.3	1,094	2.9	2,150	4.5	3,319	7.0	5,420
1.4	1,156	3.0	2,220	4.6	3,396	7.2	5,615
1.5	1,219	3.1	2,290	4.7	3,474	7.4	5,815
1.6	1,282	3.2	2,361	4.8	3,552	7.6	6,025
1.7	1,346	3.3	2,432	4.9	3,630	7.8	6,235
1.8	1,410	3.4	2,503	5.0	3,708	8.0	6,450
1.9	1,475	3.5	2,575	5.2	3,865	8.5	7,020
2.0	1,540	3.6	2,647	5.4	4,023	9.0	7,630
2.1	1,606	3.7	2,719	5.6	4,183	9.5	8,280
2.2	1,672	3.8	2,792	5.8	4,345	10.0	8,970
2.3	1,739	3.9	2,866	6.0	4,510	10.5	9,670
2.4	1,806	4.0	2,940	6.2	4,680	11.0	10,370
2.5	1,874	4.1	3,015	6.4	4,860	11.5	11,070
2.6	1,942	4.2	3,090	6.6	5,040	12.0	11,770
2.7	2,011	4.3	3,166				

Estimated monthly discharge of Menominee River near Iron Mountain, Mich., September, 1902, to December 31, 1905.

[Drainage area, 2,415 square miles.]

Date.	Discharge.			Run-off.	
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.
	Sec.-feet.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*
1902.					
September (4-30)	1,772	1,032	1,295	0.536	0.535
October	2,625	1,094	1,596	.661	.762
November	5,306	1,806	2,829	1.17	1.30
December	2,647	1,282	1,909	.790	.911
1903.					
April	6,780	1,705	5,175	2.39	2.[illegible]4
May	11,560	4,698	7,496	3.57	3.10
June	8,020	1,540	3,417	1.57	1.41
July	6,670	1,806	3,553	1.70	1.[illegible]
August	7,630	2,467	4,049	1.94	1.[illegible]
September	10,650	2,575	5,091	2.35	2.11
October	6,130	2,719	4,057	1.94	1.68
November	3,669	1,874	2,505	1.16	1.04
December	2,719	1,705	2,150	1.03	.890
1904.[a]					
April	8,150	2,683	3,995	1.64	1.[illegible]
May	11,770	3,630	7,879	3.76	3.[illegible]
June	8,410	2,575	4,791	3.21	1.[illegible]
July	3,396	1,094	2,196	1.05	.90[illegible]
August	3,242	1,032	2,125	1.01	.[illegible]
September	3,669	1,410	2,488	1.15	1.03
October	6,725	1,840	3,650	1.74	1.51
November	3,591	1,378	2,293	1.06	.949
December	2,199	1,672	1,838	.877	.76[illegible]
1905.[a]					
April	7,140	4,265	5,282	2.19	2.44
May	9,250	2,503	6,810	2.82	3.25
June	9,250	1,806	5,011	2.07	2.31
July	7,140	1,573	3,850	1.59	1.83
August	3,090	1,540	2,130	.882	1.02
September	6,450	2,080	3,284	1.36	1.52
October	2,611	1,772	2,163	.896	1.03
November	2,432	1,410	2,204	.913	1.02
December	2,539	1,378	2,085	.863	.99[illegible]

[a] Ice conditions January, February, and March. No estimate made.

The following table of drainage areas of Menominee River at various points is compiled from Water-Supply and Irrigation Paper No. 83:

Menominee River drainage areas.

	Square miles.
Brule River above Iron River	170.0
Iron River above mouth	94.7
Brule River, including Iron River	264.7
Brule River above Paint River	305.0
Paint River at mouth	738.5
Brule River at junction with Michigamme River	1,044.0

	Square miles.
Michigamme River at mouth	723.7
Menominee River at junction of Brule and Michigamme rivers	1,767.7
Menominee River above junction with Pine River	1,833.0
Pine River	586.0
Menominee River, including Pine River	2,419.0
Menominee River above Sturgeon River	2,538.0
Sturgeon River at mouth	396.0
Menominee River, including Sturgeon River	2,934.0
Menominee River above junction with Pemebonwon River	2,993.0
Peme Bon Won River	163.0
Menominee River, including Pemebonwon River	3,156.0
Menominee River above junction with Pike River	3,274.0
Pike River	292.0
Menominee River, including Pike River	3,566.0
Menominee River above Little Cedar River	3,792.0
Little Cedar River	149.0
Menominee River, including Little Cedar River	3,941.0
Menominee River at mouth	4,113.0

WATER POWERS.

GENERAL CONDITIONS.

Principally because of the opening up of the many rich and valuable iron mines of this region, and the resulting extensive railroad building, the valley of Menominee River has had a rapid development. The following railroads at present have extensions in this territory: Chicago, Milwaukee and St. Paul; Chicago and Northwestern; Minneapolis, St. Paul and Sault Ste. Marie; and Wisconsin and Michigan. All of them cross the Menominee one or more times, and several are near enough to run short spurs to the important water-power sites. The developed water power is at present used for the most part in mining and for the operation of lumber, paper, and pulp mills.

Menominee River varies in width from 200 to 600 or 700 feet far up toward the headwaters. For the first 7 miles from the junction of the Brule and Michigamme there are no heavy rapids, but, in the language of the lumberman, there is "strong water" all the way and probably many good water-power sites.

BAD WATER RAPIDS.

The first notable rapids, known as the Bad Water rapids, occur 7 miles below the head of the river, in sec. 27, T. 40 N., R. 19 E., at a point where the river, 100 feet wide, descends 5 feet over a ledge of rock. While definite information is lacking, it is likely that a dam could be built here, giving a head of 10 feet.

TWIN FALLS.

About 3½ miles below Bad Water rapids, in sec. 2, T. 39 N., R. 19 E., are the Twin Falls, about one-half mile apart. The vertical fall in each case is 12 feet, but the adjacent rapids are sufficient to increase the total descent to 28 feet.

PINE RIVER RAPIDS.

For 6 miles below the foot of Twin Falls the total descent of the river is but 20 feet, and the only rapids worthy of note are those extending for about five-eighths of a mile on both sides of the mouth of Pine River. Here an island divides the river into two channels with rocky bed. The descent of the rapids at this point is said to be 6 feet, but as the banks are high a dam could develop more than this. Pine River increases the drainage area by 586 square miles.

HORSE RACE RAPIDS.

The most important rapids between Twin Falls and Big Quinnesec Falls, called the Horse Race, are found in sec. 7, T. 38 N., R. 20 E., both above and below the Chicago, Milwaukee and St. Paul Railroad bridge. These rapids consist of two pitches, the upper of about 20 and the lower of 8 feet descent, separated by about 2,000 feet of less swift water. As the banks are high and the river narrow, it seems likely that a dam could be economically constructed here to develop about 40 feet of head. This site is only 3 miles from Iron Mountain, Mich.

BIG QUINNESEC FALLS.

A little over 7 miles below the mouth of Pine River, and 4 miles from Quinnesec, are the Big (Upper) Quinnesec Falls. These are located in sec. 6, T. 38 N., R. 20 E.

At Upper Quinnesec Falls the river narrows to hardly more than 50 feet wide (map measurement), between rocky banks of igneous origin. Immediately at the foot of the falls the river widens out and about 800 feet below is 700 feet across. On the Wisconsin side the banks are 80 to 100 feet high and on the Michigan side 30 to 40 feet.[a]

Below the falls the river descends only 2 feet to the mile for a distance of about 3 miles. At present only 54 feet of the total head is improved, one-half of the power being used to compress air for the supply of the Chapin Iron Mines at Iron Mountain, 3½ miles distant. The remaining portion is to be harnessed in 1905 and used for operating mines at Norway 9 miles away. On account of the local conditions it is unlikely that much more than the present head can be economically developed.

LITTLE QUINNESEC FALLS.

Four miles below in sec. 10, T. 38 N., R. 20 E., are the Little (lower) Quinnesec Falls which, together with the upper falls, described above, form the most important powers on the river. For the greater portion of the distance between the upper and lower Quinnesec Falls there is comparatively quiet water. The greater part of the descent of 24 feet in this distance occurs in the lower 2 miles. Above the upper and below the lower falls the banks are generally high near the river, but between these falls the hills recede from the river an average distance of about one-half mile and are separated from it by a flat and in some places swampy area.

Maj. T. B. Brooks, who reported on the geology of this district, considered that the shore deposits indicated the presenee of a lake at a comparatively recent date.

Above Little Quinnesec Falls the river runs southwest, but at the foot of the falls it suddenly turns at right angles and runs southeast, the water surging down an incline of about 45° and then plunging into the comparatively still water of the basin below. The total fall is 62 feet. A short distance above the falls the river is 250 feet wide, but narrows down at the pitch to about 50 feet. The falls are hemmed in by great masses of greenstone and schist rock. Along the Michigan side a steep cliff of greenstone at least 140 feet high forms the bank for a distance of a mile or more. A smaller, but similar, rib of rock forms the Wisconsin bank for about 700 feet.

Formerly Little Quinnesec Falls were partially developed under 25 feet head for wood-pulp grinding; but in 1898 they were redeveloped by the Kimberly & Clark Company for wood-pulp and paper manufacturing. A ledge of rock, which is used for a bridge pier, divides the falls into two channels. The present development gives a net head of 62 feet, equivalent to 8,370 theoretical horsepower. An actual installation of turbines, generating 5,800 horsepower, consumes all the available power.

SAND PORTAGE RAPIDS.

These rapids lie between Little Quinnesec Falls and the mouth of Sturgeon River. They receive this name because the Indians, in making their "carry" around part of them,

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[a] Tenth Census, vol. 17.

passed over a large amount of sand. The rapids are scattered along a distance of 6 miles, in which space there is a descent of 60 feet. About half of this amount is concentrated in the 1½ miles between the falls and the old cable bridge or ferry below. As the topographic map shows very high banks, fairly close together, a head of 25 feet or more may some day be developed here. The Chicago and Northwestern Railway is distant only 1.5 miles.

Between the above-described dam site and a point 2.5 miles below, the river descends 27 feet. A point due south of Norway, Mich., and on the road leading from that city is probably the best location for the dam to develop this fall, but even here a dam not less than 700 feet long would probably be required.

Menominee River descends but 6 feet between this point and the mouth of Sturgeon River. This may be considered a part of the Sturgeon Falls power.

STURGEON FALLS.

From below the mouth of Sturgeon River to a point just above Pemebonwon River, a distance of 10 miles, the drainage area increases from 2,934 square miles to 2,993 square miles. In this stretch are Sturgeon Falls, one-half mile below the mouth of Sturgeon River, in sec. 22, T. 38 N., R. 21 E., Wisconsin. These falls have high rock-ledge banks, with two pitches aggregating 13 feet. By backing the water a distance of about 3 miles this head could be increased to 15 feet. At the head of the falls the river narrows to about 200 feet, but at the foot it spreads out into a broad basin. In order to use the power it will probably be necessary to blast out a race in the rocks or build a flume and locate the mill at or near the foot of the rapids.

In the next 10 miles the river descends only 17 feet, with a fairly even grade, except for two or three small rapids. The largest of these, Nose Peak rapids, is about 1,000 feet long and descends about 4 feet.

PEMENA DAM AND RAPIDS.

A logging dam which, together with the adjacent rapids, gives a fall of 14 feet in a distance of a quarter of a mile is located in sec. 24, T. 37 N., R. 21 E. The Minneapolis, St. Paul and Sault Ste. Marie Railway crosses the river 2½ miles above the dam and passes within a fraction of a mile from it. The operation of a dam at this point for lumbering purposes greatly lessens the amount of available power. At the present rate of progress, however, this dam will be needed for logging only a few more years. It has been found elsewhere in the State that river logging, except for pine, can not compete with railroad transportation.

From below Pemebonwon River to a point just below Pike River, a distance of 18 miles, the drainage area increases from 3,156 square miles to 3,566 square miles. Pemena, Chalk Hill, and White rapids occur in this distance.

About a mile above the mouth of Pemebonwon River, in sec. 8, T. 36 N., R. 21 E., the Pemena rapids begin. They extend for a distance of about 2 miles, with a total descent of 20.2 feet.[a] The river bed here is a metamorphic slaty schist, and the location is said to be favorable for a dam site. The Wisconsin and Michigan Railway runs parallel to the river at this point and is only 2 miles distant, and the Minneapolis, St. Paul and Sault Ste. Marie Railway crosses the river a few miles above.

CHALK HILL RAPIDS.

In the 11 miles between the foot of Pemena rapids and the head of White rapids the river descends 38 feet, the grade being even except for three small rapids of from 3 to 6 feet each. Chalk Hill rapids, the most important of these three, are located in sec. 6, T. 35 N., R. 21 E. They run over a slaty rock at a point said to be suitable for a dam, and if developed in connection with other falls about half a mile above would give a total head of 8 feet or more.

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[a] This statement is based on an accurate profile of the river, prepared by Mr. T. W. Orbison, C. E., from his actual surveys. The statement made in the Tenth Census, vol. 17, p. 61, that the total fall is 70 feet, is evidently an error.

WHITE RAPIDS.

Four miles above the mouth of Pike River, in sec. 19, T. 35 N., R. 21 E., are the White rapids. The bed of the river is said to be gravel and bowlders, and the banks are high enough to give a head of 30 feet, thus developing the fall for 3 miles. Even above this limit the river descends 10 feet in 1¼ miles, as will be seen from the profile (p. 51). A head of 30 feet at ordinary low water would develop 5,350 theoretical horsepower.

From below Pike River to a point just above Little Cedar River, a distance of 25 miles, the drainage area increases from 3,566 to 3,792 square miles.

All the rapids thus far described have been over the pre-Cambrian crystalline rocks. In the next 28 miles the river crosses the Cambrian sandstone and "Lower Magnesian" limestone. No falls or rapids worthy of note occur until Grand rapids are reached, immediately above the mouth of Little Cedar River, in sec. 5, T. 33 N., R. 22 E. These rapids are caused by a descent over hard "Trenton" limestone, underlain by softer strata. They have a fall stated at 25 feet in a length of 3 miles, but of this fall only that in the lower 2 miles, amounting to 18 feet, can be cheaply developed. Both the Wisconsin and Michigan and the Chicago, Milwaukee and St. Paul railways pass within 2 or 3 miles of this site.

From below the mouth of Little Cedar River to the mouth of the Menominee, 23 miles, the drainage increases from 3,941 to 4,113 square miles.

TWIN ISLAND RAPIDS.

These rapids are situated about 7 miles below the Grand rapids and 16 miles from the mouth of the river. They extend for three-fourths of a mile and are said to descend 10 feet. The two islands lie one below the other, dividing the river into east and west channels. The bed of the river is limestone, the banks are steep, and a dam could be built across each channel to the islands. The total length of such dams is estimated at about 700 feet. A sawmill with a 6-foot head once occupied the east channel.

SCHAPPIES RAPIDS.

Located about 5 miles from the mouth of Menominee River, in T. 31 N. and between Rs. 22 and 23 E., Schappies rapids extend for a distance of about a mile. During the winter of 1897 a survey was made of these rapids by a competent engineer, Mr. C. B. Pride, at a time of extreme low water. He found a discharge of 2,370 second-feet and determined that a head of 18 feet could be economically obtained. This power belongs to the Menominee River Boom Company. The Chicago, Milwaukee and St. Paul Railway is located about 3 miles distant.

MARINETTE DAMS.

The last series of rapids is found at Marinette, Wis., near the mouth of the Menominee. The natural channel probably had about 12 feet descent here, but the Menominee River Boom Company built three dams, one above another, the upper one backing the water to the foot of Schappies rapids. The first of these dams, 850 feet long, located about 3 miles from the mouth of the river, in T. 30 N. and near the line between Rs. 23 and 24 E., develops a head of 7 feet.[a] Power is applied to two paper and pulp mills owned by the Marinette and Menominee Paper Company and also to a flouring mill. No statement of the turbine installation at the paper mills is made, but that at the flouring mill is 95 horsepower.

The third dam from the mouth is located on the west line of sec. 1, T. 30 N., R. 23 E. This dam is 940 feet long and has a head of 18 feet. The middle or second dam is located about a quarter of a mile below the third dam and is 700 feet long, with a head of 7 feet. It is used for boom purposes only. The Marinette and Menominee Paper Company mill

[a] Data regarding the Marinette dams furnished by the owners.

is located just below this dam, but it takes power through a canal from the third dam. Its turbines therefore work under a total head of about 24 feet.

The owners of these three dams state that each could be raised from 5 to 10 feet higher than at present.

TRIBUTARIES OF MENOMINEE RIVER.

The notable Wisconsin tributaries of Menominee River are Brule, Pine, Pemebonwon, and Pike rivers.

Brule River courses in a bed composed mostly of gravel and bowlders of the drift, and for this reason has few vertical falls, one of 10 feet being said to exist at its mouth. It is described as having a series of rapids or "strong water" for its entire length of 42 miles. Its total drainage area, including that of Paint River, is 1,044 square miles.

The following table gives a fairly complete profile of Brule River:

Profile of Brule River, Wisconsin, from its mouth to sec. 23, T. 41 N., R. 14 E.[a]

No.	Station.	Distance— From mouth.	Distance— Between points.	Elevation above sea level.	Descent between points. Total.	Descent between points. Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Brule, Wis. (C. & N. W. bridge)	7.0		1,260		
2	½ mile below section line 22-23, T. 41 N., R. 15 E.	24.0	±17.0	1,411	151	8.8
3	Center of bend E. ¼ stake, sec. 31, T. 41 N., R. 15 E.	29.5	5.4	1,431	20	3.7
4	¼ mile west of east line, sec. 24, T. 41 N., R. 14 E.	31.6	2.1	1,468	37	18.0
5	0.4 mile below dam. Noted below	33.1	1.5	1,490	22	14.6
6	Above dam 800 feet east of ¼ post, sec. 22-23, T. 41 N., R. 14 E.	33.5	.4	1,507	17	42.5
7	¼ mile last of section line 22-23, T. 41 N., R. 14 E.	35.5	2.0	1,520	13	6.5

[a] Authority: No. 1, Chicago and Northwestern Railway; Nos. 2-7, U. S. Geol. Survey.

Pine River, the largest tributary lying wholly in Wisconsin, has a total length of 53 miles and drains an area of 586 square miles.

In the first half mile from its mouth the current is very rapid[b]; in the next 12 or 13 miles the fall is comparatively slight, and in the next 3 miles there are two falls of 8 feet each 1,000 feet apart, half a mile of strong water, succeeded by another fall of 12 feet, then, half a mile above, a fall of 40 feet. Sixty feet above this is a logging dam belonging to the Menominee River Improvement Company.[b]

The length of Pike River is 48 miles.

[b] Tenth Census.

DAMS ON MENOMINEE RIVER AND TRIBUTARIES.

The location and height of dams on Menominee River and tributaries in Wisconsin are shown in the following table:

Dams on Menominee River and tributaries in Wisconsin.

Dam.	Section.	Township.	Range.	Height of dam.
Menominee River:				*Feet.*
1	6	30	24	7
2	1	30	23	7
3	32	31	22	14
Pemena dam	24	37	21	12
Pike River:				
1	8	35	21	9
2	16	35	20	13
North Branch of North Branch Pike River	28	37	18	13
North Branch Pike River:				
1	32	36	20	9
2	20	36	20	13
South Branch Pike River:				
1	19	35	20	13
2	31	36	19	9
3	35	36	18	11
4	29	36	18	10
5	17	36	18	6
Pine River:				
1	30	39	16	9
2	11	39	15	10
3	10	39	14	10
4	36	40	13	9
Brule River:				
1	5	40	17	7
2	19	41	16	8
3	15	42	13	8
Wheeler dam	23	41	14	10

PESHTIGO RIVER.

In length, grade, shape, and size of drainage area Peshtigo River closely resembles its neighbor, the Oconto. It descends an average of nearly 10 feet to the mile, but few of its powers have as yet been developed, because this region is very thinly populated. The only powers reported are two at Peshtigo. A dam with a 10-foot head, owned by the Peshtigo Lumber Company, supplies the power for a sawmill, which has turbines of 1,390 horsepower installed. A flouring mill of 50 horsepower is also located at Peshtigo.

The next important development is a power known as "High Falls" in sec. 1, T. 32 N., R. 18 E. In a distance of 165 feet the river descends 46 feet. A dam 1,000 feet long would increase this to 55 feet. In a recent report on this power by a competent engineer, it is stated that a dam 200 feet long 1½ miles above this point would create an immense reservoir. Both dam sites are on the pre-Cambrian rock, and the banks are of clay and sand.

A few miles below, in sec. 9, T. 32 N., R. 19 E., are the Grindstone Rapids, with a fall of 25 feet. The banks at this point are said to be high and steep. The Wisconsin Geological Survey map shows a descent of 35 feet in sec. 10, T. 33 N., R. 18 E. A dam at Ellis Junction creates a large pond and furnishes a head of 12 feet, which was formerly used to run a sawmill. It is now proposed to increase this head to 24 feet and to use the power for a new pulp mill.

Between Ellis Junction and the mouth the Chicago, Milwaukee and St. Paul and the Chicago and Northwestern railways are adjacent to the river, which is still being used for lumbering purposes. Besides the above-described dams, logging dams are located in sec. 10, T. 33 N., R. 18 E., and in sec. 22, T. 34 N., R. 18 E., with heads of 10 and 8 feet, respectively. The following table shows the profile of the river:

Profile of Peshtigo River from its mouth to near North Crandon.

Station.	Distance from mouth.	Elevation above sea level.	Authority.
	Miles.	*Feet.*	
Mouth of river		581.3	United States engineers.
Peshtigo	18	594.7	Wisconsin and Michigan Rwy.
Do	+18	619.7	Chicago and Northwestern Rwy.
West of Ellis Junction	48	658.0	Do.
Near North Crandon	140	1,620.0	Minneapolis, St. Paul and Sault Ste. Marie Rwy.

OCONTO RIVER.

GENERAL CONDITIONS.

Oconto River rises in a number of small lakes and swamps in the plateau region, at an elevation of about 1,530 feet above the sea. In its length of 87 miles it descends 945 feet. In the upper 35 miles of its course the river flows over the crystalline rocks, and here is found about two-thirds of its total fall. Upon leaving the crystalline rocks the river flows nearly due south for 20 miles over the Cambrian sandstones. At Underhill it turns abruptly and flows nearly due east, crossing the "Lower Magnesian" and "Trenton" limestones and joining Lake Michigan near Oconto. The profile of the river is shown in the following table:

Profile of Oconto River, Wisconsin, from its mouth to Wabena.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Chicago and Northwestern Railway bridge, Oconto	2		581		
2	Chicago, Milwaukee and St. Paul Railway bridge, Oconto	7	5	590	9	1.8
3	Stiles	13	6	614	24	4.0
4	Underhill	33	20	770	156	7.8
5	Surings	44	11	791	21	1.9
6	One mile south of mountain	60	16	916	125	7.8
7	Two miles north of mountain	63	3	941	25	8.3
8	Wabena	87	24	1,526	585	24.3

The most important powers are found in the last 33 miles of its course, in which distance the river descends 190 feet.

WATER POWERS.

STILES.

The first dam above the mouth of the Oconto River is located at Stiles, in sec. 34, T. 28 N. R. 20 E., where a dam 400 feet long, with 11-foot head, furnishes power for saw and pulp mills owned by the Anson Eldred Company. This company has installed turbines of 500

[a] Authority: Nos. 1 and 4-8, Chicago and Northwestern Railway; Nos. 2 and 3, Chicago, Milwaukee and St. Paul Railway.

horsepower. It is reported that by constructing a dike about 450 feet long the head could be increased to 18 feet.

OCONTO FALLS.

The most important concentration of fall on the river, about 100 feet, occurs in the "Lower Magnesian" limestone at Oconto Falls, in sec. 25, T. 28 N., R. 19 E. A dam owned by the Falls Manufacturing Company has a head of 37 feet and supplies power for a large paper and pulp mill. The company has installed turbines rated at 1,370 horsepower, besides 400 steam horsepower. About a quarter of a mile farther up is located a dam of 19-foot head, which furnishes power for a large pulp mill, belonging to the Union Manufacturing Company. Seven turbines rated at 940 horsepower are installed. These run twenty-four hours every day except Sunday. Only half a mile below the Falls Manufacturing Company's dam are some important rapids, where an excellent power is available. It is estimated that a dam 250 feet long would develop a head of nearly 40 feet. This power is owned by E. A. Edmonds, who has a charter for a dam at this point with a head of 27.5 feet. The Chicago and Northwestern railway furnishes excellent shipping facilities at all the Oconto Falls powers described above.

PULCIFER DAM.

The last dam used for power purposes is located in sec. 6, T. 27 N., R. 18 E., and furnishes power for a gristmill. It is also used for logging purposes.

MISCELLANEOUS POWERS

The following table gives the location and extent of the most important developed and undeveloped water powers on the Oconto River:

Water powers on Oconto River.

No.	Location.	Estimated head.[a]	H. P. installed.	Use.
	DEVELOPED POWERS.	Feet.		
1	Stiles, sec. 34, T. 28 N., R. 20 E	11	500	Saw and pulp mill.
2	Oconto Falls, sec. 25, T. 28 N., R. 19 E	37	1,370	Paper and pulp mill.
3	Oconto Falls, sec. 26, T. 28 N., R. 19 E	19	940	Pulp mill.
4	Pulcifer, sec. 6, T. 27 N., R. 18 E	12	45	Flouring mill and driving.
5	Sec. 25, T. 31 N., R. 16 E	12	....	Driving only.
6	Sec. 4, T. 31 N., R. 16 E	10	....	Do.
7	Sec. 23, T. 32 N., R. 16 E	10	....	Do.
8	Sec. 30, T. 33 N., R. 17 E	12	....	Do.
9	Sec. 5, T. 33 N., R. 16 E	10	....	Do.
10	Sec. 1, T. 33 N., R. 15 E	10	....	Do.
11	Sec. 11, T. 32 N., R. 16 E	10	....	Do.
12	Sec. 34, T. 33 N., R. 16 E	10	....	Do.
13	Sec. 30, T. 33 N., R. 16 E	10	....	Do.
14	Sec. 27, T. 33 N., R. 15 E	12	....	Do.
15	Sec. 18, T. 31 N., R. 17 E	10	....	Do.
16	Sec. 33, T. 32 N., R. 17 E	10	....	Do.
17	Sec. 21, T. 32 N., R. 17 E	10	....	Do.
18	Sec. 23, T. 30 N., R. 16 E	10	....	Do.
19	Sec. 16, T. 30 N., R. 16 E	6	....	Do.
	UNDEVELOPED POWERS.			
20	Oconto, sec. 23, T. 28 N., R. 21 E	12	....	
21	Oconto Falls, sec. 31, T. 28 N., R. 20 E	40	....	
22	Sec. 34, T. 28 N., R. 18 E	15	....	
23	Sec. 23, T. 31 N., R. 16 E	20	....	

[a] The first four heads are reported by owners; the remainder are estimated by Mr. W. A. Holt, of the Holt Lumber Co., Oconto.

WOLF RIVER SYSTEM.

GENERAL CONDITIONS.

Wolf River rises in a number of lakes about 25 miles south of the Michigan boundary and flows in a general southerly direction, entering upper Fox River at a point about 10 miles west of Lake Winnebago. Though nominally a branch of Fox River, it is in reality the master stream, having over three times the discharge. Wolf River receives all its important tributaries from the west and at points relatively near its mouth. It has been elsewhere noted (p. 64) that there is much evidence that the river formerly ran west and joined Mississippi River through the present Wisconsin River Valley between Portage and Prairie du Chien.

In the upper half of its course Wolf River has formed its bed in the pre-Cambrian crystalline rocks, and in this distance the descent of the river is very rapid. At the Chicago and Northwestern railway crossing, 2 miles west of Lenox, the river has an elevation of 1,562 feet above the sea. In the 80 miles between this point and Shawano the river descends 774 feet, or 9.7 feet per mile. This steep gradient causes many rapids and falls. Lumbering dams have been maintained on the upper river at the following points:[a] Sec. 9, T. 33 N., R. 12 E.; Lilly dam, sec. 34, T. 33 N., R. 13 E.; sec. 10, T. 31 N., R. 14 E.; sec. 25, T. 31 N., R. 14 E., and at several other places lower down. In the 40 miles above Shawano small undeveloped powers of 10 to 15 feet head are of frequent occurrence.

Shawano, the head of navigation on the river, and county seat of Shawano County, has a population of 2,000. A dam is located at this point, with a head of 12 feet. It is used to grind wood pulp. Shawano also marks the point of transition from the pre-Cambrian to the Cambrian sandstone. It is at this point that the river crosses the old coast line of Lake Michigan and enters the region of red clay. Below Shawano the stream is sluggish, its descent being only about 42 feet to Lake Winnebago, a distance of about 80 miles. The banks are low, and in high water the surrounding flats are all covered, the river sometimes expanding at time of heavy freshets to several miles in width. For obvious reasons there can be no water powers in this lower region.

The profile of Wolf River for 160 miles of its course is shown in the following table:

Profile of Wolf River, Wisconsin, from mouth to near Lenox.

Station.	Distance from mouth.	Elevation above sea level.	Authority.
	Miles.	*Feet.*	
Winneconne		746.4	United States Engineers.
New London	33	749.5	Chicago and Northwestern Railway.
Shawano	80	788.0	Do.
Lenox	160	1,562.5	Do.

[a] Wisconsin Geological Survey maps.

RUN-OFF.

The following tables showing gage-height observations and discharge measurements at Winneconne and near Northport, on Wolf River, are from data published by the United States Geological Survey:

Discharge measurements of Wolf River at Winneconne, Wis., in 1903.

Date.	Hydrographer.	Gage height.	Discharge.
		Feet.	*Second-feet.*
January 5 [a]	L. R. Stockman	5.50	94
January 24 [a]	do	5.30	1,4[illegible]
February 20	do	5.00	1,2[illegible]
March 24	do	6.60	9,9[illegible]
April 15	do	6.90	3,[illegible]
May 11	do	6.70	3,5[illegible]
June 20	do	6.40	3,194

[a] River frozen.

Mean daily gage height, in feet, of Wolf River at Winneconne, Wis., January 1 to July 25, 1903.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.
1	5.50	5.30	4.80	7.10	6.60	7.00	6.10
2	5.50	5.30	4.80	7.20	6.65	7.00	6.10
3	5.50	5.30	4.80	7.20	6.70	7.00	6.10
4	5.50	5.20	4.80	7.10	6.65	6.90	6.10
5	5.50	5.20	4.80	7.10	6.60	6.80	6.10
6	5.50	5.20	4.90	7.10	6.65	6.80	6.10
7	5.50	5.20	4.90	7.05	6.70	6.85	6.20
8	5.50	5.20	4.90	6.90	6.70	6.80	6.20
9	5.50	5.20	4.90	6.80	6.70	6.80	6.20
10	5.50	5.10	5.00	6.95	6.70	6.80	6.30
11	5.50	5.10	5.00	7.10	6.80	6.70	6.30
12	5.50	5.10	5.10	7.00	6.80	6.70	6.30
13	5.50	5.10	5.25	7.00	6.80	6.60	6.30
14	5.50	5.10	5.30	6.90	6.80	6.60	6.30
15	5.50	5.00	5.60	6.80	6.80	6.60	6.30
16	5.50	5.00	5.70	6.85	6.80	6.50	6.30
17	5.50	5.00	5.80	6.90	6.80	6.50	6.30
18	5.40	5.00	5.90	6.80	6.80	6.45	6.40
19	5.40	5.00	6.00	6.80	6.80	6.45	6.40
20	5.40	5.00	6.20	6.75	6.80	6.40	6.40
21	5.40	4.90	6.30	6.70	6.80	6.40	6.30
22	5.40	4.90	6.40	6.80	6.80	6.40	6.30
23	5.40	4.90	6.50	6.80	6.80	6.30	6.20
24	5.40	4.90	6.60	6.80	6.85	6.30	6.20
25	5.40	4.90	6.70	6.80	6.90	6.20	6.10
26	5.40	4.80	6.80	6.80	6.90	6.20	....
27	5.30	4.80	6.90	6.70	7.05	6.10	....
28	5.30	4.80	6.90	6.70	6.90	6.10	....
29	5.30	....	6.90	6.65	7.00	6.10	....
30	5.30	....	7.00	6.60	7.00	6.10	....
31	5.30	....	7.10	....	7.00	....	....

Discharge measurements of Wolf River near Northport, Wis., in 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
		Feet.	*Square feet.*	*Feet per second.*	*Feet.*	*Second-feet.*
April 5	F. W. Hanna	182	2,642	2.64	7.03	6,965
May 27	S. K. Clapp	171	2,198	1.8	4.65	3,964
June 17	M. S. Brennan	151	2,553	1.97	6.42	5,032
July 15	do	176	2,300	1.69	5.06	3,885
August 16	do	176	2,053	1.26	3.01	2,594
September 22	F. W. Hanna	172	1,978	1.41	3.6	2,781

Mean daily gage height, in feet, of Wolf River near Northport, Wis., April 6 to December 30, 1905.

Day.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1		3.40	6.00	3.30	2.00	1.60	1.55	1.30	1.70
2		3.60	5.00	3.00	2.30	2.40	1.40	1.40	1.60
3		3.80	5.00	3.40	2.20	2.60	1.35	1.50	1.40
4		4.00	4.90	3.60	2.10	2.20	1.30	1.60	1.50
5		4.20	4.60	3.80	2.30	3.10	1.15	1.70	1.60
6	6.90	4.40	5.60	3.30	2.20	3.40	1.10	1.95	1.75
7	6.80	4.60	5.40	4.30	2.40	3.60	.90	2.10	1.80
8	6.70	4.80	5.30	4.60	2.90	3.30	.85	2.30	1.90
9	6.60	4.80	5.80	4.60	3.30	3.40	.70	2.50	2.10
10	6.50	5.00	5.80	4.90	4.00	3.60	.65	2.70	2.00
11	6.40	5.00	5.80	5.00	3.50	3.80	.60	2.60	1.90
12	6.30	5.20	5.90	5.20	3.60	3.50	.50	2.40	1.95
13	6.10	5.60	6.10	5.10	3.60	2.80	.35	2.30	1.80
14	6.00	5.80	6.40	5.10	3.00	2.50	.10	2.20	1.60
15	5.80	5.60	6.40	4.90	3.00	2.70	.25	2.10	1.40
16	5.60	5.50	6.60	4.20	3.50	2.90	.40	1.80	1.20
17	5.50	5.30	6.50	4.60	3.50	2.80	.75	1.50	1.20
18	5.20	5.30	6.40	4.30	3.30	3.00	.90	1.40	1.10
19	5.20	5.20	6.40	4.80	3.20	3.40	1.15	1.30	1.00
20	4.90	5.00	6.20	4.60	3.00		1.50	1.20	1.00
21	4.80	4.80	6.00	4.45	2.80	2.90	2.20	1.10	1.00
22	4.80	4.60	5.80	4.20	2.40	2.80	2.60	1.00	1.00
23	4.30	4.60	5.60	4.10	2.50	3.70	2.90	.90	.90
24	4.10	4.80	5.30	4.00	2.30	3.60	3.20	.80	.90
25	4.00	5.00	5.10	3.30	2.00	3.40	3.40	.60	.75
26	3.80	5.00	4.70	3.60	1.80	3.25	3.30	.40	.60
27	3.60	4.60	4.40	2.80	1.60	3.10	3.20	.20	.50
28	3.50	4.30	4.00	2.50	1.40	2.90	3.00	.60	.50
29	3.50	5.40	3.80	2.30	1.20	2.75	2.10	.80	.40
30	3.40	5.60	3.50	2.20	1.10	2.35	2.30	1.90	.40
31		5.80		2.00	1.00		2.00		

TRIBUTARIES OF WOLF RIVER.

The lower part of the Wolf River drainage area is more thickly settled than the upper, and as a result the tributaries which occupy this lower portion are rather fully developed. This is especially true of Embarrass, Little Wolf, and Waupaca rivers.

WATER POWERS.

The following table shows the water powers on Wolf River and its tributaries:

Water powers on Wolf River and its tributaries.

Location and stream.	Owner and use.	Head.	H. P.
		Feet.	
Manowa, sec. 15, T. 23 N., R. 13 E., Little Wolf River.	Little Wolf River Lumber Co., grist, lumber, electric light.	10	300
Littlewolf, sec. 34, T. 23 N., R. 13 E., Little Wolf River.	Booth & Smith, grist, lumber, electric light.	9	60
Scandinavia, south branch Little Wolf River..	Henry Peterson, feed mill...........	8	25
Sec. 22, T. 23 N., R. 11 E., south branch of Little Wolf River.	J. I. Wralstatt, feed mill..............	9	50
Phlox, sec. 26, T. 30 N., R. 12 E., Red River.....	J. Kaufman, saw and planing mill	14	75
Mount Morris, sec. 16, T. 19 N., R. 11 E., Rattlesnake Creek.	Wm. Kemp, grist mill................	12	44
Wittenberg, sec. 10, T. 27 N., R. 11 E., Embarrass River.	Viking Lumber Co., sawmill	12	75
North branch of Embarrass River............	N. M. Edwards, sawmill..............	13	50
Sec. 7, T. 26 N., R. 13 E., Embarrass River ...	N. M. Edwards, undeveloped.........	20	....
Embarrass, sec. 5, T. 25 N., R. 15 E., Embarrass River.	Decker & Beedle, lumber and planing mill.	8	115
Sec. 23, T. 26 N., R. 13 E., middle branch of Embarrass River.	Theo. Boettner, flouring mill..........	9	200
Sec. 15, T. 27 N., R. 15 E., north branch of Embarrass River.	Seiber & Dumke, sawmill	13	192
Sec. 23, T. 28 N., R. 12 E., north branch of Embarrass River.	L. A. Weikel, saw, planing, and feed mill.	16	116
Pilla, sec. 9, T. 26 N., R. 14 E., Embarrass River.	Grosskopt, saw and planing mill.....	13	60
Sec. 9, T. 27 N., R. 12 E., middle branch of Embarrass River.	Buckstaff Lumber Co., power house burned.	10	....
Waupaca, sec. 32, T. 22 N., R. 12 E., Crystal River.	Waupaca woolen mills...............	8	35
Waupaca, sec. 20, T. 22 N., R. 12 E., Waupaca River.	A. G. Nelson, planing and grist mill...	6½	65
City of Waupaca, Waupaca River.............	Electric Light Co....................	18	200
Do..	Undeveloped.........................	15	....
Sherman, sec. 18, T. 22 N., R. 11 E., Waupaca River.	Brooks & Root, flouring mill.........	7	100
Weyauwega, sec. 4, T. 21 N., R. 13 E., Waupaca River.	Weed Gunnard, flour, planing, and electric light.	10	480
Waupaca, Waupaca River	C. Gurines, brick manufacture........	8	30
Amherst, Spring Creek......................	N. Howard, feed mill.................	10	20
Rural, sec. 10, T. 21 N., R. 11 E., Arbor Creek...	J. Ashmun, flouring and saw mill....	9	96
Gresham, sec. 3, T. 27 N., R. 14 E., Red River...	A. G. Schmidt, sawmill..............	11	100
Sec. 6, T. 27 N., R. 15 E., Red River...........	Undeveloped.........................	....	....
Sec. 19, T. 27 N., R. 14 E., Red River..........	.....do..............................	....	....
Sec. 18, T. 28 N., R. 14 E., Red River..........	.....do..............................	....	....
Sec. 15, T. 26 N., R. 10 E., Little Wolf River ..	Little Wolf River Lumber Co., logging	7	0
Sec. 7, T. 25 N., R. 11 E., Little Wolf River ...	.....do..............................	7	0
Sec. 5, T. 24 N., R. 13 E., Little Wolf River ...	.....do..............................	7	0
Sec. 9, T. 33 N., R. 12 E., Wolf River..........	Used for logging....................	....	....
Sec. 34, T. 33 N., R. 13 E., Wolf River.........	.....do..............................	....	....
Sec. 10, T. 31 N., R. 14 E., Wolf River.........	.....do..............................	....	....
Sec. 25, T. 31 N., R. 14 E., Wolf River.........	.....do..............................	....	....

WISCONSIN RIVER SYSTEM.

TOPOGRAPHY AND DRAINAGE.

Because of its length, its great drainage area, and its central location Wisconsin River is preeminently the main river of the State.

Like the Flambeau, the headwaters of Wisconsin River are found in an intricate network of lakes and swamps occupying the flat plateau region near the northern boundary. Its extreme source is found in Lake Vieux Desert, a body of water about 10 square miles on the line separating the northern peninsula of Michigan from Wisconsin, at about 1,650 feet above sea level. The general course of the river for the first 300 miles is south. At a point near Portage it turns abruptly westward, and in the next 100 miles flows nearly west, joining Mississippi River at Prairie du Chien, only 40 miles from the southern boundary of the State.

The drainage basin includes 12,280 square miles, with an average width of 50 miles and a length of about 225 miles. The apportionment of this drainage area among the several tributaries of Wisconsin River is shown in the following table:

Distances and drainage areas of Wisconsin River.

River.[a]	Distance. From source.	Distance. Between stations.	Drainage area above station.
	Miles.	*Miles.*	*Sq. miles.*
Pelican, above mouth		60	940
Pelican, mouth	60	0	1,202
Tomahawk	85	25	2,111
Prairie	113	28	2,697
Rib, above mouth		23	3,192
Rib, mouth	136	0	3,690
Eau Claire	138	2	4,114
Eau Pleine, above mouth		20	4,268
Eau Pleine, mouth	158	0	4,645
Little Eau Pleine	166	8	5,005
Plover	184	18	5,300
Yellow, above mouth		64	6,448
Yellow, mouth	248	0	7,394
Lemonweir	259	11	8,172
Baraboo	292	33	9,095
Wisconsin	407	115	12,280

[a] Station is at mouth of river unless otherwise stated.

Because of its long traverse from the extreme northern to the extreme southwestern part of Wisconsin the topography of the basin includes nearly every form found in the State. Like the upper Chippewa Valley, the northern half is a densely wooded region of hard and soft timber except where cleared for farming. The woods gradually give way to a semi-prairie region with a gently undulating surface, but with occasional decided ridges both of rock and glacial origin. A very striking surface feature toward the southern part is found in the "Baraboo quartzite" ranges, which have an elevation of from 400 to 700 feet above the surrounding country. These ranges comprise two main ridges from 4 to 6 miles apart, extending nearly east and west in the section of country west of Portage for about 25 miles, but uniting and ending abruptly on the west side of the valley, near Portage. The angle of the river at this point seems due to its effort to secure a passage around this rock barrier.

Through a portion of the city of Portage and southward, the river can hardly be said to have an eastern divide. Fox River approaches within 1½ miles of the Wisconsin at this point, only a low marsh intervening. Even this marsh has a slope of about 3 feet toward Fox River. At the present time levees at this and other points prevent the Wisconsin at times of high water from overflowing into Fox River. These levees for a distance of several miles compel the river to flow along the contour instead of in the direction of maximum slope.

The reasons for this and other peculiarities of its valley are interestingly discussed in Geology of Wisconsin, (vol. 3):

It is evident that such an uncertain divide as this can not have formed one of the original permanent features of the drainage of the region, but as the disposition of the surface soil is due to glacial action, modified by subsequent erosion and transportation, this may be fairly attributed to such a cause. The rampart of limestone which compels the lower Wisconsin to flow west does not stop south of Portage, but continues east and north, although less prominent, forming an eastern barrier to the flow of the Wolf River. The course of the upper Fox to Lake Winnebago is sluggish, consisting largely of marshes and lake-like expansions. On account of the depression of the divide at Portage, the continuation of the southern barrier northeast, the small slope of the upper Fox, the large trough of the Wisconsin below Portage, which it is unable to occupy, while above the river is more nearly in proportion to its channel of drainage, and finally the evidently modern outlet for the Wolf and the upper Fox through the lower Fox—the conclusion is reasonable, if not inevitable, that at one time the Lake Winnebago system drained southwest into the Mississippi and the Wolf was the true continuation of the Wisconsin above Portage, while the present upper Wisconsin was merely a tributary of the main stream.

LAKE ELEVATIONS AND RESERVOIR SITES.

Attention has elsewhere been called (p. 15) to the opportunity of increasing the low-water flow of the northern rivers by the construction of dams near the headwaters for use as reservoirs. The opportunity for such a system on Wisconsin River is especially good, because the ownership of the lands to be flooded is in the hands of a comparatively few corporations and a beginning has already been made. For example, a well-built dam at the foot of the Tomahawk chain of lakes, which impounds water covering many square miles of reservoir, has been used for several years to regulate the stage of the river for the mills below the mouth of the Tomahawk. In scores of cases the dams are already constructed for logging purposes and need only to be kept in repair to be of service for power regulation when they are no longer needed for their original purpose, as will soon be the case.

It has been proposed to build or maintain dams at the following points: Lake Vieux Desert, sec. 17, T. 42 N., R. 11 E.; Twin Lakes, sec. 19, T. 41 N., R. 11 E.; Eagle Lakes, sec. 31, T. 40 N., R. 10 E.; Sugarcamp Lakes, sec. 17, T. 39 N., R. 9 E.; Buckataban Lakes, sec. 24, T. 41 N., R. 9 E.; Little St. Germain Lake, sec. 2, T. 39 N., R. 8 E.; Big St. Germain Lake, sec. 18, T. 39 N., R. 8 E.

At many if not most of the larger lakes near the headwaters, logging companies have long maintained dams, which some day will serve the double purpose of reservoirs and sources of power. A list of some of these lakes, together with their elevation above the sea, as determined by United States engineers, is given in the following table:

Lakes at headwaters tributary to Wisconsin River.

Name of lake.	At headwaters of—	Elevation above sea level.
		Feet.
Eagle	Eagle River	1,582.0
Catfish	....do	1,583.0
Cranberry	....do	1,583.5
Long	....do	1,592.2
Planting Ground	....do	1,592.2
Fish	....do	1,592.2
Medicine	....do	1,592.2
Stone	....do	1,592.2
Dog	....do	1,592.2
Big	....do	1,592.2
Pelican	Pelican River	1,590.0
Tomahawk	Tomahawk River	1,562.2
Island	....do	1,560.4
Keawasogan	....do	1,560.4
Mud	....do	1,553.4
Squirrel	....do	1,542.9

The following table [a] gives dimensions and other data of eight reservoir sites surveyed by United States engineers as an aid to navigation on Mississippi River:

Proposed United States Government reservoirs on Wisconsin River.

Name.	Location.			Elevation of low water at dam site.	Maximum dimensions.				Reservoir.		Area of watershed.
					Dam.		Dike.				
	Section.	Township.	Range.		Length.	Height.	Length.	Height.	Area.	Capacity.	
				Feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Sq. mi.*	*Cubic feet.*	*Sq. mi.*
Pelican........	6	36 N.	9 E..	1,520.83	800	28	3,625	15	13.45	5,153,180,527	301.0
Sugarcamp....	17	39 N.	9 E..	1,562.00	235	12.5	260	4	5.00	1,356,284,160	60.0
Otter rapids...	36	40 N.	9 E..	1,578.07	1,300	22	700	5	30.74	7,389,727,488	447.0
Tomahawk....	7	39 N.	6 E..	1,554.67	190	12	......	......	13.46	2,226,113,036	101.5
Squirrel.......	1	38 N.	5 E..	1,521.78	315	17	......	......	5.30	1,338,163,200	56.0
Rice...........	9	35 N.	6 E..	....	1,100	14	......	......	6.00	1,043,516,880	396.0
Vieux Desert..	17	42 N.	11 E.	....	......	......	......	......	7.00	400,000,000	19.0
Twin Lakes...	19	41 N.	11 E.	....	......	......	......	......	6.50	650,000,000	30.0
									87.45	19,556,985,291	1,410.5

Subsequent to this report two of these dams, at Rhinelander (Pelican) and Tomahawk, have been constructed by private enterprise for power purposes; several others have been constructed with reduced heads. It will be noted that the proposed Government reservoirs have a total area of 87.45 square miles and a drainage area of 1,410½ square miles. It was proposed to fill the reservoirs during the spring freshets and then allow the water to escape at times of low water. The United States engineers estimated that these reservoirs would maintain a flow of 3,000 second-feet for three months of the year. Such a flow would nearly double the present low-water flow of the river and its resulting water power. Incidentally the use of such reservoirs would to a large extent serve to reduce the dangers of high floods, both to dams and to overflowed lands. It would, in fact, tend to restore the regimen of the river to that which it possessed before deforesting and cultivation began to transform a great primeval forest region into cleared and well-cultivated fields.

PROFILE.

According to the United States engineers, the elevation of Lake Vieux Desert is about 1,650 feet, while the elevation of the mouth of Wisconsin River at Prairie du Chien is 604 feet at low water or 625 feet at high water. This gives a total descent of about 1,046 feet in an estimated length of 429 miles, or about 2½ feet per mile. About 634 feet of this fall occur in the 150 miles between Rhinelander and Nekoosa, an average of 4.23 feet per mile. This descent is concentrated at many places, producing a large number of valuable water powers, many of which have been improved and used by important industries.

The fall in the main tributaries is even greater in many cases than that in the parent stream, and owing to this fact, and also to the absence of lakes and swamps, it is likely that their discharge is subject to great extremes.

[a] Rept. Chief Eng. U. S. Army, 1880, p. 1655.

A statement in detail of the profile of Wisconsin River is given in the following table:

Profile of Wisconsin River from its mouth to Lake Vieux Desert.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river			604.0		
2	Sauk City	90.0	90.0	746.0	142.0	1.5
3	Merrimac	102.0	12.0	764.0	18.0	1.5
4	Portage	118.0	16.0	790.0	26.0	1.63
5	Kilbourn, railroad bridge	138.0	20.0	814.0	24.0	1.2
6	Sec. 36, T. 15 N., R. 5 E., north line	147.0	9.0	833.0	19.0	2.1
7	Peterwell bridge, opposite Necedah	174.0	27.0	875.3	42.3	[illegible]
	Nekoosa dam:					
8	Below	208.0	34.0	918.9	43.6	1.2[illegible]
9	Above			936.6	17.7	
	Port Edwards dam:					
10	Below	212.5	4.5	938.5	1.9	.4
11	Above			955.5	17.0	
	South Centralia dam:					
12	Below	214.0	1.5	957.3	1.8	1.2
13	Above			969.3	12.0	
	Grand Rapids dam:					
14	Below	216.5	2.5	979.8	10.5	4.2
15	Above			1,002.0	22.2	
	Biron dam:					
16	Below	220.5	4.0	1,005.5	3.5	.9
17	Above			1,016.3	10.8	
	Lower paper mill south of Stevens Point:					
18	Below	233.0	12.5	1,032.4	16.1	1.3
19	Above			1,044.0	11.6	
	Upper paper mill south of Stevens Point:					
20	Below	233.5	.5	1,045.5	1.5	3.0
21	Above			1,058.8	13.3	
22	Stevens Point, Wisconsin Central bridge	236.0	2.5	1,063.8	4.0	1.6
23	Sec. 23, T. 24 N., R. 7 E.	240.0	4.0	1,075.8	13.0	3.2
24	Knowlton bridge, Chicago, Milwaukee and St. Paul Rwy	257.0	17.0	1,092.2	16.4	.97
25	Sec. 8, T. 26 N., R. 7 E.	260.5	3.5	1,097.4	5.2	1.5
26	Sec. 31, T. 27 N., R. 7 E., south line	264.5	4.0	1,104.0	6.6	1.65
27	Mosinee rapids, foot, sec. 29, T. 27 N., R. 7 E., south line	266.0	2.0	1,105.8	1.8	.9
28	Mosinee dam, above	266.5	.5	1,124.6	18.8	37.6
29	Black Creek, mouth of	270.5	4.0	1,125.9	1.3	.3
30	Cedar Creek, mouth of	274.0	3,5	1,130.6	4.7	1.34
31	Eau Claire River, mouth of	279.0	5.0	1,138.6	8.0	1.6
32	Rib River, mouth of	280.5	1.5	1,142.8	4.2	2.8
33	Lower Wausau bridge	283.0	2.5	1,151.0	8.2	3.3
	Wausau dam:					
34	Below	283.5	.5	1,171.0	20.0	40.0
35	Above			1,177.7	6.7	
	Brokaw dam:					
36	Foot	289.0	5.5	1,182.7	5.0	.9
37	Crest			1,194.7	12.0	

[a] Authority: Nos. 1 (low-water elevation) and 53-57, United States engineers; 2 and 3, Major Warren; 4-35, Wisconsin water-power survey by the U. S. G. S. and State authorities; 36-52, levels run by C. B. Pride in 1900 for the Wisconsin River Valley Advancement Association; 56, Chicago and North-western Ry.

Profile of Wisconsin River from its mouth to Lake Vieux Desert—Continued.

No.	Station.	Distance. From mouth.	Distance. Between points.	Elevation above sea level.	Descent between points. Total.	Descent between points. Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
38	Pine River, mouth	298.0	9.0	1,212.7	18.0	2.0
	Merrill:					
39	Lindore dam, foot	304.0	6.0	1,214.7	2.0	.33
40	Lindore dam, crest			1,227.7	13.0	
41	Upper dam, crest	305.0	1.0	1,233.7	6.0	6.0
42	Bill Cross rapids, foot	314.0	9.0	1,245.7	12.0	1.3
43	Grandfather rapids, foot	318.0	4.0	1,272.2	26.5	6.6
44	1.5 miles above, head	319.5	1.5	1,361.7	89.5	6.0
45	Grandmother rapids, foot	321.2	1.7	1,370.7	9.0	5.3
46	Gilbert Station	326.7	5.5	1,409.7	39.0	7.1
	Tomahawk dam:					
47	Foot	328.7	2.0	1,412.7	3.0	1.5
48	Crest			1,425.7	13.0	
49	Nigger Island	344.7	16.0	1,449.4	23.7	1.48
50	Whirlpool rapids, head	346.7	2.0	1,464.8	15.4	7.7
51	Hat rapids, foot	351.7	5.0	1,477.4	12.6	2.5
	Rhinelander dam:					
52	Foot	357.7	6.0	1,523.2	45.8	7.6
53	Crest			1,553.2	30.0	
54	Otter rapids, head	392.7	35.0	1,570.7	17.5	.5
55	Sec. 30, T. 41 N., R. 10 E	402.7	10.0	1,592.7	22.0	2.2
56	Sec. 6, T. 41 N., R. 10 E	416.7	14.0	1,644.0	51.3	3.66
57	Lake Vieux Desert	429.0	12.3	1,650.0	+ 6.0	.5

GEOLOGY.

All that part of the Wisconsin River basin above Nekoosa, including over half the entire drainage, is underlain by pre-Cambrian rocks. North of Merrill this region has been covered so deeply by drift that the rock rarely outcrops except in the river bed. These rocks, by presenting a barrier to further erosion, cause numerous rapids; in fact, all the water powers, with but a single exception,[a] are found in the pre-Cambrian area. Below Nekoosa the pre-Cambrian rocks give way to the softer Cambrian sandstone, the disintegration of which has made the bed of the river one succession of shifting sandbars, almost without interruption, to its mouth. North of Nekoosa this sandy belt rapidly narrows and, at Merrill, 90 miles above, almost entirely disappears, being replaced by the clayey loams and loamy clays. North of Tomahawk the clays are replaced again by sandy soils containing gravel and by bowlders and glacial drift.[b] In the 60 miles below the city of Tomahawk the tributaries of Wisconsin River flow mainly through a clayey-loam soil, except for a narrow strip adjacent to the main stream, where, as before stated, the sandy soil predominates.

RAINFALL AND RUN-OFF.

The United States Geological Survey has maintained regular gaging stations at Necedah and Merrill since November, 1902. As the rainfall during 1904 was very close to the average rainfall for the past thirty years, the run-off data for this year are especially valuable.

[a] Kilbourn, in the Cambrian sandstone.
[b] Weidman, Samuel, Wis. Geol. Nat. Hist. Survey, Bull. 11, pl. 1.

Rainfall records for this drainage area are given elsewhere in this report. The following tables give the run-off data:

Discharge measurements of Wisconsin River near Necedah, Wis., in 1902, 1903, 1904, and 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
		Feet.	*Square feet.*	*Feet per second.*	*Feet.*	*Second-feet.*
1902.						
December 2	L. R. Stockman				4.90	3,[illegible]
December 23	do				5.40	3,534
1903.						
January 13 [a]	do	280	2,617	1.18	5.65	2,[illegible]
February 5 [a]	do	284	2,360	1.26	5.80	2,[illegible]
March 5 [a]	do	284	2,411	1.09	5.80	2,[illegible]
March 26	Johnson and Stockman		5,405	3.94	11.05	21,290
April 2	L. R. Stockman	220	4,206	2.42	7.55	10,190
April 28	do	309	3,860	1.84	6.50	7,12[illegible]
June 12	do	281	3,282	1.79	6.00	5,[illegible]
July 7	do	316	4,708	4.43	10.50	20,[illegible]
August 19	do	302	2,832	2.46	6.20	6,[illegible]
September 4	do	276	2,463	2.05	5.30	5,047
October 12	do	314	3,871	3.23	9.43	12,500
1904.						
January 12 [a]	E. Johnson, jr	286	2,031	1.33	4.60	3,[illegible]
May 11	do	317	4,685	3.65	9.60	17,110
May 23	Johnson and Hanna	314	3,717	2.67	7.05	9,921
July 16	E. Johnson, jr	294	3,525	1.66	5.80	5,[illegible]
September 21	do	294	1,823	2.08	4.92	3,[illegible]
October 14	F. W. Hanna	449	6,216	5.71	13.35	[b] 34,420
1905.						
April 4	S. K. Clapp		5,777	5.07	12.33	29,29[illegible]
May 25	do	317	4,437	3.23	7.65	13,3[illegible]
June 12	M. L. Brennon	437	6,017	4.99	12.9	30,[illegible]
August 9	do	314	3,846	2.4	6.85	9,[illegible]

NOTE.—Width is the actual width of water surface, not including piers. Area of section is the total area of the measured section, including both moving and still water.

[a] Frozen. [b] Add to this discharge 3,000 second-feet overflow.

Mean daily gage height, in feet, of Wisconsin River near Necedah, Wis., December 2, 1902, to December 31, 1905.

Day.	1902.	1903.											
	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1		5.90	5.75	5.75	7.70	6.65	10.55	4.70	4.80	5.20	7.10	5.70	5.[illegible]
2	4.90	5.90	5.70	5.60	7.55	8.30	9.85	4.60	4.95	5.50	6.60	5.55	6.20
3	4.95	5.80	5.90	5.85	7.35	9.35	8.85	7.70	4.75	5.40	6.80	5.70	6.90
4	5.10	5.75	5.80	5.80	7.50	9.75	8.15	8.90	4.80	5.30	6.80	5.75	7.10
5	4.85	5.60	5.75	5.75	7.40	9.95	7.60	10.10	4.85	5.30	7.05	5.55	6.[illegible]
6	4.75	5.70	5.90	5.80	7.25	10.15	7.40	10.00	5.65	5.40	8.30	5.45	6.[illegible]
7	4.70	5.65	5.80	5.90	7.15	10.05	7.15	10.60	6.65	5.60	9.05	5.30	6.[illegible]
8	4.30	5.45	5.70	5.50	7.20	9.70	6.85	10.60	7.75	6.10	8.95	5.50	6.70
9	4.85	5.60	5.60	5.50	7.10	9.30	6.65	9.70	8.00	6.10	9.15	5.35	6.50
10	5.25	5.50	5.80	6.25	7.25	8.80	6.55	8.40	7.70	6.80	9.80	5.30	6.40
11	5.20	5.45	5.75	6.40	7.05	8.25	6.20	7.80	7.50	7.30	9.80	5.25	6.30
12	5.40	5.50	5.65	7.05	6.90	8.15	6.00	7.50	7.20	7.30	9.35	5.30	6.00
13	5.25	5.65	5.90	7.65	6.80	8.45	6.15	7.10	6.90	7.20	8.90	5.30	[a] 4.10
14	5.30	5.75	5.80	6.75	6.75	9.05	5.85	6.70	6.70	8.60	8.30	5.30	4.40

[a] River frozen December 13 to 31.

Mean daily gage height, in feet, of Wisconsin River near Necedah, Wis., December 2, 1902, to December 31, 1905—Continued.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
15	5.35	5.45	5.75	7.30	6.80	9.80	5.70	6.55	6.70	10.90	7.90	5.35	4.50
16	5.65	5.80	5.65	7.75	6.95	10.10	5.45	6.25	6.40	12.50	7.65	5.25	4.40
17	5.65	5.65	5.65	8.35	7.10	9.90	5.35	6.00	6.20	13.40	7.55	5.35	4.00
18	5.30	5.55	5.55	8.70	7.25	9.35	5.60	6.10	6.40	14.60	7.25	4.90	4.40
19	5.50	5.45	5.75	8.85	7.10	8.70	5.45	5.90	6.10	14.60	6.95	4.90	4.30
20	5.45	5.75	5.70	10.00	6.90	8.30	5.25	6.00	5.70	14.60	7.00	5.00	5.00
21	5.30	5.65	5.70	11.40	6.50	7.95	5.15	5.90	5.90	13.80	6.95	5.10	4.80
22	5.30	5.55	5.65	12.70	6.55	7.90	4.90	5.60	5.40	12.70	6.55	5.05	4.90
23	5.40	5.85	5.55	13.55	6.30	7.75	5.20	5.40	5.10	11.40	6.40	4.95	4.90
24	5.60	5.80	5.70	12.85	6.20	7.45	4.95	5.20	5.10	10.60	6.40	5.20	4.80
25	6.40	5.80	5.65	11.80	6.05	7.35	4.70	5.30	5.40	9.90	6.30	5.20	4.70
26	6.30	5.65	5.65	10.90	6.10	7.60	4.80	5.30	5.20	8.70	6.10	5.05	4.70
27	6.60	5.85	5.70	10.05	6.35	8.00	4.75	5.00	5.30	8.15	6.05	5.00	4.90
28	6.15	5.80	5.85	9.35	6.50	8.70	4.80	5.10	5.30	7.95	5.95	5.15	4.80
29	6.05	5.70	......	8.95	6.85	9.55	4.70	5.00	5.20	7.65	6.00	5.00	4.90
30	6.20	5.80	......	8.50	6.60	10.55	4.85	4.90	5.00	7.55	5.80	5.40	4.90
31	6.00	5.80	......	8.00	......	11.00	......	4.80	5.00	......	5.70	......	4.90

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
1	[a] 6.00	5.10	5.30	[b] 6.60	9.80	11.20	6.30	4.50	4.80	6.60	7.10	4.80
2	5.70	5.20	5.30	6.30	9.20	10.00	6.50	4.70	4.80	6.45	6.90	4.80
3	5.90	5.10	5.30	6.50	8.90	9.10	6.50	4.90	4.80	6.20	6.60	4.80
4	5.60	5.00	5.30	6.90	8.40	8.60	6.30	4.80	4.80	6.10	6.70	(c)
5	6.00	5.20	5.30	6.60	8.00	8.40	6.20	4.70	4.30	7.00	6.50	......
6	6.00	5.20	5.30	6.80	7.80	9.00	6.30	4.70	5.80	6.00	6.40	......
7	5.90	5.20	5.30	7.00	7.40	9.90	6.00	4.30	6.30	5.90	6.10	......
8	6.10	5.00	5.40	7.20	......	10.50	5.90	4.40	5.90	6.30	6.10	......
9	6.10	5.20	5.40	7.50	7.40	10.50	6.10	4.80	5.70	6.42	6.60	......
10	6.00	4.90	5.30	7.90	7.90	9.80	6.30	4.90	5.70	6.70	6.10	5.10
11	5.90	5.10	5.30	8.80	9.50	9.00	6.40	5.30	5.30	8.40	5.90	......
12	5.10	5.20	5.30	9.80	10.50	8.30	6.80	5.30	5.70	10.10	5.20	......
13	5.10	5.10	5.30	9.80	10.50	7.80	7.10	5.40	5.30	12.00	5.50	......
14	5.20	5.20	5.20	9.40	9.90	7.50	7.00	5.30	5.50	13.20	5.50	......
15	5.20	5.10	5.30	8.70	9.40	7.20	6.50	5.30	5.60	13.00	5.60	......
16	5.30	5.20	5.20	8.30	9.20	6.80	5.90	5.70	4.90	11.90	5.80	......
17	5.20	5.20	5.20	7.70	9.00	6.90	5.80	5.00	5.20	10.30	5.50	5.50
18	5.10	5.10	5.20	7.30	8.50	6.70	5.50	5.10	5.30	9.40	5.30	......
19	5.30	5.10	5.10	7.50	8.00	6.50	5.80	5.00	5.90	9.00	4.80	......
20	5.00	5.00	5.00	7.50	7.70	6.20	5.50	5.00	5.70	8.40	5.00	......
21	5.20	5.10	4.90	7.70	7.40	5.90	5.60	5.00	4.80	7.90	5.00	......
22	5.20	5.10	5.00	7.70	7.20	5.80	5.30	4.70	4.90	8.00	5.30	......
23	5.20	5.10	5.00	7.50	7.10	6.20	5.00	5.10	4.80	8.50	5.40	......
24	5.10	5.10	4.80	7.60	7.00	5.70	4.80	4.80	4.70	8.50	5.30	6.00
25	5.00	5.20	5.00	8.00	7.50	6.00	4.50	4.90	4.85	8.30	4.90	......
26	5.10	5.30	5.00	9.30	8.10	5.70	4.80	4.90	4.80	8.30	5.50	......
27	5.00	5.40	5.20	10.30	9.40	6.10	4.90	5.00	6.70	7.90	5.10	......
28	5.10	5.20	5.20	10.90	10.60	5.80	4.80	5.90	7.40	7.60	4.80	......
29	5.20	5.10	5.20	10.70	11.90	6.10	4.70	4.60	7.40	7.50	5.00	......
30	5.20	......	5.50	10.50	12.60	6.00	4.80	5.00	7.00	7.40	5.00	......
31	5.10	......	5.80	......	12.30	......	4.70	4.70	......	7.00	......	6.00

[a] River frozen January 1 to March 31. Ice, average thickness, 10 inches.
[b] Ice conditions April 1 to 12.
[c] River frozen December 4 to 31. Ice 1 foot to 2 feet thick.

Mean daily gage height, in feet, of Wisconsin River near Necedah, Wis., December 2, 1902, to December 31, 1905—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1905.												
1	(a)	(a)	(a)	13.30	5.95	6.50	7.30	5.50	5.90	6.00	5.80	[b] 5.40
2				13.30	6.10	6.40	7.30	5.30	6.00	5.70	5.50	[illegible]
3				12.80	6.10	6.40	7.50	5.20	6.20	5.60	5.30	5.10
4		6.00	5.70	12.40	6.00	6.30	7.50	5.20	6.00	5.50	5.40	5.30
5				11.90	6.00	7.70	7.50	5.60	6.10	5.40	5.50	5.60
6				11.60	6.50	8.30	8.10	5.30	6.10	5.40	5.40	5.80
7	6.00			11.80	6.60	11.00	8.60	5.40	6.55	5.40	5.40	6.80
8				11.90	6.70	12.50	9.10	6.60	6.20	5.40	5.40	[illegible]
9				11.40	6.90	15.00	8.70	6.90	5.70	5.20	5.50	[illegible]
10				10.60	7.00	17.00	8.30	7.10	5.60	5.30	5.50	[illegible]
11			6.15	9.90	7.00	16.00	7.60	6.70	5.50	5.00	5.60	[illegible]
12				9.30	7.50	13.00	7.40	6.50	5.30	4.90	5.50	8.40
13		6.00		9.00	8.30	11.90	7.00	6.70	5.40	4.70	5.50	7.70
14	6.00			8.60	8.50	11.50	6.60	6.40	5.50	4.70	5.50	7.70
15				8.40	8.30	11.20	6.70	6.20	5.50	5.10	5.30	7.40
16				8.00	8.60	10.40	6.50	5.90	5.30	5.10	5.30	7.60
17				7.80	9.30	9.70	6.30	6.00	5.40	5.30	5.10	[illegible]
18				7.50	9.80	9.50	6.50	5.90	5.50	5.30	5.20	7.30
19				7.10	9.80	9.60	6.30	5.80	6.90	5.60	5.20	7.30
20		6.00		6.70	9.70	11.20	6.30	5.60	7.40	5.60	5.30	7.30
21	6.10		5.00	6.60	9.30	12.40	6.30	5.50	8.20	5.80	5.20	7.20
22			5.00	6.60	8.80	12.30	6.10	5.70	8.40	6.30	4.90	7.10
23			5.00	6.50	8.30	11.00	5.90	5.70	8.40	6.70	4.90	7.10
24			5.30	6.40	8.00	9.80	5.70	5.50	7.80	7.00	4.90	6.90
25		6.00	5.60	6.30	7.70	8.80	5.75	5.70	7.20	6.80	4.80	[illegible]
26			6.80	6.00	7.20	8.30	6.00	5.90	6.80	6.70	5.10	[illegible]
27			7.10	6.15	7.10	8.00	5.50	5.30	6.50	6.60	5.10	7.10
28	6.00		8.30	6.00	7.00	7.80	5.10	5.00	6.00	6.40	5.50	6.50
29			9.30	5.95	6.70	7.40	5.30	5.80	5.90	6.20	5.30	6.40
30			10.70	5.90	6.80	7.00	5.10	5.70	6.00	6.20	5.40	6.30
31					6.60		5.30	5.70		6.00		6.10

[a] River frozen over January 1 to March 20. Gage heights are to water surface in a hole in the ice. Thickness of ice, 2 to 2.5 feet.
[b] No ice record for December.

Rating table for Wisconsin River near Necedah, Wis., from March 10 to July 5, 1903.[a]

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
4.6	3,400	5.9	5,680	7.2	9,160	8.8	14,160
4.7	3,540	6.0	5,900	7.3	9,460	9.0	14,800
4.8	3,690	6.1	6,130	7.4	9,760	9.2	15,440
4.9	3,840	6.2	6,370	7.5	10,060	9.4	16,080
5.0	4,000	6.3	6,620	7.6	10,360	9.6	16,720
5.1	4,160	6.4	6,880	7.7	10,670	9.8	17,360
5.2	4,320	6.5	7,150	7.8	10,980	10.0	18,000
5.3	4,490	6.6	7,430	7.9	11,290	10.5	19,600
5.4	4,670	6.7	7,710	8.0	11,600	11.0	21,200
5.5	4,860	6.8	8,000	8.2	12,240	11.5	22,920
5.6	5,060	6.9	8,290	8.4	12,880	12.0	24,670
5.7	5,260	7.0	8,580	8.6	13,520	13.0	28,360
5.8	5,470	7.1	8,870				

a Flood in July changed channel.

Rating table for Wisconsin River near Necedah, Wis., from July 6 to December 12, 1903.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet*	*Second-feet.*
4.8	4,200	6.1	6,730	7.4	10,440	9.2	16,480
4.9	4,350	6.2	6,970	7.5	10,760	9.4	17,160
5.0	4,510	6.3	7,220	7.6	11,080	9.6	17,840
5.1	4,680	6.4	7,480	7.7	11,410	9.8	18,520
5.2	4,860	6.5	7,750	7.8	11,740	10.0	19,200
5.3	5,040	6.6	8,030	7.9	12,070	10.5	20,900
5.4	5,230	6.7	8,320	8.0	12,400	11.0	22,600
5.5	5,430	6.8	8,620	8.2	13,080	11.5	24,300
5.6	5,630	6.9	8,920	8.4	13,760	12.0	26,000
5.7	5,840	7.0	9,220	8.6	14,440	12.5	27,700
5.8	6,050	7.1	9,520	8.8	15,120	13.0	29,400
5.9	6,270	7.2	9,820	9.0	15,800	14.0	32,800
6.0	6,500	7.3	10,130				

Rating table for Wisconsin River near Necedah, Wis., from January 1 to December 31, 1904.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
4.0	1,800	5.4	4,880	6.7	8,500	9.0	15,400
4.1	2,000	5.5	5,130	6.8	8,800	9.2	16,000
4.2	2,200	5.6	5,380	6.9	9,100	9.4	16,600
4.3	2,400	5.7	5,640	7.0	9,400	9.6	17,200
4.4	2,600	5.8	5,900	7.2	10,000	9.8	17,800
4.5	2,810	5.9	6,170	7.4	10,600	10.0	18,400
4.6	3,020	6.0	6,440	7.6	11,200	10.5	19,900
4.7	3,240	6.1	6,720	7.8	11,800	11.0	21,400
4.8	3,460	6.2	7,010	8.0	12,400	11.5	23,610
4.9	3,690	6.3	7,300	8.2	13,000	12.0	25,860
5.0	3,930	6.4	7,600	8.4	13,600	12.5	28,230
5.1	4,150	6.5	7,900	8.6	14,200	13.0	30,750
5.2	4,390	6.6	8,200	8.8	14,800	13.5	38,450
5.3	4,630						

Rating table for Wisconsin River near Necedah, Wis., from January 1 to December 31, 1905.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
4.00	1,800	5.50	5,130	7.00	9,400	9.80	17,800
4.10	2,000	5.60	5,380	7.20	10,000	10.00	18,400
4.20	2,200	5.70	5,640	7.40	10,600	10.50	19,900
4.30	2,400	5.80	5,900	7.60	11,200	11.00	21,400
4.40	2,600	5.90	6,170	7.80	11,800	11.50	23,610
4.50	2,810	6.00	6,440	8.00	12,400	12.00	25,860
4.60	3,020	6.10	6,720	8.20	13,000	12.50	28,230
4.70	3,240	6.20	7,010	8.40	13,600	13.00	30,750
4.80	3,460	6.30	7,300	8.60	14,200	13.50	38,450
4.90	3,690	6.40	7,600	8.80	14,800	14.00	46,200
5.00	3,920	6.50	7,900	9.00	15,400	15.00	61,800
5.10	4,150	6.60	8,200	9.20	16,000	16.00	77,500
5.20	4,390	6.70	8,500	9.40	16,600	17.00	93,300
5.30	4,630	6.80	8,800	9.60	17,200	18.00	109,200
5.40	4,880	6.90	9,100				

The last table is applicable only for open-channel conditions. It is based on 23 discharge measurements made during 1902-1905. It is well defined between gage heights 4.5 feet and 10.5 feet. The table has been extended beyond these limits. From gage height 6.3 feet to 11 feet the rating curve is a tangent, the difference being 300 per tenth. Above 11 feet the bank overflows, which causes the discharge to increase at a greater rate per foot.

Estimated monthly discharge of Wisconsin River near Necedah, Wis., 1903 to 1905.

[Drainage area, 5,800 square miles.]

Date.	Discharge.			Run-off.		Rainfall [a]
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1903.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
January			[b] 2,600	0.45	0.52	0.36
February			[b] 2,550	.44	.46	.91
March [c]	30,450		11,859	2.04	2.35	2.[illegible]
April	10,670	6,015	8,322	1.43	1.60	4.[illegible]
May	21,200	7,570	14,492	2.50	2.88	6.[illegible]
June	19,760	3,540	6,897	1.19	1.33	1.[illegible]
July	21,240	3,400	9,022	1.56	1.80	6.11
August	12,400	4,125	6,648	1.15	1.33	6.[illegible]
September	34,840	4,860	15,832	2.73	3.05	5.[illegible]
October	18,520	5,840	10,586	1.83	2.11	2.[illegible]
November	5,945	4,350	5,007	.86	.96	1.0[illegible]
December 1-12 [d]	9,520	[e] 5,630	[e] 7,798	[e] 1.34	[e] .60	.88
1904.						
January						.33
February						1.[illegible]
March						1.[illegible]
April	21,100	7,300	12,830	2.21	2.47	2.0[illegible]
May	28,720	9,400	15,250	2.63	3.03	6.[illegible]
June	22,280	5,640	11,350	1.96	2.19	4.8[illegible]
July	9,700	2,810	5,926	1.02	1.18	3.28
August	6,170	2,400	3,845	.663	.764	3.21

[a] Rainfall for 1903 is the average of the recorded precipitation at the following stations: Antigo, Koepenick, Stevens Point, Wausau, Amherst, Grand Rapids, and Medford. That for 1904 includes the same stations, except Medford and adding Minocqua and Prentice.
[b] Estimated.
[c] March 1 to 9, inclusive, estimated.
[d] River frozen December 13 to 31.
[e] Twelve-day period.

Estimated monthly discharge of Wisconsin River near Necedah, Wis., for 1903 to 1905—Continued.

Date.	Discharge.			Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1904.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
September	10,600	2,400	5,227	.901	1.01	4.53
October	33,830	6,170	13,590	2.34	2.70	5.70
November	9,700	3,460	5,698	.982	1.10	.25
December						1.86
1905.[a]						
March 21-30		20,500	3,920	9,037	1.56	0.58
April		35,370	6,170	15,790	2.72	3.04
May		17,800	6,305	11,060	1.91	2.20
June		93,300	7,300	23,320	4.02	4.48
July		15,700	4,150	8,711	1.50	1.73
August		9,700	3,920	6,099	1.05	1.21
September		13,600	4,630	7,419	1.28	1.43
October		9,400	3,240	5,748	.991	1.14
November		5,900	3,460	4,667	.805	.898
December		14,800	4,150	8,888	1.53	1.76
The year						34.87

[a] No estimate for ice period.

Discharge measurements of Wisconsin River at Merrill, Wis., in 1902, 1903, 1904, and 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
1902.		*Feet.*	*Square feet.*	*Feet per second.*	*Feet.*	*Second feet.*
November 17	L. R. Stockman				7.6	9,015
December 10	do				3.8	1,394
1903.						
January 20[a]	do	310	718	1.91	4.05	1,376
February 16[a]	do	310	669	1.86	3.70	1,250
March 20	do	344	2,639	3.78	8.90	9,995
May 7[b]	do	332	2,232	3.54	6.85	7,893
June 17[b]	do	308	1,269	1.78	4.72	2,258
July 13	do	305	1,424	2.10	5.70	2,993
August 22[b]	do	283	1,115	2.36	5.00	2,638
September 11	E. C. Murphy	343	1,759	3.19	6.66	5,614
October 24	L. R. Stockman	334	1,594	2.61	6.08	4,159
1904.						
May 12[b]	E. Johnson, jr	334	2,220	3.71	7.85	8,242
June 5	do	334	2,286	4.19	8.25	9,587
July 15[b]	do	334	1,366	1.98	5.30	3,107
September 21	do	312	1,210	1.91	5.01	2,312
October 14	F. W. Hanna	327	2,333	4.42	8.25	10,323
November 30[a]	E. Johnson, jr	306	1,237	1.85	4.97	2,294
1905.						
April 10	S. K. Clapp	334	2,189	3.84	7.8	8,396
May 26	do	324	1,679	2.69	6.25	4,519
June 10	M. S. Brennan	334	2,334	4.06	8.17	9,478
July 10	do	332	1,596	2.73	6.48	4,357

[a] Partly frozen. [b] Affected by log jam.

Mean daily gage height, in feet, of Wisconsin River at Merrill, Wis., November 16, 1902, to December 31, 1905.

Day.	1902.		1903.											
	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1			3.65	3.80	3.70	6.75	6.65		7.05	4.30		6.50	5.85	[illegible]
2			3.70	3.80	3.70	6.70	8.30		7.65	4.30	5.50	5.90	5.60	[illegible]
3			3.85	3.85	3.80	6.55			7.65	4.90	5.70	6.90	5.30	5.25
4			3.80	3.80	3.75	6.65			8.70	5.60	5.80	7.85	5.30	[illegible]
5			3.75	3.85	3.80	6.75			8.80	6.55	6.00	8.00	5.30	5.25
6			3.70	3.85	3.70	6.70			8.70	7.45	6.10	7.80	5.75	5.15
7			3.75	3.90	3.80	6.80			8.30	7.35	6.10	8.85	5.30	[illegible]
8			3.70	3.80	3.75	6.90			8.10	7.25	6.50	8.55	5.10	[illegible]
9			3.65	3.90	3.75	6.75			7.70	(a)	6.60	8.35	5.20	5.15
10		4.00	3.65	3.85	3.90	6.70			7.60		6.80	8.20	5.10	5.05
11		3.95	3.70	3.55	4.05	6.75			7.50		6.90	7.70	5.30	4.95
12		4.00	3.70	3.85	4.20	6.55			7.30		9.10	7.25	5.25	4.70
13		4.00	3.80	3.75	4.75	6.05			6.30		9.40	7.35	5.25	4.90
14		3.85	3.75	3.70	5.00	6.70			5.50		10.00	7.10	5.25	[illegible]
15		3.65	3.65	3.75	5.05	6.75			6.00		11.10	7.10	5.05	5.30
16	3.60	3.85	3.50	3.80	5.05	6.80			5.40		11.50	6.75	5.50	[illegible]
17	3.80	3.85	3.70	3.90	5.50	6.85			5.50		10.80	6.60	5.35	5.00
18	2.50	3.90	3.70	3.75	5.55	6.75			5.40		10.10	6.60	4.65	5.10
19	2.05	3.85	4.20	3.55	7.90	7.10		5.15	5.15		9.40	6.60	4.75	5.10
20	1.90	4.05	4.00	3.70	8.35	7.20		5.20	5.55		8.90	6.35	4.85	5.20
21	1.90	3.80	3.85	3.65	8.30	7.10		5.55	4.65		8.50	6.35	4.75	5.30
22	1.55	3.80	4.00	3.85	8.00	6.80		5.60	4.90		8.10	6.35	4.55	4.80
23	1.05	3.75	4.00	3.45	8.25	6.75		6.00	5.40		7.90	6.40	4.60	4.90
24	.90	3.85	4.00	3.65	7.50	6.80		6.45	5.10		7.70	6.15	4.70	[illegible]
25	1.05	4.05	4.10	3.70	7.35	7.05		6.40	4.65		7.10	6.05	4.90	5.00
26	.55	3.95	4.05	3.70	7.00	6.10		5.85	5.10		7.20	6.00	5.35	[illegible]
27	.15	3.90	4.00	3.60	6.65	6.35		5.35	4.10		7.20	5.85	5.25	[illegible]
28	.05	3.80	3.85	3.65	6.05	6.50		5.35	4.50		7.05	5.75	4.85	[illegible]
29	.10	3.70	3.95		6.80	6.85		6.30	5.60		5.40	5.85	4.85	[illegible]
30		3.70	3.90		6.45	6.60		6.75	4.50		6.05	5.95	5.05	[illegible]
31		3.70	3.85		6.70				4.30			5.60		[illegible]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
1	5.90	5.65	5.90	5.90	7.55	7.55	6.75	5.15	5.05	6.15	6.70	4.85
2	6.00	5.65	5.95	5.85	7.10	7.25	6.80	5.20	4.90	6.25	6.60	5.20
3	6.05	5.60	5.90	5.90	6.80	7.30	6.60	5.10	5.95	6.70	6.25	4.85
4	6.10	5.55	5.90	5.85	7.05	7.70	6.20	5.00	7.80	5.85	5.90	4.85
5	6.10	5.70	5.90	5.90	7.30	8.05	6.25	5.05	6.90	6.40	5.75	4.75
6	6.00	5.70	5.85	6.40	6.75	8.30	6.20	5.25	6.25	5.90	5.70	4.95
7	5.75	5.60	5.95	6.35	6.75	7.80	6.20	5.20	7.05	5.60	5.55	[illegible]
8	5.80	5.80	5.85	6.65	7.05	7.85	6.35	6.65	7.00	6.70	5.15	[illegible]
9	5.50	5.80	5.90	7.20	8.40	7.55	6.70	5.10	6.75	7.75	4.70	[illegible]
10	5.85	5.75	5.90	7.15	8.20	7.35	6.95	5.20	5.90	10.10	4.55	[illegible]
11	5.55	5.85	5.90	7.15	7.90	7.00	7.20	5.30	6.15	10.40	4.40	4.65
12	5.70	5.75	5.90	6.75	7.95	7.55	6.55	6.20	6.60	10.15	4.75	4.45
13	5.75	6.10	5.80	6.80	7.70	7.25	5.45	7.15	6.15	9.05	5.90	4.80
14	5.70	5.55	5.80	6.65	7.95	6.50	6.00	5.50	5.95	8.30	6.00	[illegible]
15	5.55	5.60	5.90	6.35	7.70	6.20	5.75	5.30	5.95	7.55	4.60	5.20

a Chain gage stolen.

Mean daily gage height, in feet, of Wisconsin River at Merrill, Wis., November 16, 1902, to December 31, 1905—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
16	5.60	5.65	5.85	5.85	7.90	6.10	5.60	5.70	5.90	7.15	4.50	5.30
17	5.60	5.95	5.90	6.45	7.40	6.05	5.80	6.65	5.90	6.90	4.55	5.20
18	5.55	5.90	5.75	6.30	6.55	6.10	6.05	5.90	5.95	6.90	4.55	5.20
19	5.65	5.90	5.65	6.35	6.75	6.50	4.50	5.75	5.90	6.95	4.75	5.10
20	5.60	6.15	5.70	6.15	6.75	6.20	4.60	6.40	5.20	6.25	5.05	5.05
21	5.70	5.85	5.75	6.05	6.85	6.05	5.10	6.10	5.35	6.35	5.10	5.20
22	5.65	5.95	5.80	6.05	7.05	6.10	5.20	6.20	5.05	6.80	5.25	5.70
23	5.60	5.90	5.35	6.10	7.05	6.05	5.30	6.15	5.50	6.65	5.10	5.25
24	5.50	5.90	5.75	7.00	7.05	7.25	4.90	5.85	6.80	6.55	4.95	5.20
25	5.85	5.90	6.00	8.10	8.10	5.60	4.50	5.80	7.10	7.10	4.75	5.45
26	5.55	5.90	5.85	8.35	10.10	6.00	4.60	5.70	7.75	7.20	4.95	5.05
27	5.55	5.90	5.55	8.45	10.60	6.25	5.40	6.00	7.15	6.90	5.45	5.60
28	5.55	5.90	5.70	8.50	9.80	7.25	5.10	5.50	7.20	6.75	5.15	5.95
29	5.55	5.95	5.80	8.20	9.05	6.80	5.50	5.90	6.40	6.85	5.15	5.60
30	5.55	......	5.70	7.75	8.60	6.30	5.70	6.35	6.10	6.65	4.85	5.50
31	5.50	......	5.80	......	7.95	......	5.50	5.05	......	6.25	......	5.15
1905.												
1	5.20	5.25	5.35	8.90	5.15	5.80	7.40	5.05	6.10	6.20	6.20	5.20
2	4.95	5.00	5.35	8.80	5.70	5.70	7.30	5.45	6.20	6.00	6.30	5.70
3	5.20	4.90	5.20	9.20	6.20	5.90	7.50	5.35	6.15	6.20	5.60	5.60
4	5.70	5.15	5.40	8.90	6.10	5.50	7.60	5.35	6.70	6.00	5.25	5.80
5	5.35	5.50	5.25	8.80	6.25	7.60	8.00	5.95	6.70	6.05	5.80	5.40
6	5.40	5.35	5.35	8.80	6.40	10.40	8.20	6.10	6.25	6.00	5.35	5.25
7	5.20	5.15	5.55	8.80	6.45	10.00	7.70	6.85	6.00	5.30	5.10	5.55
8	5.45	5.25	5.15	8.20	7.20	9.00	7.80	6.00	6.25	5.30	5.55	5.55
9	5.15	5.10	5.20	8.60	6.70	9.00	8.00	6.25	6.10	4.30	5.45	5.35
10	5.05	5.40	5.65	7.80	6.45	8.40	7.20	6.40	6.30	4.90	5.90	5.75
11	5.35	5.20	5.45	7.40	6.95	8.50	6.85	6.05	6.15	6.15	5.80	5.70
12	5.50	4.95	5.05	7.20	7.20	8.40	7.05	5.75	6.30	6.80	5.15	5.55
13	5.50	5.20	5.45	7.40	6.90	7.80	6.20	6.00	6.65	6.45	5.30	5.60
14	5.90	5.25	4.70	7.00	7.40	7.80	6.30	6.15	6.40	6.55	5.00	5.60
15	6.00	5.15	4.70	6.90	7.60	7.60	6.80	5.90	6.05	5.90	5.20	5.35
16	5.95	5.50	4.95	7.20	7.60	8.10	6.50	6.05	6.40	6.15	5.65	5.70
17	5.75	5.70	5.05	7.40	7.80	10.40	6.30	5.90	6.60	5.85	5.70	6.15
18	5.85	5.65	5.25	7.00	7.80	10.60	6.55	6.40	6.45	6.25	5.75	5.50
19	5.65	5.55	5.25	6.45	7.50	10.60	5.05	6.25	6.90	6.65	5.85	5.60
20	5.15	5.35	5.25	6.45	7.30	9.60	5.65	6.20	7.35	6.85	5.45	5.65
21	5.15	4.30	5.50	6.45	7.00	9.20	6.45	5.90	6.70	6.70	5.75	5.75
22	5.60	5.20	4.95	6.05	6.75	8.60	6.00	6.80	7.30	6.70	5.75	5.75
23	5.80	5.75	4.95	5.95	6.80	8.50	5.50	6.50	6.90	6.80	5.75	5.45
24	6.05	5.80	5.35	5.60	6.40	8.00	5.40	4.80	6.80	6.15	5.60	5.55
25	6.15	5.60	4.55	5.75	6.45	7.60	5.20	5.55	6.50	6.30	5.70	5.90
26	5.85	5.15	5.75	6.20	6.30	6.90	5.10	6.35	6.25	6.70	5.20	6.25
27	5.85	5.65	6.05	5.90	6.35	7.60	5.65	6.40	6.55	6.55	4.80	5.90
28	5.25	5.35	7.40	5.65	6.35	7.50	5.80	6.25	6.15	6.90	5.15	5.60
29	5.10	......	8.00	5.95	6.25	6.75	5.60	5.75	5.95	6.50	5.65	5.85
30	5.05	......	8.60	5.45	6.25	7.05	5.75	6.00	6.45	5.70	5.50	5.70
31	5.15	......	8.50	......	6.00	......	4.30	6.80	......	5.80	......	5.40

NOTE.—No ice record at this station.

Rating table for Wisconsin River at highway bridge near Merrill, Wis., from June 17, 1903, to December 31, 1904.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-ft.*	*Feet.*	*Second-ft.*	*Feet.*	*Second-ft.*	*Feet.*	*Second-ft.*
4.5	1,485	5.5	3,225	6.5	5,485	8.0	9,565
4.6	1,645	5.6	3,425	6.6	5,725	8.2	10,225
4.7	1,805	5.7	3,635	6.7	5,975	8.4	10,885
4.8	1,970	5.8	3,855	6.8	6,225	8.6	11,545
4.9	2,140	5.9	4,075	6.9	6,475	8.8	12,205
5.0	2,310	6.0	4,305	7.0	6,725	9.0	12,865
5.1	2,485	6.1	4,535	7.2	7,245	9.5	14,515
5.2	2,665	6.2	4,765	7.4	7,785	10.0	16,165
5.3	2,845	6.3	5,005	7.6	8,345	10.5	17,815
5.4	3,035	6.4	5,245	7.8	8,935	11.0	19,465

Estimated monthly discharge of Wisconsin River at Merrill, Wis., for 1904.

[Drainage area, 2,630 square miles.]

Date.	Discharge.			Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1904.	*Sec. ft.*	*Sec. ft.*	*Sec. ft*	*Sec. ft.*	*Inches.*	*Inches.*
January	4,535	3,225	3,664	1.39	1.60	0.33
February	4,655	3,330	3,749	1.43	1.54	1.30
March	4,305	2,945	3,889	1.48	1.71	1.49
April	11,220	3,970	6,242	2.37	2.64	2.01
May	18,140	5,610	8,935	3.40	3.92	6.[illegible]
June	10,560	3,425	6,472	2.46	2.74	4.81
July	7,245	1,485	3,957	1.51	1.74	3.[illegible]
August	7,110	2,310	3,766	1.43	1.65	3.[illegible]
September	8,935	2,140	5,000	1.90	2.12	4.[illegible]
October	17,480	3,425	7,343	2.79	3.22	3.70
November	5,975	1,410	2,800	1.06	1.18	[illegible]
December	4,195	1,490	2,566	.976	1.12	1.[illegible]
The year	18,140	1,410	4,865	1.85	25.18	34.[illegible]

RAILROADS.

The railway facilities will be discussed in connection with each power, but in general it may be said that they are excellent. The willingness of the railroads to go where there is an assured traffic is seen at Nekoosa. Since the construction of the paper and pulp mill at this point three different railroads have extended their lines to the mill.

The land is being rapidly cleared and made into farms, especially during the past five years. This fact insures the certain and steady extension of the railroads in this region.

WATER POWERS.

In the first 138 miles above its mouth Wisconsin River occupies a wide, sandy valley, entirely devoid of any falls or rapids, and showing a very uniform descent of only 1¼ feet per mile.

KILBOURN.

The first water power is found at Kilbourn, where the river flows across the Cambrian sandstone in a narrow, deep gorge known as "The Dell," 100 to 600 feet wide and 40 to 70 feet deep. The drainage area of Wisconsin River at Kilbourn is about 8,200 square miles. According to discharge measurements made by United States engineers, the low-water discharge is 3,000 second-feet. A dam with a crest 3 feet above low water was for many years operated here, under an old charter, the power being used for a flouring mill. This was burned down over thirty years ago and since that time no use has been made of the power.

The Madison Traction Company, of Madison, Wis., has a charter for a 15-foot dam above the ordinary low-water level, with the privilege of 2 feet of flashboards, giving a head of 17 feet or more. It has been proposed to build an electric railroad from Madison northward via Kilbourn, as an extension of the present Madison street railway system, and the plans contemplate using this water power to drive the dynamos.

The river continues to flow in the Cambrian sandstone for the next 70 miles, until Nekoosa is reached. Although the river descends over 105 feet in this distance the fall is so evenly distributed that good water-power sites are lacking. At Nekoosa, however, for the first time, we find the river flowing in the hard pre-Cambrian crystalline rock. In the next 8½ miles above, the river has a descent of 83 feet, nearly all of which is improved by 5 dams. These dams furnish power for 5 large modern paper and pulp mills and will be described in order, beginning below.

NEKOOSA.

A rock crib dam at Nekoosa develops a head of nearly 20 feet. This power is used to operate a modern paper and sulphite mill, one of the largest on the river, owned by the Nekoosa Paper Company. An installation of 37 turbines is reported, developing a total of 4,560 actual horsepower for twenty-four hours per day. The drainage area of the river at this point is about 5,700 square miles.

PORT EDWARDS.

About 4½ miles farther upstream is another fully developed power owned by the John Edwards Manufacturing Company. A head of 18 feet is here available. Turbine wheels, to the number of 28, develop 3,860 actual horsepower, which is used to run a large paper and pulp mill. Two miles farther upstream is the Centralia Pulp and Water Power Company's dam, with an average head of 13 feet. Turbines of 1,460 horsepower are here installed, according to the company's report, all used in the manufacture of paper and pulp.

GRAND RAPIDS.

One of the largest and most complete paper and pulp mills in the entire State, owned by the Consolidated Paper and Power Company, is located on the west side of the river, within the city limits of Grand Rapids. This mill was erected in 1902 and its installation of paper-making machinery has all the recent important improvements. Before this mill was constructed there was a total descent of 30.8 feet between the foot of Biron dam, 4 miles above, and the Grand Rapids bridge. Of this amount the new masonry and concrete dam of the Consolidated Paper and Power Company develops a head of about 25 feet. Turbines of 6,500 horsepower are already installed, flume space being also provided for the development of an additional 1,000 horsepower for future expansion. Prior rights to 500 horsepower developed by this dam are owned by the Grand Rapids Milling Company, which uses it in the manufacture of flour.

The Pioneer Wood and Pulp Company has certain rights to about 600 or 800 horsepower "when the stage of the river will permit," which has meant about ten months each year. This power is used by the company for grinding wood pulp. The Grand Rapids foundry also has rights to about 40 horsepower from the same dam. The milling company and the foundry both receive their power from the Consolidated Company in consideration for power previously owned by them and displaced by the present dam.

The above-described four paper mills have the advantage of competition in freight rates incident to being served by each of the following railways: The Chicago and Northwestern; Chicago, Milwaukee and St. Paul; Green Bay and Western; and Wisconsin Central.

About 4 miles above Grand Rapids is located the dam of the Grand Rapids Paper and Pulp Company. A head of from 10 to 12 feet, depending on the stage of the water, is reported with turbines already installed of 3,063 horsepower. This company is served by the Green Bay and Western Railroad. In the 13 miles between the crest of the Biron dam and the foot of the next one above, near Stevens Point, Wisconsin River descends 16 feet. The only rapids in this distance is one of 3½ feet called "Crooked Rift," about 4 miles above the Biron dam. The greater part of this fall properly belongs to the Biron power and is largely developed by the splash boards of that dam.

STEVENS POINT.

Owing to the peculiar topography of the river valley between Nekoosa and Stevens Point, whereby the adjacent tributaries flow for long distances parallel to the main river, and to the decided narrowing of the river valley between these points, the discharge of Wisconsin River at Stevens Point does not differ greatly from that at Nekoosa. The drainage area at Stevens Point is about 5,600 square miles.

In the city of Stevens Point and immediately south of it are found three developed powers and one undeveloped. Of the former, the lower two are owned and operated by the Wisconsin River Paper and Pulp Company. One of its dams is located in the NE ¼ sec. 17, T. 23 N., R. 8 E., just below the mouth of Plover River, and supplies a head of 9 feet. At this point a large island occupies the middle of the river and is made use of in the construction of the dam. The company has installed turbines rated at 1,370 horsepower. One-half mile above this dam, at a point where the river is much narrower, is located the second dam belonging to this company, giving an average head of 16 feet. Here are installed 18 turbines rated at 4,660 horsepower. This power, as well as that derived from the dam below, is used in the manufacture of pulp and paper. Both mills are located on the east or right bank and have good shipping facilities. Above the last-described power and about a mile below the dam next above is an undeveloped power of about 7-foot head belonging to the same company. As the river is wide at this point, a very long dam would be required to develop this power, but its location in the city would make it very valuable.

The third dam is located within the limits of the city of Stevens Point and is owned by the Jackson Milling Company. This dam develops an average head of 7 feet. The owners have installed only 3 turbines, rated at 140 horsepower, which is used to run a flour and feed mill. By building a new dam 1,000 feet below the present one, with a crest of 10 feet, a 12-foot head could easily be obtained without flooding. On account of its location in a growing city of 10,000 people, it would seem that all this power could easily find takers at remunerative rates.

BATTLE ISLAND.

In the 19 miles between the head of the upper dam at Stevens Point and the bridge of the Chicago, Milwaukee and St. Paul Railway near Knowlton, according to railroad levels, the river descends 30 feet. In this distance there is only one opportunity for the development of water power, namely, at Battle Island, in sec. 28, T. 26 N., R. 7 E. From the foot of the rapids at Mosinee to Battle Island, according to a survey, there is a fall of 20 feet. The banks at this point are said to be high, so that a dam could be economically built with a head of 20 feet. The Wisconsin Valley division of the Chicago, Milwaukee and St. Paul Railway is distant less than a mile from this site, and rock and timber are very abundant and near at hand; in fact, the bed and banks of the river are in rock.

MOSINEE.

An easily developed power, one of the best on the river, is found at Mosinee, in sec. 31, T. 27 N., R. 7 E. It is owned by the Joseph Dessert Lumber Company. About forty years ago a flooding dam with a head of 5 or 6 feet was built here, and it has since been rebuilt several times. The dam was located near the head of the rapids, probably because of the ease of construction due to a large island in the river at this point. Later a sawmill was built on the right bank, thereby securing a head of about 12 feet. At the present time this mill is run by steam power, and no use is made of the water power. An effort is now being made to interest capital to develop this power to its maximum amount for a proposed paper and pulp mill. This will require a new dam. Such a dam could be made to develop a head of 20.7 feet by flooding a small marsh above. The high banks and the bed of the river are in the hard crystalline rock.

ROTHCHILDS.

In the 18 miles between the east quarter stake of sec. 35, T. 29 N., R. 7 E., below the mills at Wausau, and the crest of the Mosinee dam, Wisconsin River descends 28 feet.[a] A considerable portion of this fall is concentrated in rapids in sec. 24, T. 28 N., R. 7 E. at a place called Rothchilds. The right bank is steep, but the left bank is much less so. A dam could be built here which would develop a head of nearly 20 feet but it would need to be long. Rib and Eau Claire rivers, with drainage areas of 500 and 423 square miles, respectively, enter Wisconsin River from opposite sides but a short distance above Rothchilds. This place is 7 miles from Wausau and is reached by the Chicago, Milwaukee and St. Paul Railway. During the year 1903 Wausau capitalists made earnest efforts to acquire the necessary flowage rights for the improvement of this power, but the owners of the land were unwilling to sell at the rates offered and the project was dropped.

WAUSAU.

Only a portion of the valuable water power located in the city of Wausau has been developed. A high granite island, nearly a quarter of a mile long, occupies the middle third of the river at this place, the main dam being built from the head of this island to the right bank, a distance of about 350 feet. The guard lock is located on the opposite channel, at the site of the Scott Street Bridge, and is about 300 feet long.

Near the head of the island is located the McEchroy roller mill. Three turbines, installed under an average head of only 7½ feet, develop 296 actual horsepower, which is ample for this mill as at present equipped.

About 1,000 feet below the guard lock are situated the Alexander Stewart Lumber Company's planing and saw mills, working under heads of 9 and 11 feet, respectively. Four turbines, rated at 200 horsepower, are installed. The planing mill runs ten and the sawmill twenty hours a day. The company also has 350 steam horsepower.

About 1,300 feet below the guard lock is located the plant of the Wausau Paper Mills Company, which takes its power from the old dam, but because of the location so far below has the advantage of the additional fall in the river. This gives an average head of 14 feet. The company has installed 12 turbines, rated at about 3,600 actual horsepower, and in addition has 500 steam horsepower. This mill runs twenty-four hours a day.

During the past year the Wausau Electric Company has acquired rights to two-ninths of the total flow of the stream, and has blasted a new tailrace out of the solid rock for a distance of 300 or 400 feet, thereby increasing the head to 22½ feet. This company has as yet installed only one pair of turbines, rated at 700 horsepower, but intends to double

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[a] U. S. Geol. Survey topographic map.

this in two years. The Stewart Lumber Company owns three-ninths and D. L. Plumer four-ninths of the total flow of the river.

Wausau is a city of about 13,000 inhabitants and is the county seat of Marathon County. The Marshfield branch of the Chicago and Northwestern Railway crosses Wisconsin River at this point, and the city is served also by the Chicago, Milwaukee and St. Paul Railway.

BROKAW.

In the 20 miles (by river) between the foot of the lower dam at Merrill and the head of the Wausau dam Wisconsin River descends about 55 feet, 35 feet of this being between Wausau and the mouth of Pine River.[a] The only portion of this fall at present developed is at Brokaw, where a dam with a head of 12 feet furnishes power for a large paper and pulp mill. Twelve turbines, rated at 3,964 horsepower, are installed. Brokaw is about 6 miles above Wausau, in sec. 3, T. 29 N., R. 6 E., and is reached by the Chicago, Milwaukee and St. Paul Railway.

TRAPP RAPIDS.

About 4 miles above Brokaw, near the mouth of Trapp River, are the Trapp rapids.[b] The bed of the river is in the hard crystalline rocks, and, according to the topographic map, both banks are about 30 or 35 feet high and the river 600 feet wide. A head of 18 or 20 feet could probably be developed here by a dam. The nearest city is Merrill, a place of over 9,000 inhabitants, distant only 8 miles by the Chicago, Milwaukee and St. Paul Railway.

MERRILL.

In the city of Merrill are two dams. The lower one, which has recently been repaired and partially rebuilt, is located between lots 1 and 3, sec. 12, T. 31 N., R. 6 E., and gives an average head of 14 feet. This power is owned by the Merrill Electric Light Company, which has installed and uses 600 horsepower. The remainder of the power is leased to the Lindore Paper Company, which in 1904 blasted out a new tailrace about 600 feet below the dam and has here installed 23 turbines under a 14-foot head, rated at 2,230 horsepower.

The second dam within the city limits of Merrill is located in sec. 10, T. 31 N., R. 6 E., about 2¼ miles above the paper mill, and is used for boom purposes only. It has a length of about 475 feet and develops an average head of 8 feet. A similar dam with an 8-foot head is located about 2 miles above, between secs. 8 and 9, T. 31 N., R. 6 E., and is also used only for boom purposes. Both dams are owned by the Wisconsin River Driving Association. As these dams are of little use at present, owing to the decline of the lumber interests, a company is now being formed to greatly improve the two powers by the construction of a new dam, to be located between secs. 9 and 16, T. 31 N., R. 6 E. It is stated that a head of 24 feet can be obtained here to run a new paper mill.

Wisconsin River is joined at Merrill by Prairie River. Between Merrill and Rhinelander Tomahawk and Pelican rivers add their waters from an aggregate drainage area of 3,300 square miles.

BILL CROSS RAPIDS.

The next power in order above Merrill is found at Bill Cross Rapids, in sec. 13, T. 32 N., R. 5 E., not far from the east quarter stake. Between this point and the foot of Grandfather Rapids the river descends 26½ feet. As the banks are reported high at this point it is probable that a head of 20 to 24 feet could be obtained. This dam site is distant 5 miles from the Chicago, Milwaukee and St. Paul Railway.

[a] U. S. Geol. Survey topographic map.
[b] The power and riparian rights at these rapids are owned by G. D. Jones, Neal Brown, and C. Mathie, of Wausau, Wis.

A. GRANDFATHER RAPIDS, WISCONSIN RIVER.

Ninety feet fall in 1½ miles.

B. BRUNETT FALLS, CHIPPEWA RIVER.

GRANDFATHER RAPIDS.

In the 53 miles between the foot of the upper dam at Merrill and the foot of the Rhinelander dam the river has a natural descent of 277 feet, an average of 5.2 feet per mile. In this stretch, besides several other fine powers, are included Grandfather Rapids, the largest water power on the river, developed or undeveloped. These rapids begin in the NE. ¼ sec. 30, T. 33 N., R. 6 E., and extend to the SW. ¼ sec. 31, a distance of 1½ miles, and are the most noted rapids on the river. A view of them is shown in Pl. III, *A*. The descent in this distance is 89½ feet. The high bank and the bed of the river are in the hard pre-Cambrian rock. For nearly thirty years the Wisconsin River Logging Association has maintained three logging dams on these rapids. It is probable that the cheapest method of developing this power would be to construct three dams of 30 feet head each, and that the power could be best employed by paper mills. The site is about midway in the 20 mile stretch from Merrill to Tomahawk.

About 1.5 miles above Grandfather Rapids are some small rapids where a 8.9-foot dam would back the water to the foot of Grandmother Rapids.

GRANDMOTHER RAPIDS.

From the foot of the present Tomahawk dam to the foot of Grandmother Rapids, Wiconsin River descends 41 feet, 6½ feet of which are concentrated at these rapids in a distance of 40 rods. According to a survey, 39 feet can be developed here. One dam site should be near the south line of sec. 10, T. 33 N., R. 6 E., which is distant only 2½ miles from the Chicago, Milwaukee and St. Paul Railway at Irma.

TOMAHAWK DAM.

This dam, which has a head of 13.2 feet, is located in the SW. ¼ sec. 10, T. 34, N., R. 6 E. It has a total pondage of over 4 square miles, the largest on the river, backing up the water in the main river for about 6 miles, as well as in the tributaries. The steadying effect which this dam exerts, together with that of several dams on adjoining lakes, must be very beneficial. This power has been used for several years for running a large paper mill located on the left bank and reached by spur tracks of the Chicago, Milwaukee and St. Paul Railway. The installation is 650 horsepower. During the summer of 1904 another paper mill was erected on the opposite bank, taking its power from the same dam.

PINE CREEK RAPIDS.

From the foot of Whirlpool Rapids to the backwater of the Tomahawk dam, a distance of 10 miles, the river has a nearly even descent of 23½ feet, 20 feet of which could be developed by one or possibly two dams. A 20-foot dam located about a mile east of the city of Tomahawk, in the SE. ¼ sec. 25, T. 35 N., R. 6 E., would back the water up into Pine River. Most of the land thus to be overflowed belongs to lumbering companies or to the Bradley Company, of Tomahawk, Wis. This dam site is less than half a mile from the Marinette, Tomahawk and Western Railway.

WHIRLPOOL RAPIDS.

These rapids extend from the west line of sec. 12, T. 35 N., R. 7 E., to the north line of Lincoln County, a distance of about 2 miles, in which the river descends 15.4 feet. Between the head of Whirlpool Rapids and the foot of Hat Rapids there is a descent of 12.63 feet. A suitable dam at the foot of Nigger Island, in sec. 12, T. 35 N., R. 7 E., would develop a head of 28 feet. The banks are said to be high, with an abundance of rock and timber adjacent to the dam site. The drainage area at this point is 1,300 square miles. Three different railroad lines are located within 3 or 4 miles of this site, and Tomahawk, a city of 2,500 population, is 7 miles west.

HAT RAPIDS.

Between the mouth of Pelican River and the foot of Hat rapids, in sec. 27, T. 36 N., R. 8 E., the Wisconsin descends about 22 feet. As the banks are high, a dam in sec. 27 between lots 4 and 5 could be made to develop about 20 feet of head. The drainage area at this point is 1,220 square miles. The Rhinelander Power Company has been formed to develop this power, and from Mr. A. W. Sheldon, Rhinelander, Wis., its attorney, the following facts were learned. A recent survey shows that the concrete dam should be located 13 rods north of the south line of sec. 27, T. 36 N., R. 8 E. It will be 264 feet long, with earthen dikes, in addition, of 80 and 250 feet. Such a dam would create a head of 20.3 feet. The site is only 5 miles from Rhinelander, a city of over 5,000 inhabitants, reached by both the Chicago and Northwestern and the Minneapolis, St. Paul and Sault Ste. Marie railways. The power could be either used at the site for a paper mill or electrically transmitted to Rhinelander for lighting and power purposes. The latter is stated as the present intention of the owners. An officer of the company states that all the contracts for construction and machinery have been let, and that the plant is expected to be in operation by September, 1905.

RHINELANDER DAM.

Between the foot of the present dam of the Rhinelander Paper and Pulp Company, in the city of Rhinelander, and the foot of Otter rapids, in sec. 36, T. 40 N., R. 9 E., a distance of about 35 miles, the river descends 79.2 feet. The dam develops 30 feet of this descent, and the power is used to run one of the largest paper and pulp mills on the river. The company has installed turbines rated at a total of 3,000 actual horsepower and has also 1,200 steam horsepower. The daily capacity of this mill is 45 tons of finished paper, 40 tons of pulp, and 40 tons of sulphite pulp.

The river above this point has a drainage area of about 940 square miles. Above Rhinelander the river banks are lower and the opportunities for developing large powers few. In the 35 miles between Rhinelander and the source there are two rapids, called Rainbow rapids and Otter rapids. In this distance, according to the United States engineers,[a] between the head of Otter rapids and a point about a mile above the mouth of Pelican River, the descent of Wisconsin River is only 57 feet, or about 1.62 feet per mile.

RAINBOW RAPIDS.

These rapids are of small extent. They are located in sec. 6, T. 38 N., R. 8 E., and a head of 6 to 10 feet could be secured.

OTTER RAPIDS.

The most important power above Rhinelander is at Otter rapids, where a logging dam with a head of about 10 feet was early constructed. The rapids proper descend 16 feet,[a] so that a head of this amount or more could be developed. The dam site is between lots 6 and 8, sec. 36, T. 40 N., R. 9 E. The drainage area above this point is about 500 square miles.

According to the Chicago and Northwestern Railway, the Wisconsin River at Conover, in sec. 9, T. 41 N., R. 10 E., has an elevation of 1,644 feet above the sea. This would give a fall of 66 feet in the 24 miles between Conover and the head of Otter rapids.

TRIBUTARIES OF WISCONSIN RIVER.

GENERAL STATEMENT.

The watershed line on each side of the Wisconsin Valley is between 300 and 400 feet above the main river, and as the tributaries have to descend this distance in a length of 50 or 60 miles they have many rapids and available powers. In the upper portion of

[a] Rept. Chief Eng., U. S. Army, 1881, p. 1824.

their courses the tributaries flow over the hard pre-Cambrian rock, giving many rapids. The lower valleys, however, are filled by continued erosion, so that with few exceptions, no powers are found here.

The length and drainage area of certain streams tributary to Wisconsin River are shown in the following table:

Principal tributaries of Wisconsin River.

River.	Length.	Drainage area.
	Miles.	*Sq. miles.*
Pelican	25	262
Tomahawk	50	714
Rib	50	498
Eau Claire	50	423
Eau Pleine	50	377
Yellow	70	946
Lemonweir	50	588
Baraboo	70	655
Kickapoo	75	760

Only Kickapoo, Baraboo, and Lemonweir rivers and their branches have been as yet fully or even largely developed, but the present rapid settlement of this northern region is fast bringing a demand for the utilization of these valuable water-power resources. While these powers are small as compared with those on the main river, in the aggregate they are large, and their wide distribution makes them of still greater value. In some cases, because of the ease with which they can be developed and controlled, manufacturers seem to prefer them to the larger but more expensive powers on the parent river. An example of this is seen in the present power developments on Prairie River.

ST. GERMAIN RIVER.

Although but 20 miles long, St. Germain River has at least three good dam sites located as follows: (1) SW. ¼ sec. 31, T. 41 N., R. 8 E.; (2) near the outlet of Big St. Germain Lake, sec. 32, T. 40 N., R. 8 E.; and (3) near the northeast corner of sec. 18, T. 39 N., R. 8 E. At the second dam site a head of 20 feet and at the third site a head of 26 feet are reported as feasible.

TOMAHAWK RIVER.

This river rises in about 40 lakes with elevations of from 1,540 to 1,575 feet above the sea, the largest of which is Tomahawk Lake, with an area of 7 square miles. The river joins the Wisconsin at Tomahawk after a course of about 50 miles.

The dam in Wisconsin River at Tomahawk backs the water in Tomahawk River to an elevation of 1,442 feet, so that the remaining descent is about 120 feet, or 2.4 feet per mile, nearly half of which is concentrated in four rapids. Only one of these has been developed for power purposes, the dam being located about 2 miles above the mouth of the river, where a head of about 18 feet is obtained. At present only 300 horsepower are here utilized, in a tannery belonging to the United States Leather Company.

Eight miles above this dam, in lots 5 and 6, sec. 21, T. 36 N., R. 6 E., are the Prairie rapids, with a descent of 20 feet; 10 miles above, in lots 1 and 4, sec. 17, T. 37 N., R. 6 E., are the Halfbreed rapids, with descent of 8 feet; and 12 miles still farther upstream, in sec. 27, T. 38 N., R. 5 E., are the Cedar rapids, with descent of 12 feet.

PELICAN RIVER.

This river rises in a series of lakes, the largest being known by the same name, at an elevation of 1,590 feet above the sea. The river flows west and joins the Wisconsin near

Rhinelander, after descending about 50 feet in its length of 25 miles. The following table shows the location of promising dam sites, none of which are as yet developed:

Dam-site locations on Pelican River.

	Possible head (feet).
Between lots 4 and 6, sec. 4, T. 36 N., R. 10 E.	6 to 8
SW. ¼ sec. 17, T. 36 N., R. 10 E.	6
Between lots 3 and 4, sec. 26, T. 36 N., R. 9 E.	10
Between lot 1, sec. 21, and lot 1, sec. 22, T. 36 N., R. 9 E.	12

PRAIRIE RIVER.

Although Prairie River has a drainage area of only 214 square miles and is without lakes at its upper headwaters, its water powers are of sufficient importance to have already attracted capital for their development. At the eastern limits of the city of Merrill a dam 200 feet long is being rebuilt so as to give a head of 21 feet. This dam is owned by the Prairie River Power and Boom Company. Nine miles northeast, in sec. 13, T. 32 N., R. 7 E., at a point where the river has worn a deep channel in the rocks, forming dalles, a masonry dam to furnish a head of 72 feet is now being built by the same company. This power will be transmitted electrically to the lower dam for use in a paper mill now under construction.

In sec. 14, T. 33 N., R. 8 E., are smaller dalles, where a head of 20 feet may be obtained.

RIB RIVER.

Rib River rises in two small lakes, the larger of which, Rib Lake, has an elevation of about 1,556 feet. After a course of about 50 miles, the Rib joins Wisconsin River a mile below the city of Wausau. Its total descent is 400 feet, an average of 8 feet to the mile. A considerable part of this descent is concentrated in the middle third of its length.

The first power is found at Marathon, 10 miles from its mouth, where a dam about 80 feet long develops a head of 18 feet. About 5 miles above, in the city of Rib Falls, a dam 100 feet long develops a head of 20 feet. In sec. 24, T. 30 N., R. 4 E., there is an undeveloped power with a head of 18 feet.

EAU CLAIRE RIVER.

The Eau Claire enters Wisconsin River 2 miles below the mouth of Rib River, and from the opposite (eastern) side. It has a smaller drainage area than that of the Rib, and a much larger proportion of its descent is distributed in its lower part.

A total of 148 feet is concentrated at the following points, given in order from the mouth of the river.[a]

Dam-site locations on Eau Claire River.

Location.	Head.	Remarks.
	Feet.	
Schofield, sec. 12, T. 28 N., R. 7 E.	12	Developed (old mill abandoned).
Manser's, sec. 10, T. 28 N., R. 8 E.	25	Developed, but only part used.
Old Kelley, sec. 13, T. 28 N., R. 8 E.	25	Developed for logging.
Barnards rapids, sec. 23, T. 29 N., R. 9 E.	22	Undeveloped.
The Dalles, sec. 7, T. 29 N., R. 10 E.	40	Do.
Three Rolls, sec. 34, T. 30 N., R. 10 E.	12	Do.
Little rapids, sec. 22, T. 30 N., R. 10 E.	12	Do.

The first three powers are adjacent to the Chicago and Northwestern Railway, and are used chiefly for boom purposes.

[a] Authority: D. L. Plummer, C. E.

EAU PLEINE RIVER.

This river has a narrower and smaller drainage area than either the Rib or the Eau Claire and is entirely devoid of lakes. Like the latter, it has considerable descent concentrated in its lower reaches, one power with a 15-foot head being located within 2 miles of its mouth. Following is a summary of its powers:

Dam-site locations on Eau Pleine River.

Location.	Head.	Remarks.
	Feet.	
Sec. 18, T. 26 N., R. 6 E	15	Undeveloped.
Sec. 24, T. 26 N., R. 6 E	15	Do.
Sec. 13, T. 27 N., R. 3 E	15	Do.
Sec. 4, T. 27 N., R. 3 E	10±	Developed.
Sec. 24, T. 28 N., R. 2 E	10±	

BLACK RIVER.

TOPOGRAPHY AND DRAINAGE.

Black River, hemmed in by the Chippewa on the west and the Wisconsin on the east, is restricted to a long and narrow watershed of about 2,270 square miles, [a] with an average width of only 20 miles. At one point the branches of Chippewa River extend to within a quarter of a mile of Black River. Unlike that of the Chippewa, about a third of the Black River drainage area is in the comparatively level sandstone region, so that the maximum watershed available for water powers, namely, at Black River Falls, is only 1,570 square miles.[a] The watershed narrows rapidly as the river is ascended, and at Neillsville, 22 miles in an air line from Black River Falls, the drainage area is reduced to only 729 square miles. [a] Were it not for this small watershed, the steep gradient of the river and its high, rocky banks would insure large water powers. Black River rises at an elevation of about 1,400 feet above sea level, and after a sinuous course of over 140 miles joins Mississippi River at La Crosse. The total descent in this distance is 772 feet, with details as shown in the following table:

Profile of Black River from its mouth near La Crosse to near Withee.[b]

No.	Station.	Distance. From mouth.	Distance. Between points.	Elevation above sea level.	Descent between points. Total.	Descent between points. Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	La Crosse (near)			628		
2	Black River Falls:					
	Below dam	55.0	55.0	749	121	2.2
3	Above dam	55.0	.0	763	14	
4	Chicago, St. Paul, Minneapolis and Omaha Railroad bridge	58.0	3.0	766	3	1.0
5	Halls Creek, mouth of	61.6	3.6	776	10	2.8
6	Halcyon	67.0	5.4	793	17	3.1
7	Hatfield railroad bridge	71.2	4.2	838	45	10.4
8	East Forks, mouth of	74.2	3.0	846	8	2.7
9	Dells dam, below	77.5	3.3	874	28	8.5

[a] Census report, vol. 17, 1880, p. 87.
[b] Authority: No. 1 (low-water elevation), Mississippi River Commission; 2 to 22, Joint Survey of Wis. Geol. and Nat. Hist. Survey and United States Geological Survey.

Profile of Black River from its mouth near La Crosse to near Withee—Continued.

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
10	Wedges Creek, mouth of	78.5	1.0	893	19	19.0
11	Cunningham Creek, mouth of	84.8	6.3	909	16	2.5
12	Center sec. 22, T. 24 N., R. 2 W.	86.8	2.0	929	20	10.0
13	O'Neill Creek, Neillsville	0.8	4.0	989	60	15.0
14	Bridge, secs. 9 and 16, T. 25 N., R. 2 W	98.8	8.0	1,034	45	5.6
15	Bridge, secs. 21 and 28, T. 27 N., R. 2 W	103.5	4.7	1,070	36	7.9
16	Bridge, Fairchild and Northeastern Rwy	107.8	4.3	1,094	24	5.6
17	Site New Greenwood dam	109.3	1.5	1,105	11	7.3
18	Between secs. 27 and 28, T. 27 N., R. 2 W	110.3	1.0	1,107	2	2.0
19	Hemlock dam, 600 feet below	113.5	3.2	1,132	25	8.0
20	Hemlock dam, above	113.6	.1	1,151	19	....
21	Bridge, secs. 20 and 29, T. 29 N., R. 2 W	119.6	6.0	1,167	16	2.7
22	Bridge, Wisconsin Central Rwy., west of Withee	125.1	5.5	1,187	20	3.6

In the 55 miles below the city of Black River Falls the river flows through the sandstone country in a wide valley with low banks, making dam construction very expensive, if not entirely impracticable. In the 40 miles next above Black River Falls the river has worn its bed into the hard, crystalline rocks, which rise from 10 to 60 feet or more from the water, frequently in nearly vertical walls. The descent in this distance is 337 feet, nearly 9 feet to the mile. It is only in this stretch that important water powers occur. In the upper third of the valley the crystalline rocks frequently outcrop, but the resulting rapids are of less importance. The United States Geological Survey maintained a gaging station on Black River at Melrose for nine months in 1903, but as the station proved unsatisfactory it was abandoned August 1, 1903. Such measurements and observations as were taken are given below:

Discharge measurements of Black River near Melrose, Wis., in 1903.

Date.	Hydrographer.	Gage height.	Discharge.
		Feet.	*Second-feet.*
January 15	L. R. Stockman	4.30	*a* 598
February 7	do	4.30	*a* 518
April 4	do	5.90	2,982
May 1	do	11.00	10,931
June 13	do	3.90	842

a Frozen.

Mean daily gage height, in feet, of Black River near Melrose, Wis., December 4, 1902, to August 1, 1903.

Day.	1902.	1903.							
	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.
1	....	5.05	4.10	4.30	5.10	11.00	7.60	3.60	3.75
2	....	5.00	4.10	4.35	4.85	10.00	....	6.70	....
3	....	4.90	4.10	4.40	5.30	10.25	....	11.20	....
4	3.75	4.75	4.10	4.45	5.65	10.50	....	10.90	....
5	3.95	4.60	4.10	4.60	5.90	9.65	6.00	13.00	....
6	4.00	4.60	4.20	4.75	6.50	9.05	4.70	12.30	....
7	3.80	4.50	4.20	(a)	6.65	8.15	4.40	10.20	....
8	4.35	4.50	4.20	6.25	6.50	7.00	4.30	7.90	....
9	4.35	(a)	4.20	8.20	6.20	6.95	4.25	6.90	....
10	4.30	4.40	4.20	9.30	5.50	6.55	4.00	7.40	....
11	4.35	4.40	4.30	9.70	5.60	6.10	4.00	8.70	....
12	4.20	4.40	4.25	10.75	....	6.65	3.95	7.20	....
13	4.20	4.40	4.20	12.05	5.45	10.60	3.95	6.70	....
14	4.10	4.40	4.20	12.55	5.60	12.00	3.80	6.20	....
15	4.15	(a)	4.20	11.55	5.95	10.90	3.80	5.80	....
16	4.10	4.30	4.10	9.85	5.85	9.15	3.80	5.30	....
17	4.00	4.30	4.15	9.40	6.05	7.80	3.70	4.50	....
18	4.00	4.30	4.00	10.35	5.60	6.55	3.70	4.20	....
19	4.05	4.30	3.95	11.95	5.00	6.50	3.70	4.10	....
20	4.25	4.20	3.90	13.40	5.15	6.40	3.70	4.00	....
21	4.60	4.20	3.90	12.90	4.50	6.30	3.70	4.00	....
22	4.95	4.20	4.00	11.40	4.65	5.90	3.70	4.00	....
23	5.80	4.20	4.00	9.65	4.30	6.50	3.70	3.90	....
24	6.05	4.20	4.00	8.05	4.30	5.70	3.60	3.90	....
25	5.85	4.20	4.05	7.65	4.35	5.80	3.60	3.90	....
26	5.80	4.20	4.10	6.65	4.65	5.95	3.50	3.75	....
27	5.65	4.20	4.20	6.02	.85	8.40	3.50	3.90	....
28	5.50	4.20	4.35	6.55	5.00	11.85	3.50	4.20	....
29	5.35	4.20	....	5.70	5.65	12.60	3.50	4.00	....
30	5.20	4.20	....	6.55	6.80	10.95	3.50	3.80	....
31	....	4.10	....	5.30	....	9.50	....	3.75	....

[a] Observer absent.

A gaging station was established by the United States Geological Survey at Neillsville April 7, 1905, and the following data have been collected:

Discharge measurements of Black River at Neillsville, Wis., in 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
		Feet.	*Square feet.*	*Feet per second.*	*Feet.*	*Second-feet.*
April 7	Hanna and Clapp	192	1,021	3.5	7.7	3,279
May 24	S. K. Clapp	165	471	2.18	4.95	1,024
June 13	M. S. Brennon	192	945	3.15	7.26	2,978
July 11	.....do	161	392	1.56	4.25	612
August 11	.....do	151	242	.93	3.3	225
September 25	F. W. Hanna	163	419	1.86	4.35	780

NOTE.—*Width* is the actual width of water surface, not including piers. *Area of section* is the total area of the measured section, including both moving and still water.

Mean daily gage height, in feet, of Black River at Neillsville, Wis., for 1905.

Day.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	....	3.4	3.7	4.4	2.7	3.3	3.5	3.7	4.0
2	....	3.4	3.3	4.4	2.6	3.2	3.5	3.5	4.2
3	....	4.1	3.2	4.9	2.6	3.5	3.4	3.5	3.8
4	....	5.3	7.7	6.5	2.6	3.4	3.4	3.5	3.7
5	....	5.2	14.2	8.4	2.9	3.6	3.0	3.5	3.8
6	8.2	4.9	19.8	8.0	2.7	3.2	3.0	3.7	3.5
7	7.7	5.0	16.5	6.8	4.2	3.1	3.1	4.1	3.5
8	6.9	4.6	11.5	5.9	4.0	3.0	2.7	4.1	3.4
9	6.2	4.6	8.8	5.3	4.0	2.9	2.4	3.9	3.3
10	6.0	5.9	7.6	4.7	3.5	2.8	3.1	3.8	3.4
11	5.7	6.6	8.6	4.2	3.3	2.7	3.0	3.7	3.4
12	5.5	6.7	8.0	3.8	3.3	2.8	3.0	3.7	3.5
13	5.1	6.2	7.1	3.9	3.3	2.7	3.0	3.6	3.4
14	4.8	10.7	6.2	4.0	3.3	2.7	3.0	3.5	3.4
15	4.6	10.1	5.5	4.8	3.2	4.3	4.0	3.4	3.5
16	4.3	9.2	5.8	4.5	3.0	6.0	4.9	3.4	3.4
17	3.9	8.7	11.2	4.0	2.9	6.0	5.4	3.4	3.3
18	3.8	8.2	10.7	3.8	3.0	6.1	5.5	3.4	3.0
19	4.2	6.6	8.6	4.2	3.0	8.6	5.6	3.4	3.0
20	3.9	6.0	7.0	4.3	3.0	8.3	6.6	3.3	3.2
21	3.2	5.3	6.0	4.0	3.2	7.5	6.9	3.2	3.1
22	3.1	5.1	5.2	3.8	3.5	6.3	6.5	3.2	3.1
23	3.1	4.9	4.5	3.3	3.4	5.8	5.9	3.2	3.3
24	3.5	4.7	4.1	3.1	3.6	4.7	5.5	3.5	3.5
25	3.4	4.3	3.9	3.1	3.4	4.2	5.0	4.2	3.7
26	3.4	4.2	3.7	3.0	3.3	3.9	4.6	4.6	3.5
27	3.4	4.1	3.5	2.9	3.2	3.8	4.4	4.5	3.4
28	3.4	3.9	3.3	2.9	3.0	3.7	4.1	4.3	3.4
29	3.4	3.9	3.3	2.8	3.4	3.6	3.9	3.9	3.4
30	3.4	3.8	3.5	2.8	3.5	3.8	3.7	3.7	3.5
31	....	3.8	....	2.7	3.3	....	3.6	....	3.5

NOTE.—No ice record at this station.

Rating table for Black River at Neillsville, Wis., from April 6 to December 31, 1905.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
3.00	150	4.80	990	6.60	2,310	8.40	4,120
3.10	177	4.90	1,055	6.70	2,395	8.50	4,230
3.20	205	5.00	1,120	6.80	2,480	8.60	4,340
3.30	235	5.10	1,185	6.90	2,570	8.70	4,460
3.40	267	5.20	1,250	7.00	2,660	8.80	4,580
3.50	301	5.30	1,315	7.10	2,750	8.90	4,700
3.60	338	5.40	1,385	7.20	2,850	9.00	4,820
3.70	379	5.50	1,455	7.30	2,950	9.10	4,940
3.80	424	5.60	1,525	7.40	3,050	9.20	5,060
3.90	473	5.70	1,600	7.50	3,150	9.30	5,180
4.00	525	5.80	1,675	7.60	3,250	9.40	5,300
4.10	579	5.90	1,750	7.70	3,350	9.50	5,420
4.20	635	6.00	1,825	7.80	3,460	9.60	5,540
4.30	692	6.10	1,905	7.90	3,570	9.70	5,660
4.40	750	6.20	1,985	8.00	3,680	9.80	5,780
4.50	810	6.30	2,065	8.10	3,790	9.90	5,900
4.60	870	6.40	2,145	8.20	3,900	10.00	6,020
4.70	930	6.50	2,225	8.30	4,010		

The above table is applicable only for open-channel conditions. It is based on six discharge measurements made during 1905. It is well defined between gage heights 3.3 feet and 7.7 feet. Beyond the limits of the table the discharge is only approximate.

Estimated monthly discharge of Black River at Neillsville, Wis., for 1905.

Month.	Discharge in second-feet.		
	Maximum.	Minimum.	Mean.
April (6-30)	3,900	177	1,036
May	6,910	267	1,768
June	23,060	205	3,840
July	4,120	80	884
August	635	60	229
September	4,340	80	918
October	2,570	20	750
November	870	205	392
December	635	150	292

WATER POWERS.

It is many years since Black River was used for lumbering, and as the surrounding country is well settled, it seems likely that the near future will see a demand for the available water powers. These powers, while not of the largest, are so situated as to be cheaply developed. The river has no large tributaries, but nearly all its numerous small feeders are now developed and used to run grist and saw mills. At the present time several projects are being exploited which look to the employment of these powers by interurban electric railroads and other enterprises in near-by cities.

BLACK RIVER FALLS.

The first dam in the river is at Black River Falls and is of timber construction. The power developed is owned by the city of Black River Falls, with turbines working under a head of 13 feet, and by J. J. McGillivray, with turbines under a head of 16 feet. The present tailrace could be lowered 3 or 4 feet, and the crest of the dam could be raised the same amount without flooding. This improvement would give a total head of 20 feet. The turbines now installed develop about 345 horsepower, which is used to run an electric-light plant, a sash and door mill, a wagon shop, and a gristmill.

About 1½ miles below the above-described dam is the site of an old sawmill dam, 300 feet long, which at one time was made to develop a head of 7 feet.

BLACK RIVER FALLS TO NEILLSVILLE.

Because of the high, rocky banks and high gradient of this river, dams of 15 to 20 feet head could be installed nearly every 2 or 3 miles between Black River Falls and Neillsville, but only a few of the largest undeveloped powers will be described.

The first dam site above Black River Falls is located near the east line of sec. 2, T. 21 N., R. 4 W., just below the Chicago, St. Paul, Minneapolis and Omaha Railway bridge. At this point the rocky banks form a narrow gorge and are high enough to furnish a head of 30 feet or more. By the use of a short canal this head could probably be increased. This site belongs to the Black River Improvement Company, of La Crosse, Wis. Another undeveloped power, similar in all respects, for which a charter has been granted, is at Halcyon, in sec. 16, T. 22 N., R. 3 W. A 30-foot dam here would back the water nearly to Hatfield, 3 miles above. A still more important dam site is located at Hatfield, just above the bridge of the Green Bay and Western Railroad. According to surveys made recently it is possible to obtain here a head of 50 feet, which could be increased to about 85 feet by means of a long canal. Such a dam would create a large pondage by backing up the water for 7 miles. This would cover up dam sites in sec. 35, T. 23 N., R. 3 W., and also the "Dells dam" in sec. 18, T. 23 N., R. 2 W., near the mouth of Wedges Creek. At the latter site a head of 25 feet could be easily secured.

In the 6 miles below Neillsville, between the mouths of O'Neill and Cunningham creeks, the river descends 80 feet, 42 feet of which can be easily developed at Ross Eddy rapids, where a large part of this gradient is concentrated. It has been proposed to build a crib dam 250 feet long, with a crest of 18 feet, at the head of these rapids, and then conduct the water through a canal 95 rods long (in earth), thereby cutting off a long bend of the river and giving a total fall of 42 feet.[a] The outlet of such a canal would provide a favorable power site, free from any injury from ice jams.

NEILLSVILLE.

The last important undeveloped water power, known as Westons Rapids and owned by V. Huntzicker, of Neillsville, Wis., is located in sec. 2, T. 24 N., R. 2 W., about 1½ miles above Neillsville. From the head of these rapids near the north line of the NW. ¼ sec. 2, to the south line of same section, a distance of about a mile, the river descends 21.2 feet.[a] The owner proposes to locate a crib dam 250 feet long, with a crest of 18 feet, near the center of the section, and by making use of a canal in earth 600 feet long to obtain a head of 24 feet. A franchise has recently been obtained from the city of Neillsville for the employment of this power in lighting the city and for other purposes.

HEMLOCK DAM.

The most important developed power on the upper river is in sec. 15, T. 27 N., R. 2 W. This dam, called the Hemlock dam, has a head which averages 12 feet. Four turbines are installed here, with a total of 175 horsepower, used to run a roller flouring mill. The dam was originally erected for lumbering purposes.

Because of the unusually steep gradient in the branches of Black River a water power of from 10 to 20 feet can be located at frequent intervals on these streams. Several of the many mills in such locations report an available head of from 35 to 40 feet. In nearly every case timber and rock are found near the dam sites.

RAILROADS.

That portion of Black River containing the important powers is fairly well served by railroads. The river is crossed by the Chicago, St. Paul, Minneapolis and Omaha Railway four times, and once each by the Wisconsin Central Railway and the Green Bay and Western Railroad.

CHIPPEWA RIVER SYSTEM.

TOPOGRAPHY AND DRAINAGE.

The Chippewa drainage system has its source in over a hundred lakes, large and small, with many connecting swamps, near the Michigan boundary and only 20 miles from Lake Superior. The drainage area has a length of 180 miles, a maximum width of 90 miles, and an average width of nearly 60 miles. The general direction of the drainage, except in the extreme western part, is toward the southwest. Chippewa River unites with the Mississippi at the foot of Lake Pepin, after a course of 267 miles. The total area drained by the river is 9,573 square miles, of which about 6,000 include the most unsettled region of northern Wisconsin. This area includes the richest forests of the State, of both soft and hard timber. Although lumbering operations have been very active here for many years, considerable pine timber still remains, chiefly at the upper headwaters, but it is fast disappearing. Most of the large tracts of pine lands are owned by large corporations, and many of them are reached by long lines of logging railroads, which in many cases have been purchased by the trunk-line railroads and made a part of their systems. The extensive use of such railroads has greatly relieved the rivers of the burden of transporting logs, and correspondingly added to the value of the rivers for water-power purposes.

The main line of drainage runs very nearly along the central line of the basin, but the name of Chippewa River is not given to this continuation of the principal stream. The

[a] Authority: C. Stockwell, county surveyor.

river divides 112 miles from the mouth; one branch, the prolongation of the line of drainage, called the Flambeau, rises in the lakes near the Michigan line, at an elevation of a little over 1,600 feet above the sea; the other branch, rising farther west and flowing more directly south, receives the name Chippewa. The Flambeau drains 1,983 square miles, while Chippewa River, above their junction, drains only 1,777 square miles. About 56 miles above this junction the Chippewa again divides into East and West branches, the one flowing from the northeast, the other from the north, draining, respectively, 278 and 480 square miles.

The lakes of this region are situated in two widely separated groups, one in the extreme northeastern part, at the headwaters of Flambeau River, and the other in the northwestern part, at the headwaters of what is known as the main stream and of Red Cedar River. The remainder of the area is almost devoid of lakes. The wooded regions, however, include very large areas of cedar and tamarack swamps.

GEOLOGY.

The pre-Cambrian crystalline rocks form the underlying strata in the area above Chippewa Falls, while below that point they are replaced by the Cambrian sandstone. The entire area above Chippewa Falls is covered with glacial drift, so that the rock appears only in the river bed. The country is level or rolling. In the southern part of the area the rivers have eroded deeply into the drift and rock, but in the northern portion they have not cut much below the surface.

With only a few exceptions (the most notable one at Eau Claire) all the many and important water powers on Chippewa River are found in the region of the pre-Cambrian crystalline rocks, but because of the deep drift the powers on the upper streams occur as bowlder rapids.

PROPOSED RESERVOIR SITES.

According to detailed surveys made by United States engineers, this drainage area is favored with an unusual number of excellent sites for reservoirs. A list of these sites, with valuable data concerning them, is given in the following table:

Proposed United States Government dams on Chippewa River. [a]

Location and name.	Length.		Maximum height.		Drainage area above reservoir.
	Dam.	Dike.	Dam above low water.	Dike.	
East Branch Chippewa River:	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Sq. miles.*
Bear Lake	1,015	200	19.5	8.5	244.5
Little Chief Lake	710		24.0		57.6
West Branch Chippewa River:					
Moose Lake	1,235	160	25.7	1.5	214.3
Pakwawang Lake	900		25.5		257.2
Court Oreilles	260	100	6.5	5.0	114.0
Chippewa River, Paint Creek	620		22.0		3,943.1
Total	4,740	460			4,830.7
Butternut Creek, Butternut Lake	336		10.0		40.0
Manitouish River, Rest Lake	250	75	15.0	2.5	211.6
North Fork Flambeau, Bear Creek	2,500	2,000	15.0	10.5	154.5
Dore Flambeau:					
Round Lake	170	250	10.0	10.0	63.0
Squaw Lake	250		9.0		39.0
Turtle River, Park Lake	297		15.0		174.0
Grand total	8,543	2,785			5,512.8

[a] Rept. Chief Eng. U. S. Army, 1880, p. 1648.

Proposed United States Government dams on Chippewa River—**Continued.**

Location and name.	Supply (one-third of 30 inches rainfall).	Capacity of reservoir.	Surplus over reservoir capacity.	Supply from reservoir for 90 days.	Cost of dam and dike.
East Branch Chippewa River:	*Cubic feet.*	*Cubic feet.*	*Cubic feet.*	*Sec.-feet.*	
Bear Lake	5,677,951,910	1,113,148,856	4,564,803,054	143.1	$25,92[illegible]
Little Chief Lake	1,337,627,935	771,332,009	566,295,926	99.2	40,7[illegible]
West Branch Chippewa River:					
Moose Lake	4,976,626,153	2,[illegible]01,783,402	1,234,725,814	260.0	45,[illegible]90
Pakwawang Lake	5,972,880,292	7,692,997,229		989.3	66,4[illegible]
Court Oreilles	2,647,388,621	2,647,388,621	....	340.4	2,4[illegible]
Chippewa River, Paint Creek	91,569,456,760	505,336,720	91,064,120,040	65.0	60,[illegible]
Total	112,181,931,671	14,751,986,837	97,429,944,834	1,897.0	240,6[illegible]
Butternut Creek, Butternut Lake	928,908,288	585,446,400	343,461,888	75.3	5,2[illegible]6
Manitouish River, Rest Lake	4,897,100,264	1,840,000,000	757,813,112	236.6	7,665
North Fork Flambeau, Bear Creek	3,107,280,000	5,406,567,152		695.3	47,5[illegible]
Dore Flambeau:					
Round Lake	1,382,304,000	1,303,036,416	79,267,584	167.6	10,550
Squaw Lake	864,230,400	731,808,000	132,422,400	94.1	4,000
Turtle River, Park Lake	4,026,198,428	620,782,720	3,405,415,708	79.8	9,941
Grand total	127,387,953,051	25,239,627,525	102,148,325,526	3,245.7	325,[illegible]

It will be seen from the above table that the systematic operation of these proposed reservoirs for this purpose would increase the ordinary low-water flow of the river by 3,245 second-feet for ninety days a year, thus about doubling the present available water power of the river. Estimated upon a run-off of one-fourth of the annual rainfall, assumed at 30 inches, this increase would be 2,800 second-feet for ninety days.

Experiments now being carried on by the Government in Minnesota on five similarly constructed dams will doubtless determine whether the reservoir system at the headwaters of the Mississippi will be extended to include any of the above proposed dams. Probably the main obstacle to building such reservoirs at the present time by the Government is the fact that, owing to the settling up of this region, the land has now become very valuable. The total cost would seem to be prohibitive. That the owners of water powers are in favor of such Governmental control is certain. Besides adding to the amount of power, such a system would prevent, in large measure, the danger to dams by floods. The building of even a part of these dams would have marked economic value. Already private enterprise has developed some of the smaller of these reservoirs.

RAILROADS.

The logging interests of the river are controlled by the Chippewa Falls Lumber and Boom Company, with headquarters at Chippewa Falls, a thriving city of about 10,000 population. The largest city of this region is Eau Claire, population 17,517, situated at the junction of Eau Claire and Chippewa rivers. This city has numerous manufactories and sawmills, and is quite a railroad center. From its mouth to Chippewa Falls, Chippewa River is paralleled by the Chicago, Milwaukee and St. Paul Railway, and between Eau Claire and Chippewa Falls by the Chicago, St. Paul, Minneapolis and Omaha and the Wisconsin Central railways, besides an electric line. Chippewa River, above Chippewa Falls, is reached by the Chicago, St. Paul, Minneapolis and Omaha Railway for a distance of about 25 miles. In addition, the drainage area is crossed east and west by the Minneapolis, St. Paul and Sault Ste. Marie Railway and north and south by the Wisconsin Central Railway.

Several railroad lines are projected or being built in this section, and the agricultural and manufacturing interests are fast supplanting that of lumber. Where the timber has been cut the land is being taken up by settlers, so that there is but little second-growth timber. The people seem prosperous, and numerous companies are on the point of investing large sums in the manufacturing interests of the neighborhood, thereby utilizing the undeveloped water powers.

RAINFALL AND RUN-OFF.

The extensive forests of this area combine with the numerous lakes and swamps to give a naturally uniform flow by preventing the rapid escape of the rainfall into the streams. Since 1903 the United States Geological Survey has maintained gaging stations near Eau Claire, on Chippewa River, and at Ladysmith, on the Flambeau. As a result of the operation of logging dams, the minimum discharge is found to be only 1.6 per cent of its maximum discharge for the year. The following tables give discharge data of Chippewa River at Eau Claire, covering the period from November 14, 1902, to August 12, 1905, and also a monthly summary of the same.

Discharge measurements of Chippewa River at highway bridge, Shawtown, near Eau Claire, Wis., 1902 to 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height.	Discharge.
1902.		*Feet.*	*Square feet.*	*Feet per second.*	*Feet.*	*Second-feet.*
November 13	L. R. Stockman		3,666	3.88	8.70	11,134
December 6	do		2,809	1.03	4.45	2,871
December 28	do		2,793	1.09	4.60	3,063
1903.						
January 17	L. R. Stockman		2,509	.79	4.15	[a] 1,979
February 10	do		2,315	.77	3.80	[a] 1,778
March 9	do		2,877	1.32	4.85	[b] 3,818
April 6	do				7.40	10,688
May 5	do		5,726	4.62	11.85	26,458
June 15	do		3,105	1.64	4.70	4,107
July 10	do		4,761	3.61	9.25	17,167
August 20	do		2,372	1.83	5.13	4,336
September 5	do		3,626	2.21	6.20	8,032
October 13	do		4,637	3.25	8.77	15,087
November 24	do		2,281	1.54	4.90	3,511
1904.						
January 11[a]	E. Johnson, jr	310	2,429	.99	3.80	2,454
May 14	do	385	4,272	3.42	8.40	14,610
May 24	Johnson and Hanna	370	4,074	3.10	7.60	12,630
June 7	E. Johnson, jr	426	5,815	4.52	11.25	26,270
July 13	do	354	3,770	2.10	6.55	7,918
August 28	do	322	2,766	.82	4.20	2,274
September 19	do	329	3,122	1.47	5.25	4,581
October 12	F. W. Hanna	495	7,118	5.43	14.80	38,680
October 13	do	457	6,137	4.76	13.10	29,200
November 29	E. Johnson, jr	324	2,847	.80	4.44	2,281
1905.						
May 22	S. K. Clapp	200	4,004	3.66	8.80	16,110
June 14	M. S. Brennan	427	5,131	3.83	10.72	19,665
July 12	do	355	3,585	2.09	6.55	7,489
August 12		335	3,062	1.29	5.00	3,948

[a] Frozen. [b] Partly frozen.

NOTE.—Width is the actual width of water surface, not including piers. Area of section is the total area of the measured section, including both moving and still water.

Mean daily gage height, in feet, of Chippewa River near Eau Claire, Wis., November 14, 1902, to December 31, 1905.

Day.	1902.		1903.											
	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1			4.30	3.70	3.20	6.85	12.80	(a)	7.65	5.05	5.90	6.90	5.55	(a)
2			4.35	4.25	3.75	7.15	13.10	(a)	8.05	5.20	6.45	7.00	5.55	(a)
3			4.15	4.10	3.85	7.45	12.15	(a)	10.60	5.85	5.65	7.35	5.45	(a)
4			4.20	4.05	3.90	7.95	11.85	(a)	13.50	6.85	5.85	9.90	5.50	(a)
5		4.50	4.40	4.10	3.75	7.85	11.55	(a)	15.30	7.50	5.75	11.25	5.25	(a)
6		4.45	3.90	4.05	3.75	7.40	10.90	(a)	15.20	9.10	6.45	11.65	5.20	[illegible]
7		4.00	4.15	4.15	3.85	7.50	10.30	6.85	13.75	9.00	5.85	11.60	5.15	[illegible]
8		4.05	4.40	4.15	4.10	7.90	9.20	5.45	11.30	11.05	6.30	11.15	5.10	[illegible]
9		4.05	4.20	4.00	4.60	8.55	9.15	6.45	10.30	6.70	8.00	11.35	5.05	[illegible]
10		4.10	4.40	3.85	5.05	8.00	7.65	5.95	9.40	7.55	8.05	10.95	5.00	4.70
11		4.25	4.80	3.80	5.80	7.60	8.95	6.50	10.10	7.25	9.15	9.95	5.00	[illegible]
12		4.20	4.65	3.85	6.00	8.15	12.00	5.90	9.95	6.75	12.85	9.45	5.00	[illegible]
13		4.45	4.75	3.90	7.05	7.70	13.25	9.25	8.90	6.85	14.00	9.00	5.05	4.70
14	13.70	4.25	4.60	3.90	8.20	7.65	13.40	3.75	8.10	6.90	16.75	8.80	5.30	[illegible]
15	10.20	4.55	4.85	3.90	8.05	7.80	13.25	4.65	7.80	9.65	17.85	8.45	5.40	[illegible]
16	12.40	4.25	4.30	4.50	7.00	7.80	11.85	4.95	7.20	5.10	18.50	7.75	4.95	[illegible]
17	13.05	4.30	4.20	4.10	7.00	7.50	10.45	4.95	6.50	6.80	17.45	7.70	4.95	[illegible]
18	12.60	4.10	4.30	4.15	7.55	6.75	9.90	4.90	8.80	6.65	15.50	7.40	4.25	4.65
19	11.35	4.30	4.65	4.30	11.80	6.65	9.15	4.30	5.95	6.00	13.45	7.55	4.40	4.65
20	9.60	4.25	4.40	4.25	13.95	6.85	9.15	5.15	6.70	5.10	11.80	7.05	4.15	[illegible]
21	8.50	4.30	4.50	4.20	13.65	6.80	9.30	4.70	6.15	5.15	10.50	7.05	4.20	[illegible]
22	7.55	4.25	4.35	3.35	12.65	6.65	9.50	4.20	5.70	7.50	9.95	6.75	4.35	[illegible]
23	7.40	4.50	4.35	3.80	11.70	6.40	9.05	4.20	5.60	5.60	9.15	6.70	4.85	4.70
24	7.15	4.70	4.45	4.15	10.45	6.40	9.10	4.15	6.00	5.10	7.80	6.55	4.95	[illegible]
25	7.00	4.30	3.50	4.05	9.40	8.60	9.85	4.25	9.20	5.15	7.60	6.30	4.95	[illegible]
26	6.45	4.90	4.20	3.90	8.75	6.55	10.20	4.15	5.25	5.25	7.00	6.15	4.90	4.40
27	6.20	5.10	4.10	3.95	8.40	7.15	12.50	6.75	5.05	4.70	7.65	5.95	4.85	[illegible]
28	6.00	4.60	4.10	3.85	7.75	7.00	15.15	4.20	5.20	5.45	7.05	6.10	4.60	[illegible]
29	5.75	4.50	3.85	......	7.60	7.30	16.70	4.60	5.15	5.60	7.05	6.10	......	[illegible]
30	5.55	4.85	4.15	......	7.15	11.70	16.10	4.95	5.15	5.20	7.05	6.10	......	[illegible]
31	......	4.50	4.25	......	6.80	......	......	......	5.10	5.60	......	5.90	......	3.40

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.[b]												
1	[c]4.90	......	......	6.25	9.00	8.45	8.10	4.42	4.58	6.31	6.79	[illegible]
2	[c]5.15	......	......	6.72	8.65	7.65	8.52	4.80	5.05	5.24	6.84	[illegible]
3	......	......	......	6.50	8.63	7.65	8.28	4.38	8.18	8.27	6.80	[illegible]
4	[c]5.00	......	......	6.52	8.28	8.85	8.02	4.78	7.52	5.65	6.61	[illegible]
5	......	......	[d]4.80	6.60	8.10	9.00	10.32	4.32	8.30	5.30	5.91	[illegible]
6	......	[e]4.80	......	7.17	8.03	10.95	10.12	4.12	7.30	5.27	6.41	[illegible]
7	......	......	......	7.60	10.13	11.30	9.63	3.45	7.52	4.82	6.08	[illegible]
8	......	......	......	9.10	8.50	10.85	8.93	4.62	7.12	9.12	5.35	[illegible]
9	[c]4.80	......	......	9.67	8.83	9.85	7.22	5.10	6.72	7.86	5.35	[illegible]
10	......	......	......	9.72	9.35	8.92	7.10	5.42	7.95	13.35	5.60	[illegible]
11	......	......	......	9.70	9.25	7.95	7.20	5.35	5.03	15.07	5.36	[illegible]
12	......	......	[e]4.95	9.32	8.85	7.80	6.72	4.38	5.28	14.93	5.02	4.34
13	......	[d]5.00	......	9.05	8.78	7.80	6.60	4.12	5.42	13.15	6.27	[illegible]
14	......	......	......	8.82	10.20	7.45	5.88	3.50	5.35	11.38	4.82	[illegible]

[a] Observer absent.
[b] River frozen over January 1 to March 18, 1904, but open about 200 to 300 feet above and one-fourth mile below bridge.
[c] Ice 2.0 feet thick at gage; 1.0 foot in middle of channel.
[d] Ice 2.5 feet thick at gage; 2.5 feet in middle of channel.
[e] Ice 2.0 feet thick at gage; 2.0 feet in middle of channel.

Mean daily gage height, in feet, of Chippewa River near Eau Claire, Wis., November 14, 1902, to December 31, 1905—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
15	......	......	......	8.50	8.00	7.80	5.07	3.80	5.38	10.30	6.26	4.34
16	[a] 5.00	......	......	7.55	8.20	8.35	5.55	4.55	5.42	9.17	5.42	4.32
17	......	......	......	7.25	7.55	6.25	5.35	4.82	7.10	8.10	5.30	4.15
18	......	......	......	7.50	7.22	5.50	5.60	4.65	4.80	8.00	5.47	4.19
19	......	[b] 5.10	4.45	8.38	7.55	5.10	5.50	4.78	4.89	7.08	5.20	4.53
20	......	......	4.07	8.20	6.93	6.20	5.10	7.20	4.27	6.85	4.98	4.29
21	......	......	4.45	8.13	10.30	6.50	5.47	5.25	4.30	8.35	5.28	4.34
22	......	......	4.37	7.45	6.30	5.90	5.05	4.75	4.35	9.25	5.23	4.37
23	[a] 5.00	......	4.32	8.05	6.83	5.80	4.75	4.68	5.10	9.42	5.74	4.55
24	......	......	5.62	8.50	7.33	5.50	3.93	4.60	4.24	9.00	4.77	4.38
25	......	......	5.95	9.65	9.20	8.95	4.07	5.00	5.76	8.78	4.94	3.31
26	......	......	6.45	10.63	12.00	5.55	4.90	5.40	8.18	7.81	5.10	4.19
27	......	[b] 5.30	6.10	10.45	13.48	7.75	4.80	8.15	7.61	8.02	4.85	4.55
28	......	......	5.72	9.18	13.63	7.60	5.00	3.58	6.93	7.22	4.55	(c)
29	[d] 4.70	......	5.05	9.55	12.02	7.75	4.88	5.55	9.81	7.55	4.54	......
30	......	......	5.10	9.03	10.67	7.50	6.45	4.92	6.65	7.30	4.46	......
31	......	......	5.32	......	9.20	......	5.00	4.52	......	6.85	......	......
1905.												
1	(e)	......	......	12.00	6.80	6.30	6.80	6.80	6.50	5.15	5.70	4.75
2	4.36	......	......	10.20	5.80	6.80	7.40	5.75	6.20	5.55	6.15	5.15
3	......	......	......	11.20	5.90	6.50	6.30	4.35	5.30	7.80	5.45	4.85
4	......	......	......	10.50	6.50	8.20	6.20	4.35	6.10	5.55	5.75	5.65
5	......	4.80	4.20	10.40	7.50	12.10	6.90	4.75	6.80	4.90	5.65	4.40
6	......	......	......	10.80	6.60	19.20	10.40	5.10	6.50	4.90	5.75	5.40
7	......	......	......	10.20	7.30	......	10.60	5.25	6.10	7.50	6.10	5.40
8	......	......	......	9.80	7.60	19.60	11.30	5.05	5.65	5.00	6.30	5.90
9	4.40	......	4.50	9.20	7.30	17.30	10.10	5.30	5.75	4.85	6.05	5.85
10	......	......	......	8.90	8.80	14.50	7.00	5.90	5.10	4.80	6.10	5.35
11	......	......	......	8.70	7.80	13.00	8.10	4.90	4.90	5.50	6.50	5.30
12	......	......	4.30	7.40	7.90	12.60	6.90	5.45	5.35	7.90	5.50	5.30
13	......	5.30	4.30	5.75	7.50	11.50	6.90	5.10	6.90	6.20	5.70	5.30
14	......	......	4.25	5.65	9.50	10.00	7.20	5.35	5.85	5.90	5.80	5.25
15	4.80	......	4.10	6.10	10.70	9.40	7.10	5.70	5.40	5.25	5.20	5.00
16	......	......	4.50	5.75	12.20	8.80	7.60	4.45	5.55	6.40	6.20	4.60
17	......	......	4.50	6.20	12.90	8.70	6.80	4.45	7.70	7.35	5.50	4.70
18	......	5.36	4.65	7.00	12.00	10.20	6.50	5.60	10.70	7.80	6.00	4.80
19	......	......	4.40	5.60	10.60	12.20	6.60	5.25	7.00	7.90	5.50	4.70
20	......	......	4.45	5.45	10.20	11.30	6.70	7.40	10.10	8.95	5.45	4.70
21	......	......	4.55	5.40	9.20	10.50	6.00	7.30	10.80	8.50	5.40	4.65
22	4.67	......	5.50	5.40	8.60	9.10	6.40	8.90	10.30	8.65	4.60	4.85
23	......	......	6.20	5.05	8.60	9.00	5.70	5.85	9.20	8.35	5.20	4.55
24	......	......	7.10	5.30	8.00	8.80	6.10	6.20	8.40	7.90	5.00	4.60
25	......	4.95	7.80	5.30	8.10	8.20	5.75	6.40	6.50	7.55	5.20	4.10
26	......	......	8.90	6.30	7.50	7.30	5.55	8.30	8.50	7.20	5.70	4.80
27	......	......	10.40	5.50	7.70	7.50	4.90	5.30	6.00	7.10	6.05	4.60
28	......	......	11.80	4.80	7.00	8.70	4.45	5.00	6.00	7.00	5.70	4.65
29	5.17	......	13.20	5.05	7.10	7.80	5.30	7.80	7.90	6.60	5.70	4.55
30	......	......	13.60	4.85	7.20	5.75	4.45	6.20	6.20	6.50	5.00	4.70
31	......	......	12.90	......	6.90	......	4.35	6.20	......	6.40	......	4.70

[a] Ice 2.0 feet thicker at gage; 1.0 foot in middle of channel.
[b] Ice 2.5 feet thick at gage; 2.5 feet in middle of channel.
[c] River frozen December 28 to 31.
[d] Ice 2.0 feet thick at gage; 2.0 feet in middle of channel.
[e] River frozen entirely across at gage January 1 to February 28; March 1 to 17, ice gradually disappeared. Thickness of ice, 2 to 2.5 feet. Gage heights are to water surface in a hole in the ice.

Rating table for Chippewa River near Eau Claire, Wis., from November 30, 1902, to March 12, 1903.[a]

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
3.2	840	4.0	1,985	4.7	3,370	5.4	5,150
3.3	940	4.1	2,165	4.8	3,610	5.5	5,410
3.4	1,055	4.2	2,345	4.9	3,850	5.6	5,670
3.5	1,190	4.3	2,535	5.0	4,110	5.7	5,930
3.6	1,335	4.4	2,735	5.1	4,370	5.8	6,190
3.7	1,490	4.5	2,940	5.2	4,630	5.9	6,450
3.8	1,655	4.6	3,150	5.3	4,890	6.0	6,710
3.9	1,825						

[a] To be used only when river is frozen.

Rating table for Chippewa River near Eau Claire, Wis., from March 12, 1903, to December 1, 1903.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
3.8	2,160	5.7	6,290	7.6	11,310	11.0	23,310
3.9	2,340	5.8	6,530	7.7	11,610	11.2	24,070
4.0	2,530	5.9	6,770	7.8	11,910	11.4	24,830
4.1	2,730	6.0	7,010	7.9	12,210	11.6	25,500
4.2	2,930	6.1	7,270	8.0	12,510	11.8	26,350
4.3	3,130	6.2	7,530	8.2	13,150	12.0	27,110
4.4	3,330	6.3	7,790	8.4	13,790	12.5	29,010
4.5	3,540	6.4	8,050	8.6	14,450	13.0	30,910
4.6	3,760	6.5	8,310	8.8	15,130	13.5	32,810
4.7	3,980	6.6	8,570	9.0	15,810	14.0	34,710
4.8	4,200	6.7	8,830	9.2	16,530	14.5	36,610
4.9	4,420	6.8	9,090	9.4	17,250	15.0	38,510
5.0	4,640	6.9	9,350	9.6	17,990	15.5	40,410
5.1	4,860	7.0	9,610	9.8	18,750	16.0	42,310
5.2	5,090	7.1	9,890	10.0	19,510	16.5	44,210
5.3	5,330	7.2	10,170	10.2	20,270	17.0	46,110
5.4	5,570	7.3	10,450	10.4	21,030	17.5	48,010
5.5	5,810	7.4	10,730	10.6	21,790	18.0	49,910
5.6	6,050	7.5	11,010	10.8	22,550		

Rating table for Chippewa River near Eau Claire, Wis., from January 1 to December 31, 1904.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
4.0	1,780	5.1	4,390	6.4	8,100	9.0	16,680
4.1	1,980	5.2	4,660	6.6	8,720	9.5	18,380
4.2	2,180	5.3	4,930	6.8	9,350	10.0	20,080
4.3	2,390	5.4	5,200	7.0	9,990	10.5	21,780
4.4	2,610	5.5	5,480	7.2	10,650	11.0	23,480
4.5	2,840	5.6	5,760	7.4	11,310	11.5	25,210
4.6	3,080	5.7	6,040	7.6	11,970	12.0	26,960
4.7	3,330	5.8	6,320	7.8	12,630	13.0	30,500
4.8	3,590	5.9	6,610	8.0	13,290	14.0	34,480
4.9	3,850	6.0	6,900	8.5	14,980	15.0	40,000
5.0	4,120	6.2	7,490				

Rating table for Chippewa River near Eau Claire, Wis., from January 1 to December 31, 1905.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
3.50	750	5.40	4,830	7.60	10,290	11.40	22,410
3.60	960	5.50	5,050	7.80	10,870	11.60	23,160
3.70	1,170	5.60	5,280	8.00	11,450	11.80	23,950
3.80	1,380	5.70	5,510	8.20	12,030	12.00	24,750
3.90	1,590	5.80	5,740	8.40	12,610	12.20	25,550
4.00	1,800	5.90	5,970	8.60	13,200	12.40	26,350
4.10	2,010	6.00	6,200	8.80	13,800	12.60	27,150
4.20	2,220	6.10	6,430	9.00	14,400	12.80	27,950
4.30	2,430	6.20	6,660	9.20	15,000	13.00	28,750
4.40	2,640	6.30	6,900	9.40	15,620	13.20	29,560
4.50	2,850	6.40	7,140	9.60	16,260	13.40	30,690
4.60	3,070	6.50	7,380	9.80	16,920	13.60	31,240
4.70	3,290	6.60	7,630	10.00	17,600	13.80	32,110
4.80	3,510	6.70	7,880	10.20	18,280	14.00	33,000
4.90	3,730	6.80	8,130	10.40	18,960	14.20	33,900
5.00	3,950	6.90	8,390	10.60	19,640	14.40	34,800
5.10	4,170	7.00	8,650	10.80	20,320	14.60	35,700
5.20	4,390	7.20	9,180	11.00	21,000	14.80	36,600
5.30	4,610	7.40	9,720	11.20	21,690		

The above table is applicable only for open-channel conditions. It is based on 15 discharge measurements made during 1904-5. It is well defined between gage heights 5 feet and 13 feet. The table has been extended beyond these limits.

Estimated monthly discharge of Chippewa River at Eau Claire, Wis., 1902 to 1905.

[Drainage area, 6,740 square miles.]

Date.	Discharge.			Run-off.		Rainfall.[a]
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1902.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
November 14-29	....	....	14,835	2.20	1.39	5.82
December 5-31	....	....	2,789	.41	.41	1.92
1903.						
January	3,730	1,190	2,593	.38	.44	.45
February	2,940	995	2,023	.30	.31	.86
March	34,520	840	11,573	1.72	1.98	2.28
April	25,970	8,050	11,240	1.67	1.86	3.07
May [b]	44,970	11,460	24,761	3.67	4.23	6.45
June [c]	36,990	2,070	8,720	1.29	1.44	1.95
July	39,650	4,750	14,698	2.18	2.51	7.70
August	23,500	3,980	8,602	1.28	1.48	5.35
September	51,810	6,170	19,584	2.90	3.24	7.58
October	25,780	6,770	13,524	2.01	2.32	3.57

[a] Rainfall for 1902 and 1903 is the average of the recorded precipitation at the following stations: Butternut, Hayward, Medford, Barron, and Eau Claire; that for 1904 includes the same stations with the addition of Stanley and Prentice.
[b] May 31 estimated.
[c] 1 to 6, inclusive, estimated.

Estimated monthly discharge of Chippewa River at Eau Claire, Wis., 1902 to 1905—Continued.

Date.	Discharge.			Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.	Per square inch.	Depth.	
	Sec.-feet.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
1903.						
November	5,930	2,830	4,562	0.68	0.76	0.96
December	3,980	1,055	2,855	.42	.48	.64
The year	51,810	840	10,395	1.54	21.05	41.77
1904.						
January	....	....	....	....	....	.51
February	....	....	....	....	....	1.05
March 19-31	8,255	1,920	4,622	.686	.332	1.56
April	22,220	7,640	14,550	2.16	2.41	2.01
May	32,900	7,790	16,960	2.52	2.90	4.33
June	24,510	4,390	12,600	1.87	2.09	6.14
July	21,170	1,647	8,525	1.26	1.45	3.13
August	13,790	650	3,778	.561	.647	4.27
September	19,430	2,264	7,801	1.16	1.29	4.86
October	40,400	3,642	15,170	2.25	2.59	5.59
November	9,478	2,748	5,576	.827	.923	.17
December 1-27	2,960	380	2,230	.331	.332	1.79
The year	....	....	....	....	....	35.41
1905.						
March 18-31	....	31,240	2,640	13,510	2.00	1.04
April	....	24,750	3,510	10,184	1.51	1.68
May	....	28,350	5,740	12,666	1.88	2.17
June	....	60,520	5,625	20,368	3.02	3.26
July	....	22,050	2,535	8,626	1.28	1.48
August	....	14,100	2,535	5,867	.870	1.00
September	....	20,320	3,730	8,970	1.33	1.48
October	....	14,250	3,510	8,041	1.19	1.37
November	....	7,380	3,070	5,437	.807	.900
December	....	5,970	2,010	3,821	.567	.654

WATER POWERS.

CHIPPEWA BELOW JUNCTION OF FLAMBEAU RIVER.

Topography and drainage.—The following descriptions of the water powers on Chippewa River between its mouth and the junction with Flambeau River were largely obtained from a manuscript report of a hypsometric survey of this part of the river made by the United States Geological Survey during the summer of 1903.[a] Between the mouth of the river and Chippewa Falls a very careful primary level was run, while between Chippewa Falls and the mouth of the Flambeau, in addition to taking levels, a topographic survey was made of the river bank and the area immediately adjacent. Between the mouth of the Chippewa and that of the Eau Claire, a distance of 48.4 miles, this survey showed that there was a descent at low water of about 106 feet, or about 2.3 feet per mile. Because of the uniformity of this low gradient, and also because of the width of the stream and of the adjacent bottom lands, there are no opportunities for water powers until Eau Claire is reached. Details of descent and apportionment of drainage areas are shown in the following tables:

[a] The survey of that portion of the river between Watkins Landing, Minnesota, and Chippewa Falls, Wis., was under the charge of Geographer J. H. Renshaw. Above Chippewa Falls the work was in charge of Geographer H. M. Wilson.

Profile of Chippewa River from its mouth to sources of East and West branches.[a]

No.	Station.	Distance. From mouth.	Distance. Between points.	Elevation above sea-level.	Descent between points. Total.	Descent between points. Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Reeds Landing	0	0	[b]680.0 / [c]664.0	......	0
2	Shawtown	45.5	45.5	770.0	106.0	2.3
3	Eau Claire River, mouth	48.8	3.3	770.0	.0	.0
	Dalles paper mills:					
4	Foot of dam	49.4	.6	772.0	2.0	3.3
5	Head of dam	49.4	10.0	793.0	21.0	....
	Chippewa Falls:					
6	Foot of dam	64.4	14.5	806.0	14.0	1.0
7	Head of dam	64.4	50.0	839.0	33.0	....
8	Yellow River, mouth	69.9	5.5	852.0	13.0	2.4
	Eagle rapids:					
9	Foot	72.4	2.5	854.0	2.0	.8
10	Head	73.6	1.2	867.0	13.0	10.8
11	Water level	75.1	1.5	871.0	4.0	2.7
12	Rapids, foot	77.4	2.3	881.0	10.0	4.3
	Jim Falls:					
13	Foot	80.1	2.7	901.0	20.0	7.4
14	Head	81.0	.9	936.0	35.0	39.0
	Colton rapids:					
15	Foot	82.3	1.3	942.0	6.0	4.6
16	Head	83.6	1.3	945.0	3.0	2.3
17	Bob Creek	87.3	3.7	954.0	9.0	2.4
	Chevalley rapids:					
18	Foot	90.1	2.8	961.0	7.0	2.5
19	Head	91.3	1.2	966.0	5.0	4.2
	Brunett Falls:					
20	Foot	91.4	.1	967.0	1.0	10.0
21	Head	92.4	1.0	993.0	26.0	26.0
22	Fisher River, mouth	93.9	1.5	995.0	2.0	1.3
	Holcombe rapids:					
23	Foot	97.1	3.2	1,004.0	9.0	2.7
24	Foot of dam	97.6	.5	1,020.0	16.0	32.0
25	Head of dam, water level	97.6	.0	1,036.0	16.0	....
26	Deertail Creek, mouth	104.1	6.5	1,036.0	.0	.0
27	Flambeau River junction	107.7	3.6	1,050.0	14.0	4.0
28	Bruce, sec. 28, T. 32 N., R. 6 W	124.2	16.5	1,059.0	9.0	.5
29	East and West branches junction	162.7	38.5	1,280.0	221.0	5.6
	EAST BRANCH.					
30	Goose Eye rapids head (foot Little Chief Lake)	164.7	2.0	1,323.4	43.4	21.7
	Snaptail rapids (Hunters Lake):					
31	Foot	166.7	2.0	1,325.2	1.8	.9
32	Head	168.2	1.5	1,368.8	43.6	29.0
33	Blaisdells Lake	170.7	2.5	1,374.5	5.7	2.3
	Cedar rapids:					
34	Foot	173.2	2.5	1,404.0	29.5	11.8
35	Head	175.7	2.5	1,420.0	16.0	6.4
36	Bear Lake	178.2	2.5	1,432.9	12.9	5.1

[a] Authority: Nos. 1, Mississippi River Commission; 2-27, U. S. Geol. Survey; 28, David Kirk; 29-47 U. S. engineers.
[b] High water.
[c] Low water.

Profile of Chippewa River from its mouth to sources of East and West branches.—Continued.

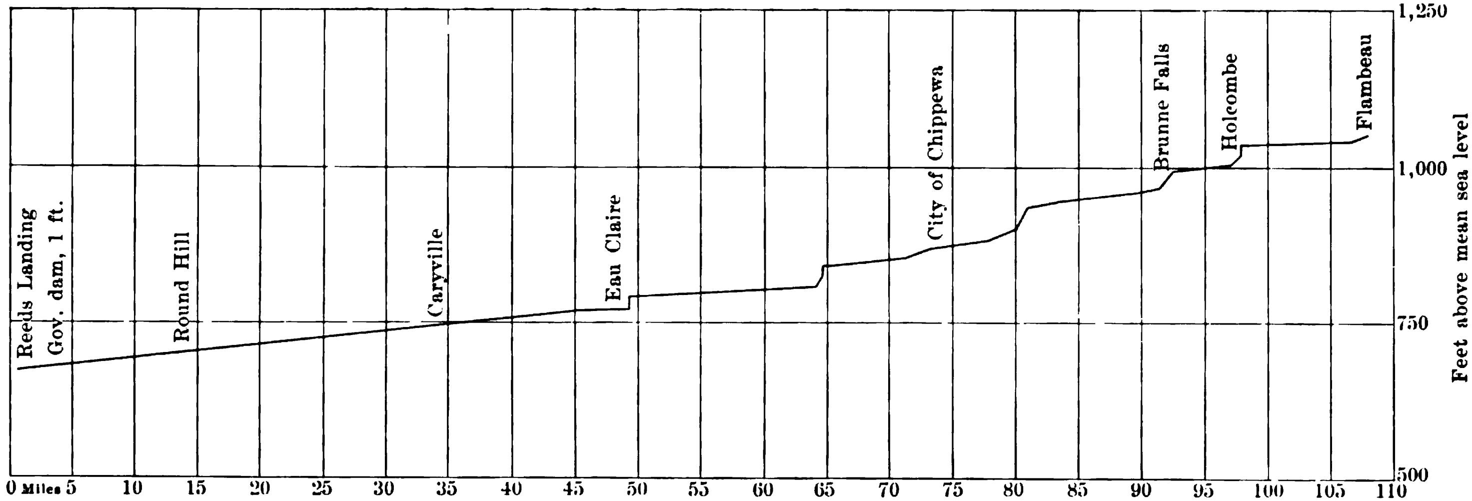
No	Station.	Distance.		Elevation above sea-level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
	EAST BRANCH.—Continued.	*Miles.*	*Miles.*	*Feet.*	*Feet.*	*Feet.*
37	River, water level	181.7	3.5	1,442.0	9.1	2.6
38	Pelican Lake	186.7	5.0	1,462.0	20.0	4.0
39	River, water level, sec. 19, T. 42 N., R. 2 W	190.2	3.5	1,463.8	1.8	.5
40	Glidden Station	201.7	11.5	1,509.3	45.5	4.0
41	Source of river	223.7	22.0±	....	....	......
	WEST BRANCH.					
42	Proposed U. S. dam	164.5	1.8	1,286.0	6.0	3.3
43	Pakwawang Lake	168.7	6.0	1,287.2	1.2	.2
	Moose Lake:					
44	Proposed U. S. dam	178.7	10.0	1,358.8	71.6	7.2
45	Water level	178.7	.0	1,361.9	3.1	......
46	Partridge Crop Lake	185.7	7.0	1,384.4	22.5	3.2
47	Source of river	205.7	20.0±	....	....	......

Distances and drainage areas of Chippewa River.

River.[a]	Distance from the junction of East and West branches, map measure.	Drainage area above station.
	Miles.	*Sq. miles.*
East and West branches (junction)	0	757
Court Oreilles	14	[illegible]
Thornapple (above mouth)	36	[illegible]
Flambeau:		
Above mouth	53	1,777
Mouth	53	3,751
Yellow:		
Above mouth	90	4,928
Mouth	90	5,384
Eau Claire:		
Above mouth	113	5,768
Mouth	113	6,639
Red Cedar:		
Above mouth	142	7,004
Mouth	142	8,981
Chippewa	165	9,573

[a] Station is at mouth of river, unless otherwise stated.

Eau Claire.—The first dam site is located about 2½ miles below the mouth of Eau Claire River. According to a recent survey by the city engineer, a head of 7 feet could be obtained here. On account of its proximity to the city of Eau Claire, this power would have especial value. Before improvement there were two rapids in the river between Eau Claire and Chippewa Falls, one 1.25 miles above the Eau Claire, called the Lower Dalles, with a descent of 10½ feet in a little over 2 miles; the other about 4 miles below Chippewa Falls, called the Upper Dalles, with a descent of 9 feet in about 2 miles.



PROFILE OF CHIPPEWA RIVER FROM REEDS LANDING, MINN., TO FLAMBEAU, WIS.

The dam 2 miles above Eau Claire, owned by the Dells Paper and Pulp Company, is of the square-timber, crib type on a sandstone foundation. It is about 600 feet long, 19 feet high, 3 feet wide at the top, and with a base of about 8 feet. Eight splash boards are used on the crest when necessary, giving a head of 26 feet. It would be possible to increase the height of the dam so as to develop 32 feet, and a bill authorizing this increase is now (March, 1905) pending before the State legislature. Such a dam would back the water nearly to Chippewa Falls, 15 miles above, greatly adding to an already very large pondage. This is the most important manufacturing plant on the river. The turbine installation is reported as follows:

Dells Paper and Pulp Company's turbine installation, 2 miles above Eau Claire.

Purpose.	Horsepower.
Paper mill	1,396
Pulp mill	4,918
Electric light and power	1,632
Waterworks	300
	8,246

Chippewa Falls.—In the 14½ miles between the Dells dam and Chippewa Falls no power sites are found, the river having a nearly uniform slope of 1 foot to the mile. At the latter place, however, is a wooden dam 800 feet long, with a head of 30 feet, owned by the Chippewa Falls Lumber and Boom Company. This dam supplies power for a large sawmill and also a plant furnishing the city of Chippewa Falls with water and electric light. The dam could be made several feet higher, as the local conditions are favorable, but this would interfere with a proposed plant at Paint Creek rapids, 2½ miles upstream, to which point the water now backs. The owners have developed only about 20 feet of head, but this could be increased to the full head of 30 feet by blasting and cleaning out the river to the wagon bridge below. The power and light company leases 1,000 horsepower, using a head of 29 feet.

The next rapids, known as Paint Creek rapids, are 2½ miles above the Chippewa Falls dam. A flooding dam 526 feet long, with a crest 10½ feet above low water, was formerly maintained here. A dam about 800 feet long, with a head of 14 feet, could be constructed at the foot of the rapids at this point. The banks and bed appear to be sand, intermingled with large bowlders. Stone for construction is abundant and near at hand, and it is likely that a rock foundation could be easily obtained.

Eagle rapids, 4½ miles farther upstream, in lot 3, sec. 16, T. 29 N., R. 8 W., is a good site for a dam, owned by F. G. & C. A. Stanley, of Chippewa Falls. A dam 60 feet long and 20 feet high would back the water three fourths of a mile above the city of Chippewa Falls, where O'Neils Creek enters from the west. One mile above the mouth of O'Neils Creek, in sec. 10, T. 29 N., R. 8 W., is a gorge 700 feet wide, where a 25-foot dam would have solid sandstone for foundations and abutments and would back the water almost to the foot of Jim Falls, 5 miles above. Such a dam would develop 5,000 theoretical horsepower.

Jim Falls.—Near the small station of Jim Falls, on the Chicago and Northwestern Railway, occurs the best opportunity for water-power development on Chippewa River. It is owned by W. L. Davis, of Eau Claire. Formerly an old flooding dam was located here. The river flows over a series of granite ledges 1 to 4 feet high, while the banks seem to be of the same rock, covered by a few feet of sandy soil. This power is now under development, a company having purchased all the land needed. The proposed dam, 28 feet high, will be located at the head of the rapids. It is designed to furnish power for a pulp mill near the foot. The total head obtained by this plant will be 55 feet. Fig. 4 shows the plan of the proposed development. Water is to be conducted from the dam by a canal extending on the left bank for a distance of about 5,000 feet to high bluffs 100 feet from the river bank. The power house will be on the river bank immediately below. The dam will back the water nearly to Brunett Falls, 9½ miles above, and will cover the Colton and Chevalley rapids.

Brunett Falls.—One of the best powers on Chippewa River, and one most cheaply developed, is found at Brunett Falls (Pl. III, *B*), located in sec. 18, T. 31 N., R. 6 W. It belongs

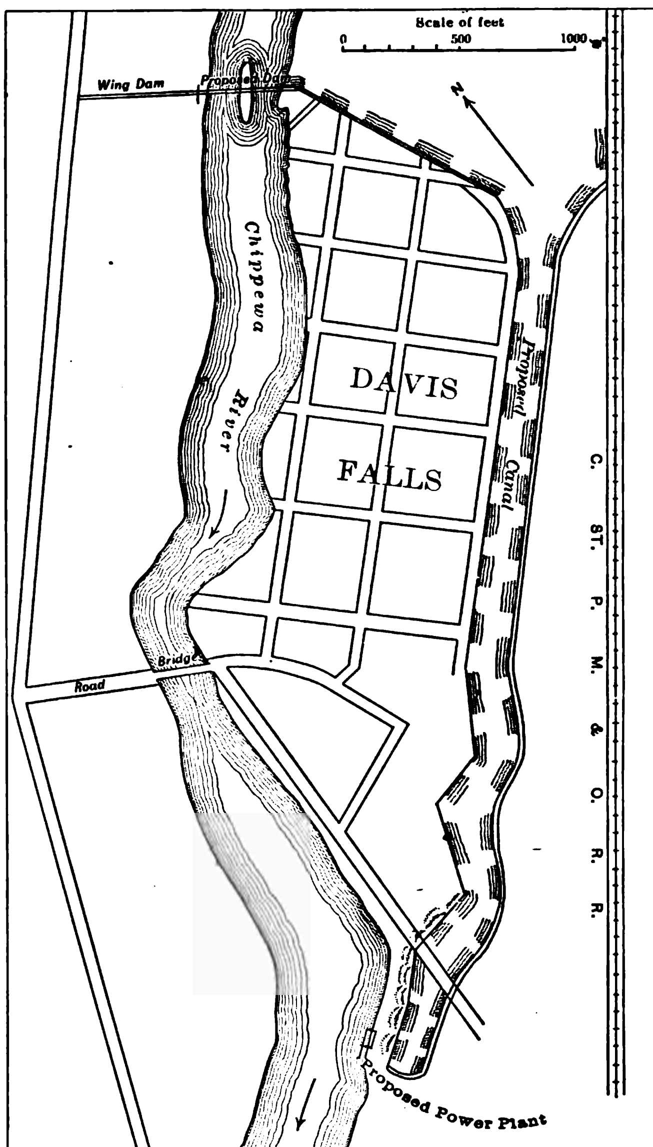


FIG. 4.—Plan of proposed water-power development at Jim Falls (Davis Falls).

to Cornell University, which also owns the adjacent land as well as the water rights. The best location for the dam would be about 650 feet above the foot of the rapids, where a 35-foot dam would back the water up to the rapids at Holcombe, 5½ miles above. The river at the dam site is narrow (70 or 80 feet), while the banks are high, granite ledges.

A dam here would create a large reservoir. It is stated that the plans contemplate a dam 200 feet long. A steel wagon bridge has recently been built across the river immediately below the dam site.

Holcombe dam.—The next power is at Holcombe, about 3 miles below the mouth of Jump River, where the Chippewa Falls Lumber and Boom Company maintains a timber dam, with a head of about 17 feet. This is the third dam that has been built here, the others having been washed out by freshets. As the lumber interests are fast declining, the present dam is being allowed to decay. For power purposes it should be replaced by a more substantial structure. The river here has a rock bottom, with rather low clay sides, but an 18-foot dam could be constructed on the site of the present structure, which, together with a 15-foot dam at the foot of the rapids just below (sometimes called Little Falls), would develop about all the head at this point and would not flood any more valuable lands above. This would back the water above Deertail Creek and furnish considerable storage.

Mouth of Flambeau.—Of the 14 feet of descent in Chippewa River between Holcombe and the mouth of the Flambeau 10 feet are concentrated in the first mile below the latter point. It is very likely that a dam on this reach would easily develop 15 feet of head.

It is worthy of note that all the water powers on Chippewa River thus far described are reached by one or more railroads. Because of their availability many of the above powers are likely to be developed in the near future. Their importance is emphasized by the following statement: Of the 244 feet descent in the Chippewa between Chippewa Falls and the mouth of the Flambeau, 116 feet are concentrated in 5 falls and rapids. The building of 10 dams would economically develop a total of 213 feet head in this distance of 43 miles. When fully developed these powers will rival in importance the extensive developments on lower Fox River between Appleton and Green Bay.

BRANCHES AND UPPER WATERS.

Topography and drainage.—The following statements in regard to the water powers of upper Chippewa River, not being based on a hydrographic survey, are necessarily incomplete. Statements concerning profile, etc., are based on the survey and maps of this region made in 1880 by United States engineers in connection with the reservoir surveys. Distance and drainage area data are shown in the following table:

Length and drainage area of the upper tributaries of Chippewa River.

River.	Length (map measure).	Drainage area.
	Miles.	*Sq. miles.*
West Branch of Chippewa	35	480
East Branch of Chippewa	60	278
Court Oreilles	20	176
Flambeau	155	1,983
Jump	65	721
Yellow	65	458
Eau Claire	65	899
Red Cedar	95	1,957

In the 16½ miles between the mouth of the Flambeau and Bruce Chippewa River descends only 0.8 foot per mile, but in the 38½ miles between Bruce and the confluence of East and West branches of the Chippewa, in sec. 2, T. 39 N., R. 6 W., the river descends 216 feet, an average of 5.6 feet per mile. This steep gradient is certain to produce many good powers. This reach is, however, devoid of railroads except a few logging roads. One of these undeveloped powers, called Belills Falls, is located in sec. 26, T. 38 N., R. 7 W. Its owner, the John Arpin Lumber Company, reports that this power is capable of producing a head of about 30 feet. It is near Radison, on the Chicago, St. Paul, Minneapolis and Omaha Railway.

East Branch of Chippewa.—Three important rapids occur in East Branch of Chippewa River. Between Little Chief Lake and the confluence of East and West branches, a distance of 2.7 miles, there is a descent of 43 feet. Between these points there is a series of rapids, "the bed of the river being literally paved with bowlders. The banks are from 10 to 20 feet high and the drift a reddish clay." These are known as the Goose Eye rapids. Two or three dams could develop a head of about 40 feet.

Above Hunters Lake, in secs. 22 and 23, T. 40 N., R 5 E., occur the Snaptail rapids, with an aggregate descent of 43.6 feet.

Cedar rapids, the last of importance on this branch, with a descent of 16 feet, are located in sec. 9, T. 40 N., R. 4 W., and in the 2 miles above. The total descent between Blaisdell and Bear lakes is about 58 feet, all in a distance of 7½ miles. Between Bear and Little Chief lakes the banks vary from 4 to 50 feet in height. A logging dam has been maintained at the head of the rapids, in sec. 26, T. 41 N., R. 4 W., which had a height of 10 feet. Measurements made here by United States engineers on June 20 and July 12, 1879, with the river respectively 0.6 and 2.1 feet above low-water mark, showed a discharge of 381 and 472 second-feet. The river at this point is 153 feet wide.

West Branch of Chippewa.—West Branch of the Chippewa River has a drainage area of 480 square miles, or 200 square miles more than East Branch, but its descent is considerably less rapid. The river has its source in several large lakes at about 1,380 feet above sea level. The first undeveloped power is located about 1½ miles above the confluence of the two branches, in sec. 34, T. 40 N., R. 6 W., where the hills approach within 900 feet. The river at this point has a width of 121 feet, and here United States engineers made surveys for a dam with a head of 25½ feet, which gave a very large reservoir area. A 15-foot head could probably be obtained at reasonable expense. Four measurements made by United States engineers on August 6, 1879, at a stage only 0.2 foot above low water gave a mean discharge of 360 second-feet, or 0.75 second-feet per square mile of drainage area. This large low-water run-off is double that estimated for this drainage area The excess may be explained by the steadying action of the large lakes near the headwaters of this river.

In the 10 miles between Moose and Pakwawang lakes West Branch descends 71.6 feet, including a series of rapids with sluggish water between. The banks are generally from 20 to 30 feet high, with clay soil.[a]

Court Oreilles River.—Court Oreilles River has its source at an elevation of 1,287 feet in a lake of the same name. The group of lakes forming its headwaters have a total area of about 16 square miles. A dam at this outlet would need to have a length of 260 feet to secure a head of 5 feet, and would store a supply sufficient to deliver 255 second-feet for ninety days at times of low water. The river is from 50 to 60 feet wide, and in the first 3 miles of its course is sluggish. Thence to its mouth it furnishes a series of rapids, with still reaches between. The most important rapids, known as the Court Oreilles, are situated within 3 miles from the mouth of the river, which at this point flows over ledges of the pre-Cambrian rocks. The river is crossed at its middle point by the Chicago, St. Paul, Minneapolis and Omaha Railway, where the water surface has an elevation of 1,240 feet. This shows a descent of 47 feet in 10 miles between this point and the lake. The lower half of the river is reached by the above railway. Unlike either East Branch, West Branch, or any other neighboring branches of the Chippewa, Court Oreilles River drains a region with a very open sandy soil. A measurement made by United States engineers, October 25, 1879, at a stage 0.3 foot above low water, showed a discharge at the mouth of Lake Court Oreilles of only 25 second-feet from a drainage area of 114 square miles. It seems likely that, because of the character of the soil, part of the run-off escapes underground to the west into Namekagon River.

[a] Rept. Chief Eng. U. S. Army, 1880, p. 1562.

Upper powers.—Because of their present isolation from railroads, the chief use of dams which have been maintained on the upper headwaters of Chippewa River would lie in their operation as reservoirs to improve the powers below. Their location is shown in the following table:

Dams on upper waters of Chippewa River.[a]

	Location.	Dimensions.[b] Height.	Length.	Reservoir capacity.
		Feet.	*Feet.*	*Cubic feet.*
	Chippewa River:			
1	NW. ¼ sec. 28, T. 32 N., R. 6 W	21	625	133,333,000
2	Sec. 22, T. 33 N., R. 8 W	9		153,331,000
3	Sec. 28, T. 32 N., R. 6 W	17		334,536,000
	West Branch:			
4	SW. ¼ SW. ¼ sec. 32, T. 42 N., R. 5 W	8	123	
5	Sec. 12 T. 42 N., R. 5 W	*20	*300	
6	NE. ¼ SE. ¼ sec. 14, T. 41 N., R. 6 W	7	347	430,000,000
7	Outlet to Pokegama Lake, NW. ¼ NW. ¼ sec. 32, T. 40 N., R. 6 W	8	108	
8	Little Chief River, NE. ¼ NE. ¼ sec. 26, T. 40 N., R. 7 W	6	142	
9	East Branch, NW. ¼ SE. ¼ sec. 26, T. 41 N., R. 4 W	10	564	300,000,000
	Thornapple River:			
10	Sec. 10, T. 35 N., R. 6 W	*18	*800	
11	Sec. 4, T. 36 N., R. 5 W	*18	*400	
12	Sec. 20, T. 38 N., R. 4 W	*12	*250	
13	Sec. 4, T. 38 N., R. 4 W	*15	*250	
14	Brunett River, sec. 17, T. 38 N., R. 5 W	*15	*325	
15	Torch River, sec. 16, T. 42 N., R. 4 W	*20	*300	

[a] Authority: Nos. 1-4 and 6-9, United States engineers; 5 and 10-15, Chippewa Lumber and Boom Company.

[b] Dimensions marked with an asterisk (*) were estimated by the owner, The Chippewa Lumber and Boom Company.

TRIBUTARIES OF CHIPPEWA RIVER.

FLAMBEAU RIVER.

Drainage and water powers.—In size of drainage area Flambeau River ranks first among the tributaries of the Chippewa. Indeed, because of its central location in the drainage basin, it might properly be regarded as the prolongation of the main stream itself. Regardless of its size, however, its water power must, in large part, continue for some time unused, because of its forested location and its lack of railroad facilities. The settling of this area will eventually justify the extension of present railroads and the building of new ones. Flambeau River is crossed near its mouth (at Ladysmith) by the Minneapolis, St. Paul and Sault Ste. Marie Railway, near its center (at Park Falls) by the Wisconsin Central Railway, and at its upper headwaters by the Chicago and Northwestern Railway. Between Park Falls and Ladysmith is a reach of 70 miles unserved by railroad, and yet with no point at a greater distance than 15 miles from the present railroads. It is significant that the two points with transportation facilities, Ladysmith and Park Falls, have established large paper and pulp mills and other manufactories. The unusually steady flow, the soft water, and the proximity of almost unlimited quantities of pulp wood should make this river a center of the paper and pulp industry. Transportation alone is lacking.

Flambeau River has its source in the largest number of lakes and connecting swamps with the greatest aggregate storage capacity of any river in the State. This storage capacity has been increased in many cases by lumbering dams built at the lake outlets, but as yet many opportunities for the storing of surplus waters remain unimproved. These lakes lie in the highest portion of the State, at elevations varying from 1,560 to 1,650 feet or more above the sea. The levels show that the river descends 570 feet in a distance of 150 miles, or about 3.8 feet per mile. A large part of this fall is known to be concentrated at numerous falls and rapids. In the 19 miles between the mouth of the river and Ladysmith the descent is 42 feet. A company has recently been formed to construct a dam with a head of 20 feet at a point 6 miles below Ladysmith, in sec. 18, T. 34 N., R. 6 W., and the work of construction is already begun. The next developed power above the mouth is found at Ladysmith, where a timber dam 350 feet long develops a head of 16 feet. This power is used to run a paper and pulp mill and also for the manufacture of wooden ware.

There are no developed powers on Flambeau River for 70 miles above Ladysmith, but a fall of 353 feet in this distance insures many undeveloped powers. Two of these, Little Falls and Big Falls, are of special importance. The former is located in the NW. ¼ sec. 21, T. 35 N., R. 5 W., and is owned by A. J. McGilvary and B. D. Viles, of Chippewa Falls. A 15-foot dam at the head of the first rapids would give a head of about 25 feet at the foot of the rapids a short distance below. Big Falls, owned by the John Heim Company, of Tony, Wis., is located 6 miles above Little Falls, in sec. 35, T. 36 N., R. 5 W. There is a descent of 25 feet here in a short distance, concentrated in three pitches. A view of one is shown in Pl. V, *A*. No accurate survey has been made of either fall, but the owner of Big Falls estimates that a 25-foot dam at the head of the rapids and a canal about five-eighths of a mile to the end of the rapids would give a 60-foot head. Both falls occur over ledges of pre-Cambrian crystalline rock.

At Park Falls the Flambeau Paper Company has constructed two dams; one, half a mile above the railroad crossing, in sec. 13, T. 40 N., R. 1 W., and one about a mile below, in sec. 25, T. 40 N., R. 1 W. Each dam furnishes an average head of 16 feet. The upper plant has installed 13 turbines, rated at 1,300 horsepower, while at the lower plant about 1,100 horsepower has been installed.

There are other rapids in secs. 28, 32, and 33, T. 41 N., R. 1 E., and levels taken by United States engineers showed a fall here of 24 feet in 2 miles. Again, in secs. 3 and 4, T. 41 N., R. 2 E., below the junction of Turtle and Flambeau rivers, is a similar fall of 25 feet. Above this point the river is much smaller and has lower gradient, though bowlder rapids are of frequent occurrence.

The lack of railroad transportation on this watershed will postpone the utilization of its many large water powers until the region is more thickly settled and better served by railroads.

Profile.—No Government surveys have been made in the 46 miles above Big Falls, so that reliable data regarding water powers along this portion of the river are almost entirely lacking. Above Park Falls United States engineers have run levels in connection with the reservoir surveys, thus furnishing valuable hypsometric data. Information concerning the river profile from mouth to headwaters, with the exception noted above, is fairly complete, and is summarized in the following table:

Profile of Flambeau River from its mouth to Boulder Lake.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river	0.0	....	1,050.0	....	....
2	SW. ¼ sec. 34, T. 34 N., R. 7 W	7.0	7.0	1,064.0	14.0	2.0
3	Ducomon rapids, NW. ¼ sec. 23, T. 34 N., R. 7 W	11.0	4.0	1,070.0	6.0	1.5
4	New dam, foot of rapids	15.0	4.0	1,081.0	11.0	2.7
5	SW. ¼ sec. 1, T. 34 N., R. 6 W	15.75	.75	1,088.4	7.4	10.0
6	Ladysmith, below dam	24.25	8.5	1,099.0	10.6	1.25

[a] Authority: No. 1-26, U. S. Geol. Survey; 27-30, U. S. engineers. Because of an error in the assigned elevation of the initial bench mark, 15 feet is added to the U. S. engineer elevation to correct to sea-level datum.

A. LOWER PITCH OF BIG FALLS, FLAMBEAU RIVER.

B. COPPER FALLS, BAD RIVER.

Profile of Flambeau River from its mouth to Boulder Lake—Continued.

No	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
7	Ladysmith, above dam	24.25	0.0	1,115.3	16.3	....
8	NW. ¼ sec. 25, T. 35 N., R. 6 W	28.0	3.75	1,115.4	.1	....
9	Little Falls, foot of	32.0	4.0	1,131.4	16.0	4.0
10	Little Falls, head of (sec. 21, T. 35 N., R. 5 W.)	32.8	.8	1,147.4	16.0	20.0
11	NE. ¼ sec. 15, T. 35 N., R. 5 W	36.8	4.0	1,166.7	19.3	4.8
12	Big Falls, foot of (NW. ¼ sec. 2, T. 35 N., R. 5 W.)	40.3	3.5	1,177.0	10.3	[a] 3.0
13	NW. ¼ sec. 8, T. 39 N., R. 1 W	86.2	45.9	1,421.8	244.8	5.3
14	South line sec. 33, T. 40 N., R. 1 W	91.2	5.0	1,429.6	7.8	1.5
15	Sec. 35, T. 40 N., R. 1 W., west line of	94.2	3.0	1,438.0	8.4	2.8
16	Below dam, sec. 25, T. 40 N., R. 1 W., west line of	95.0	54.7	1,454.0+	....	....
17	Above dam	95.0	0.0	1,470.0+	....	....
18	Park Falls railroad bridge, west line sec. 24, T. 40 N., R. 1 W	96.6	1.6	1,470.0	....	....
19	Below tail race upper dam, Park Falls	98.3	.5	1,466.8	2.8	5.6
20	Above upper dam, Park Falls	98.5	.2	1,481.0	14.2	....
21	Backwater, upper dam	104.3	5.8	1,482.5	1.5	.2
22	Center sec. 28, T. 41 N., R. 1 E	107.1	2.8	1,499.2	16.7	6.0
23	Sec. 12, T. 41 N., R. 1 E., W. ¼ stake	112.5	5.4	1,510.8	11.6	2.0
24	Sec. 4, T. 41 N., R. 2 E., W. ¼ stake	115.8	3.3	1,516.0	6.2	1.8
25	Turtle River, mouth	119.0	3.2	1,541.4	25.4	7.6
26	Manitowish River, junction of Bear Creek	134.0	15.0	1,568.0	26.6	1.8
27	Rest Lake, mouth of (sec. 8, T. 42 N., R. 5 E.)	146.0	12.0	1,587.0	19.0	1.6
28	Island Lake, inlet of	153.5	7.5	1,592.0	5.0	.66
29	Boulder Lake	163.0	9.5	1,625.0	33.0	3.5

Rainfall and run-off.—Like all the northern rivers of the State the minimum flow of Flambeau River occurs in severe midwinter weather, or during very dry summers in the months of July and August. At present there are not sufficient discharge data covering periods when the river is frozen to construct an accurate rating curve for such periods. Because of the extensive forest and the numerous lakes and swamps, an ordinary flow of 0.8 second-foot per square mile of drainage area would seem conservative. By the proper regulation of present dams at the headwaters it is likely that this discharge could be considerably increased.

In February, 1903, the United States Geological Survey established an observing station at the Ladysmith dam, and has taken daily gage readings since. Discharge measurements are taken by current meters and are being continued so that in time an accurate estimate of the river's discharge will be available. The following tables give such daily observations:

discharge measurements, and computations as have become available since the establishment of the station, and also a record of rainfall for the corresponding period:

Discharge measurements of Flambeau River near Ladysmith, Wis., for 1903, 1904, and 1905.

Date.	Hydrographer.	Width.	Area of section.	Mean velocity.	Gage height	Discharge.
		Feet.	*Square feet.*	*Feet per second.*	*Feet.*	*Second-feet.*
1903.						
February 13[a]	L. R. Stockman	325	472	1.64	16.20	773
March 19[b]	do	366	1,871	1.77	18.95	3,312
April 8	do	349	1,330	2.80	17.40	3,727
May 6	do	361	1,927	3.70	18.97	7,113
June 16	do	342	703	1.91	16.00	1,345
July 11	do	342	1,430	2.95	18.10	4,222
August 21	do	342	995	2.69	16.85	2,681
September 10	E. C. Murphy	364	1,579	3.36	18.05	5,303
October 23	L. R. Stockman	348	1,271	3.07	17.21	3,996
1904.						
May 16	E. Johnson, jr.	350	1,333	3.15	17.88	4,203
June 3	do	350	1,448	2.99	17.45	4,321
August 29	do	349	733	2.07	16.06	1,517
September 20	do	343	702	2.21	16.01	1,554
October 12	F. W. Hanna	364	1,653	3.37	18.58	5,588
1905.						
April 8	S. K. Clapp	129	1,537	3.49	18.27	5,367
May 23	do	357	1,292	2.69	17.60	3,474
June 14	M. S. Brennan	354	1,232	2.67	17.35	3,288
July 12	do	353	1,015	2.54	16.80	2,576
August 12	do	345	623	1.84	15.66	1,144
September 23	F. W. Hanna	353	1,404	3.02	17.75	4,236

[a] Frozen. [b] Log jam below.

Mean daily gage height, in feet, of Flambeau River near Ladysmith, Wis., February 15, 1903, to December 31, 1905.

Day.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1903.											
1		16.15	17.00	18.30	19.80	15.65		17.00	17.20	16.00	15.90
2		16.60	16.80	18.40	19.65	16.15		16.70	17.30	16.25	15.75
3		16.50	16.90	18.60	18.95	17.25		16.80	17.60	15.85	15.95
4		16.10	16.90	19.05	18.60	18.10		16.30	19.65	15.85	15.60
5		16.30	17.05	19.10	18.10	18.90		16.80	19.70	15.85	15.80
6		16.50	16.40	19.10	17.55	19.05		16.90	19.35	15.85	15.80
7		16.60	16.90	19.10	17.55	19.20		16.90	19.25	15.90	16.95
8		16.10	17.45	18.80	17.30	18.85		17.30	19.25	15.65	15.80
9		16.50	17.35	18.70	16.95	18.70		18.20	19.30	15.70	14.70
10		16.50	17.25	17.95	16.60	18.60	18.20	18.20	19.35	15.85	16.50
11		16.05	17.25	18.25	16.75	18.75	18.00	18.00	18.95	16.00	16.35
12		16.45	17.30	18.80	16.80	18.55	17.90	18.40	18.65	15.85	16.30
13		16.35	17.25	19.55	16.30	18.30	17.80	19.00	18.45	15.85	16.50
14		16.35	17.50	19.80	16.15	17.85	17.70	19.80	18.25	15.80	16.45
15	16.00	16.60	17.40	19.65	16.35	17.70	17.50	20.40	17.90	15.80	16.55
16	16.10	16.15	17.20	19.55	16.50	17.65	17.30	20.50	17.85	15.75	16.35
17	16.05	16.20	16.95	19.40	16.05	17.60	17.30	20.50	17.80	15.80	16.40
18	16.00	16.30	17.00	19.45	16.05	17.35	17.20	20.30	17.50	15.00	16.70
19	15.90	18.25	16.90	19.05	15.85	17.35	17.00	20.00	17.35	15.50	16.60

Mean daily gage height, in feet, of Flambeau River near Ladysmith, Wis., February 15, 1903, to December 31, 1905—Continued.

Day.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1903.											
20	16.00	20.35	16.85	19.20	15.80	17.20	17.00	19.70	17.25	15.50	16.67
21	15.90	19.30	16.90	19.25	15.85	17.15	17.10	19.30	17.25	15.45	16.67
22	16.05	18.50	16.65	18.85	15.95	16.70	16.90	18.90	17.15	15.25	16.67
23	16.00	18.45	16.65	19.15	15.65	16.70	16.70	18.50	17.05	15.40	16.66
24	16.25	17.30	17.30	18.90	15.90	16.70	16.80	18.20	17.00	15.65	17.00
25	16.00	17.60	17.30	19.00	15.60	16.80	17.10	18.00	16.80	15.45	16.69
26	15.95	17.25	17.25	19.55	15.85	(a)	16.80	17.70	17.00	15.55	16.66
27	16.40	17.00	17.15	20.60	15.60	(a)	17.00	17.85	16.80	15.85	16.50
28	16.25	17.10	17.20	21.45	15.70	(a)	16.90	17.50	16.55	15.80	16.10
29	....	17.00	17.40	21.45	15.95	(a)	16.80	17.30	16.65	15.80	16.80
30	....	16.75	18.45	21.20	15.80	(a)	16.70	17.20	16.05	15.85	16.80
31	....	16.60	....	21.45	....	(a)	16.80	....	16.20	....	16.70

Day.	Jan.[b]	Feb.[c]	Mar.[b]	Apr.[c]	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.[d]
1904.												
1	16.75	16.75	17.00	16.90	18.70	17.50	17.58	15.15	16.25	(e)	17.25	15.65
2	16.95	16.75	17.05	17.20	18.55	17.40	17.77	15.40	16.30	(e)	17.30	15.05
3	16.75	16.10	17.00	16.80	18.60	17.42	17.70	(f)	16.38	(e)	17.20	15.95
4	16.85	16.80	17.05	16.90	18.35	17.43	19.90	(f)	17.78	(e)	16.90	14.50
5	17.00	16.90	17.15	16.85	18.45	18.00	18.82	15.60	17.65	16.05	16.70	15.15
6	16.50	16.70	16.95	16.80	18.60	18.02	18.88	15.72	17.20	16.05	16.80	15.30
7	16.50	16.80	16.90	17.10	18.60	18.25	18.75	15.13	17.28	16.10	16.30	14.87
8	16.65	16.75	17.15	17.25	18.85	18.27	18.75	15.40	17.30	16.10	16.20	15.78
9	16.65	16.75	16.90	17.05	19.20	17.90	18.05	15.75	17.03	17.05	16.17	15.57
10	16.70	16.65	17.50	17.00	19.15	17.22	17.95	15.92	16.00	18.70	16.15	15.55
11	16.60	16.70	17.40	17.10	18.90	17.10	17.70	15.85	16.40	18.65	16.05	15.25
12	16.55	16.95	17.05	17.10	18.80	17.25	17.25	16.00	16.32	18.60	15.55	15.77
13	16.50	17.00	17.30	17.15	18.35	17.15	16.40	15.90	16.45	18.50	15.60	15.55
14	16.70	16.70	17.20	17.05	18.15	17.12	16.30	15.90	16.30	18.43	15.45	15.30
15	16.70	16.90	17.20	17.25	18.01	16.60	16.12	15.85	16.15	18.30	15.82	15.35
16	16.75	16.95	17.00	17.20	17.95	16.55	16.15	16.02	16.05	17.85	15.28	15.45
17	16.60	16.55	17.15	17.10	18.01	16.35	16.03	15.90	16.05	17.20	15.55	15.35
18	16.60	17.10	17.20	17.05	18.01	16.25	16.00	15.85	16.13	16.95	15.72	15.30
19	16.65	16.95	17.15	17.00	18.03	16.32	15.60	15.95	16.15	17.15	15.70	15.50
20	16.30	16.95	17.05	16.85	17.05	15.95	15.80	15.75	16.00	17.25	15.60	15.57
21	16.75	17.55	17.05	16.85	17.03	15.88	15.95	16.00	16.00	17.60	15.45	15.70
22	16.75	16.90	17.15	16.65	17.01	16.15	15.85	16.20	15.90	17.80	15.82	15.72
23	16.75	17.00	16.85	17.20	17.04	15.95	15.80	16.35	15.95	17.75	15.27	15.68
24	16.65	16.60	17.15	17.20	17.06	16.35	15.70	16.45	15.95	17.75	15.55	15.60
25	16.70	17.00	16.95	18.00	18.40	16.55	15.85	16.65	16.40	17.85	15.72	15.65
26	16.70	16.90	16.95	18.40	19.00	16.70	16.15	16.45	16.40	17.75	15.70	15.70
27	16.45	17.00	16.95	18.45	19.40	16.95	16.76	16.45	16.40	17.65	15.55	16.10
28	16.65	16.95	17.15	18.50	19.30	17.05	16.75	16.20	16.45	17.55	15.40	15.75
29	16.60	16.95	17.05	18.50	18.80	17.05	15.70	16.10	16.40	17.65	14.95	15.80
30	16.75	....	16.45	18.90	18.40	17.20	15.25	16.27	16.45	17.70	15.55	16.30
31	16.65	....	17.20	....	17.80	....	15.55	16.15	....	17.22	....	16.40

[a] Chain gage stolen.
[b] Frozen from January 1 to March 30, when ice begins to break. Ice varied from 6 to 18 inches in thickness.
[c] Ice conditions March 31 to about April 10.
[d] Ice conditions during December.
[e] Weight gone.
[f] Key lost; no gage height taken on August 3 and 4.

Mean daily gage height, in feet, of Flambeau River near Ladysmith, Wis., etc.—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1905.												
1	(a)	......	......	18.80	16.8	17.40	17.70	15.80	16.90	16.55	16.40	16.15
2	......	......	......	18.40	16.8	16.25	17.60	15.90	16.60	16.55	16.25	15.90
3	......	......	......	19.00	16.8	16.40	17.50	15.80	16.90	16.35	16.15	15.85
4	......	16.4	16.80	19.20	17.0	16.80	17.60	15.70	17.00	16.25	16.05	16.20
5	......	......	......	18.90	17.3	17.80	18.60	15.75	17.00	16.20	16.20	15.95
6	......	......	......	18.40	17.6	18.70	19.00	15.35	16.90	16.15	16.15	16.10
7	16.3	......	......	18.40	17.6	19.60	19.20	15.35	16.60	15.80	16.20	16.45
8	......	......	......	18.20	17.4	19.30	18.80	15.30	16.70	15.95	16.15	16.15
9	......	......	......	18.10	17.1	18.90	18.50	15.50	16.50	15.80	16.05	16.25
10	......	......	......	17.80	17.4	18.70	18.20	15.30	16.40	15.95	15.95	16.25
11	......	16.5	16.00	17.40	17.5	18.60	17.40	15.40	16.20	15.55	16.05	15.90
12	......	......	......	17.20	17.8	18.60	17.00	15.55	16.20	16.20	16.30	15.80
13	......	......	......	17.20	17.8	17.60	17.00	15.40	16.25	16.15	16.30	15.90
14	16.6	......	......	18.20	18.0	17.50	17.00	15.32	16.25	16.10	16.20	15.70
15	......	......	......	18.10	18.3	17.60	16.70	15.45	16.45	16.40	16.10	15.70
16	......	......	......	17.60	18.2	17.70	16.60	15.60	16.80	16.75	16.25	16.20
17	......	......	......	17.20	18.4	18.00	16.35	15.55	17.10	16.75	16.05	16.40
18	......	16.7	16.90	16.55	18.6	19.70	16.35	15.90	17.05	17.00	15.90	15.70
19	......	......	......	16.55	18.6	19.60	16.40	16.80	17.70	17.65	15.90	15.95
20	......	......	......	16.45	18.4	19.40	16.15	16.90	17.80	17.20	15.85	15.45
21	16.7	......	......	16.55	18.0	19.10	16.10	17.10	18.20	17.65	15.90	15.65
22	......	......	......	17.00	18.0	18.90	16.20	16.90	18.40	17.35	15.70	15.60
23	......	......	......	16.30	17.4	18.60	15.90	16.60	17.80	17.45	15.75	15.65
24	......	......	16.80	16.35	18.0	18.40	15.50	16.45	17.60	17.30	15.60	15.70
25	......	16.6	16.45	16.15	17.6	18.00	15.90	16.80	17.30	17.10	16.00	16.75
26	......	......	16.35	16.05	17.4	18.00	15.75	16.35	17.20	16.85	16.60	16.10
27	......	......	16.25	17.40	17.2	18.00	15.80	16.25	16.80	16.85	16.10	15.60
28	16.6	......	17.10	17.40	18.0	17.70	15.80	16.80	16.80	16.80	16.60	16.00
29	......	......	17.90	17.00	17.8	17.70	15.75	17.00	16.70	16.70	16.15	15.90
30	......	......	18.20	16.80	17.6	17.70	15.70	17.20	16.20	16.55	15.90	16.15
31	......	......	18.60	......	17.4	......	15.55	17.00	......	16.30	......	16.10

a River frozen entirely across January 1 to March 23. March 11-23 there was water on the ice. Gage heights are to water surface in a hole in the ice. The following comparative readings were also made:

Date.	Water surface.	Top of ice.	Thickness.
	Feet.	*Feet.*	*Feet.*
January 7	16.3	16.4	0.7
January 14	16.6	16.8	1.3
January 21	16.7	16.9	1.4
January 28	16.6	16.9	1.5
February 4	16.4	16.9	1.7
February 11	16.5	16.9	1.7
February 18	16.7	17.1	2.0
February 25	16.6	16.9	1.8
March 4	16.8	17.8	1.8

Rating table for Flambeau River near Ladysmith, Wis., from March 19, 1903, to December 1, 1903. [a]

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
15.0	530	16.3	1,765	17.6	4,280	18.8	6,920
15.1	555	16.4	1,925	17.7	4,500	18.9	7,140
15.2	600	16.5	2,085	17.8	4,720	19.0	7,360
15.3	665	16.6	2,245	17.9	4,940	19.2	7,800
15.4	745	16.7	2,405	18.0	5,160	19.4	8,240
15.5	825	16.8	2,575	18.1	5,380	19.6	8,680
15.6	915	16.9	2,755	18.2	5,600	19.8	9,120
15.7	1,010	17.0	2,960	18.3	5,820	20.0	9,560
15.8	1,110	17.1	3,180	18.4	6,040	20.2	10,000
15.9	1,220	17.2	3,400	18.5	6,260	20.4	10,440
16.0	1,340	17.3	3,620	18.6	6,480	20.6	10,880
16.1	1,465	17.4	3,840	18.7	6,700	21.0	11,760
16.2	1,610	17.5	4,060				

[a] Made from measurements between gage heights 16 and 18.95 feet. Curve above and below these points is approximate. To be used only for open river.

Rating table for Flambeau River near Ladysmith, Wis., from January 1, 1904, to December 31, 1904.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
15.0	567	16.0	1,399	17.0	2,841	18.4	5,291
15.1	596	16.1	1,542	17.1	2,990	18.6	5,704
15.2	637	16.2	1,686	17.2	3,143	18.8	6,120
15.3	690	16.3	1,830	17.3	3,300	19.0	6,539
15.4	755	16.4	1,974	17.4	3,461	19.2	6,959
15.5	832	16.5	2,118	17.5	3,626	19.4	7,379
15.6	921	16.6	2,262	17.6	3,795	19.6	7,799
15.7	1,022	16.7	2,406	17.8	4,145	19.8	8,219
15.8	1,135	16.8	2,550	18.0	4,511	20.0	8,639
15.9	1,260	16.9	2,695	18.2	4,893		

Rating table for Flambeau River near Ladysmith, Wis., from January 1 to December 31, 1905.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
15.00	600	16.20	1,735	17.40	3,510	18.50	5,770
15.10	670	16.30	1,855	17.50	3,700	18.60	5,980
15.20	745	16.40	1,980	17.60	3,890	18.70	6,190
15.30	825	16.50	2,110	17.70	4,090	18.80	6,400
15.40	910	16.60	2,245	17.80	4,300	18.90	6,610
15.50	1,000	16.70	2,385	17.90	4,510	19.00	6,820
15.60	1,090	16.80	2,530	18.00	4,720	19.20	7,240
15.70	1,185	16.90	2,680	18.10	4,930	19.40	7,680
15.80	1,285	17.00	2,835	18.20	5,140	19.60	8,120
15.90	1,390	17.10	2,995	18.30	5,350	19.80	8,560
16.00	1,500	17.20	3,160	18.40	5,560	20.00	9,000
16.10	1,615	17.30	3,330				

The above table is applicable only for open-channel conditions. It is based on discharge measurements made during 1903–1905. It is not very well defined.

Estimated monthly discharge of Flambeau River near Ladysmith, Wis., for 1903, 1904, and 1905.

[Drainage area, 2,120 square miles.]

Date.	Discharge.			Run-off.		Rainfall [a]
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1903.	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Inches.*	*Inches.*
January						0.46
February [b]			860	0.41	0.21	.90
March	10,330	833	2,736	1.29	1.49	2.77
April	6,150	1,925	3,266	1.54	1.72	3.09
May	12,750	5,050	8,187	3.86	4.45	6.04
June	9,120	915	2,749	1.30	1.45	1.64
July [c]			4,598	2.17	2.02	8.70
August [d]			3,431	1.62	1.33	5.66
September	10,660	1,765	5,777	2.72	3.03	8.53
October	8,900	1,400	4,807	2.27	2.62	3.26
November	1,685	530	1,054	.50	.56	.94
December						.87
The year						43.25
1904.						
January						.46
February						1.11
March						1.76
April [e]	6,339	2,334	3,389	1.60	1.78	1.77
May	7,379	2,856	5,183	2.44	2.81	4.64
June	5,034	1,234	2,890	1.36	1.52	5.64
July	8,429	662	2,834	1.34	1.54	2.14
August [f]	2,334	607	1,336	.630	.726	5.01
September	4,109	1,200	2,056	.970	1.08	4.70
October [f]	5,912	1,470	3,517	1.66	1.91	5.41
November	3,300	555	1,416	.668	.745	.19
December [e]	1,974	390	951	.449	.518	2.39
The year						35.43
1905. [g]						
March 24-31		5,980	1,795	3,384	1.60	.476
April		7,240	1,558	3,867	1.82	2.03
May		5,980	2,530	4,090	1.93	2.22
June		8,340	1,795	5,223	2.46	2.74
July		7,240	1,000	2,950	1.39	1.60
August		3,160	825	1,669	.787	.905
September		5,560	1,735	2,839	1.34	1.50
October		3,990	1,045	2,305	1.09	1.29
November		2,245	1,090	1,616	.762	.850
December		2,045	955	1,449	.683	.787

[a] Rainfall for 1903 is the average of the recorded rainfall at Butternut, Medford, and Eau Claire that for 1904 omits Eau Claire and adds Prentice and Minocqua.
[b] February 15 to 28, inclusive.
[c] July 1 to 25, inclusive.
[d] August 10 to 31, inclusive.
[e] Estimates April and December made as if open channel.
[f] Discharge estimated for August 3 and 4 and October 1 to 4.
[g] No estimate for ice period.

Tributaries of Flambeau River.—Dore Flambeau River, the south branch of the Flambeau, rises at an elevation of 1,582 feet above the sea, in a group of a dozen lakes, the largest being Long Lake. Its total drainage area is 742 square miles. After a very rapid course of about 60 miles it joins the main stream in sec. 31, T. 37 N., R. 3 W. In the 27 miles between Long Lake and Fifield the river descends 146 feet, or 5.4 feet to the mile. Its gradient below Fifield has not been determined, but it is known to have many important falls and rapids. One of the largest of these, located in sec. 33, T. 37 N., R. 3 W., has a total fall of 35 feet. Owing to its lakes and swamps this river has a much more uniform flow than any of the Chippewa tributaries farther south. Dams are maintained by the Chippewa River Improvement Company at the outlet of Long Lake and at Fifield. The same company maintains logging dams on Elk River in sec. 11, T. 37 N., R. 2 W., and also in sec. 14, T. 37 N., R. 1 W., with flowage of 1½ and 2½ square miles, respectively. These and other logging dams within this drainage area are listed in the following table:

Logging dams maintained on tributaries of Flambeau River.[a]

No.	Location.	Dam.	
		Height.	Length.
	Dore Flambeau River:	*Feet.*	*Feet.*
1	Sec. 7, T. 39 N., R. 1 E; sec. 24, T. 40 N., R. 1 E	16	350
2	Sec. 16, T. 38 N., R. 1 W		
3	Secs. 23-26, T. 40 N., R. 3 E	15	400
4	Flambeau Lake, sec. 2, T. 40 N., R. 4 E	6	24
	Manitowish River:		
5	Sec. 9, T. 42 N., R. 5 E	13	400
6	Sec. 24, T. 42 N., R. 6 E	17	300
7	Sec. 15, T. 42 N., R. 7 E	15	250
	Elk River:		
8	Sec. 11, T. 37 N., R. 2 W	10	450
9	Sec. 14, T. 37 N., R. 1 W	10	
10	Trout River, sec. 14, T. 41 N., R. 6 E	4	
11	Bear Creek, sec. 2, T. 40 N., R. 4 E		

[a] Authority: Nos. 1 and 3, Wm. Irving, manager, Chippewa Lumber and Boom Co.; 4-8, Flambeau Lumber Co.; 9, J. R. Davis Lumber Co.; 10 and 11, E. S. Shepard. Owners: Nos. 1, 3, and 5-7, Chippewa Lumber and Boom Co.; 2, Lugar Lumber Co.; 4, Flambeau Lumber Co.; 8 and 9, Chippewa River Improvement Co.

RED CEDAR RIVER.

Drainage.—An area of 1,957 square miles in the extreme western part of Chippewa Valley is drained by Red Cedar River (sometimes called the Menomonie), which, unlike the other large tributaries of Chippewa River, does not reach the main stream until within a few miles from its mouth. Except at its headwaters, Red Cedar River drains a region underlain by the Cambrian sandstone. As a result, the greater part of the area has a sandy soil. A narrow belt of clayey loam, increasing in width southward, extends along the western limit of this area. The drainage area occupies the U-shaped region included between two terminal moraines, one near the eastern and one near the western border, which unite at the upper headwaters, giving rise to numerous lakes. Four of the largest of these have an area of about 20 square miles.

Profile.—A study of the profile of Red Cedar River shows that its total descent in the 90 miles above its mouth is 470 feet, or 5.2 feet per mile. This gives opportunity for a large number of water powers. There are about 25 old logging dams on the river, besides about an equal number of sawmills and flouring mills. The following table has been compiled from actual surveys by competent engineers and from checked railroad levels:

Profile of Red Cedar River from its mouth to Red Cedar Lake.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river	0	....	705.0	....	....
2	Dunnville	2.0	2.0	723.4	18.4	9.2
	Downsville dam:					
3	Foot	7.8	5.8	739.0	15.6	2.7
4	Crest	7.8	.0	758.2	19.2	
5	Irving	13.0	5.2	766.4	8.2	3.1
	Menomonie dam:					
6	Foot	16.6	3.6	788.3	21.9	
7	Crest	16.6	.0	803.9	15.6	6.0
8	"Omaha" bridge	18.9	2.3	806.7	2.8	
	Cedar rapids dam:					
9	Foot	23.4	4.5	823.3	16.6	3.7
10	Crest	23.4	.0	842.0	18.7	5.3
11	Hay River, mouth	30.2	6.8	859.3	17.3	
12	Colfax	35.0	4.8	895.0	35.7	7.4
13	Cameron (2 miles west)	70.0	35.0	1,068.0	173.0	5.0
14	Railroad crossing	74.0	4.0	1,116.0	48.0	12.0
15	Cedar Lake dam, sec. 22, T. 37 N., R. 10 W	90.0	16.0	1,191.0	75.0	4.7
16	Dam in sec. 25, T. 37 N., R. 10 W	96.0	6.0	....	....	....

[a] Authority: No. 1, Chicago, Milwaukee and St. Paul Railway; 2-11, O'Keef & Orbison, Appleton Wis.; 12, Wisconsin Central Railway; 13, Minneapolis, St. Paul, and Sault Ste. Marie Railway; 14 and 15, Chicago, St. Paul, Minneapolis and Omaha Railway.

A study of this table shows that Red Cedar River has a high gradient, averaging 5½ feet per mile in the last 74 miles, with frequent concentrations of descent. No gagings of the river have been made. Tributaries entering the river from the west flow through a clayey-loam soil, but the upper and eastern portions of the drainage area have a sandy-loam soil. It is therefore likely that this river has a fairly uniform flow. The decline of the lumbering interests greatly increases the value of the Red Cedar River as a power producer.

Water powers and dams.—In the 30 miles below Hay River the Red Cedar descends 154.3 feet, and as this region borders the prairie region and is thickly settled, the six powers here included will probably be developed to the full extent in the near future. This development includes: (1) The construction of a dam at Dunnville, 2 miles above the mouth of the river, giving a head of 15.6 feet and an estimated 1,685 horsepower; (2) the raising of the present dam at Downsville 4 feet, giving a total head of 23.2 feet and an estimated 2,480 horsepower; (3) the construction of a dam at Irving, with a total head of 21.9 feet, giving an estimated 2,260 horsepower; (4) the raising of the present dam at Menomonie 2.8 feet, thus obtaining a total head of 18.4 feet and an estimated 1,800 horsepower; (5) the building of a new dam near the "Omaha" bridge, 2.8 miles above Menomonie, with a head of 16.6 feet and an estimated 1,700 horsepower; (6) the raising of the present dam at Cedar rapids 21.3 feet, giving a total head of 40 feet and an estimated 3,800 horsepower.[a] Recently all the powers owned by Knapp, Stout & Co., including many of the most valuable on the river, have been

[a] This statement is based on a careful survey for the owners made by O'Keef & Orbison, hydraulic engineers, of Appleton, Wis., and an estimated run-off of 0.461 second-foot per square mile.

acquired by the Wisconsin Power Company, of Chicago, Ill. The location of 10 dams owned by this company is shown in the following table:

Dams on Red Cedar River owned by the Wisconsin Power Company.

Location.	Head.	Amount of flowage.	Authority.
	Feet.	*Cubic feet.*	
Sec. 25, T. 37 N., R. 10 W	14.0	1,674,000,000	U. S. engineers.
Sec. 2, T. 36 N., R. 10 W	7.0	405,000,000	Do.
Sec. 25, T. 36 N., R. 10 W	12.0	135,000,000	Do.
Sec. 30, T. 36 N., R. 9 W	10.0		
Sec. 29, T. 36 N., R. 9 W	10.0		
Sec. 13, T. 34 N., R. 10 W	12.0	40,500,000	Do.
Sec. 30, T. 33 N., R. 10 W	10.0	810,000,000	Do.
Downsville	19.0	....	J. W. Orbison.
Menomonie	15.5	....	Do
Cedar Falls	18.7	....	Do.

Railroads.—Between the mouth of Red Cedar River and Menomonie the Chicago, Milwaukee and St. Paul Railway closely parallels the river. In this stretch of 17 miles are situated the most important powers. Above Menomonie the drainage is crossed by the Chicago, Milwaukee and St. Paul, the Chicago, St. Paul, Minneapolis and Omaha, the Wisconsin Central, and the Minneapolis, St. Paul and Sault Ste. Marie railways.

EAU CLAIRE RIVER.

Ranked in order of its drainage area (900 square miles), Eau Claire River is third among the tributaries of the Chippewa. The greater part of this area is underlain by the Cambrian sandstone, and all except the upper headwaters drain a sandy-loam soil, as will be seen from Pl. II. Like most of the neighboring rivers, the Eau Claire has been an important lumbering stream, with many flooding dams. Very few water powers have been utilized. The first developed water power is about 500 feet from the mouth of the river, where a dam 300 feet long develops a head of 11 feet to run a linen mill, which uses only part of the power thus furnished. About 3,000 feet farther upstream is a second dam, with an average head of 13½ feet, owned by the Northwestern Lumber Company. An installation of turbines of 420 horsepower is reported. This is used in running a sawmill, a machine shop, and dynamos. The same company reports the three following lumbering dams on this river, but none of the resulting water power is utilized at the present time. In the NW. ¼ NE. ¼ sec. 14, T. 27 N., R. 9 W., is a dam with a 7-foot head, capable at ordinary low water of furnishing 210 theoretical horsepower. In the SW. ¼ NE. ¼ sec. 13, T. 27 N., R. 8 W., is a timber dam with a head of 8 feet, which could easily and cheaply be increased to 20 feet, thus producing at ordinary low water 540 theoretical horsepower. The third dam, with a present head of 20 feet, is reported in the SW. ¼ SW. ¼ sec. 5, T. 26 N., R. 6 W. This dam has not been used for many years and is much in need of repairs. There are many other opportunities for developing water powers on the Eau Claire River, as well as on its tributaries.

JUMP RIVER.

As its name would imply, Jump River is a very rapid stream, with numerous falls and rapids, making a descent of nearly 500 feet in its entire length of 65 miles. Its drainage area of 720 square miles is a long and narrow one, and with only a few unimportant exceptions is devoid of lakes and swamps. As a result the river has a very uneven flow as compared with the Flambeau, which stream it resembles in flowing through a valley whose soil is a clayey loam. The main portion of the Jump River valley has no railroads and is sparsely settled. A branch of the Wisconsin Central is now being built across this drainage. The most important falls on the river, 35 feet in height, are in sec. 20, T. 34 N., R. 2 W., about 1 mile east of the junction of North and South forks, but there are numerous other dam sites of 15 to 20 foot head, which will doubtless be utilized when this section is settled.

YELLOW RIVER.

The drainage area of Yellow River is 460 square miles, distributed in a long, narrow valley. The lower half of the valley has a sandy soil, the upper part a clayey loam. While the gradient of Yellow River is not so great as that of its neighbor, Jump River, it has a rapid current. As in the case of other rivers in this region the only dams built were for logging purposes. The Miller dam is said to be the only one remaining. Three other dams, one at Colburn, one in sec. 7, T. 29 N., R. 5 W., and one at Cadott, have all been carried away by floods. The river is crossed by three railroads.

SMALLER TRIBUTARIES.

Chippewa River has a host of smaller tributaries, nearly all of which, because of their rapid currents and high, rocky banks, can be cheaply developed. Duncan Creek is a good example of what can be done with this class of tributaries. Although only 25 miles long, it has five dams with an aggregate head of 68 feet. Four gristmills, with a total turbine capacity of over 500 horsepower, take their power from this creek. Below the "Star mills," in the city of Chippewa Falls, is an unimproved power of 14-foot head; and immediately below this site is a dam with a 9-foot head, belonging to the Gatzian Shoe Manufacturing Company. The significant point regarding powers of this class is that they are cheaply improved and very widely distributed. The locations of some of them are shown in the following table:

Dams on smaller tributaries of Chippewa River.

Location.	Owner and use.	Head.	Installation.
		Feet.	*H. P.*
Arkansaw Creek, Arkansaw	Mills & Son, gristmill	12	25
Bass Creek, Afton	Wm. Denoger, flouring mill.	9	40
Bear Creek, Durand	Durand roller mill, flour	18	73
Bridge Creek:			
Augusta	Dells Milling Co., flour	20	60
Sec. 18, T. 26, R. 6 W	J. P. Waddell	20	(a)
Duncan Creek:			
Chippewa Falls	Gotzian Shoe Co	9	(b)
Do	Leinenkugel Brewing Co	14	(a)
Do	Leinenkugel Co., flour	16	350
Sec. 31, T. 29 N., R. 8 W	Glen mills, flour	20	100
Sec. 24, T. 29 N., R. 9 W	G. W. Lockin, Tilden flouring mills.	10	70
Sec. 8, T. 30 N., R. 9 W	Bloomer mills, flour	12	30
Eighteenmile Creek, Colfax	J. A. Anderson & Son, grist and saw mill.	14	60
Hay River, Prairie farm	P. F. Milling Co., grist	9	
Jump River:			
Sec. 20, T. 34 N., R. 2 W		35	(a)
Westboro			
Lowes Creek, sec. 4, T. 26 N., R. 9 W	W. J. Davis	c 30	

a Undeveloped. b Unused. c Could be raised 8 feet.

Dams on smaller tributaries of Chippewa River—Continued.

Location.	Owner and use.	Head.	Instal-lation.
O'Neals Creek, west branch:		*Feet.*	*H. P.*
Sec. 26, T. 31 N., R. 9 W	Wm. Durch, grist and saw mill.	8	30
Near mouth	F. G. & C. A. Stanley, saw-mill.	22	50
Eagle Point	M. Rosmus, electric light	12	150
Otter Creek, Eau Claire	R. Clark, flour	18	95
Pine Creek:			
Lucas	T. Teegarden, grist and saw mill.	12	80
Sand Creek	A. F. Johnson, grist and saw mill.	6	96
Dalles	J. A. Anderson, grist and saw mill.	8	50
Plover River:			
Shantytown	S. Y. Bentley, sawmill	7	40
Jordan	A. Van Orden	14	100
Bevent	do	7	(a)
Rock Creek, sec. 22, T. 27 N., R. 11 W	D. W. Andrews, flour	35	75
Tiffany Creek, Boyceville	A. A. Hoyr & Bro., grist	9	30

[a] Undeveloped.

ST. CROIX RIVER SYSTEM.

TOPOGRAPHY AND DRAINAGE.

St. Croix River rises at an elevation of 1,010[a] feet, in St. Croix Lake, on the Lake Superior divide, only 20 miles from Lake Superior. The lower two-thirds of its length forms a part of the Minnesota boundary. In its total length of 168 miles it descends 344 feet, all but 20 feet of which is in the upper 116 miles, making the average for this upper portion nearly 3 feet per mile. This slope is fully six times the slope of Mississippi River above Minneapolis, and, according to United States engineers, has an important bearing on the relatively large run-off as compared with Mississippi Valley above. Another important feature of this region is its relatively small number of lakes, these forming only 3 per cent of the total drainage area as compared with 11 per cent in Mississippi Valley above Minneapolis.[a] Evaporation on lake surfaces is probably nearly equal to the precipitation for the corresponding period. The total drainage area comprises 7,576 square miles, the greater part of which is in Wisconsin. The Wisconsin portion has a width of 50 miles on its northern margin and extends southwesterly toward Mississippi River, a distance of about 150 miles.

The topography may be described under three heads—(1) the level area, (2) the rolling and swelling hill districts, and (3) the knoll and basin combination. The first includes the so-called "barrens" which border the streams and some elevated plateaus, together with smaller scattered areas. The third class may be described as a belt lying near the southeastern watershed and stretching from the vicinity of Lake Namekagon southwestward to the St. Croix. The second class includes most of the territory which remains.[b]

Marshes are quite as infrequent as the lakes and occur only on the river bottoms. Not half of the lakes are visibly connected with the rivers, but because of the open soil they are likely to have underground connection. There are usually lumbering dams on such lakes as have outlets, and these lakes, together with the numerous smaller depressions, play an important part in the preventing of freshets. The lakes of this region arrange themselves into two groups—one, lying mostly in the "barrens," adjacent and parallel to the upper St.

[a] Repts. Chief Eng. U. S. Army, 1881, 1883.
[b] Geol. Wisconsin, vol 3, 1880, p. 370.

Croix and extending southwest from its source to the point where the stream turns southward, and a second group in the extreme southeastern portion of this region, occurring in the depressions of the "Kettle moraine." As the water of this region flows almost exclusively over the crystalline rocks and sandstones, or the drift derived from them, it is in general soft, though usually amber colored. Springs are very common, many of the lakes being fed almost entirely by them. They are especially frequent in the Cambrian sandstone and tend to equalize the flow of all the streams.

The apportionment of drainage areas is shown in the following table:

Distances and drainage areas of St. Croix River.

River.[a]	Distance from source (map measure).	Drainage area above station.
	Miles.	*Sq. miles.*
St. Croix, source		
Eau Claire:		
Above mouth	6.5	117
Mouth	6.5	224
Namekagon	38.0	1,451
Yellow	50.0	2,064
Clam:		
Above mouth	64.0	2,428
Mouth	64.0	2,844
Kettle:		
Above mouth	75.0	3,046
Mouth	75.0	4,139
Snake	79.0	5,097
Wood	84.0	5,261
Sunrise	100.0	5,657
St. Croix, St. Croix rapids	120.0	6,202
Apple	138.0	6,951
Willow	151.0	7,301
St. Croix, mouth	168.0	7,576

[a] Station is at mouth of river, unless otherwise stated.

PROFILE.

The following table gives, upon the authority of United States engineers, elevations above the sea and gradients per mile of St. Croix River at twenty points between its mouth and its source:

Profile of St. Croix River from its mouth to St. Croix Lake.

Station.	Distance.		Elevation above sea level.	Descent between points.	
	From mouth.	Between points.		Total.	Per mile.
	Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
Prescott, mouth of river	0.0	....	[a] 667.0	....	....
Kinnikinnic River, mouth	5.0	5.0	668.0	1.0	0.2
Apple River, mouth	28.0	23.0	672.0	4.0	.2
Osceola	42.0	14.0	683.0	11.0	.8
St. Croix Falls (head of navigation)	48.0	6.0	687.0	4.0	.7
Trade River, mouth	60.0	12.0	753.0	6.6	5.5
Sunrise River, mouth	65.0	5.0	758.5	5.5	1.1
Rush City, ferry	75.0	10.0	773.0	14.5	1.4
Sec. 35, T. 38 N., R. 20 W	79.0	4.0	±782.0	9.0	2.2
Snake River, mouth	86.0	7.0	±790.0	8.0	1.1
Kettle River rapids, foot	89.0	3.0	±801.0	11.0	3.7
Kettle River, mouth	90.0	1.0	±816.0	15.0	15.0
Kettle River rapids, head (proposed U.S. dam, sec. 2, T. 39 N., R. 19 W)	93.0	3.0	±850.0	34.0	11.3
Clam River, mouth	101.0	8.0	±868.0	18.0	2.2
Sec. 1, T. 40 N., R. 18 W	103.5	2.5	874.0	6.0	2.4
Yellow River, mouth	115.0	11.5	888.0	14.0	1.2
Namekagon River, mouth	127.0	12.0	908.0	20.0	1.7
Moose River, mouth	139.0	12.0	1,001.0	93.0	7.7
Sec. 35, T. 44 N., R. 13 W.:					
Below dam	144.0	5.0	1,001.5	.5	.1
Above dam	144.0	.0	1,005.3	3.8	....
St. Croix Lake	160.0	16.0	1,010.0	4.7	.3

[a] Low-water elevation.

GEOLOGY.

Almost the entire watershed has been glaciated to such an extent that outcrops, except near the rivers, are very infrequent. According to the reports of the Wisconsin Geological Survey, the central and by far the greater portion of this area is underlain by the pre-Cambrian crystalline rocks known as the "Keweenawan." This belt narrows toward the south, giving way both on the east and west to the Cambrian sandstones. These pre-Cambrian crystalline rocks intersect St. Croix River at St. Croix Falls, and because of their greater hardness have caused the falls and rapids—the most important on the entire river—which extend for 6 or 7 miles above the city of Taylors Falls, Minn.

RAINFALL AND RUN-OFF.

The United States Geological Survey has maintained a gaging station 3½ miles above St. Croix Falls, Wis., since 1903. The gage heights are referred to four iron pins on the right bank just below the gaging station, the elevations of which are referred to the datum of the bench marks of the St. Croix River survey. Their elevations are as follows.

	Feet.
Pin No. 1	732.08
Pin No. 2	734.54
Pin No. 3	736.10
Pin No. 4	737.57

A large number of measurements were obtained during 1903, and the gage was read daily by V. H. Caneday. Discharge measurements were made from a boat held in place by a wire cable stretched across the river between two trees. The initial point for soundings is a vertical rod on the left bank. The channel is straight for about 800 feet above and 1,000 feet below the station, while the banks are high and can not overflow. The section is regular, smooth, and permanent, and the velocity is never sluggish, making this on the whole a station at which good results are obtainable. The drainage area at this point is 6,370 square miles.

Discharge measurements of St. Croix River near St. Croix Falls, Wis., in 1903.

Date.	Hydrographer.	Gage height.	Discharge.
1903.		*Feet.*	*Second-feet.*
May 22	E. Johnson, jr	4.00	10.747
August 11	W. R. Hoag	2.70	7.470
October 9	L. R. Stockman	3.84	10.244

Discharge data relating to St. Croix River near St. Croix Falls, Wis., obtained through the United States Geological Survey, have been supplemented by data supplied by Loweth & Wolf, civil engineers, of St. Paul, Minn. The results are embodied in the following table:

Daily discharge, in second-feet, of St. Croix River near St. Croix Falls, Wis., January 10, 1902, to December 31, 1904.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1902.												
1	....	1,820	2,425	2,910	3,930	5,150	6,690	2,270	1,725	2,330	5,190	2,480
2	....	1,875	2,442	2,840	4,090	5,010	4,490	1,740	1,730	2,390	3,950	2,555
3	....	1,930	2,460	300	3,910	4,480	4,830	1,790	1,680	2,395	3,290	....
4	....	1,700	2,300	400	3,920	9,798	4,700	1,825	1,840	1,685	4,740	....
5	....	1,760	2,370	2,750	3,935	11,871	7,350	1,870	1,700	2,150	3,910	....
6	....	1,755	2,420	2,515	4,900	10,956	5,200	2,035	2,560	2,445	4,180	....
7	....	1,750	2,270	2,280	3,980	10,610	12,106	2,260	2,550	2,390	4,030	....
8	....	1,770	....	2,280	4,880	....	11,693	1,980	2,220	2,290	4,740	....
9	....	1,765	....	2,190	4,560	9,261	11,137	1,660	4,110	2,050	4,500	....
10	1,895	1,760	....	2,110	4,590	10,468	12,947	1,990	3,500	930	3,290	....
11	1,910	1,750	....	1,990	4,450	6,810	8,978	3,970	1,720	2,950	2,960	....
12	1,860	1,750	....	1,870	5,850	7,600	7,980	1,680	1,500	1,950	3,195	....
13	1,850	1,815	....	1,468	6,150	8,289	6,700	1,120	1,640	2,040	4,300	2,370
14	1,685	1,870	....	2,065	5,250	4,780	6,060	1,015	1,550	2,000	4,530	2,240
15	1,765	1,990	....	2,020	4,780	6,350	5,780	1,570	1,355	1,915	4,900	2,150
16	1,775	1,990	....	2,170	4,875	4,220	4,860	1,590	1,355	800	4,600	[illegible]
17	1,795	1,990	....	2,070	4,820	3,420	4,380	1,560	1,540	845	4,700	2,015
18	1,880	1,990	....	5,190	4,940	3,580	3,800	1,500	1,480	3,600	4,580	2,110
19	1,860	1,990	4,650	1,510	5,065	6,350	5,210	1,510	1,120	1,940	5,160	2,180
20	1,920	1,990	4,995	1,005	5,300	3,780	2,850	1,500	510	1,925	3,690	2,085
21	1,875	1,990	4,650	500	5,870	960	3,405	1,480	2,800	2,040	4,665	2,040
22	1,930	1,990	4,600	5,540	7,080	3,300	3,530	1,480	2,070	1,980	4,160	2,000
23	1,860	2,027	4,035	440	9,600	3,400	3,600	1,575	2,540	2,040	4,250	[illegible]
24	1,950	2,065	3,470	510	7,250	6,000	3,185	1,495	2,365	850	4,060	....
25	1,985	2,110	3,110	1,050	6,420	3,560	2,560	1,405	1,065	1,100	3,720	....
26	1,975	2,180	3,117	2,760	5,585	4,145	7,250	3,850	1,135	2,300	3,555	....
27	1,950	2,260	3,125	3,025	5,760	4,380	850	1,860	1,120	2,310	3,680	....
28	1,930	2,480	3,125	3,290	5,090	4,200	750	1,740	3,050	2,660	3,060	....
29	1,920	....	3,125	3,480	6,070	2,550	2,515	1,465	2,210	2,890	3,050	2,080
30	1,905	....	3,037	3,750	4,930	4,690	2,515	5,995	2,310	1,875	2,050	2,060
31	1,890	....	2,950	....	5,290	....	2,610	1,795	....	2,840	....	2,045

Daily discharge, in second-feet, of St. Croix River near St. Croix Falls, Wis., January 10, 1902, to December 31. 1904—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1903.												
1....	2,055	1,950	1,940	6,770	8,920	7,680	251	4,570	....	....	....	....
2....	1,940	1,935	1,920	9,800	9,555	10,420	3,030	4,800	....	....	....	....
3....	1,940	1,760	1,920	10,750	....	(9,490)	....	5,050	....	....	....	....
4....	1,910	1,830	1,960	12,220	....	8,560	....	6,170	....	....	....	....
5....	1,880	1,900	1,965	....	15,200	7,910	....	6,710	....	....	....	....
6....	1,930	2,015	1,885	10,350	15,611	7,340	8,640	1,600	....	....	....	....
7....	1,945	1,930	1,990	8,850	15,176	(6,805)	....	7,900	....	....	....	....
8....	2,010	1,915	2,050	11,645	13,835	6,270	8,880	(7,600)	....	....	....	....
9....	1,930	1,895	2,110	17,975	12,150	6,010	10,155	(7,280)	....	....	....	....
10....	1,850	1,945	....	16,438	....	5,760	10,870	7,170	....	....	....	....
11....	1,875	1,950	....	18,272	9,245	5,160	11,630	4,830	....	....	....	....
12....	1,900	1,880	....	20,166	16,157	6,190	(10,437)	5,510	....	....	....	....
13....	1,930	1,975	....	....	....	7,320	9,245	5,340	....	....	....	....
14....	1,950	1,930	....	....	....	(6,910)	7,250	5,350	....	....	....	....
15....	1,980	1,870	....	15,382	....	6,500	7,200	(4,792)	....	....	....	....
16....	1,870	1,840	....	14,080	....	5,825	6,915	4,235	....	....	....	....
17....	1,770	1,870	4,030	12,800	....	5,130	6,790	4,150	....	....	....	....
18....	1,815	1,970	4,530	....	....	5,755	6,035	3,460	....	....	....	....
19....	1,870	1,850	6,480	12,540	....	4,300	(5,592)	3,580	....	....	....	....
20....	1,780	1,700	9,890	10,260	13,830	....	5,150	4,360	....	....	....	....
21....	1,980	1,745	11,440	....	13,790	....	4,540	4,740	....	....	....	....
22....	1,730	1,830	11,460	....	10,580	1,542	4,375	....	....	....	....	....
23....	1,820	1,910	11,480	....	11,230	2,700	3,900	3,220	....	....	....	....
24....	1,800	1,950	10,740	6,800	11,665	2,710	1,830	....	....	....	....	....
25....	1,865	1,945	9,660	9,740	12,100	2,640	5,590	....	....	....	....	....
26....	1,930	1,820	10,100	9,265	9,580	2,545	(4,670)	....	....	....	....	....
27....	1,990	1,880	9,530	8,790	12,020	(2,475)	3,750	....	....	....	....	....
28....	2,050	1,970	8,725	10,460	12,640	(2,420)	4,770	....	....	....	....	....
29....	1,980	....	8,590	10,080	11,420	2,360	4,730	....	....	....	....	....
30....	1,835	....	8,445	8,925	10,640	907	4,485	....	....	....	....	....
31....	1,970	....	8,160	....	(9,160)	....	4,570	....	....	....	....	....
1904.												
1....	2,390	2,110	2,580	5,560	8,400	6,340	6,170	840	....	3,950	8,780	1,690
2....	2,390	2,090	2,570	6,130	7,590	5,520	5,850	1,080	4,530	(3,840)	8,040	1,760
3....	....	2,060	2,520	(7,000)	7,540	6,050	(3,630)	1,480	4,610	3,720	7,590	2,210
4....	....	2,040	2,390	8,080	7,480	7,950	1,410	3,460	(4,750)	3,360	6,780	(2,400)
5....	....	2,080	2,290	9,873	7,380	(12,560)	(3,010)	2,250	4,900	11,310	3,280	2,620
6....	....	2,070	2,390	12,390	8,290	17,180	4,610	1,990	4,870	1,240	(4,230)	2,740
7....	3,660	(2,040)	2,490	15,930	8,790	17,920	4,780	2,040	5,040	(2,800)	5,230	2,890
8....	3,140	2,020	2,600	16,900	(10,320)	17,460	4,610	2,100	4,690	4,690	5,440	2,970
9....	2,810	2,160	2,590	18,300	(11,850)	15,650	4,970	2,210	4,600	(3,400)	5,700	2,770
10....	(2,820)	2,110	2,560	(16,600)	13,370	12,940	2,960	2,100	(4,030)	2,120	4,900	2,820
11....	2,840	2,000	2,590	15,060	11,300	12,610	950	2,000	(3,460)	10,430	5,330	(2,820)
12....	2,600	2,160	2,640	14,010	9,490	(12,070)	3,480	2,300	2,820	15,020	5,600	2,830
13....	2,340	2,000	2,650	10,590	8,550	11,530	3,860	2,340	(2,380)	14,270	(5,540)	2,500
14....	2,660	(2,140)	2,660	7,910	8,980	11,320	3,750	(1,750)	1,940	13,800	5,470	2,420
15....	2,680	(2,280)	2,700	12,560	8,650	7,880	3,890	1,150	2,150	12,560	5,250	2,220
16....	2,630	2,430	2,740	10,010	8,310	8,540	3,990	950	3,480	....	4,970	2,380
17....	(2,410)	2,430	2,690	(9,460)	7,280	7,628	(2,530)	1,430	3,190	10,060	4,770	2,330
18....	2,200	2,460	2,700	(8,920)	7,820	8,140	1,080	3,370	(3,160)	10,760	4,570	2,320
19....	2,480	2,410	2,750	8,380	6,860	8,710	1,140	1,920	3,140	10,310	4,480	2,300
20....	2,460	2,450	(a)	7,850	5,250	9,280	3,760	2,240	2,890	12,710	(4,340)	2,380

[a] March 20 to 25, ice going out.

Daily discharge, in second-feet, of St. Croix River near St. Croix Falls, Wis., January 10, 1902, to December 31, 1904—Continued.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1904.												
21....	2,440	(2,370)	....	7,490	6,390	6,730	3,420	(3,760)	2,750	(15,700)	4,190	2,160
22....	2,630	2,290	....	7,530	(6,900)	5,630	3,170	5,290	2,380	18,700	4,020	2,440
23....	2,620	2,330	....	11,260	(7,500)	5,820	3,270	4,390	2,490	(18,010)	4,000	2,340
24....	(2,570)	2,230	....	(10,810)	8,000	4,960	(2,240)	2,520	2,700	17,330	4,120	2,3[illegible]
25....	2,520	2,280	....	10,360	7,790	5,190	1,210	2,970	(3,240)	16,180	3,720	(2,4[illegible])
26....	2,330	2,410	3,090	11,170	8,760	(3,380)	1,050	2,510	3,790	15,540	3,710	2,450
27....	2,390	2,460	(3,370)	11,230	8,030	1,570	2,580	2,480	3,330	12,710	(3,300)	2,4[illegible]
28....	2,280	2,480	3,660	10,769	7,390	4,850	2,780	(2,230)	3,500	12,910	2,890	(2,440)
29....	2,270	2,520	3,300	10,850	(6,700)	5,330	2,720	1,960	3,580	10,590	2,800	(2,430)
30....	2,250	....	3,770	9,490	6,060	5,320	2,810	2,260	(3,880)	(10,410)	(2,250)	(2,400)
31....	....	....	4,510	....	6,440	....	....	(3,000)	....	10,230	....	2,380

Estimated monthly discharge of St. Croix River at St. Croix Falls, Wis., for 1902, 1903, and 1904.

[Drainage area, 6,370 square miles.]

Date.	Discharge. Maximum.	Discharge. Minimum.	Discharge. Mean.	Run-off. Per square mile.	Run-off. Depth.	Rainfall.[a]
	Sec.-feet.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
1902.						
January	1,980	1,680	1,880	0.31	0.36	0.90
February	2,480	1,700	1,880	.31	.36	.54
March	5,000	2,380	3,300	.60	.69	.6[illegible]
April	5,560	200	2,220	.37	.41	2.0[illegible]
May	9,000	4,100	2,020	.33	.38	3.0[illegible]
June	11,870	960	5,950	.99	1.14	3.3[illegible]
July	12,106	760	5,500	.92	1.06	6.30
August	6,000	1,020	1,860	.31	.36	2.82
September	4,100	400	1,860	.31	.36	3.2[illegible]
October	3,600	800	2,000	.33	.38	1.55
November	5,200	2,550	4,080	.68	.77	2.78
December	2,550	2,020	2,100	.35	.40	2.1[illegible]
The year	12,106	200	2,912	.48	6.67	29.77
1903.						
January	2,040	1,740	1,920	.32	.37	.5[illegible]
February	2,020	1,700	1,880	.31	.36	.[illegible]
March	11,480	1,960	5,500	.92	1.05	1.97
April	20,185	5,800	12,000	2.00	2.30	2.7[illegible]
May	16,157	8,920	12,700	2.12	2.43	5.7[illegible]
June	7,900	906	5,050	.84	.96	1.[illegible]
July	11,620	251	6,360	1.06	1.21	6.75
August	7,900	1,600	4,850	.81	.92	4.77
September	14,918	6,960	11,750	1.96	2.26	7.[illegible]
October	29,611	5,740	12,780	2.13	2.44	4.11
November	7,000	850	4,270	.71	.81	.6[illegible]
December	3,440	2,340	2,740	.46	.52	.9[illegible]
The year	29,611	[b]251	6,816	1.14	15.63	3[illegible]

[a] This is the average of the recorded precipitation at Barron, Duluth, Grantsburg, Hayward, Os[illegible] and St. Paul.

[b] Low water due to the manipulation of a lumbering dam a few miles above.

Estimated monthly discharge of St. Croix River at St. Croix Falls, Wis., for 1902, 1903, and 1904—Continued.

Date.	Discharge.			Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.	Per square mile.	Depth.	
1904.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Inches.*	*Inches.*
January	2,840	2,200	2,600	.43	.48	.64
February	2,480	2,000	2,238	.37	.42	1.18
March	4,510	2,290	2,832	.47	.53	1.19
April	18,300	5,560	10,748	1.79	2.01	1.65
May	13,370	5,250	8,176	1.36	1.53	3.78
June	17,920	4,850	8,868	1.48	1.66	5.58
July	5,850	950	3,145	.52	.58	4.64
August	3,460	840	2,334	.39	.44	3.84
September	5,040	1,940	3,544	.59	.66	5.75
October	18,700	1,240	10,560	1.76	1.98	5.47
November	8,780	2,800	4,843	.80	.90	.05
December	2,970	1,690	2,441	.40	.45	.99
The year	18,700	840	5,194	10.36	11.65	34.77

WATER POWERS.

FALL.

In the lower 48 miles of its course the St. Croix River has its bed in the Cambrian sandstone or "Lower Magnesian" limestone, principally the former, which it has succeeded in wearing down nearly to base level, giving steamboat navigation from Taylors Falls, Minn., to Mississippi River. Its descent in this distance of 48 miles is only 20 feet at low stages, nearly all of which is found in the upper half between Stillwater and Taylors Falls. At Stillwater, 223 miles above the mouth of the river, the sandstone bluffs rise steeply on either side to a height of 150 to 200 feet, and the river rapidly narrows. The bluffs continue, generally with a flat on one side, between Taylors Falls and Stillwater. In the 24 miles below Stillwater the river averages about half a mile in width, with a maximum of 7,000 feet at the expansion of the river known as St. Croix Lake, below Stillwater. For several miles here, according to reports of United States engineers, the river is almost without gradient.

The portion of the St. Croix above Taylors Falls abounds in undeveloped powers. Except near the headwaters of St. Croix, Totogatic, and Namekagon rivers and a small area served by a branch line of the Northern Pacific, running to Grantsburg, this region is without railroad facilities. The following detailed description of the main river above St. Croix rapids, taken from the Tenth Census, 1880, gives the most trustworthy information of the region obtainable:

From the mouth of the Eau Claire to that of the Namekagon River there is a descent of 100 feet, or 4 feet per mile, and many rapids occur, among which Copper Mine rapids may be mentioned. Above the mouth of the Namekagon the ordinary low-water power under a head of 10 feet would be 150 horsepower. The Namekagon River increases this to 600 horsepower.

In the 12 miles from the mouth of the Namekagon to the Yellow River the total fall is 20 feet, including Big Island rapids, State Line rapids, and Bishops rapids. Each of the first two is described as affording fine opportunities for developing water powers. At Big Island rapids the river runs close to the bluffs on the left bank, but a dam would need to extend some distance across the flat on the right.

From the mouth of the Yellow River to the head of Kettle rapids, a distance of 21 miles, the average slope is 1.8 feet per mile, there being no rapids of special importance. It is very probable that available water-power sites can be found in this section.

ST. CROIX RAPIDS.

The St. Croix rapids offer fine opportunities for water power, and were used at one time, but now the river flows unemployed. There is a total descent of 55 feet in the 6 miles which may be included under the name of St. Croix rapids. Several local names are indefinitely applied at different points. At the foot are Taylors Falls, about three-quarters of a mile above are St. Croix Falls, then Turtle Falls, etc. Strictly speaking, there are no falls in the entire distance, only a more rapid decline in the bed at certain places.

The village of Taylors Falls is situated in Minnesota, at the foot of the rapids, about 50 miles above the mouth of the river, at the head of navigation. St. Croix Falls, a village of Wisconsin, is situated upon the slope overlooking the river from that side, nearly opposite Taylors Falls. Directly below the rapids the river enters the Dalles of the St. Croix, where for half a mile or more it passes between vertical cliffs of trap rock with sharp edges and bold angles. Just above the entrance into the Dalles the water way is so contracted that when the river is high the water forms a fall of nearly 5 feet before it can overcome the resistance, but there is no very rapid descent there in low water. It is to this portion of the river that the name of Taylors Falls is given.

Above the Dalles the rock continues in the bed, and to a certain extent in the banks of the river, but the valley spreads considerably. On the Minnesota side the bank rises steep from the river for 30 or 40 feet at the lower part of the rapids. Back from this for several hundred feet is a nearly level plain swampy in places; and bounding this are the bluffs, rising fully 100 feet higher. At the foot of the rapids the plain narrows and is lost in the Dalles. On the Wisconsin side, in the vicinity of St. Croix Falls, the slope is rather more uniform up to the general level of the country. At the entrance into the Dalles the river is scarcely more than 100 feet wide. At St. Croix Falls, three-quarters of a mile above Taylors Falls, it is between 200 and 300 feet wide, the average width of the river in this part of its course.

The portion of the rapids known as St. Croix Falls presents the most favorable site for improvement of the power, and here a dam was once built and sawmills were run. The bed is solid rock, and the banks rise abruptly from the river on both sides. On the Minnesota side a large, high mass of trap rock stands out in the channel and forms a natural abutment for a dam; on the Wisconsin side the rock bank rises to a considerable height above the water in a rib, and back of it is a depression which leads to the slope upon which the village of St. Croix Falls is situated. The improvement, long since gone to ruin, consisted of a dam built across the river at the point described, and a race blasted through the rock in the line of the depression on the Wisconsin side and then carried down the slope along the river front, giving a head of 25 or 30 feet. The dam was a very extensive structure, raising the water to a height of 25 feet when in good condition. It was 300 feet long, 24 feet wide at the top, and only 60 feet wide at the base. . . . The same natural facilities exist for developing the water power as formerly. . . . If the dam were built so as to give a head of about 40 feet, which is practicable, a race could be carried down the plain on the Minnesota side for a long distance as readily as on the Wisconsin shore. The pond would probably back the water 4 or 5 miles, and would not overflow much land. With the ordinary low flow the power under a head of 30 feet is 7,811 theoretical horsepower, and under a head of 40 feet, 10,415 theoretical horsepower. With the yearly average flow it is 17,266 theoretical horsepower under a head of 30 feet, and 23,021 theoretical horsepower under 40 feet. There is about 5 feet of fall in the river from the site of the dam to Taylors Falls. Here is an excellent site for the construction of a dam, which would scarcely be more than 100 feet long, but the vertical cliffs come close to the river just below, leaving only room for a small steamboat landing, without space to erect extensive manufactories.

KETTLE RIVER RAPIDS.

The Kettle River rapids are, next to the St. Croix rapids, the most prominent on the river. They start $2\frac{1}{2}$ miles above the mouth of the Kettle River, which enters from the west, and end $1\frac{1}{2}$ miles below it. In this length of 4 miles the total fall is 49 feet, of which 34 feet is above the mouth of the Kettle River. Two islands from 1 mile to 2 miles long divide the river into two channels. The bed of the river is solid rock and it is practical to build several dams. Above the mouth of the Kettle River a head of 10 feet would afford 1,280 theoretical horsepower, with the ordinary low-water flow, and below the entrance of the Kettle River 1,737 theoretical horsepower, under the same conditions of flow, according to the estimates previously given.

Above the mouth of the Snake River, which enters $4\frac{1}{2}$ miles below the Kettle River, there is 11 feet of fall from the foot of the rapids. Between Snake River and St. Croix rapids are the following rapids: The Otter Slide, just below the mouth of the Snake, the ordinary low-water power of which, under a head of 10 feet, is 2,140 theoretical horsepower; the Horse Race, 1 mile below; the Baltimore rapids a mile below the mouth of the Wood River, the ordinary low-water power of which, under a head of [illegible] feet, is 2,220 theoretical horsepower; the Upper Big Rock rapids, about 1 mile below them; and the Yellow Pine rapids, about 3 miles above the mouth of the Sunrise River. The amount of fall at each of these rapids can not be determined from the data at hand. The total fall from the mouth of Snake River to St. Croix rapids is 111 feet and the average slope is 2.64 feet per mile. This must furnish opportunities to develop power with what will be a reasonable expense at some time in the future.

TRIBUTARIES OF ST. CROIX RIVER.

LENGTH AND DRAINAGE.

The length and drainage area of the principal tributaries of St. Croix River, including those entering from the western (Minnesota) side, are shown in the following table:

Principal tributaries of St. Croix River.

River.	Length (map measure).	Drainage area.
	Miles.	*Sq. miles.*
Eau Claire	25	107
Namekagon	85	1,002
Yellow	50	310
Clam	50	416
Kettle (Minnesota)	70	1,093
Snake (Minnesota)	78	937
Wood	30	168
Apple	55	427
Willow	35	246

YELLOW RIVER.

Yellow River rises in a large lake called Mud Lake, at an elevation of 1,085 feet,[a] and after a sinuous course of 50 miles joins the St. Croix at a point only half this distance from the source and at an elevation of 888 feet. This gives a descent of 197 feet, an average of nearly 4 feet per mile. This high gradient results in rapids at frequent intervals throughout its entire course. The slope in the upper third of its length is about 120 feet. Here springs and creeks are numerous. The river is known to have a remarkably constant stage, the natural rise and fall during the year varying only from 1½ to 3½ feet. This fact may be attributed to the springs and to the regulating effect of the large lakes, especially Yellow Lake, through which it flows. "Its valley is generally narrow, being from 200 to 800 feet in width, although in some places it widens into tamarack marshes of considerable extent. The first banks have a general elevation of 15 feet above low water, running back into high, broken ridges, covered with white Norway and jack pine. Little stone and few bowlders are found until reaching the rapids below Yellow Lake, which are almost continuous to the mouth of the stream."[a]

Near the mouth of the river the banks are high. A dam could be built in sec. 27, T. 41 N., R. 16, which would develop a head of 25 feet or more and still not back the water up to the Yellow Lake dam. This power could be combined in the same plant with that furnished by Loon Creek, which enters Yellow River near the proposed dam. Loon Creek is said to descend 50 to 75 feet in a distance of 1½ miles, and is therefore of considerable importance. A dam could also be located in Yellow River about a mile above Yellow Lake, which would develop a head of 20 feet by overflowing some good meadow lands between Yellow and Devils lakes.

[a] Rept. Chief Eng. U. S. Army, 1880.

The following profile of Yellow River suggests the possibility of developing other powers on this river because of its high gradient in ranges 14 and 13:

Profile of Yellow River from its mouth to Mud Lake dam.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river			888.0		
2	Yellow Lake dam	7.0	7.0	928.0	40.0	5.7
3	SW. ¼ sec. 2, T. 39 N., R. 16 W	15.0	8.0	938.4	10.4	1.3
4	Rice Lake dam (SW. ¼ sec. 16, T. 39, N., R. 14 W)	34.0	19.0	969.4	31.0	1.6
5	SE. ¼ sec. 25, T. 39 N., R. 14 W	39.5	5.5	994.4	25.0	4.5
6	Sec. 31 (near north ¼ stake), T. 39 N., R. 13 W	40.5	1.0	1,004.8	10.4	10.4
7	SW. ¼ sec. 32, T. 39 N., R. 13 W	41.5	1.0	1,011.6	6.8	6.8
8	Harts (SE. ¼ sec. 5, T. 38 N., R. 13)	42.5	1.0	1,019.0	8.4	8.4
9	Sec. 36 (near north-south ¼ line), T. 39 N., R. 13 W	47.5	5.0	1,046.8	27.8	5.6
10	Spooner	49.0	1.5	1,058.0	11.2	7.5
11	Mud Lake dam (above)	52.0	3.0	1,085.0	27.0	9.0

[a] Authority: Nos. 1-9 and 11, U. S. engineers; 10, Chicago, St. Paul, Minneapolis and Omaha Rwy.

Important logging dams are described by United States engineers as follows:

Logging dams on Yellow River.

Name.	Location.	Head.	Capacity.	Remarks.
		Feet.	*Cubic feet.*	
Mud Lake dam	Sec. 27, T. 39 N., R. 12 W	7.5	475,000,000	
Hector dam	Sec. 10, T. 38 N., R. 13 W	7.5		Small capacity.
Rice Lake dam	Sec. 20, T. 39 N., R. 14 W	10.0	700,000,000	Head could be increased to 15 feet.
Yellow Lake dam	Sec. 7, T. 40 N., R. 16 W	18.0	1,400,000,000	Raises water in Yellow Lake 3 feet.

EAU CLAIRE RIVER.

Eau Claire River has its source in lakes of the same name at an elevation of 1,122 feet[a] above sea level. These lakes are surrounded by high banks, so that at small expense a dam could be constructed at their outlet and made to store surplus waters, thus adding greatly to all water power on the river. In its short length of 25 miles this river descends 118 feet, including several rapids, 46 feet of this descent being concentrated in the first 6 miles below Eau Claire Lakes. The total drainage area of the river is 107 square miles.

APPLE RIVER.

Apple River, like the Willow, occupies a comparatively well-settled valley. It drains an area of 427 square miles. The Wisconsin Central, the Chicago, St. Paul, Minneapolis and Omaha, and the Minneapolis, St. Paul and Sault Ste. Marie railways are distant 1 to 5 miles from the river, the last-named road crossing it near Amery. The river has its source in 20 or more lakes, the largest 6 miles long and one-half to three-fourths of a mile wide. These lakes tend to equalize and increase the summer flow. The long and severe winters cause the minimum flow during the months of January and February.

Formerly most of the dams on Apple River were used in connection with logging operations, but the timber is now practically all cut. Flouring mills have been maintained at

[a] Rept. Chief Eng. U. S. Army, 1883.

a number of points, and at others the power is used for electric lighting. There are several projects at the present time which look to large improvements of some of these powers. The river in the first and last thirds of its course runs through the Cambrian sandstone, while its middle third is through the "Lower Magnesian" limestone. In the lower third of its course the river flows over a rocky bed between rocky banks, giving ideal conditions for dams. Most of the larger powers occur in this stretch, and some of these, developed and undeveloped, are described below:

1. The first power on the river is an undeveloped one located about 1½ miles from its mouth. A dam at this point would give a head of 15 feet.

2. The second power, owned by the St. Croix Power Company, is located about 2 miles from the mouth. Here a concrete dam of the arch type, 250 feet long and 47 feet high, develops a head of 82 feet.

3. Four miles from the mouth is a gristmill with a head of 11 feet, owned by E. E. Mason.

4. The next dam, located in sec. 35, T. 31 N., R. 19 W., develops a head of 18 feet.

5. Another dam, located in sec. 31, T. 31 N., R. 18 W., with a head of 22 feet, is owned, under the name of the Apple River Power Company, by the Western Gas and Investment Company of Chicago, which also owns No. 4, described above.

6. A dam 12 miles above the mouth of Apple River gives a head of 29 feet. The discharge at this point is about 80 per cent of the total flow measured at the mouth. This power is transmitted electrically to New Richmond, where it is used by mills and elevators.

The powers on Apple River of less importance are described in the following table:

Minor water powers on Apple River.

Location.	Owner and use.	Head.	Remarks.
Above mouth:		*Feet.*	
13 miles	H. L. Bixby, flour	11	Developed.
13½ miles	M. C. Duggles & Jewett	8	Undeveloped.
15½ miles (Star Prairie)	H. L. Bixby		Do.
25½ miles	J. C. Schnyder, flour	12	Developed.
Sec. 17, T. 12. N, R. 13 W	Winger & Winger	2	Do.
One-half mile above last site	J. Stucky, gristmill	12	Do.
Amery	Northern Supply Co., elevators.	12	One-half total discharge developed.
Blakes Lake	Blake	12	Developed; can be made 18 feet.

There are many other powers above Blakes Lake, with heads of from 6 to 20 feet, mostly old logging dams in poor condition. When the region becomes more settled some of these powers will be improved.

The following data on the discharge of Apple River for the year 1903 are furnished by John Pearson, superintendent of the St. Croix Power Company, Somerset, Wis. The computations are based on the capacity of turbines located at a point 2 miles from the mouth of Apple River. The average daily discharge for each month is as follows:

Estimated daily discharge of Apple River near Somerset, Wis., for 1903.

Month.	Discharge.	Month.	Discharge.	Month.	Discharge.
	Sec.-feet.		*Sec.-feet.*		*Sec.-feet.*
January	258	May	860	September	690
February	239	June	468	October	660
March	600	July	492	November	362
April	555	August	380	December	324

WILLOW RIVER.

Willow River, one of the smaller tributaries of the St. Croix, has a high gradient, due to the fact that its bed lies in the "Lower Magnesian" limestone for its entire length. It drains an area of only 246 square miles and has a length of about 35 miles. In the lower two-thirds of this distance, between Hudson and Jewett Mills, it descends 213 feet, giving many opportunities for water power. Many of these powers are improved, as the river traverses a fairly rich and well-settled country and is paralleled for a considerable distance either by the Wisconsin Central or the Chicago, St. Paul, Minneapolis and Omaha Railway. The powers are here briefly described in order, beginning at the mouth:

1. A timber dam at Hudson 100 feet long gives a head of 16 feet, and with improved machinery would develop 117 horsepower at ordinary low water. A part of this power is used occasionally for electric light when the power described as No. 3 is short of water.

2. Two miles from the mouth of Willow River a dam formerly developed a 9-foot head and was used for driving a flouring mill. At present this dam is washed out.

3. The 130-foot dam of the Willow River Electric Light and Power plant, 3½ miles from the mouth of the river, gives a head of 22 feet, sufficient to develop 200 theoretical horsepower at ordinary low-water flow. The power is used to generate electricity for lighting the city of Hudson, Wis., and for pumping its water supply.

4. A timber dam 100 feet long, 5½ miles from the mouth of Willow River, gives a head of 24 feet, sufficient to develop about 125 horsepower. This power is used for a flouring mill. About 1,200 feet below this dam there is a fall of about 47 feet, and at this point a new dam could be erected, which could be made to include the 24-foot dam above, giving a total head of 71 feet. Such a dam would need to be about 26 feet high and about 70 or 80 feet long. By carrying the water a short distance below in a penstock, a total head of 105 feet could be secured, sufficient to develop about 600 horsepower at ordinary flow of water. This site, being where the river bed changes from the "Lower Magnesian" limestone to the Cambrian sandstone, affords ideal conditions for a dam. The town of Burkhardt, on the Chicago, St. Paul, Minneapolis and Omaha Railway, is located about a mile distant.

5. Seven miles from the mouth of Willow River a 100-foot timber dam gives a head of 16 feet. This power is used to run dynamos.

6. Rapids occur 8½ miles from the mouth of Willow River. A dam 125 feet long at this point, located at comparatively small expense in a narrow limestone gorge, could be made to develop a head of 22 feet.

7. At a point about 11 miles from the mouth of Willow River the Boardman flouring mills were formerly located. The 80-foot timber dam at this point was washed out some time ago, but the mill still stands. If the dam were replaced, a head of 16 feet or more could be easily developed. All the above powers on Willow River are owned by C. Burkhardt, who has the right of flowage wherever needed along this stretch of 11 miles, giving an aggregate descent of nearly 200 feet.

8. The next power on Willow River is located at New Richmond. A timber dam 40 feet long, owned by the New Richmond roller mills, develops a head of 18 feet.

9. The last dam on this stream is located at Jewett, 5 miles east of New Richmond. Power afforded by a 10-foot head is owned by P. Newell & Hennesey and used in a feed mill and sawmill. Above this point Willow River is too small for water-power use.

CLAM RIVER.

Clam River drains an area of 416 square miles. It is formed by two branches—North Fork and South Fork—which unite near the center of the drainage area just above Clam Lake. The river descends about 350 feet in a total length of 50 miles, and, as much of this high gradient is concentrated at rapids, several good opportunities are offered for development. The river flows through a comparatively thinly settled region, which as yet has no railroads. Several railroads, however, cross the margins of the drainage. The following statements regarding its principal water powers are based on information given the writer by Edward L. Peet, editor of the Journal, Grantsburg, Burnett County.

A large, unimproved water power exists in T. 40 N., near the line between Rs. 17 and 18 W. At this point the banks of Clam River are 80 to 150 feet high, and the land which would be flooded is low and of little value. Above the proposed dam the valley bottom will average half a mile wide, with a few expansions to 1½ miles. The bed of the river is clay and bowlders, mixed with sand. Plenty of timber for the construction of a dam grows in the swamps close at hand. Bowlders are also abundant at the dam site. The levels taken on a recent survey show that this power could be improved in the following ways: A dam 6 rods long at the range line would give a head of 20 feet. A dam 10 rods long built farther downstream would produce a head of 35 feet. By adding a 6-foot embankment for a distance of 20 rods this head could be increased to 28 feet; or a dam 60 rods long could be built across the valley with an average height of 40 feet and a maximum height of 85 feet. If the water were conducted by canal a distance of about a mile to the lowlands adjacent to St. Croix River, turbines could be installed with a head of 100 feet. This dam site is distant only 3 miles from other large, undeveloped powers on St. Croix and Yellow rivers, with which it could be easily and cheaply connected by electric transmission.

About half a mile below Clam Lake there is now a logging dam with a head of about 20 feet which raises the water in the lake 3 or 4 feet. This dam impounds the water from a drainage area of 283 square miles. United States engineers reported that a dam would need to be 560 feet long at this point to produce a head of 25 feet. Such a dam would have a capacity of 4,670,786,000 cubic feet,[a] and if properly regulated could be made to greatly increase the amount and value of the powers below. The engineers found that the bed of the river consisted of sand from 3 to 20 feet, at which depths soundings indicated hard materials, supposed to be clay and gravel.

Another large water power is found at Clam Falls, in sec. 13, T. 37 N., R. 16 W., where the river falls over a wide ledge of the "Keweenawan" rocks. A dam at this point impounds the drainage from an area of 45 square miles and develops a head of 34 feet. Between Clam Falls and Clam Lake the slope is small and the river valley half a mile to 1½ miles wide. The river profile is shown in the following table, compiled from surveys made by United States engineers:

Profile of Clam River from its mouth to Clam Falls.

Station.	Distance.		Elevation above sea level.	Descent between points.	
	From mouth.	Between points.		Total.	Per mile.
	Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
Mouth of river			868		
St. Croix, road crossing	6.0	6.0	881	13	2.2
Clam Lake, mouth	19.0	13.0	947	66	5.1
Sec. 35, T. 38 N., R. 16 W., south line	29.0	10.0	967	20	2.0
Clam Falls	32.5	3.5			

NAMEKAGON AND TOTOGATIC RIVERS.

Namekagon River rises in a large lake of the same name near the divide in the watersheds of Chippewa and Bad rivers. Its drainage area is second in extent of all the St. Croix tributaries. Namekagon Lake is formed by six or more connected lakes, occupying parts of 14 sections and surrounded by extensive cedar and tamarack marshes. In the upper 60 miles of its course the river is generally narrow and swift, stretches of rapids over

[a] Rept. Chief Eng. U. S. Army 1880, p. 1619.

pre-Cambrian crystalline rock being frequent.[a] There are also several vertical falls of 2 to 4 feet, which, together with the rapids, furnish good opportunities for water powers. The banks are high on either side, stretching away into high, broken ridges and sand barrens covered with timber. In the remaining 25 miles of its length the river is from 100 to 200 feet wide. In this reach it descends 130 feet, including several sharp pitches and rapids, the principal of which are Little and Big Bull rapids and Dupee flats. The average slope of the river is 5.3 feet per mile.

A good location for a dam is found 4 miles above the mouth of the river, where the high gravel banks approach within 600 feet. A head of 20 feet or more could be obtained here without overflowing much land, impounding the drainage from 1,000 square miles. With the ordinary low-water flow estimated at one-third of a second-foot per square mile, this would produce 740 theoretical horsepower. Because of the storage effect of the present dams above this point, the river at this site might be made to produce nearly 1,000 horsepower. Another good location for a dam is found at Veazie, on the Chicago, St. Paul, Minneapolis and Omaha Railway. By overflowing 6,000 acres, mostly railroad and Government land, a head of 30 feet could be obtained, according to United States engineers. A dam of 15 feet head would cause little overflow. Such a dam would have the run-off from about 800 square miles and at ordinary low water would produce 275 theoretical horsepower. Small dams are located at Stinnett and at the outlet of Lake Namekagon. A dam owned by the Hayward Electric Light and Power Company, located near Hayward, develops 200 horsepower and is used for light and power purposes in that city.

Additional information regarding undeveloped powers is given in the following profile:

Profile of Namekagon River from its mouth to Cable, Wis.[a]

No.	Station.	Distance.		Elevation above sea level.	Descent between points.	
		From mouth.	Between points.		Total.	Per mile.
		Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
1	Mouth of river			±908.0		
2	Sec. 33, T. 43 N., R. 14 W., east side	4.0	4.0	917.8	9.8	2.4
3	Totogatic River, mouth	5.0	1.0	918.0	.2	.2
4	McKinzie Creek, mouth, sec. 28, T. 42 N., R. 13 W.	13.0	8.0	944.0	26.0	3.2
5	Stuntz Brook, mouth, sec. 27, T. 42 N., R. 13 W.	15.0	2.0	952.0	8.0	4.0
6	N. E. ¼ sec. 34, T. 41 N., R. 13 W	16.0	1.0	958.0	6.0	6.0
7	NW. ¼ sec. 6, T. 40 N., R. 12 W	19.5	3.5	990.0	32.0	9.0
8	Sec. 18, T. 40 N., R. 12 W., near center	21.5	2.0	1,004.5	14.5	7.2
9	Sec. 30, T. 40 N., R. 12 W., near center	24.0	2.5	1,024.2	19.7	7.9
10	SW. ¼ sec. 27, T. 40 N., R. 12 W	25.5	1.5	1,025.2	1.0	.7
11	Veazie, sec. 36, T. 40 N., R. 12 W	28.5	3.0	1,039.0	13.8	4.6
12	River Jordan, mouth, sec. 21, T. 40 N., R. 11 W.	35.5	7.0	1,058.0	19.0	2.7
13	Spring Brook, mouth, sec. 15, T. 40 N., R. 11 W.	37.0	1.5	1,068.0	10.0	6.6
14	Chippenacia Creek, mouth, sec. 33, T. 40 N., R. 10 W	43.0	6.0	1,115.0	47.0	7.8
15	Stinnett	45.0	12.0	1,136.0	21.0	10.5
16	Little Puckanance	59.0	14.0	1,218.0	82.0	5.9
17	Cable, Bayfield County	70.0	11.0	1,303.0	85.0	7.7

[a] Authority: Nos. 1-14, and 16, U. S. engineers; 15 and 17, Chicago, St. Paul, Minneapolis and Omaha Railway.

In its length of 55 miles, Totogatic River, the principal tributary of the Namekagon, descends 350 feet. It enters the main stream only 5 miles above its mouth. The region is high and precipitous, with frequent ledges of pre-Cambrian crystalline rock and bowlders. As a result, the stream forms for miles a series of rapids with many vertical falls of 10 feet or more. Many logging dams already exist, the most important being located

[a] Simar, V. B., Asst. U. S. Engineer: Rept. Chief Eng. U. S. Army, 1880, p. 1616.

as follows: Sec. 13, T. 42 N., R. 10 W.; sec. 6, T. 42 N., R. 10 W.; and sec. 12, T. 43 N., R. 10 W. A good site for a dam is near the outlet of Gilmore Lake, in sec. 9, T. 42 N., R. 12 W.; and another in sec. 12, T. 42 N., R. 12 W. The following profile of Totogatic River is compiled from surveys made by United States engineers:

Profile of Totogatic River from its mouth to NE. ¼ sec. 15, T. 42 N., R. 9 W.

Station.	Distance.		Elevation above sea level.	Descent between points.	
	From mouth.	Between points.		Total.	Per mile.
	Miles.	*Miles.*	*Feet.*	*Feet.*	*Feet.*
Mouth of river			918.0		
Sec. 13, T. 42 N., R. 13 W., dam	11.5	11.5	975.5	57.5	5.0
NE. ¼ sec. 10, T. 42 N., R. 12 W	20.0	8.5	1,008.8	23.3	2.7
NE. ¼ sec. 3, T. 42 N., R. 10 W	37.0	17.0	1,168.4	159.6	9.4
NE. ¼ sec. 13, T. 42 N., R. 10 W	40.0	3.0	1,241.6	73.2	24.4
NE. ¼ sec. 15, T. 42 N., R. 9 W	50.0	10.0	1,251.6	10.0	1.0

MINOR STREAMS.

Osceola Creek.—Emptying into St. Croix River a few miles south of Willow River is a small stream known as Osceola Creek. In the city of Osceola, near its mouth, is a water power with a head of 90 feet, owned by the Osceola Mill and Elevator Company. This dam furnishes the power to run a mill with a capacity of 175 barrels per day. One-fourth of a mile above is another dam with a head of 26 feet.

Kinnikinnic River.—A small river emptying into St. Croix River only 5 miles above its mouth bears this name. Its gradient is so high that there are a number of good sites for water powers. The descent in 10 miles is 190 feet. The following is a tabulated statement of its water power:

Water powers on Kinnikinnic River. [a]

No.	Location.	Owner and use.	Head.	Estimated horsepower.	Remarks.
			Feet		
1	2 miles from mouth	N. Kohl, flouring mill	10	70	Timber dam.
2	5 miles from mouth				Good dam location.
3	7 miles from mouth		20		
	River Falls:				
4	3 miles below		14		
5	1 mile below	City waterworks	15	60	Timber dam, 9 by 120.
6	River Falls	do	39	140	Timber dam.
7	do	Geo. Fortune, mill and elevator	8	40	Timber dam, 4 by 210.
8	do	Prairie mill and elevator	14	60	Timber dam, 12 by 180.
9	7 miles above River Falls.	Clapp's mill	10		Dam out.
10	South Branch, sec. 1, T. 27 N., R. 19 W.	W. H. Putnam, feed and flour	50	30	Timber dam, 26 by 114.
11	1 mile above No. 10	Glass Bros., manufacturers	14		
12	Balsom Lake	J. W. Park, lumber and flour		180	

[a] Figures are low-water estimates. Nos. 1 and 5-12 developed; 2-4, undeveloped.

LAKE SUPERIOR DRAINAGE SYSTEM.

TOPOGRAPHY.

The watershed which limits the area of Lake Superior drainage in Wisconsin varies in elevation (above the level of Lake Superior) from 600 feet near the Minnesota line to over 1,000 feet near the Michigan line. Its average distance from Lake Superior is only 30 miles. For this reason the rivers are comparatively small; but owing to the fact that their high gradient, 600 to 1,000 feet, is largely concentrated at a few points, they offer many opportunities for water-power development. From a point near the center of the watershed a wide and nearly flat table-land, of which Bayfield Peninsula and the Apostle Islands form the northern prolongation, separates the drainage into eastern and western sections of nearly equal area. In both of these sections three distinct belts of topography are usually distinguished. The southernmost belt consists of a plateau in large part covered with swamps and lakes and is so flat that in many cases the water from the same swamps and lakes may flow either north to Lake Superior or south to the Mississippi.

From this flat watershed the descent northward is gradual until a range of mountains from 600 to 900 feet above the level of Lake Superior is reached. The northern slope of these mountains is much steeper than their southern slope, forming a marked though not continuous escarpment.

In the western section these mountains, known as Douglas Copper Range, reach a height of 400 to 600 feet above the lake and have a width of 1 to 4 miles. They extend in an east-northeast direction, gradually merging into the Bayfield moraine. From the crest of the mountains there is a sudden descent of 300 to 400 feet, caused by a faulting of the rocks. The Lake Superior rivers break through the ridges at this point, and here the greatest opportunities for water-power development are to be found.

In the eastern section the mountains, called the Penokee Iron Range, extend from a point on the Michigan boundary, 12 miles from Lake Superior, in a southwesterly direction for about 35 miles, gradually merging into the plateau. As in the western section, many falls and rapids occur in breaking through the hard "Huronian" rocks of which the range is composed. Smaller falls continue for a distance of 5 to 6 miles after crossing the Penokee Range, or until the Copper Range has been crossed.

To the north of the highlands and extending with a gradual slope northward to the shores of Lake Superior lies a plain with a width of 5 to 15 miles. Its northern portion reaches an elevation of 100 to 200 feet above Lake Superior or 700 to 800 feet above the sea. The entire belt is underlain by till and deep layers of red clays sometimes mixed with sand. The rivers, both large and small, have cut deep and narrow banks in the clay soil. As a result the surface is carved in every direction by narrow water courses whose steep sides have a height of 25 to 100 feet, making railroad and highway construction expensive. Very few swamps are found in this lowland area. Because of the gradual slope of the shallow rivers opportunities for water-power development in this belt are rare. In many cases, however, there are important falls at the immediate mouths of the rivers and over the red sandstone.

WATER POWERS.

CHARACTER.

Owing to the fact that the rivers of the Lake Superior system in Wisconsin have a total fall of 400 to 1,000 feet in the narrow belt of 30 miles separating the plateau region in which they rise from Lake Superior, their currents are characteristically rapid. As a result the rainfall is quickly discharged, the streams alternating between small creeks and torrential rivers. While the storage of surplus waters is important everywhere in the State for the economical development of water power, it is here doubly so. The fact that the most important falls and rapids are in the upper half of the drainage area increases the difficulty of storing a large proportion of the rainfall. With a storage of less than 5 to 15 per cent of the rainfall most of the rivers would furnish at low water an insignificant flow.

Rainfall data regarding this drainage area are scanty, but sufficient to show that the rainfall increases from the lake to the highlands. This fact is strikingly shown by the precipitation map published by the United States Weather Bureau and shown in fig. 1 (p. 16). It is here seen that the rainfall increases southward at the average rate of about 5 inches every 25 miles, the maximum not being reached until after the highlands are passed. This fact has an important bearing on the value of the water powers, because, as already stated, it necessitates the location of reservoirs to a large extent in this region of greatest rainfall. The most important water powers occur near the Copper ranges and the Penokee Iron Range, where future mining operations may render them of much economic importance.

ST. LOUIS RIVER.

Although the water powers of St. Louis River lie outside the State, they are located so near the Wisconsin boundary that development contemplates their extensive use in Supe-

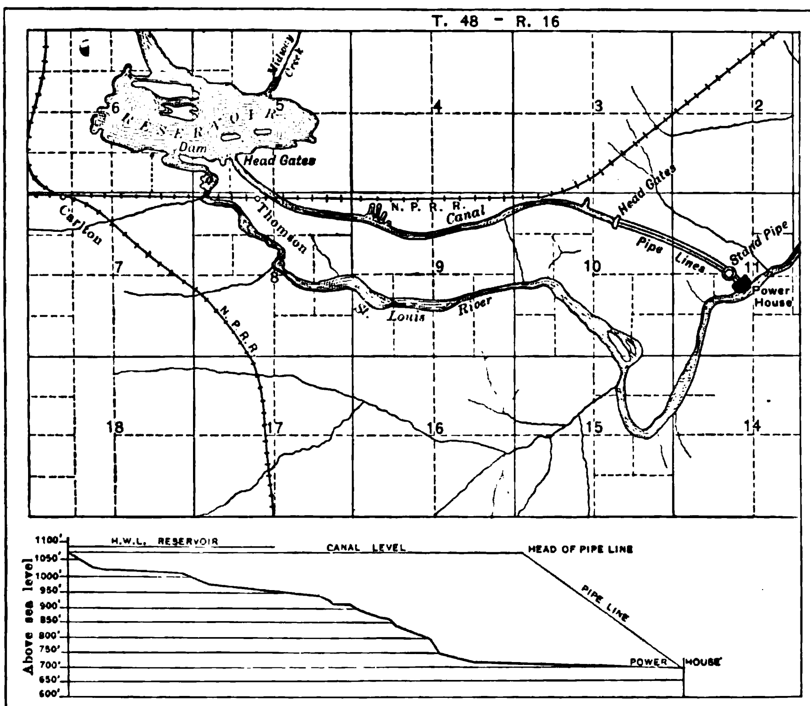


FIG. 5.—Plan of canal of Great Northern Power Company, St. Louis River.

rior and other Wisconsin cities. An important feature of St. Louis River is the concentration of its descent in the lower reaches, where its volume is greatest. This provides opportunities for water power which if distributed among its smaller tributaries would be in large part wasted. The upper portions of St. Louis River are sluggish, flowing through many lakes and swamps, but as the waters near the lake their speed is increased until at a point about 22 miles from Lake Superior, just above Fond du Lac, there is a series of falls and rapids extending 6 miles upstream from a point 2 miles from the Wisconsin boundary. In this distance of 6 miles the river descends 456 feet in a series of wild leaps over the upturned ledges of slate rock, forming a water power which has few superiors in the West. This power and the riparian rights are owned by the Great Northern Power Company. Mr.

F. A. Cokefair, chief engineer of the company, furnishes the following statement under date of January 23, 1904:

One steel gravity dam 36 feet high and 620 feet long has already been constructed near the village of Thompson. This dam conserves the water in a reservior of about 1 square mile of area, from which the water is led through a canal 2½ miles long, 62 feet wide, and 15 feet deep. (Fig. 5.) From the terminus of the canal the water is taken by iron pipes for a distance of about a mile and delivered under a head of 365 feet at the power house midway between Thompson and Fond du Lac. The capacity of this canal is sufficient to develop 100,000 horsepower. Final plans and designs are now completed, and work on the first station for the ultimate development of 100,000 horsepower will begin in the early spring. Bids have been asked from leading manufacturers for the first three wheels of 12,500 horsepower capacity each, the largest units ever yet built and similar to those in use at Niagara Falls. Transmission lines will carry the power to the neighboring cities of Duluth and Superior, and even farther to the Mesabi and other iron ranges, where it will augment or displace steam power.

NEMADJI AND BLACK RIVERS.[a]

Unlike other rivers of the Lake Superior watershed, Nemadji River flows northeast instead of north and does not rise in an elevated region. As a result it is devoid of important rapids or falls suitable for water power.

Black River, the most important tributary of the Nemadji, rises in an elevated country, its source being in a lake on the Minnesota boundary. It flows north and empties into Nemadji River about 10 miles from Lake Superior at an elevation of only 20 feet above the lake. In the upper two-thirds of its length Black River flows through many tamarack and cedar swamps, which give to its waters a distinct color and taste. Up to about 4 miles from the Douglas Copper Range it occupies a wide valley with small descent. As this range is approached the valley narrows and its gradient increases. In the SE. ¼ sec. 28, T. 47 N., R. 14 W. the hard layers of the "Keweenawan" rocks cross the river, producing a vertical fall of 31 feet. A total head of 160 feet[b] could easily be obtained here for a dam site. As Black River has a drainage area of 80 square miles above these falls, an assumed run-off of 0.4 second-foot per square mile gives 560 theoretical horsepower. A company was formed some time ago to improve this power, and a franchise was secured from the city of Superior for lighting by electricity, but no construction has yet been done. The water at the head of the upper rapids is 387 feet above Lake Superior; at their foot, 50 yards beyond, the elevation is 227 feet. From this point the river passes for nearly a mile through a gorge 100 to 170 feet deep, below which the walls of the gorge are less elevated above the stream, but the current is very rapid until it joins Nemadji River 4 miles below. From the foot of Black River Falls to the junction with the Nemadji the total descent is 200 feet, an average of 50 feet to the mile.

BOIS BRULE RIVER.

Though over 33 miles long, Bois Brule River has a drainage area of only 200 square miles, practically all of which is in the highland district. It rises in a swamp, near St. Croix Lake, at an elevation of 420 feet above the level of Lake Superior. In sec. 15, T. 46 N., R. 10 W., at the Dalles, Bois Brule River is only 25 feet wide, with banks of clay and bowlders averaging 8 feet in height. Near this point there are swift rap ds, with a total descent of about 15 feet in 200 yards. Similar rapids about 3 miles farther north, near the township line, continue as far as the mouth of Nebagemain River, the most important tributary of the Bois Brule, in sec. 27, T. 47 N., R. 10 W. For the next 10 or 12 miles the current is very sluggish until the head of the lower rapids is reached, in sec. 26, T. 48 N., R. 10 W. From this point to within 1½ miles of Lake Superior rapids and small falls (the largest being 4 or 5 feet in height) occur almost continuously. These descend an aggregate of 200 feet over "Keweenawan" eruptives and sandstones. By constructing dams at the outlets of Lakes Nebagemain and Minnesung the surplus water could be held

[a] The authority for most of the statements concerning the Lake Superior rivers is Prof. R. D. Irving: Geology of Wisconsin, vol. 3, 1880.
[b] Sweet, E. T., Geol. Wisconsin, vol. 3, 1880, p. 319.

back and used at times of low water, thus adding greatly to the value of the water powers on the river. At present there are no dams. Mr. Howard Thomas, city engineer of Superior, Wis., states that the normal discharge of this river is 100 second-feet, and that at several points heads of 40 feet could be obtained by dams between bluffs or with dams and flumes along the banks. Such a head would give 450 theoretical horsepower. Because of its comparatively small watershed and the fact that the river is fed very largely by springs it is not subject to freshets.

MONTREAL AND GOGOSHUNGUN RIVERS.

For nearly its entire length Montreal River forms a part of the Michigan-Wisconsin boundary. It rises in a tangle of lakes and tamarack swamps near the boundary line at an elevation of about 1,600 feet above sea level, or 1,000 feet above Lake Superior. Its length is 50 miles, the highest gradient being concentrated in the last quarter of this distance. This exception to the general rule of the Lake Superior drainage area is due to the fact that here the Penokee Iron Range and its associated highlands of the "Keweenawan" series approach Lake Superior within a distance of only 3 miles, leaving no lowland region.

About 1,300 feet from its mouth, on the north line of sec. 7, T. 47 N., R. 1 E., is a vertical fall of 35 feet over sandstone. It is stated by an officer of the Duluth, South Shore and Atlantic Railway that a head of 55 feet could be developed here by constructing a flume 100 feet long. Because of the lakes and swamps at the headwaters of this river it is likely that at least 5 per cent of the annual rainfall could be stored in reservoirs. This would give, from its 280 square miles of drainage area, an ordinary flow of 140 second-feet, equivalent, with a head of 55 feet, to 868 theoretical horsepower. In the last five-eighths of a mile of its course Montreal River descends 90 feet. The railway official mentioned above also states that another power site is located in the NW. ¼ SW. ¼ sec. 21, T. 47 N., R. 1 E., at falls of 60 feet over the crystalline rocks. As the banks are high, a 20-foot dam, with a flume 250 feet long, would develop a head of 80 feet. Both of the above powers are within 4 miles of the Duluth, South Shore and Atlantic Railway. At Ironwood, about 2 miles above these falls, the river has an elevation of 880 feet. In the 5 miles above Ironwood the river descends only 30 feet, and for the remainder of its upper reaches its current is slow. At all the rapids on this river the conditions are favorable for the building of dams.

The Gogoshungun, a branch of the Montreal, is nearly as large as the upper Montreal, being about 30 miles long. Its total descent is 500 feet. Until the river reaches the Penokee Range its current is sluggish, being bordered by swamps. In its passage through the mountains, in sec. 27, T. 46 N., R. 2 E., a number of rapids and falls occur.

BAD RIVER.

MAIN RIVER.

The sources of Bad River lie in large swamps 8 miles south of the Penokee Iron Range, at an elevation of 900 feet above the level of Lake Superior. In this distance of 8 miles its descent is 110 feet, but its course is sinuous, as may be inferred from the fact that the Wisconsin Central Railway is forced to cross it eight times. About 1½ miles above Mellen are rapids called Copper Falls, which have a total descent of about 60 feet. (Pl. V, *B*.) The river at this point has a drainage area of about 144 square miles. According to a survey, 5 per cent of the annual rainfall could be easily stored in dams near the headwaters, which should provide an ordinary flow of 68 second-feet, equivalent to 460 theoretical horsepower.

Near the Penokee Range Bad River enters a gorge of pinkish granite, narrowing in places to a width of 10 feet and descending 20 feet in 30 rods, with a total descent of 50 feet in three-fourths of a mile. The river then widens and continues with reduced grade until Penokee Gap is reached, when it again contracts. Coming into contact with the "Huronian" rocks, it flows along their strike. In the next 4 miles occur many rapids and several falls, including one of 35 feet. In the next 1,000 feet, in which the river descends 40 feet, Tylers Fork, the most important tributary, is reached. Directly at the junction Tylers Fork has a fall of 45 feet over the wall of a gorge 65 feet deep. This is in sec. 17, T. 45 N., R. 2 W. A competent engineer, reporting on this water power, states that dams could develop here a head of about 120 feet. The tributary drainage area is given at 234 square miles. On the assumption that the rainfall is only 32 inches and that reservoirs can be made to store 15 per cent of the rainfall, it was estimated that the river would furnish a continuous flow of 206 second-feet, equivalent to about 3,000 theoretical horsepower. It was proposed to conduct this power electrically to Ashland.

In the next 1,000 feet below Tylers Fork the river flows through a rocky gorge 100 feet deep, beyond which the rocks disappear and the stream flows between high banks of red clay, the ground rising rapidly on both sides. The total descent in sec. 17 is probably 135 feet. In the next 6 miles of its sinuous course, to the mouth of Maringouin River, the river descends about 30 feet to the mile. Both rivers at their confluence are broad and deep, with slow-moving, muddy currents and wide bottom lands—conditions which continue to the mouth of Bad River.

Farther north, 2½ miles from this junction, Bad River receives the waters of Potato River. At this point its elevation is 80 feet above the level of Lake Superior. In sec. 25, T. 47 N., R. 3 W., occur some small falls, of 1 or 2 feet, over red sandstone and shale, which continue for perhaps 2 miles. Below these falls Bad River continues sluggish, deep, and tortuous, with bold and high clay banks, until White River is reached. For the remainder of its course the river finds its way to Lake Superior through swamps.

TRIBUTARIES.

The principal tributaries of Bad River, named in order from its mouth, are as follows: White River entering from the west; Potato River from the east; Maringouin or Mosquito River from the west, and Tylers Fork from the east.

White River.—This river, the largest tributary of Bad River, has a total length of about 45 miles, and drains an area of 400 square miles. It rises in Long Lake, at about 700 feet above the level of Lake Superior. Most of its descent is concentrated in its upper waters, where its discharge is least. It pursues a general northeasterly course, with many windings through high and steep clay banks, like those described on Bad River. Its only considerable falls are in sec. 6, T. 46 N., R. 4 W., where the river was originally obstructed by the edges of southward-dipping rocks. A dam with a 20-foot head has been maintained here for several years, and until October, 1903, furnished the power to run a paper mill. At that time the mill burned, and it has not been rebuilt. It had turbines rated at 710 horsepower. The owner, George Davidson, reports that he has a charter for a dam with 30-foot head, to be located about 1,300 feet upstream. The main dam as planned would be 125 feet long, with an embankment 10 to 12 feet high and 900 feet long. Mr. Davidson also states that about 500 feet below the present dam there is a location for a dam with a 9-foot head. At three dam sites the bed of the river is in sandstone which extends 10 feet above the water surface. The rock is overlain with red clay.

Maringouin River.—Maringouin River, sometimes also called Maringo (Mosquito) River, has a total length of about 40 miles and drains an area of 231 square miles. Four miles from its source it crosses the Penokee Range. Here, in the NW. ¼ sec. 23, T. 44 N., R. 5 W., the river descends, in a series of three falls, a total distance of 65 feet within a few rods. The two upper falls, of 15 and 25 feet, respectively, are only 50 feet apart. Nothing but the limited amount of water prevents this from being a valuable water power. For the remainder of its course the river is devoid of falls or rapids, flowing between high clay banks.

Within 6 miles of its junction with Bad River, the Maringouin receives several rapid tributaries, the most important of which is Brunsweiler Creek. This creek rises in the same swamp with Maringouin River, but, unlike it, has important falls north of the "Huronian" hills. Until Bladder Lake is passed in sec. 11, T. 44 N., R. 4 W., the current is sluggish. The outlet of this lake is only 6 feet wide, with rock walls on either side. A dam which would greatly raise the water in the lake could be constructed here at slight expense. At the outlet of the lake there is a long series of chutes and rapids for a distance of over 6 miles. In this stretch the creek flows through a narrow valley with steep, rocky hills. The last important descent occurs near the north line of sec. 22, T. 45 N., R. 4 E., where the stream leaves the Copper Range, the slope being 30 feet in a distance of 130 feet.

Tylers Fork.—This tributary is the only one which joins Bad River before the lowlands are reached. Tylers Fork, nevertheless, has a length of 30 miles and a total descent of 700 feet. Until it reaches the Penokee Range its current is sluggish. In the NE. ¼ sec. 33, T. 45 N., R. 1 W., the river falls 20 feet over the hard "Huronian" rock. Less than a mile farther on, in sec. 28, occurs a series of low falls over black slate, the descent being 20 feet in a distance of 500 feet. On the north line of sec. 20 the river surface is 760 feet above the level of Lake Superior. In the next 10 miles of its course it descends 260 feet, but without any considerable rapids. On the west line of sec. 15, T. 45 N., R. 2 W., the elevation of the water is 485 feet. The current now becomes swifter and about a quarter of a mile below the east line of sec. 16 is a series of rapids which continues to its junction with Bad River, ending in the 45-foot fall already described (p. —). As these falls and rapids are within a mile of the Wisconsin Central Railway, they seem destined to become of some economic importance.

Potato River.—In its course of only 30 miles, Potato River has a total descent of over 900 feet. The river is small until it is joined in sec. 15, T. 46 N., R. 1 W., by Little Potato River. From this confluence a course nearly due west for 12 miles takes it to Bad River. Near the east line of sec. 17, T. 46 N., R. 1 W., at 428 feet above the level of Lake Superior, is a series of rapids followed by a series of cataracts. These rapids begin on the east line SE. ¼ SW. ¼ sec. 17, T. 46 N., R. 1 W., and are in the trap rock. In the next quarter mile abrupt descents of 10, 4, and 40 feet occur, with swift water between. A still larger fall of 60 feet or more is located near the west line of sec. 17, and as the banks are high and precipitous, a suitable dam would develop a head of nearly or quite 100 feet. On both sides of the west line of sec. 17, about 2,000 feet north of the southwest corner, is a series of bold falls having a total descent of 80 feet in a distance of 500 feet, with two leaps of 25 feet and 32 feet, respectively. The total fall in secs. 17 and 18 is 170 feet. These falls, being over solid rock of conglomerate and sandstone, furnish ideal conditions for dams. Below sec. 18 the river course is tortuous and slow.

MINOR RIVERS.

Aminicon, Middle, Poplar, and Iron rivers are small streams in Douglas County. They are all swift streams with many small falls, but are subject to great variations of flow, being insignificant at low water. A corporation known as the Iron River Water, Light and Power Company has recently constructed a dam 135 feet long, with a head of 32 feet, on Iron River, in sec. 22, T. 47 N., R. 10 W., the intention being to install turbines of 1,000 horsepower, which will be transmitted to near-by towns.

RAILROADS.

All the falls which occur near the Penokee Range on Bad River and Tylers Fork are near the Wisconsin Central Railway. Montreal and White rivers are crossed by the Duluth, South Shore and Atlantic, the Chicago and Northwestern, and the Wisconsin Central railways. The western half of the Lake Superior watershed has good transportation facilities. Branches of the Great Northern Railway cross the valley of Black River and follow the valley of Nemadji River. Besides these the drainage is crossed by the Northern Pacific, the Chicago, St. Paul, Minneapolis and Omaha, and the Minneapolis, St. Paul, and Sault Ste. Marie railways, and by minor logging roads.

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[Water-Supply Paper No. 156.]

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