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HYDRAULIC MANUAL.

CONSISTING OF

WORKING TABLES AND EXPLANATORY TEXT

INTENDED AS

A GUIDE IN HYDRAULIC CALCULATIONS AND FIELD OPERATIONS

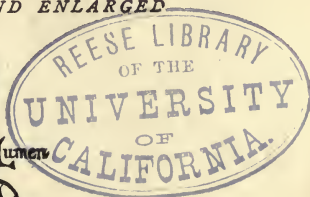
BY

LOWIS D'A. JACKSON

AUTHOR OF 'CANAL AND CULVERT TABLES' 'AID TO SURVEY PRACTICE'
'ACCENTED FIVE-FIGURE LOGARITHMS' 'MODERN METROLOGY'
AND OTHER WORKS

FOURTH EDITION.

REWRITTEN AND ENLARGED



LONDON

CROSBY LOCKWOOD AND CO.

7 STATIONERS'-HALL COURT, LUDGATE HILL

1883

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TC179
J32

LONDON: PRINTED BY
SPOTTISWOODE AND CO., NEW-STREET SQUARE
AND PARLIAMENT STREET

24540

PREFACE
TO
THE FOURTH EDITION.

IN this edition, some alterations and extensive additions have been made. Chapter I. remains generally as before, the alterations being comparatively small ; in the portion devoted to Sections of Flow, the quotation from Neville's work has been expunged, and that subject has been newly treated ; in the portion devoted to Distribution of Velocity in Section, full advantage has been taken of the deductions made by Major Allan Cunningham, and these have been inserted with his consent, but also with some modification for which he is not responsible ; the references to Box's work and to Stoddard and Dwyer's works have been entirely expunged ; and the whole chapter has been revised.

In Chapter II., a summary of the methods of gauging and of the operations of Major Allan Cunningham in his recent experiments on the Ganges Canal, has been added. This has been reprinted from 'Engineering' with the consent of the editor, and with that of Major Cunningham. This chapter has also undergone revision.

In Chapter III., the section on Irrigation in Italy, being partly statistical, has been removed, and the sections on Irrigation from Wells in India and on Indian Hydraulic Contrivances have been omitted. Other sections on Field Drainage, on the Ruin of Canals, and on Water-Meters have been added. Most of the sections in this chapter are modified reprints of articles originally written for 'Engineering,' that have been reproduced with the consent of the editor of that paper.

The working tables have been increased from a hundred to a hundred and eighty pages, and enlarged in other respects; they have also undergone some rearrangement, though not sufficient to confuse those accustomed to the use of the tables in their old form.

As the Hydraulic Statistics would render this book too bulky, they will probably be published in a separate volume after enlargement.

It may be noticed that although this book has progressively enlarged in size and come into more common use since its first edition, started in 1868, was written, the same general principles have been adhered to from the commencement; the same limited object has been kept in view; and the same opposition to old hydraulic text-books and old formulæ has been unswervingly advocated. But the methods formerly treated as Ishmaelitic by the professional public are now fully recognised as being in accordance both with sound reason and with extensive experiments; and the book has become a standard text-book in India, in Russia, and in the English-speaking colonies, as well as a familiar book of reference in England and in the United States of America.

For this development in its early stages much gratitude is due to several persons who did not spare labour in examining these principles and proposals, or in furthering their acceptance by the professional public ; among them more especially to the editors of 'Engineering,' from 1868 till now ; to General Sir Richard Strachey and Colonel Yule, Members of the Council of India ; to General Boileau, R.E., Clements Markham, and Mr. Black ; to General Gorloff, Military Attaché to the Russian Embassy, and to Admiral Possiett, Minister for Public Works ; to B. Baker, and to J. F. Schwartz and W. D. Haskoll, civil engineers ; to Mr. R. Hering, C.E., of Philadelphia, and to Mr. Theodore G. Ellis, of Hertford, Connecticut. Besides, recognition is due to several kind friends for their support and countenance ; to all the fullest acknowledgment is offered.

The typographic arrangement of the tables has been remodelled under special instructions, and will, it is hoped, render reference more convenient, and reduce risk of error in working with them.

Although the science of hydraulics has not yet arrived at such a stage that results, obtained from tables on the most modern and correct principles, can be presumed to be accurate within 5 per cent., it is yet a matter for congratulation that the errors can be now reduced to somewhere near this amount generally. Formerly, under the methods of Dubuat, De Prony, and Bazin, Beardmore, Leslie, and others, the liability to large error, 50 per cent. and more, was serious ; and English hydraulicians were behindhand in such matters. Now, the laborious experiments of Major Cunningham on large

canals have settled matters on a stable basis, and placed the English far in advance of their Continental contemporaries. A new era of steam traffic on canals for heavy goods may perhaps lead to further progress.

LONDON : *December* 1880

L. D'A. J.

The publication of Major Cunningham's large work, comprising accounts of the whole of his experiments, has caused large additions to be made to this book during its passage through the press.

LONDON : *September* 1882.

L. D'A. J.

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HYDRAULIC MANUAL.

CHAPTER I.

EXPLANATION OF THE PRINCIPLES AND FORMULÆ ADOPTED IN CALCULATION AND APPLIED IN THE WORKING TABLES.

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I. HYDRODYNAMIC THEORIES.

THE science of hydraulics, yet in its infancy, may be said to depend, as far as its practical application by the hydraulic engineer is concerned, on a combination of certain known laws with the empirical results of observation and experiment; the former few in number, and eliminated principally by the philosophers and mathematicians of the past; the latter also few, and, if we except the old observations which were carried out on a very petty and limited scale, exceedingly modern. Previous to the experiments of d'Arcy in 1856, little

was known about the velocities and discharges through pipes ; until the operations of Captains Humphreys and Abbot on the Mississippi in 1858, the discharge of large rivers was a comparatively unexplored subject ; in 1865 the experiments of Bazin led the way to a more accurate knowledge of the discharges and velocities of water in small channels and culverts, and the effects of roughness of surface and variety of material on these velocities. In 1870 Kutter and Ganguillet, from observations on Swiss hill-streams, deduced a more exact law for effect of declivity on discharge, and besides added greatly to the knowledge of effect of roughness. In 1880 the extensive experiments of Captain Allan Cunningham on the Ganges Canal had substantiated the truth of Kutter's laws when applied to very large canals, and dealt the final blow to the velocity-formulæ of all the older hydraulicians.

Before 1856 the less important subjects alone had been investigated to any practical purpose, such as the vena contracta, the discharges through small orifices, over certain forms of overfall, and through short and small pipes, the discharges from reservoirs, and the velocities in troughs 18 inches wide. There was, however, plenty of theory, and a large number of formulæ, some of them exceedingly complicated in form, mostly resulting from a number of superimposed theories, the more ancient of which were based on very limited experiments : in fact, the mode often adopted seems to have been to assume a new form of formula, and to prove it by a few partial experiments, a principle worthy of ancient soothsayers, and which, had it been further supported by traditionary and name-reverencing hydraulic schools of believers, could only have resulted in prolonged and permanent

error. Even now a reference to some works comparatively recently published in England will show formulæ to be supported by a most heterogeneous collection of experimental data ; discharges of pipes irrespective of their material or internal surface, of large and small rivers irrespective of the quality of their beds and the bends in their courses, of canals in any material, down to wooden troughs, all seem to prove the correctness of a fixed formula having an unvarying constant coefficient. Other works again, having greater accuracy of result in view, go to the opposite extreme in method and recommend the adoption of two distinct formulæ for cases in which the principles involved do not vary in the least, as for instance, in discharges through pipes with low velocities, a formula distinct from that for those with high velocities is often adopted ; this, amounting to a method of successive approximation imperfectly worked out, is almost as unfortunate as the other. From a continuance of this, however, the modern experiments have already saved us to a great extent, and further and more extended experiment will probably relieve us from it altogether.

Taken generally, the mass of hydraulic science and of hydraulic data bearing on the flow of water under various conditions, prior to about 1856, may be considered superannuated, defective, and often excessively misleading. Old hydraulic data, such as discharges of rivers, canals, and pipes, seldom can afford the means of arriving near the truth, unless accompanied both by the formulæ used by the observer, and by a large number of conditions of the case, then mostly neglected.

At present the hydraulic engineer is still quite as dependent for correctness of calculated result on the so-

called empirical data, obtained by experiment and put into convenient form, as on purely abstract theories or laws. The correct application of all known mechanical laws cannot, however, fail to be valuable in cases admitting of them; those relating purely to hydrodynamics are comparatively few, and the most important and best known of them are the four following:—

First, uniform motion.—If fluid run through any tube of variable section kept constantly full, the velocities at the different sections will be inversely as the areas, or

$$A V = A' V'.$$

This theory of uniformity of motion is also supposed to hold generally with reference to mean velocities of discharge in open channels under constant supply. This is actually little more than assuming a theoretical velocity that will fulfil the conditions of the law, in order to render calculation convenient, for there is no reason to believe that actual velocities in a tube of variable section would all vary inversely with the area of cross section.

Second, velocity of issue.—The velocity of a fluid issuing from an orifice in the bottom of a vessel kept constantly full, is equal to that which a heavy body would acquire in falling through a space equal to the depth of the orifice below the surface of the fluid, which is called the head on the orifice; or by way of formula

$$V = (2 g H)^{\frac{1}{2}}$$

where H = the head and g = force of gravity. The quantity g represents the accelerating force of gravity, which varies at different places on the earth's surface and elevations above the mean sea level, and is also affected

by the spherical eccentricity of the earth at the place, a quantity that again varies with the latitude ; above the earth's surface g varies inversely with the square of the distance from the earth's centre, below the earth's surface direct with the distance from the earth's centre ; to obtain the exact value of g , d'Aubuisson's formulæ applied to English feet are—

$$r = 20\,887\,540 (1 + 0\cdot001\,64 \cos 2l)$$

$$g = 32\cdot1695 (1 + 0\cdot002\,84 \cos 2l) \left(1 - \frac{2e}{r}\right).$$

The values of this formula for different latitudes and elevations are given in Working Table No. I., and the values of g , obtained from observation at different latitudes, are given in Table No. I. of the Hydraulic Statistics. For purposes of ordinary calculation in England, and hence throughout these tables, g is generally taken as 32·2 feet per second ; in India, however, it would be more correct to use 32·1 ; but the convenience of using English data will probably outweigh the need of this exactness until the science of hydraulics can arrive at higher accuracy.

The above theory supposes that the orifice is indefinitely small, and neglects the conditions and size of its sectional area, friction, the pressure of the atmosphere, and the resistance of the air to motion (which increases with the square of the velocity of the issuing fluid) ; the practical application of it that shows its discrepancies most strongly is the fact that the height of a jet is never equal to the head of pressure on it.

Third, general theory of flow.—This is a combination of the two previous theories in a modified form, assuming both uniform motion and the principle of

gravitation, and is best expressed in the form of a formula—

$$V = (f g R S)^{\frac{1}{2}}$$

where V = the mean velocity generated,

R = the mean hydraulic radius of the water-section,

S = the hydraulic slope or sine of the slope of the water-surface.

This formula is a simple equation of the accelerating force of gravity down an incline with the retarding force of friction at any section at right angles to the course of flow, namely :—

$$g S = \left(\frac{V^2}{Rf} \right)$$

since, for uniform motion, the total accelerating force is equal to the total resistance.

This theory is the basis of calculation of flow in full tubes, and in open channels and unfilled pipes, where the principle still holds, but f then becomes a symbolic representation of retardation due to a combination of various causes, including direct friction on the general incline at the given section.

Fourth, the principle of retardation.—This is represented by a collection of various small formulæ and methods of making allowance for loss of velocity under different conditions by a calculated head. These retardations may be introduced into any general formulæ, or may be treated separately. The ordinary sources of retardation are :—

1. Roughness of surface, varying from that of polished glass to rock-strewn or deeply-incised rocky torrent-beds ; also surface-adhesion of liquids.

2. Irregularity of form, varying from that of a recently made and trimmed rectilinear canal of one single uniform inclination and direction down to that of a river bed consisting of an infinity of heterogeneous planes and curved surfaces. Any departure from uniformity, lateral and vertical deviations and bends.

3. Varying head, inconstant pressure, diminution of supply, loss of effective head from excess of withdrawal.

4. Contraction at exit, want of perfect freedom of fall, backwater, contraction of passage, obstacles.

5. Air resistances and effect of wind ; atmospheric pressure ; differential liquid pressure internally.

6. Low specific gravity of the liquid in motion, turbidity, viscosity, and variation in weight.

7. The effect of variation of heat inducing motion in the liquid, and thus producing perturbation, and the minute effects of local change of temperature generally.

8. Absorption of velocity by yielding material, which may imperfectly deflect velocity, and partly absorb direct action.

However rigid these theories may appear in neglecting important points, they are yet generally true in the abstract, and no substitutes for them have yet been discovered ; the consequence is that all hydraulic calculations are made to depend on them, their defects being compensated by using experimental coefficients. It becomes, therefore, one of the important duties of a hydraulic engineer to apply these principles with care and circumspection, especially guarding against taking for granted the formulæ and tabular results of different calculators, which vary in form and in result to a very great extent ; some authors even giving a half more discharge than others as due to the same data. During

practical work, time forbids a lengthy examination of principles ; for this reason, therefore, this short chapter is given as an easy guide to the proper management and application to every-day wants of the Working Tables that follow.

2. NOTATION, SYMBOLS, AND UNITS OF MEASURE.

To ensure clearness and rapidity of application of these theories, it is absolutely necessary that the nomenclature should be neither doubtful nor inconvenient, that the symbols be free from confusion, and the units of time, weight, and measurement, once adopted, generally adhered to as much as possible ; this alone can cause the form of a formula to give at a glance any definite idea of the values of its terms and expressions. Decimalised measures are also necessary for the same purpose.

The English foot has been generally, though not quite exclusively, adopted in this work as the unit of length, surface, and capacity, being the measure ordinarily used for heights and depths, as well as distances in survey work, and being now more capable of extended application than either the yard, link, or inch. The footweight, or weight of a cubic foot of water at its utmost density (the English talent), has been taken as the unit of weight, being now a recognised legal standard unit. The whole system of decimal measures founded on these are on the scientific scale at 32° Fahrenheit, so as to afford exact correspondence between cubicity and weight, and to admit of facile conversion to metric values. The second has been generally taken as the unit of time, so that the numbers

expressing discharges and velocities, which often are high numbers, may be as small as possible. This has been found to be perfectly manageable in practice. In the canal departments of Northern India the engineers have succeeded in abolishing poles, yards, and inches from their plans, estimates, and calculations, and in adhering generally to the second as a unit of time; they have also, on the Bari Doab Canal, adopted the old London mile of 5 000 feet to the exclusion of the statute mile of 5 280. The league of two such miles, or 10 000 feet, being a decimal unit, is now far preferable. The acre, pole, and old Gunterian chain of 4 poles being highly inconvenient, the substitutes for these are the rod of 10 feet; the chain of 100 feet (Ramsden's); the square chain of 10 000 feet, nearly a rood; the century, or square cable, of 100 square chains; and the square league of 100 centuries. This decimal system of measures, though retaining the use of a familiar unit, and saving much needless labour in calculation, at the same time has some difficulties to contend with, the principal of which are the old habits of measuring water-supply for towns in gallons instead of cubic feet, and of using dimensions of pipes in inches, instead of tithes or tenths of a foot, estimating pressure on the square inch instead of on the square foot and square tithe; these obstacles will probably gradually disappear.

As regards the French metric system, although it is now adopted for external commerce in most civilised countries in Europe, there seems little chance of its entirely replacing our own measures. English scientific measures are naturally more convenient for an Englishman to think and calculate in, and are in closer accordance with English commercial units adopted in trade,

manufactures, and contractors' plant and appliances; besides, natural units are preferable to artificial ones.

The hydraulic engineer more especially can adopt the decimal system of measures based on the English foot with extreme convenience; nor apparently are there any very good reasons why the railway engineer should not do so also, except perhaps the tradition-loving habits of the multitude, and the meddlesome legislation in social matters under which we suffer, which enforces on him the adoption in Parliamentary plans of the whole of the old measures, with the alternative of using foreign measures. This difficulty will perhaps be eventually removed by permissive legislation, allowing the use of the complete English decimal series for all technical, engineering, and scientific purposes, apart from ordinary trade, and fixing the standards finally on the principles proposed and long advocated by the author,—namely, at a single normal temperature in vacuo, the single temperature both for material and for water being that of the maximum density of distilled water,—a method far superior to the dual temperature of the French system. In the meantime it may be remembered that decimalisation on any English units is permissive under the Act of 1878, thus actually including the whole of the English decimal scientific system; while there exists no legal prohibition of the *ad interim* temperatures 32° and 39° in vacuo used in French measure.

The advantage of adhering to one set of symbols in hydraulic formulæ, which sometimes appear very complicated, is sufficiently evident; with this view, therefore, the following general notation is drawn up. The velocity notation of the Mississippi survey is also attached for purposes of reference.

General Notation.

F = the rainfall expressed in depth.

K = catchment area drained.

Q = quantity of water discharged in cubic feet per second.

V = mean velocity of discharge in feet per second.

V_x = corresponding maximum velocity in the same section.

U = vertical velocity, or velocity past a vertical line or axis.

Y = transversal velocity, or velocity past a transversal, or transverse axis.

A = sectional area; a , a_1 , a_2 , subsidiary areas.

P = wetted sectional perimeter, exclusive of the surface-width, W .

R = the mean hydraulic radius = $\frac{A}{P}$.

R_1 = diminished hydraulic radius = $\frac{A}{P+W}$

R_2 = augmented hydraulic radius = $\frac{A}{P-W}$

S = hydraulic slope or gradient in terms of its sine = $\frac{H}{L}$;

thus $S = \frac{1}{500} = 0.002$ for a slope of 1 in 500.

L = a longitudinal length taken in the direction of flow.

H = the fall on any such length; or a vertical head of pressure.

h = difference of level of the water surface at the two ends of L .

h_l = the part of h consumed in overcoming longitudinal channel resistances, for a straight, regular course.

h_u = the part of h consumed in overcoming transverse channel resistances or irregularities.

W = a transverse width at water surface across the direction of flow.

D = a vertical depth from surface level.

B = a bed-width or bottom-width of a section.

T = total time of discharge; t , t_1 , t_2 , subsidiary times.

n = coefficient of roughness and irregularity combined.

$m = n (41.6 + 0.00281 \times \frac{1}{S})$, a combined variable.

k = coefficient for supply from catchments.

c = coefficient for mean velocity in channel discharges.

o = coefficient for orifice and overfall discharges and velocities.

g = velocity acquired by gravity in one second = 32.2 feet approximately.

When x , y , z , are rectangular co-ordinates taken with reference to flowing water, the following conventional arrangement is usual.

x is taken in the direction of flow, or longitudinally;

y is taken across the flow, or transversely;

z is taken vertically, or perpendicular to x .

All dimensions are generally in feet and decimals, and velocities and discharges are in feet and cubic feet per second. The foot-weight or talent = 1 000 ounces, is the unit of weight; its multiple is the rod-weight = 1 000 fwt. For decimal multiples and submultiples see page 14.

Velocity Notation of the Mississippi Survey.

- v = mean velocity of the river.
 V = velocity at any point in any vertical plane parallel to the current.
 V = velocity at a point 20 feet below the surface at a distance of 100 feet
^{100 20} from the base line, measured along the bank.
 U = velocity at any point in the mean of all vertical planes parallel to the
 current.
 U_m = grand mean of the mean velocities in all vertical planes parallel to
 the current.
 U_r = the mean of the bottom velocities in all such planes.
 w, V = velocity at any depth below the surface at a perpendicular distance
 $w,$ from the base line.
 V_o = velocity at the surface in any vertical plane parallel to the current.
 $V_{\frac{1}{2}D}$ and V_D = velocities at mid-depth and at the bottom in any such plane.
 V_{d_1} and V_m = the maximum and the mean velocities in any such plane.
 W = river width at any given place.
 w = perpendicular distance from the base line to any point of the water
 surface.
 $w,$ = perpendicular distance from the base line to the surface fillet moving
 with the maximum velocity.
 D = total depth of river at any given point of surface.
 d = depth of any given point below the surface.
 d_1 = depth from the surface of the fillet moving with the maximum velocity
 in the assumed vertical plane parallel to the current.
 m = depth from the surface of the fillet moving with a velocity equal to the
 mean of the velocities of all fillets in the assumed vertical plane
 parallel to the current.
 Δ = maximum or mid-channel depth.

As it may be convenient to the reader to have conversion tables at hand for reducing the quantities of water, &c., given in foreign works on hydraulics into English measure, and the converse, the following two pages are given to answer this purpose, as far as regards the English decimal system.

For other corresponding purposes, see 'Modern Metrology,' London, 1882, Lockwood, and 'Pocket Logarithms and other Tables,' London, 1880, Allen.



COMPARISON OF FRENCH AND ENGLISH DECIMAL

| English Scientific Units | In English Commercial Units at 62° Fahrenheit | French Scientific Equivalent |
|---|---|---------------------------------|
| Length | | |
| FOOT = 10 tithes (or tenths) | = 1'00029 foot | = 0'304 79 mètres |
| Rod = 10 feet | . | = 3'047 95 mètres |
| Chain = 10 rods | . | = 3'047 95 décamètres |
| Cable = 10 chains | . | = 3'047 95 hectomètres |
| League = 10 cables | . | = 3'047 95 kilomètres |
| Surface | | |
| SQUARE FOOT = 100 sq. tithes | = 1'00057 sq. ft. | = 9'289 97 déc. car. |
| Square rod = 100 sq. feet | . | = 9'289 97 mèr. car. |
| Square chain = 100 sq. rods | . | = 9'289 97 ares |
| Sq. cable or century = 100 sq. chains | . | = 9'289 97 hectares |
| Square league = 100 centrs. | . | = 9'289 97 kil. carrés |
| Capacity | | |
| Fluid mil = 1000 fl. doits | . | = 28'315 31 mill. cub. |
| Fluid ounce = 1000 fl. mils = 1 cub. tithe | . | = 28'315 31 cent. cub. |
| CUBIC FOOT = 1000 fl. ozs. = 1000 cub. tithes | = 1'00086 cub. ft. | = 28'315 31 déc. cub. |
| Cubic rod = 1000 cubic feet | . | = 28'315 31 mèr. cub. |
| Weight | | |
| Mil = 1000 doits | . | = 28'315 31 milgr. |
| Ounce (millesimal) = 1000 mils | . | = 28'315 31 grammes |
| FOOT WEIGHT or talent = 1000 ozs. | = 62'42454 lbs. | = 28'315 31 kilog. |
| Rod weight = 1000 fwt. | . | = 28'315 31 milliers |

COMPOUND UNITS.

PRESSURE.

| | | |
|--|---|--------------------------------------|
| 1 talent (or foot-weight) per sq. foot | . | = 304'794 5 kilog. per mèr. car. |
| 1 talent " " " " " | . | = 0'030 479 45 kilog. per cent. car. |
| 1 talent (or foot weight) per square tithe | . | = 30'479 45 milliers per mèr. car. |
| 1 rod-weight per square foot | . | = 304'794 5 milliers per mèr. car. |

IRRIGATION.

| | | |
|-------------------------------|---|-------------------------------------|
| 1 cubic foot per square chain | . | = 0'304 794 5 mèr. cub. per hectare |
| 1 cubic foot per century | . | = 0'003 047 9 mèr. cub. per hectare |
| 1 cubic rod per century | . | = 3'047 945 mèr. cub. per hectare |

POWER AND WORK.

| | | |
|---------------------------------------|---|----------------------------------|
| 1 foot-talent | . | = 8'630 354 2 kilogrammètres |
| 1 h.-p. = 528 foot-talents per minute | . | = 1'012 63 c.-v. force de cheval |

HEAT AND ELECTRO-MAGNETISM.

| | | |
|------------|---|--------------------------------|
| 1 foot-mil | . | = 0'008 630 35 mèrètre-grammes |
|------------|---|--------------------------------|

SIMPLE AND COMPOUND UNITS OF REDUCTION.

| | | English into French | |
|--------|---|---------------------|--------------------------------------|
| Simple | . | 0'304 794 494 | Cubic 0'028 315 312 |
| Square | . | 0'092 899 683 | Fourth power 0'008 630 354 |

SCIENTIFIC SYSTEMS AT 32° AND 39° FAHR. IN VACUO.

| French Scientific Units | | In English Commercial Units at 62° Fahrenheit | English Scientific Equivalent |
|-------------------------|---------------------------|---|----------------------------------|
| Length | | | |
| MÈTRE | = 10 décimètres . . . | = 3·281 83 feet | = 3·280 90 feet |
| Décamètre | = 10 mètres . . . | | = 3·280 90 rods |
| Hectomètre | = 10 décamètres . . . | | = 3·280 90 chains |
| Kilomètre | = 10 hectomètres. . . | | = 0·328 09 leagues |
| Surface | | | |
| MÈTRE CARRÉ | = 100 décim. car. . . | = 10·770 43 sq. ft. | = 10·764 30 square feet |
| Are | = 100 mètres carrés . . . | | = 10·764 30 square rods |
| Hectare | = 100 ares . . . | | = 10·764 30 sq. chains |
| Kilomètre carré | = 100 hectares . . . | | = 0·107 64 sq. leagues |
| Capacity | | | |
| Litre | = 1 décim. cube . . . | | = 35·316 58 fluid. ozs. |
| Décalitre | = 10 litres . . . | | = 0·353 17 cubic feet |
| Hectolitre | = 10 décalitres . . . | | = 3·531 66 cubic feet |
| MÈTRE CUBE | = 10 hectolitres . . . | = 35·346 83 cub. ft. | = 35·316 58 cubic feet |
| Weight | | | |
| Milligramme | | | = 35·316 58 doits |
| GRAMME | = 1000 milligrammes . . . | | = 3·331 658 mils |
| Kilogramme | = 1000 grammes . . . | = 2·204 62 lbs. | = 35·316 58 ounces |
| Quintal | = 100 kilogrammes . . . | | = 3·531 66 footweight |
| Millier | = 1000 kilogrammes . . . | | = 35·316 58 footweight |

COMPOUND UNITS.

PRESSURE.

| | | |
|---|---|-------------------------------------|
| 1 kilogramme per mètre carré . . . | = | 0·003 280 9 talents per sq. foot |
| 1 kilogramme per centimètre carré . . . | = | 0·328 089 9 talents per sq. tithé . |
| 1 millier per mètre carré . . . | = | 3·280 899 talents per sq. foot |
| 1 millier per centimètre carré . . . | = | 32·808 990 rod-weight per sq. foot |

IRRIGATION.

| | | |
|--------------------------------|---|------------------------------------|
| 1 mètre cube per hectare . . . | = | 3·280 899 cubic feet per sq. chain |
| ” ” . . . | = | 328·089 9 cubic feet per century |
| ” ” . . . | = | 0·328 090 cubic rods per century |

POWER AND WORK.

| | | |
|--------------------------------------|---|------------------------------|
| 1 kilogrammètre . . . | = | 0·115 870 foot-talents |
| 1 c.-v. force de cheval (4500) . . . | = | 0·987 528 h.-p. (scientific) |

HEAT AND ELECTRO-MAGNETISM.

| | | |
|----------------------|---|-----------------------|
| 1 mètre-gramme . . . | = | 115·870 154 foot-mils |
|----------------------|---|-----------------------|

SIMPLE AND COMPOUND UNITS OF REDUCTION.

| | | French into English | |
|------------------|--------------|------------------------|----------------|
| Simple | 3·280 899 | Cubic | 35·316 580 7 |
| Square | 10·764 299 3 | Fourth power | 115·870 145 02 |

3. RAINFALL, SUPPLY, AND FLOOD-DISCHARGE.

All hydraulic works of irrigation, drainage, storage, water supply, river improvement, and land reclamation, are more or less affected by the amount and periodicity of the rainfall; for many of them careful and trustworthy rainfall statistics and data are absolutely requisite; but the nature and amount of detail required vary with the nature of the work; works of storage being those that, perhaps, require the greatest amount of accurate information. In order that these local records should be sufficient to form a correct basis for the engineering data of these latter works, they should comprise observations extending over a period of ten years, or of the local period comprehending a cycle of rainfall from one season of maximum rainfall to another, including years of extreme drought; from these the following results can be deduced:—

1. The mean, maximum, and minimum monthly rainfall, from which the mean and extreme falls for each natural local season, wet, cold, and hot, can be obtained.

2. The mean and maximum daily falls in twenty-four hours, for each month in the rainy season.

3. Mean and maximum hourly falls, longest continuous falls and droughts, and special occurrences.

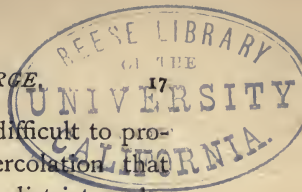
These, arranged in a convenient tabular form, are all the rainfall data that the engineer will generally require.

In most cases, also, and especially in hot climates, evaporation records are also necessary; and sometimes, too, it is advisable to possess other meteorological data, such as those of humidity, temperature, atmospheric

pressure, and wind ; and, what is often difficult to procure, some data of absorption and percolation that would be applicable to the soils of the district under consideration.

On many of the works before mentioned, the first duty of the engineer is to account for the whole of the downfall, or to discover what becomes of it all, under both ordinary and unusual circumstances, so that he may be able to deal with more certainty of knowledge with that portion of it that more intimately affects his works ; as, for instance, the bridge-builder with the floods, the engineer of storage works with the drought, and those of canals and river-improvement with both. A geographical and geological knowledge of the catchment area, whose rainfall affects the works, is hence also needful ; the boundaries of this area, its lines of watershed and drainage, its disposition as regards prevailing winds, the nature and porosity of its soil, and the amount of vegetation or cultivation on it, as well as any available records from which the quantities of water actually run off by its streams and rivers in various seasons may be arrived at, are all data necessary for establishing satisfactorily a perfect knowledge of the disposal of the whole of the rainfall under any circumstances.

In many instances it is, from want of sufficient information, utterly impossible to obtain this perfect knowledge : in others, the deficient data may be supplied by approximative deduction from the data of other places, so that a tolerably correct approximate balance may be struck between the downfall and the amount evaporated, absorbed, and run off ; in any case, however, the engineer may, with time and means at his



disposal, gauge the streams and rivers affecting his works, and make correct records of the amount of water run off in them at different seasons of the year, and in exceptional floods. Failing, however, both time and opportunity, such data have to be observed in a rapid manner that will enable him to determine this approximately ; such as the section and fall of the rivers, the depths at various stages, floodmarks, and a few velocity observations.

The results principally required are the flood or maximum discharge, in cubic feet per second, of the river or stream draining the catchment area ; its mean discharge throughout the year ; and its minimum discharge in seasons of extreme drought, as well as in its ordinary low stage ; dividing each of these by the catchment area, similar results per unit of catchment are obtained, to obtain the depth in feet of rainfall run off under each of those conditions. The relation between these quantities and the probable or approximate downpour over the catchment area can then be compared with those known to exist in other corresponding cases, and a valuable check on these important results thus obtained.

Flood discharge.—The determination of the quantity of water discharged from a catchment area in a river or stream at a time of extreme flood is a matter that is very often of the highest importance. Costly bridges have continually been sacrificed, and long lengths of canal damaged for want of sufficient attention having been paid to this subject.

When the data mentioned in the foregoing paragraphs can be obtained, and are properly handled, there is little difficulty in arriving at a generally correct result ;

but, as in many cases only some of these are forthcoming, the bases of calculation are considerably narrowed, and the various and partial modes that have to be adopted necessarily vary with the available data.

First.—If the catchment area is not very large—that is, not exceeding 400 square miles, or 100 square leagues—it may sometimes be assumed that the whole of it is simultaneously subject to the same amount of maximum downpour, and that the loss by absorption and evaporation is also tolerably uniform over the whole; if then some trustworthy data for this loss should be available, the flood discharge can be computed direct; thus:

Let F , the actual downpour in 24 hours, be 0·8 feet, and the loss by absorption and evaporation one fourth; then the effective rainfall $f=0\cdot8-0\cdot2=0\cdot6$; and the corresponding flood discharge per second, Q , from a catchment (K) of 4 square leagues, will be

$$Q = \frac{0\cdot6 \times 4 (10\ 000)^2}{24 \times 60 \times 60} = 2778 \text{ cubic feet per second.}$$

If the rainfall or the loss vary over portions of the catchment, the parts may be treated in the same way, to obtain a total value of Q through summation. For this purpose Table II., Part 3, can be used.

Second.—If the catchment area under consideration happen to form part of some large region, whose rainfall has been thoroughly investigated, and in which numerous flood discharges have been arrived at through velocity observations and computation, some general coefficient of drainage (k) may have been determined for that region. In that case the computation for flood discharge from any portion of it can be computed by formulæ.

The three best-known formulæ for this purpose are

$$(1) Q = k_1 27 (K)^{\frac{1}{2}}.$$

$$(2) Q = k_2 100 (K)^{\frac{3}{4}}.$$

$$(3) Q = k_3 1300 K (L)^{-\frac{2}{3}}.$$

In all these K is the catchment in square statute miles, Q the flood discharge in cubic feet per second; in the third L is the length of the main river or stream under consideration, in statute miles; while the coefficients k_1, k_2, k_3 , are the local drainage coefficients suitable to each formula respectively.

Formula (1) requires a very wide range of values of k_1 , and is hence inconvenient, though simple in form.

Formula (2) is preferable; it is a modification of Colonel Dickens's formula, $Q = 825 (K)^{\frac{3}{4}}$, suited to Bengal proper and Bahar; though it afterwards appeared that Formula (2) with coefficients near to $k = 8.25$ was suited to large tracts of Indian plains having an annual rainfall of from 24 to 50 inches.

It seems, however, more rational to use a coefficient more closely dependent on a similarity of general conditions, of which the maximum day's downpour is perhaps the most important. In Northern India where this latter is about 1.5 feet in or near hills, and 1.0 foot in the plains, the flood waterway allowed for bridges has generally been based on the assumption that the rainfall run off would amount to 1.0 foot in depth over the whole; and allowance has been made with these data for the flood waterway of the streams and rivers crossing both the Ganges Canal and the Sarhind Canal; in other cases, also, in Northern India, two-thirds of the depth of downpour is assumed to pass off in flood. It is hence

better to use a coefficient suitable to similar conditions of catchment area, within narrower range.

The values of k_2 for India generally lie between 1 and 24 : see coefficients in the Working Tables at Table XII., Part 1 ;—some further values of it, applicable to various river basins in India, are also given in the table of flood discharges at page [8] of the Hydraulic Statistics in the second part of this Manual. The values of the general expression, for a value of $k_2=1$, are given for catchment areas of various sizes in the Working Tables, at Table IV., Part 1, and the local coefficient can be readily applied to these quantities.

Formula (3) was deduced by Mr. Burge, of the Madras Railway, from observations in the tract through which that line passes ; and is suited to it, with a value of $k_3=1$; the conditions being that the maximum down-pour in 12 hours was 6 inches, and the area elevated from 500 to 1300 feet above mean sea level, consisting principally of unstratified rocks. It was deduced from observations on 27 bridges, of above 80 feet span, on the Madras Railway, and its results correspond closely with those of recorded flood sections ; the errors lying between 4.64 feet too high and 3.40 too low in height of section. Mr. Burge argued justly that the length of the river necessarily extends the time of the discharge, and hence diminishes the quantity passing off within a certain time ; and that also the functions of discharge, the hydraulic slope, the cross section, and the head affected by the sinuosities in greater length, are reduced by it. Admitting this, the same principle would apply not only to the main river, but also to its tributaries ; the number and conditions of the tributaries would probably be a more important consideration. Again, there

is much difficulty in saying where a main river begins ; so much so, that in the first place the introduction of an index of $\frac{2}{3}$ against a coefficient of 1300 would appear to be a needless attempt at exactitude ; and in the second place the introduction of the length of the river at all in an equation of this sort is a matter incapable of very extended application ; although in the instances from which this formula was laid down it has been very successfully introduced.

A better mode of introducing a function somewhat similar to this would be to apply the ratio of extreme breadth to extreme length of catchment area ; and introduce it in formula (2), the range of whose coefficients (k_2) for India seem to be between 1 and 24—an important step already gained. It then takes the form,

$$(4) \quad Q = k_4 \frac{B}{L} 100 (K)^{\frac{2}{3}},$$

where B = extreme breadth of catchment area.

and L = extreme length of catchment area,

and k_4 = a new coefficient,

obtaining a more tangible improvement, capable of extended application. It is unfortunate, however, that for this formula a sufficient number of values of the new coefficient are not yet forthcoming ; although in the instances in which it has been applied the improvement seems clearly manifested in reducing the range, so that for the present it is generally better to use formula (2), while in special cases the ratio can be easily introduced to obtain values of k_4 .

Third.—When coefficients of the class k_1, k_2, k_3, k_4 , are not available, and the conditions of rainfall and of absorption and evaporation are so defective as to be insuf-

ficient, direct observation of each single river or stream within the catchment becomes the sole guide. It then becomes necessary to fall back entirely on recorded flood-marks, as a means of approximating to the flood discharge ; and, after gauging the discharges of the channels in their ordinary stages, to assume the flood discharges to be proportional to them according to the ordinary formula,

$$Q = \frac{A q \cdot R^{\frac{1}{2}}}{a r^{\frac{1}{2}}},$$

where A is the sectional area up to flood-mark, R its hydraulic mean radius, and a and r are similar quantities corresponding to the discharge (q) determined under the conditions of observation in each separate channel.

4. STORAGE.

Reservoirs generally have for their object either the detention of flood water that might otherwise cause damage, as in works of river improvement, or the utilisation of it in canals, of navigation, irrigation, or driving machinery, or for town supply. For the first purpose they must, to effect their purpose, be very extensive, and strongly aided by the natural formation of the country ; for the last purpose they are, in one respect, excepting under very favourable conditions, particularly ill-fitted.

The collection of drinking-water from the surface of land needs, in the first place, a clean, uncultivated and uninhabited tract of land as a catchment area ; and in the second place, the water stored in the reservoir, which is liable to become putrescent, or seriously affected by the organisms, plants, and animalculæ that inhabit stagnant water, requires a very perfect and careful filtration, of

a sort beyond the ordinary economic powers of municipalities or public companies. Indeed, it is now asserted to be an incontrovertible fact, that it is to the tainted water of rivers and reservoirs that one-half of most preventible diseases are due, the other half being caused by want of ventilation, faulty drainage, and mistaken modes of managing sewage, or, in other words, that impure air and tainted water are the chief enemies of human life; and there is, therefore, every reason to believe that in the future, when the general public become awake to this, and acquire enough energy to throw off the incubus of vested interests in the form of water-companies, both tainted rivers and open reservoirs will be universally condemned as sources of drinking-water supply, and that the water filtered, stored, and preserved against impurity by nature in the permeable and unvitiated strata of the earth, will be considered, as it justly is, a necessary of life and health, and be drawn on in a more scientific and enlightened way than is at present usual. Another quarter of a century may show us scientific men objecting, on sanitary grounds, to the watering of our streets with such water as is now habitually and unconcernedly used in preparing our food.

It will therefore be only under conditions very favourable for clean collection and storage, or under circumstances that admit of no better alternative, that the water of storage reservoirs will be used for drinking purposes. Such water will, however, still remain valuable under ordinary circumstances, for extinguishing fires, watering streets, and many other purposes, in which it is not habitually brought into contact with the human body, and where its impurities are of little avail.

The determination of the size and dimensions of a

storage reservoir is a matter entirely governed by local circumstances and requirements. The assumptions that the area covered by it should bear a certain proportion to that of the catchment area, or that the amount of water stored should be as nearly as possible one-third of the available supply, are not by any means rules to be applied without a very large discretionary power, although there are rules laid down in various forms by different hydraulic engineers that very much resemble these. The object being the collection and retention of a certain amount of water for a definite purpose, and the circumstances being the local formation of the ground and the amount of available downpour on the catchment area, all the economic considerations depend on these points.

The intention may either be to store as much water as possible within a certain amount of expenditure of cost, or only a definite amount sufficient for a certain purpose, or to store all that can possibly be obtained with a knowledge that the extreme amount would not be enough. Again, in some cases the quality of the water and the convenience of proximity, or of cleanliness of site, may be considerations outweighing all others. If, therefore, the latter is the case, there are generally not many local conditions answering the purpose within which any choice can be made; the same may be generally said to be true with reference to the second case in which a definite amount is required. It is only therefore under special circumstances, when the object is to store and utilise as much water as possible, that much choice is left to the engineer.

Large artificial reservoirs being generally made on the natural surface of the ground, and bounded in one

direction only by an embankment of earth or a dam of masonry or brickwork, the first object is to choose a site or sites where the greatest amount of water can be stored with the shortest and least amount and length of embankment; for this purpose a river gorge, narrow and precipitous, terminating a great length of country, having a gradual fall towards it, offers the best ordinarily natural conditions; if, in addition, the lateral or transverse slope of the country is also very gradual, it becomes a large natural basin, with one narrow outlet; and if this admits of being easily dammed, an extraordinary advantage not often available presents itself.

The economy of constructing one large reservoir in preference to two or more small ones to hold the same amount would, perhaps, be evident at first sight to most people. The author has, however, met so large a number of persons that believe the contrary, that he is constrained to give the following mathematical proof of it by Graeff.

Let a single reservoir, or rather its contents when full, be supposed to consist of a number of laminæ, or layers of water, the sum of which will equal the total content, and let

H = the height of any one layer;

P and S = the perimeter and surface of its lower side;

P' and S' = the perimeter and surface of its upper side; then the volume of this layer will be

$$= aH + \frac{bH^2}{2} + \frac{cH^3}{3};$$

$$\text{where } a = S; \quad b = \frac{2P(S' - S)}{H(P' + P)}; \quad c = \frac{(S' - S)(P' - P)}{H^2(P' + P)};$$

Hence the above expression becomes

$$\begin{aligned}
 &= \frac{H}{3(P' + P)} \left\{ 3S \cdot \overline{P' + P} + P' \cdot \overline{S' - S} + 2P \cdot \overline{S' - S} \right\} \\
 &= \frac{H}{3(P' + P)} \left(P \cdot \overline{2S' + S} + P' \cdot \overline{2S + S'} \right).
 \end{aligned}$$

In the case where the lateral and longitudinal slopes of the ground are uniform, we can imagine the reservoir to consist of one only of these layers; and its contents will then represent that of the whole reservoir. In this case the height of the layer will be the extreme depth of water stored, and the quantities S and P will become indefinitely small in comparison with S' and P' , and may hence be neglected: hence the total volume of water stored $= \frac{1}{3} H S'$, and this is the volume of a reversed cone having S' for its base; a demonstration that proves how rapidly the amount of storage increases with the depth of water, or with the height of the embankment.

To the height of dams, again, there is a practical limit; earthen dams of great height require an enormous section, being consequently very costly as well as dangerous, and are in themselves difficult to manage as regards escape; masonry dams have a limit to their height, due to the pressure per unit of surface on the foundation; the highest yet built that is still standing does not exceed 164 feet, and it is very improbable that that height will be greatly exceeded for some time to come, unless iron is made to enter largely into their construction.

After choosing a site for a proposed reservoir, one of the first points requiring attention is the determination of its storage capacity up to different proposed levels of escape. For this purpose, marks are fixed at differences

of level of about 5 or 10 feet, on any convenient short-line of section; and the contours of these levels are marked out and surveyed all around the basin, in order to obtain the perimeters and areas at each contour; from these, as before shown, the contents of each lamina can be calculated, and the content up to any other contour. If, however, it be preferred to obtain this by means of a series of longitudinal and transverse sections taken up to the heights of the various contour levels, it is perhaps best to direct the former in conformity with the axis or axes of figure of the basin, and the transverse sections at right angles to them, and, as far as possible, at equal distances along them; although in some instances, unequal distances and inclined directions, more suited to the form and disposition of the ground, would give more correct results; the true values of the corresponding rectangular transverse sections can then be obtained from the oblique sectional areas by multiplying them by the cosines of their angles of obliquity. Should a winding river channel or depression form part of the basin, it is often more convenient and correct to estimate its content independently, and add it in afterwards.

The following are the three formulæ most used in obtaining the contents from the sectional areas:—

1. If there be only two sectional areas, A_1, A_2 , taken at a time, at a common distance, d ,

the contents = $\frac{1}{2}d (A_1 + A_2)$, or = $\frac{1}{3}d (A_1 + A_2 + \sqrt{A_1 A_2})$.

2. If there be three equidistant sections, A_1, A_2, A_3 , taken at a time, and their common distance is d , the con-

tents = $\frac{1}{3}d (A_1 + 4 A_2 + A_3)$.

3. If there be any even number (n) of equidistant sections, $A_1, A_2, \&c.$, up to A_n , at a common distance, d ,
the contents $= d(\frac{1}{2}A_1 + A_2 + \&c. + A_{n-1} + \frac{1}{2}A_n)$.

The accuracy of result will of course depend on the closeness of the sections, and the suitability of their positions to the general form of the reservoir.

The capacity of the reservoir being obtained, the amount of supply that can be expected annually from the catchment area may be obtained, either in total quantities or in continuous quantities as cubic feet per second, by the aid of Parts 1 and 2 of Table II. of the Working Tables; in these calculations much labour is saved by deducting, in the first place, the allowance due to evaporation and absorption on the catchment area from the rainfall given, and making use of the available or effective rainfall or rainfall run off as the basis of calculation for supply.

If a small supply alone be involved, the use of Part 1, Table III. of the Working Tables will enable the contents of the reservoir, and extent of catchment area necessary to afford the supply, to be rapidly determined. Part 2, Table III., may also be occasionally useful, where the supply is limited by the needs of an extent of land to be irrigated, or the population of a town requiring water for public purposes.

The section of waterway of escape has next to be determined; this depending on the flood discharge and the maximum downpour in twenty-four hours. In these calculations, Part 3, Table II. of the Working Tables is useful; so also are Parts 1 and 2, of Table IV., in connection with the formula already given for flood discharge.

All these are of course simply modes of calculating,

or of shortening, the calculation of the quantities of water ; the determination of them has to be left to the discretion of the engineer and the requirements of the case. Should the supply be required to maintain a certain depth of water for navigation in a canal, the seasons, the supply deficient, the loss in the canals from evaporation and filtration, and all such data, will determine the amount ;—if for irrigation, the amount of land, its quality of soil, and probable water duty ; on this latter subject information is given in Chapter III. and in the Hydraulic Statistics, where data of the waterings and water duty usual in France, Spain, Italy, and Northern and Southern India, are given. Or if the supply is required either for motive power or the public purposes of town supply, the amount and height of delivery require determining with reference to local conditions ; in such matters, therefore, no guide would be of use. Lastly, if the object is the control of floods, the whole of the physical conditions of the river and its banks, from its highest watershed down to its mouth or embouchure in the sea, will be matters affecting the amount, and the management and regulation of the storage : on this subject see the paragraph in Chapter III.

5. DISCHARGES OF STRAIGHT, UNIFORM REACHES OF OPEN CHANNELS, AND OF PIPES.

The various modes of gauging velocities and discharges are described in Chapter II. on field operations and gauging. The calculation of velocity or of discharges, under different conditions and for different data, may be considered independently of gauging. It

is important to the engineer that he should at any time be able to calculate, in a few moments, the discharge of any pipe, channel, or canal, from such sufficient data as he may possess, or obtain readily.

The number of calculated velocity formulæ, their variety, and the wonderful amount of complication in them, as well as the want of exactitude of result they give, is truly astonishing ; and when, on the other hand, one observes some engineers adhering slavishly to the tables and data of one hydraulician, others to those of another, and others again going through the conscientious, but very lengthy, course of examining everything that every hydraulician has said or done in the matter of calculation of mean velocity of discharge, one cannot but feel pained as well as surprised.

It would be quite out of place in this portion of a Manual of this description, which has for its object the supplying the engineer with information and tables for calculating his quantities and data in as rapid a way as practical correctness will allow, to enter into a detailed investigation of all these formulæ, and the reasons for setting them all aside and adhering to that adopted in preference and to the exclusion of all others ; it will, therefore, suffice for the author here to mention the reason for adopting any one formula or conclusion as it is brought forward. A comparison of the results of various hydrodynamic formulæ will be given in Chapter III., among the miscellaneous detached paragraphs.

The general formula for discharge, based on the theories mentioned in the section I of this chapter, is

$$Q = A V = A (f g R S)^{\frac{1}{2}},$$

the terms of which are given in the general notation,

page 11; the mean velocity of discharge being the smaller and more convenient quantity to deal with, for open channels and canals, and the discharge itself being the quantity more often required for pipes, sewers, and closed tubes, syphons, or tunnels of all sorts.

Taking, however, the expression for mean velocity of discharge, obtained by equating the accelerating effect of gravity down an inclined plane with the retarding effect of friction, it can be put into the form more convenient for English measures—

$$V = c \times 100 (RS)^{\frac{1}{2}},$$

where c is a variable experimental coefficient, depending on the surface, the conditions, the dimensions, and the hydraulic slope of the channel or pipe, and hence also on a further experimental coefficient n of roughness and irregularity combined, which again involves both the functions R and S : its value under extreme conditions varies from 0.25 to about 2.00.

A correct formulated determination of the value of the coefficient, c , for all conditions, is a matter that can only be said to have been even approximately arrived at in the last few years, from an examination of the experimental results of d'Arcy and Bazin on the discharges of pipes, open channels and ordinary rivers, and those of Humphreys and Abbot on the discharges of very large rivers, and on his own observations on Swiss hill-streams and channels, by Herr W. R. Kutter, of Bern.

The determination of coefficients of this type for which we are indebted to him, and tables rendering it easily found for open channels and rivers of any sort or dimensions in metric measures, are given in his valuable

articles in the 'Cultur-Ingenieur' for the year 1870. A comparison of these coefficients of Herr Kutter with recorded results, principally Indian, made, collected, and compiled by the author between 1860 and 1873, conducted to the belief that the formula of Kutter was the best extant ; but that the classification of coefficients was defective as applied to canals, and straight, uniform river-reaches.

The values of the coefficients varying so greatly in the various classes, it became necessary to reinvestigate the subject. This was done, and eventually an extension and an alteration of the classes was made by the author ; the formula was freshly worked out for English units, and the whole was set forth in detail in the author's work, 'Canal and Culvert Tables' (London, 1878, Allen).

Under this new arrangement, the values of c , the coefficient of mean velocity, are also given in this edition of this book in Part 4, Table XII.

With the aid, therefore, of these tables of coefficients (c) and the values of the expression $100 (R S)^{\frac{1}{2}}$, given in Table VII., the values of V , the mean velocity of discharge of straight and uniform reaches of canals and open channels can be rapidly determined in a few moments, according to the most improved and correct method yet known.

With the aid of the same tables of coefficients (c) and the values of the expression,

$$Q = c \times 39.27 (S d^5)^{\frac{1}{2}} \text{ when } c = 1,$$

given in Table VIII., the discharge of any full cylindrical pipe, sewer, or tunnel, may also be determined by applying suitable values of c .

These tables, to which explanatory examples are

attached, can also be used for the converse purpose of obtaining the head, diameter, hydraulic slope or hydraulic radius, due to given discharges of channels and pipes ; it will, however, be necessary for the calculator to remember that all dimensions, even diameters of pipes, are best invariably kept in feet, and that all slopes are kept in the form known as the sine of the slope, mentioned in the general notation, given in section 2 of this chapter. Should it be necessary to reduce these from gradients given in other forms, such as in feet per English mile, or as a fall of unity to a certain length, Table VI. may be used to save calculation.

The Derivation of the Coefficients.—So far for the velocity formula actually adopted, and the mode of working it in calculating results. As regards the formula itself, independently of the determination of the variable coefficient, it is none other but the Eytelwein formula, or Chezy formula, in a very much improved form, having the results of modern experiment incorporated with it. An examination of the old hydraulic formulæ for mean velocity shows that most, in fact almost all of them, were modifications of the Chezy formula, some of them adding an additional term or function, and altering the value of the experimental coefficient, but still asserting its fixity.

In the earlier editions of this Manual, written before Herr Kutter had published his valuable improvement, all the formulæ having fixed coefficients were rejected by the author, who at the same time asserted the principle that no fixed coefficient was suitable to all circumstances, and that the engineer should choose for himself a coefficient most suitable to the special circumstances, dimensions, and condition of the pipe, channel, or river, with

whose discharge he was dealing; and that the recorded results of experiment should be always consulted for the purpose of approximating as closely as possible to the special circumstances of the case under consideration. In addition to that recommendation, a mode of arriving at values of c , in cases of canals in earth, in good order, under very limited conditions, was also then mentioned. It consisted in a method of successive approximation; first, to assuming $c=1$; and then from the mean velocity v , resulting, assuming a second value of c , according to the following table, was calculated, or a second true velocity of discharge, V .

| v | c | v | c | v | c | v | c |
|-----|------|-----|------|-----|-------|-----|-------|
| 1.0 | .910 | 1.5 | .960 | 2.0 | 1.000 | 2.5 | 1.023 |
| 1.1 | .920 | 1.6 | .968 | 2.1 | 1.005 | 2.6 | 1.026 |
| 1.2 | .930 | 1.7 | .976 | 2.2 | 1.009 | 2.7 | 1.030 |
| 1.3 | .940 | 1.8 | .984 | 2.3 | 1.014 | 2.8 | 1.033 |
| 1.4 | .950 | 1.9 | .992 | 2.4 | 1.018 | 2.9 | 1.037 |
| | | | | | | 3.0 | 1.040 |

A few values of c , suitable to pipes under various velocities, were also given; but they were detached, and, from want of experiment, very insufficient. Yet the true state of the case, and the mode most advisable for adoption until investigations on a larger scale threw more light on the matter, was then clearly set forth.

Now that the experiments of d'Arcy and Bazin, of Humphreys and Abbot, and of Ganguillet and Kutter, have been reduced to one formulated expression, the labour of choosing a coefficient from general experimental records is rendered needless as far as regards ordinary canals and culverts; although it would be advantageous to experimentalise on the actual case.

As regards natural channels of rivers otherwise than those whose conditions approximate to those of canals,

the necessity of referring to records of experiment still remains, although the Kutter coefficients may be of great assistance even in this branch of the subject

The determination and tabulation of the coefficients (c) has gone through three stages of development. 1. *The first* was that made by Bazin, based on the experiments conducted by d'Arcy, by Bazin himself, and by various engineers of the French Ponts et Chaussées. The principles asserted were that the coefficient depended on two quantities or qualities only, namely, the condition of surface of the bed and banks touched by the water, and the hydraulic mean radius of the section of discharge. Four categories of coefficients were adopted.

1st. For very smooth surfaces, well-plastered surfaces in cement, and well-planed plank.

2nd. For even surfaces, ashlar, brickwork, and ordinary planking.

3rd. For rough surfaces, as rubble.

4th. For earthen channels generally

The values of an intermediate coefficient c for French measures in these four categories were—

$$(1) c_1 = 0.00015 \left(1 + \frac{0.03}{R} \right)$$

$$(2) c_2 = 0.00019 \left(1 + \frac{0.07}{R} \right)$$

$$(3) c_3 = 0.00024 \left(1 + \frac{0.25}{R} \right)$$

$$(4) c_4 = 0.00028 \left(1 + \frac{1.25}{R} \right)$$

The corresponding values of the final coefficient c for the English formula in feet may be obtained from the above values of c , by the formula

$$c = \frac{1.81}{100 (c)^{\frac{1}{2}}} = \frac{V}{100 (RS)^{\frac{2}{3}}}$$

under an arrangement that keeps the values of c within a limited range approximating to unity, and throws 100 into the old general expression for the Chezy formula.

The values of these coefficients (c), adapted to the corresponding formula in English feet, are generally as follow, in their respective categories:—

| R | c_1 | c_2 | R | c_3 | c_4 |
|-----|-------|-------|-----|-------|-------|
| 1 | 1.41 | 1.18 | 1 | 0.87 | 0.48 |
| 1.5 | 1.43 | 1.22 | 2 | 0.98 | 0.62 |
| 2 | 1.44 | 1.24 | 3 | 1.04 | 0.70 |
| 2.5 | 1.45 | 1.26 | 4 | 1.06 | 0.76 |
| 3 | 1.45 | 1.26 | 5 | 1.08 | 0.80 |
| 3.5 | 1.46 | 1.27 | 6 | 1.10 | 0.84 |
| 4 | 1.46 | 1.28 | 7 | 1.10 | 0.86 |
| 4.5 | 1.46 | 1.28 | 8 | 1.11 | 0.88 |
| 5 | 1.46 | 1.29 | 9 | 1.12 | 0.90 |
| 5.5 | 1.46 | 1.29 | 10 | 1.12 | 0.91 |
| 6 | 1.47 | 1.29 | 11 | 1.13 | 0.92 |
| 7.5 | 1.47 | 1.29 | 14 | 1.13 | 0.95 |
| 8 | 1.47 | 1.30 | 15 | 1.14 | 0.13 |
| 19 | 1.57 | 1.30 | 18 | 1.14 | 0.98 |
| 20 | 1.48 | 1.31 | 20 | 1.14 | 0.98 |

These coefficients are not correct for canals in earth generally, and are notoriously incorrect for large canals; they are useless to English engineers, excepting in so far as they afford them a knowledge of the velocities and discharges that French engineers would assume to hold with certain known conditions. In the matter of dogmatic prejudice, and mutual international recrimination, the balance between the French and the English is tolerably even; if the English are insular and coldly impassible, the French are bureaucratic and heated with vanity; yet science will progress in spite of all petty wishes, both individual and national.

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| 2.5 | 1.45 | 1.26 | 4 | 1.06 | 0.76 |
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| 3.5 | 1.46 | 1.27 | 6 | 1.10 | 0.84 |
| 4 | 1.46 | 1.28 | 7 | 1.10 | 0.86 |
| 4.5 | 1.46 | 1.28 | 8 | 1.11 | 0.88 |
| 5 | 1.46 | 1.29 | 9 | 1.12 | 0.90 |
| 5.5 | 1.46 | 1.29 | 10 | 1.12 | 0.91 |
| 6 | 1.47 | 1.29 | 11 | 1.13 | 0.92 |
| 7.5 | 1.47 | 1.29 | 14 | 1.13 | 0.95 |
| 8 | 1.47 | 1.30 | 15 | 1.14 | 0.13 |
| 19 | 1.57 | 1.30 | 18 | 1.14 | 0.98 |
| 20 | 1.48 | 1.31 | 20 | 1.14 | 0.98 |

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The values of the coefficients of discharge (c) depend on the value of (n), as well as on the hydraulic slope and hydraulic radius of the open channel under consideration, in accordance with the following formula for French measures.

$$c = \frac{23 + \frac{1}{n} + \frac{0\cdot00155}{S}}{1 + \left(23 + \frac{0\cdot00155}{S}\right) \frac{n}{\sqrt{R}}}$$

which is also given in the following form:—

$$c = \frac{z}{1 + \frac{x}{\sqrt{R}}};$$

$$\text{where } z = 23 + \frac{1}{n} + \frac{0\cdot00155}{S}; \text{ and } x = n \left(23 + \frac{0\cdot00155}{S}\right).$$

The values of c , for French measures are tabulated in Herr Kutter's book 'Die neuen Formeln für die Bestimmung der mittlern Geschwindigkeit des Wassers etc.' pages 336, 386, and 436, for the three classes in which $n=0\cdot025$, $0\cdot030$ and $0\cdot035$ respectively, and a diagram there given enables c , to be roughly read off for any conditions. The same data with complete tables of velocities and discharges suited to French measures are reprinted with the consent of Herr Kutter and attached to a translation entitled 'The New Formula for Mean Velocity of Discharge' (London, 1876, Spon).

The values of c , a corresponding coefficient suited to English feet, may at any time be easily derived from any value of c , calculated or given for French mètres by the formula

$$c = 0\cdot0181 c_p$$

It is, however, preferable to obtain English data in a

more direct manner from special English tables, as will be hereafter explained.

3. *The third* stage of development of these variable coefficients was carried out by the author of this book at the request of the Indian Government in 1877 and 1878. The general truth of the formula of Herr Kutter had previously been accepted by him, after a lengthy investigation of the principles and the recorded basic experiments ; the formula itself had also already been employed by him in the calculations for some engineering designs for Mr. John Fowler. The elasticity of the formula, however, acted both as an advantage in general applicability and as a disadvantage in choice of category or class ; almost everything centred itself in the choice of the value of n , the coefficient of roughness and irregularity ; for the effect of various values of R had been justly met in the formula, and that of various values of S had been perhaps too cautiously allowed, yet was approximately and substantially correct. A fresh independent determination of a set of values of n was therefore necessary.

The author having been for many years and in many places a persistent observer and collector of data of hydraulic experiment, having had unusually numerous opportunities since 1859 on works of irrigation, on river improvement works, on canals, and on waterworks both in South America and in Northern, and Southern India, of obtaining such information, and also having been permitted both at Calcutta, Madras, Bombay, and in London to search among official records of such works, it was hoped that enough would be forthcoming to give some limits to the application of the formula for canals by fixed values of n of independent determination.

The result of these labours and collections was successful so far as it affected canals in earth, within the range of the records, of cases that had fallen under his personal observation, and that thus admitted of little doubt as to condition.

Briefly, the results were, that none of the cases in canals in earth were below $n=0.017$, that the cases in which $n=0.025$ was approximately applicable were not canals in by any means *perfect* order, that any channels of a condition suited to $n=0.035$ were from irregularity beyond the scope of anything but excessively coarse and almost useless determination; and that a large number of cases of canals in good order happen to give a value of n not far from 0.0225 .

Five fixed classes were therefore assigned to canals in earth of various soils, and in various conditions.

- 1st $n=0.020$ for very firm, regular, well-trimmed soil.
- 2nd $n=0.0225$ for firm earth, in condition above the average.
- 3rd $n=0.0250$ for ordinary earth in average condition.
- 4th $n=0.0275$ for rather soft friable soil in condition below the average.
- 5th $n=0.030$ for rather damaged canals in a defective condition.

The attempts of the author to determine independently values of n suited to canals in artificial materials, plank, rubble, ashlar, and cement, were ineffectual from want of sufficient mention of age, quality, and condition of surface of these materials in recorded cases of experiment then forthcoming. For the special material rubble these latter did not afford quite sufficient reason for

objecting to Herr Kutter's value of $n=0.017$ for that material in a normal condition, but they did indicate a wide range of values ; as to other materials, nothing resulted on account of the reason before given ; the general conclusion was that each material should have a wider range of values of n suited to various conditions. Accepting, therefore, the normal values given by Herr Kutter as correct, the extension of their range was effected by the following arrangement.

$n=0.010$ Smooth cement, worked plaster, planed wood, and glazed surfaces in perfect order.

$n=0.013$ The materials mentioned under 0.010 when in imperfect or inferior condition. Also brickwork, ashlar, and unglazed stoneware in a good condition.

$n=0.017$ Brickwork, ashlar, and stoneware in an inferior condition. Rubble in cement or plaster in good order.

$n=0.020$ Rubble in cement in an inferior condition. Coarse rubble rough-set in a normal condition.

$n=0.0225$ Coarse dry-set rubble in bad condition.

It may be noticed that it might be considered preferable to give more simple values to n , as 1, 1.3, 1.7, 2, $2\frac{1}{4}$, etc., and to modify the general formula to suit them ; but as there is yet some doubt on this point, and as established custom must be considered also, the values have for the present been allowed to retain their original form.

Application of the coefficients.—Coefficients, velocities, and discharges suited to canals of practical dimensions and data, were worked out and tabulated in accordance with these results ; they will be found in 'Canal and Culvert Tables' (London, 1878, Allen). Tables of

the coefficients are also given in the Working Tables of this book (see Table XII.); these can be applied to the tabulated values of $100 \sqrt{RS}$, given in Table VII.; thus obtaining for any case the value of a mean velocity, from the formula

$$V = c . 100 \sqrt{RS}.$$

Also to obtain Q , the corresponding quantity of discharge, the values of A , the section of flow, or hydraulic sectional area, may be taken from Table IV., thus completing the data for the formula

$$Q = AV = A . c . 100 \sqrt{RS}.$$

A value of c may, however, be occasionally, though rarely, required for some intermediate value of n ; in that case it may be interpolated without important error, or, if accuracy be required, it should be calculated from the formula. This, after reduction of terms for direct application to English feet, has been altered into the following more convenient form:—

$$c = \frac{\sqrt{R}}{100n} \left(\frac{m + 1.811}{m + \sqrt{R}} \right)$$

where $m = n \left(41.6 + \frac{0.00281}{S} \right)$.

For the converse process of determining a value of n from given data, which is more complicated, see an example at pages 376–377 of ‘Canal and Culvert Tables,’ before mentioned.

As it is of interest to notice the effect of the values of n on the coefficient c , under ordinary hydraulic slopes of from 1 in 1000 to 1 in 10000, the two following pages of tabulated values are here given; they show that c varies there from 0.329 to 2.170, the extremes

practicable being about 0.25 and 2.50. From this it is evident that if, from unwillingness to turn over the pages of tabular quantities in this book or in the 'Canal and Culvert Tables,' it be preferred to use a fixed coefficient of unity, $c=1$, for every case of velocity in canals, the extreme error may be thrice in excess, or more than a half in diminution, while the calculated probability of ever being right approximates to zero.

The above-mentioned mode of calculating mean velocities and discharges is intended to apply generally to straight, uniform reaches of open channels. For ordinary natural channels, as of streams and rivers, it affords merely a coarse approximation, as such discharges cannot be accurately ascertained without some velocity-observation.

It will, however, be perfectly evident that the general method does not by any means preclude the application of an allowance or deduction for special circumstances. In actual fact, few channels are either perfectly straight, perfectly regular, or free from easily estimated lateral and longitudinal irregularities; variety in this particular alone may affect the amount of discharge by as much as twenty per cent., even after making allowance for loss of head by bends and obstructions. The local conditions of a channel, the wind, the amount of silt in suspension, the motion of its shoals, the change of the set of its currents, all seriously affect a discharge calculated from data that make no allowance for these circumstances. These causes of retardation are enumerated in section I of this chapter.

For canals and regular rectangular and trapezoidal channels in earth in good order, the calculated discharges will be more correct than those for deteriorated and

Coefficients of mean velocity suited to various materials, calculated for a fixed value of $S=0\cdot001$.

| R in feet | Values of n | | | | | | | |
|----------------|---------------|-------|-------|-------|-------|--------|-------|-------|
| | ·010 | ·013 | ·017 | ·020 | ·0225 | ·0250 | ·0275 | ·0300 |
| | (1) | (2) | (3) | (I.) | (II.) | (III.) | (IV.) | (V.) |
| 0·5 | 1·385 | 1·011 | 0·730 | 0·598 | 0·518 | 0·455 | 0·404 | 0·363 |
| 1 | 1·562 | 1·615 | 0·860 | 0·715 | 0·625 | 0·554 | 0·496 | 0·449 |
| 1·25 | 1·614 | 1·212 | 0·901 | 0·752 | 0·660 | 0·586 | 0·527 | 0·478 |
| 1·5 | 1·655 | 1·249 | 0·933 | 0·782 | 0·688 | 0·613 | 0·552 | 0·502 |
| 1·75 | 1·688 | 1·279 | 0·961 | 0·808 | 0·712 | 0·635 | 0·573 | 0·522 |
| 2 | 1·716 | 1·305 | 0·984 | 0·829 | 0·732 | 0·655 | 0·592 | 0·450 |
| 2·25 | 1·740 | 1·327 | 1·004 | 0·848 | 0·750 | 0·672 | 0·608 | 0·555 |
| 2·5 | 1·761 | 1·346 | 1·021 | 0·864 | 0·765 | 0·687 | 0·622 | 0·569 |
| 2·75 | 1·779 | 1·363 | 1·037 | 0·879 | 0·779 | 0·700 | 0·635 | 0·581 |
| 3 | 1·795 | 1·378 | 1·051 | 0·892 | 0·792 | 0·712 | 0·647 | 0·592 |
| 3·25 | 1·809 | 1·392 | 1·063 | 0·904 | 0·804 | 0·723 | 0·657 | 0·603 |
| 3·5 | 1·823 | 1·404 | 1·075 | 0·915 | 0·814 | 0·733 | 0·667 | 0·612 |
| 4 | 1·845 | 1·426 | 1·095 | 0·935 | 0·833 | 0·751 | 0·685 | 0·629 |
| 4·5 | 1·865 | 1·444 | 1·113 | 0·951 | 0·849 | 0·767 | 0·700 | 0·644 |
| 5 | 1·881 | 1·460 | 1·128 | 0·966 | 0·863 | 0·781 | 0·713 | 0·657 |
| 5·5 | 1·896 | 1·474 | 1·141 | 0·979 | 0·876 | 0·793 | 0·725 | 0·668 |
| 6 | 1·909 | 1·487 | 1·153 | 0·991 | 0·887 | 0·804 | 0·736 | 0·679 |
| 6·5 | 1·921 | 1·498 | 1·164 | 1·001 | 0·897 | 0·814 | 0·746 | 0·688 |
| 7 | 1·931 | 1·508 | 1·174 | 1·010 | 0·907 | 0·823 | 0·754 | 0·697 |
| 7·5 | 1·940 | 1·517 | 1·183 | 1·019 | 0·915 | 0·831 | 0·763 | 0·705 |
| 8 | 1·949 | 1·526 | 1·191 | 1·027 | 0·923 | 0·839 | 0·770 | 0·712 |
| 8·5 | 1·957 | 1·534 | 1·198 | 1·034 | 0·930 | 0·846 | 0·777 | 0·719 |
| 9 | 1·964 | 1·541 | 1·205 | 1·041 | 0·937 | 0·853 | 0·784 | 0·726 |
| 10 | 1·977 | 1·554 | 1·218 | 1·054 | 0·949 | 0·865 | 0·795 | 0·737 |
| 15 | 2·023 | 1·599 | 1·263 | 1·098 | 0·993 | 0·908 | 0·838 | 0·780 |
| 20 | 2·051 | 1·627 | 1·291 | 1·126 | 1·021 | 0·936 | 0·866 | 0·807 |

Coefficients of mean velocity suited to various materials, calculated for a fixed value of $S=0.0001$.

| <i>R</i> in feet | Values of <i>n</i> | | | | | | | |
|---------------------|--------------------|-------|-------|--------|-------|--------|-------|-------|
| | ·010 | ·013 | ·017 | ·020 | ·0225 | ·0250 | ·0275 | ·0300 |
| | (1) | (2) | (3) | (I.) | (II.) | (III.) | (IV.) | (V.) |
| 0.5 | 1.263 | 0.916 | 0.658 | 0.539 | 0.467 | 0.410 | 0.365 | 0.329 |
| 1 | 1.478 | 1.097 | 0.806 | 0.669 | 0.585 | 0.518 | 0.465 | 0.421 |
| 1.25 | 1.545 | 1.155 | 0.855 | 0.713? | 0.625 | 0.556 | 0.499 | 0.453 |
| 1.5 | 1.598 | 1.201 | 0.895 | 0.750? | 0.659 | 0.587 | 0.529 | 0.480 |
| 1.75 | 1.643 | 1.240 | 0.929 | 0.780 | 0.687 | 0.613 | 0.554 | 0.504 |
| 2 | 1.680 | 1.274 | 0.959 | 0.807 | 0.712 | 0.637 | 0.576 | 0.525 |
| 2.25 | 1.712 | 1.303 | 0.984 | 0.831 | 0.734 | 0.658 | 0.595 | 0.543 |
| 2.5 | 1.741 | 1.329 | 1.007 | 0.852 | 0.754 | 0.676 | 0.613 | 0.560 |
| 2.75 | 1.766 | 1.352 | 1.028 | 0.871 | 0.772 | 0.693 | 0.629 | 0.575 |
| 3 | 1.788 | 1.372 | 1.046 | 0.888 | 0.788 | 0.709 | 0.643 | 0.589 |
| 3.25 | 1.809 | 1.391 | 1.063 | 0.904 | 0.803 | 0.723 | 0.657 | 0.602 |
| 3.5 | 1.827 | 1.408 | 1.079 | 0.918 | 0.817 | 0.736 | 0.670 | 0.614 |
| 4 | 1.860 | 1.438 | 1.106 | 0.944 | 0.842 | 0.760 | 0.692 | 0.636 |
| 4.5 | 1.888 | 1.465 | 1.130 | 0.967 | 0.864 | 0.780 | 0.712 | 0.655 |
| 5 | 1.912 | 1.487 | 1.152 | 0.987 | 0.883 | 0.799 | 0.730 | 0.672 |
| 5.5 | 1.933 | 1.508 | 1.170 | 1.005 | 0.900 | 0.816 | 0.746 | 0.688 |
| 6 | 1.952 | 1.526 | 1.187 | 1.021 | 0.916 | 0.831 | 0.760 | 0.702 |
| 7 | 1.985 | 1.557 | 1.217 | 0.050 | 0.943 | 0.857 | 0.786 | 0.727 |
| 8 | 2.012 | 1.583 | 1.242 | 1.073 | 0.966 | 0.880 | 0.808 | 0.748 |
| 9 | 2.035 | 1.605 | 1.263 | 1.094 | 0.986 | 0.899 | 0.827 | 0.767 |
| 10 | 2.055 | 1.625 | 1.282 | 1.112 | 1.004 | 0.916 | 0.844 | 0.783 |
| 11 | 2.073 | 1.642 | 1.298 | 1.128 | 1.020 | 0.932 | 0.859 | 0.798 |
| 12 | 2.088 | 1.657 | 1.313 | 1.143 | 1.034 | 0.946 | 0.873 | 0.811 |
| 13 | 2.102 | 1.670 | 1.326 | 1.156 | 1.047 | 0.958 | 0.885 | 0.823 |
| 14 | 2.114 | 1.683 | 1.338 | 1.168 | 1.058 | 0.970 | 0.896 | 0.834 |
| 15 | 2.126 | 1.694 | 1.349 | 1.178 | 1.069 | 0.980 | 0.907 | 0.845 |
| 20 | 2.170 | 1.738 | 1.393 | 1.222 | 1.112 | 1.023 | 0.949 | 0.886 |

irregular channels; the errors due to various irregularities in the former case forming a smaller percentage. Formulæ for velocity and for discharge are, however, almost as frequently used in determining a section of canal intended to convey a certain discharge, as to obtain a discharge from data of an actual canal.

In these cases, a consideration of the various forms of section, suitable to different purposes, is also necessary. This matter has been treated and repeated in nearly the same terms in all works on hydraulics published in the last half-century; the ideas were perhaps due to laborious hydraulicians now forgotten, as they cannot be clearly traced; and little can be now added to them; but as the entire omission of the subject in this Manual might cause disappointment, section 6 will be devoted to that special subject, though its treatment will be slightly modified to suit modern notions of discharge.

The discharge of pipes.

The calculation of the discharge of pipes may be conducted either on the same principle as that of artificial channels or on that of orifices. It is extremely unfortunate that the investigations of Ganguillet and Kutter were limited to open channels, and hence the application of their principles to pipes, though rationally superior to any other mode previously adopted, cannot be conducted with the same amount of experimental record in support.

Assuming then the general formula for mean velocity of discharge—

$$V = c \times 100 (RS)^{\frac{2}{3}},$$

and adapting it to terms of the diameter of a pipe in

feet ; it becomes for full cylindrical pipes and tubes of all sorts, where $R = \frac{1}{4}d$ and d is the internal diameter,

$$V = c \times 50 (dS)^{\frac{1}{2}},$$

and as the actual discharge is the quantity more usually required direct in the case of pipes, this is—

$$\begin{aligned} Q &= AV = c \times 0.7854d^2 \times 50(dS)^{\frac{1}{2}}, \\ &= c \times 39.27(Sd^5)^{\frac{1}{2}}, \end{aligned}$$

for discharges in cubic feet per second.

The converse forms of this expression being—

$$\begin{aligned} d &= 0.23 \left(\frac{Q^2}{c^2 S} \right)^{\frac{1}{5}}, \\ H &= \frac{1}{c^2} \times 0.0648 \frac{Q^2}{d^5}, \end{aligned}$$

where H is the head in feet for a length of 100 feet, or is equal to 100 S .

The values of these quantities are given in Parts 1, 2, 3, and 4, of Table VIII., for a value of $c = 1$, and the values of c given in the table of coefficients of discharge, Table XII., can be applied ; the powers and roots of c can be taken from the Miscellaneous Tables.

With regard to these coefficients, it will be noticed that for want of sufficient experimental data, a coefficient of roughness $n = 0.010$ has been assumed as applicable to glazed or enamelled metal pipes, and one of 0.013 for ordinary metal and earthenware or stoneware pipes under ordinary conditions, but not new ; and there is every reason to believe that these assumptions are generally correct, if we compare the smoothness of surface of a glazed pipe with that of very smooth plaster in cement, and that of an ordinary pipe, in average condition, with that of ashlar or good brickwork ; in addition

to this, such few partial and limited experimental data as are available support this assumption.

In applying, however, to pipes the coefficients of discharge resulting from the foregoing formula, one would naturally be unwilling to push to extremes a principle derived from observation on open channels, and would prefer stopping at a point where the experimental data now forthcoming leave us. It would, therefore, seem imprudent at present to assume that the asserted law of coefficients holds good for an hydraulic radius R less than 0·1 foot. This limiting hydraulic radius of 0·1 foot or of 1 tithé or tenth is that of a 5-inch pipe, or a pipe having a diameter of 0·4 foot ; and in cases of falls steeper than 0·001 the corresponding coefficient for glazed pipes is 0·84, and for ordinary pipes 0·61. Hence for the present, and until further experiments have thrown more light on the subject, it may also be assumed that the coefficient of discharge for all full cylindrical pipes, having a diameter less than 0·4 feet, will be the same as for those of that diameter.

Reverting to the original formulæ for mean velocity and for discharge in pipes of all sorts,

$$V = c . 100 \sqrt{RS}$$

$$Q = AV = A . c . 100 \sqrt{RS}$$

it must be borne in mind that, though with open channels and unfilled culverts S represents the sine of the slope of the water surface, with filled pipes under low heads due to their inclinations S represents the sine of a mean hydraulic slope that is not necessarily identical with the inclination of any part of the pipes ; while if there should, in addition, be any permanent statical head of pressure on the upper entrance of the pipe, the conditions

are again changed by this further complication, and the above principle is then only partially applicable.

With siphons also that have been exhausted of air, a statical pressure of one atmosphere is added to the effective differential head.

These matters will be further explained in Section 7, devoted to the hydraulic slope.

It must also be noticed that it is merely with filled cylindrical pipes that the mean hydraulic radius is equal to one-fourth their diameter. In all other cases the value of R must be determined from the section of flow, whatever it may be, by dividing that sectional area by the wet perimeter of the bottom and sides up to water surface level. This subject will be treated in Section 6.

Bearing in mind the liabilities under these two special peculiarities, it yet remains that both S and R have certain values in connexion with pipe discharge that may be applied in the general formulæ originally given.

6. THE HYDRAULIC SECTION OR SECTION OF FLOW.

On examining the equations representing the principle of flow and of discharge (Section 1, Chapter I.), it will be noticed that the sectional area of flow, and its function the hydraulic mean radius, are both involved.

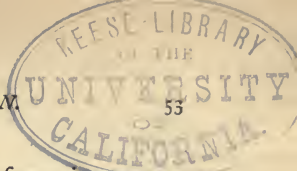
There may still remain considerable doubt whether in all cases the mean hydraulic radius, $R = \frac{A}{P}$, is the exact term for correct introduction into any general formula of the type,

$$Q = AV = A \cdot c \cdot 100 \sqrt{RS}.$$

In excessively wide and comparatively shallow sections of flow the resistance of the air on the water surface becomes an important function, and in that case, the prime hydraulic radius $R_1 = \frac{A}{P+W}$ might, as adopted by Captain Humphreys and Abbot on the Mississippi, be more suitably introduced, with a corresponding new set of coefficients c_1 in place of c . In the converse case of very narrow and very deep sections of flow, an augmented hydraulic radius $R_2 = \frac{A}{P-W}$ might be a convenient means of modification for obtaining the augmented discharges actually resulting in such sections, that are physically due to diminished total friction on the perimeter that mostly consists of the two sides.

There is, however, much doubt as to the mode and limits within which these functions could be correctly introduced ; while the two extremes of excessive width and of very great depth of section are of comparatively rare practical occurrence. A general adherence to the use of R , the mean hydraulic radius in all ordinary cases, is hence advisable, and will for purposes of convenience be assumed in this book, except where otherwise mentioned.

The relative dimensions of the hydraulic section or section of flow become important principally from two points of view ; first, when the maximum discharge possible through the section has to be considered, as in drainage-cuts, flood-escapes, and such channels where erosion from high velocity might not be a serious defect ; secondly, when in design there is sufficient scope for various forms of section that would have equal discharging powers, and among which a choice has to be made.



The conditions of the canal section of maximum discharge.

From the functions involved in the general formula of discharge

$$Q = A c . 100 \sqrt{R S},$$

it is evident that though the conditions of a complete maximum cannot be determined, those of a partial and nearly complete maximum admit of reduction in known terms. Assuming that the side slopes of the section are fixed by practical considerations of soil, &c., that the hydraulic slope is constant, and the coefficient of roughness also ; and using the following symbols :

Let t to 1 be the given ratio of the side slope.

b and d the bed width and depth of the water section.

R the mean hydraulic radius.

P the wet perimeter.

S the hydraulic slope.

Now with a trapezoidal section of any proportions,

$$R = \frac{A}{P} = \frac{d(b + td)}{b + 2d(1 + t^2)^{\frac{1}{2}}}$$

Under the condition of maximum discharge, A will be a maximum, so also will R ; and when these are temporarily constant, P will be a minimum.

$$\text{hence } \partial A = d . \partial b + b . \partial d + 2td . \partial d = 0 \quad (1)$$

$$\partial P = \partial b + 2\partial d (1 + t^2)^{\frac{1}{2}} = 0 \quad (2)$$

Subtracting (1) from (2),

$$2\partial d \left\{ (1 + t^2)^{\frac{1}{2}} - td \right\} + \partial b (1 - d) - b\partial d = 0 ;$$

substituting for δb its value, $-2\delta d (1+t^2)^{\frac{1}{2}}$,

$$2\delta d \left\{ d (1+t^2)^{\frac{1}{2}} - td \right\} - b\delta d = 0;$$

dividing by δd and reducing,

$$b = 2d \left\{ (1+t^2)^{\frac{1}{2}} - t \right\}$$

substituting this value of b , in $A = d (b + td)$,

$$A = d^2 \left\{ 2 (1+t^2)^{\frac{1}{2}} - t \right\}$$

$$d = \sqrt{A} \left\{ 2 (1+t^2)^{\frac{1}{2}} - t \right\}^{-\frac{1}{2}}$$

$$b = 2 \sqrt{A} \left\{ 2 (1+t^2)^{\frac{1}{2}} - t \right\}^{-\frac{1}{2}} \times \left\{ (1+t^2)^{\frac{1}{2}} - t \right\}$$

$$\text{and } R = \frac{A}{b + 2d (1+t^2)^{\frac{1}{2}}}$$

Then for any given value of t , the quantities d and b may be expressed in terms of \sqrt{A} with numerical coefficients; according to the following table.

The above results may also be reduced to another form of expression.

If a the angle of inclination with the horizon of the side slope be given, it is evident from the above that $t = \cotg a$, and A may be also expressed in the old form, $d^2 (\text{cosec } a + \tan \frac{1}{2} a)$; whence also corresponding values of d and b may be reduced from given values of a ; this form is, however, not practically as convenient as the former.

The geometrical figure obtained by this process is a trapezoid touching a semicircle; it has the least perimeter for a given area, and has greatest values of R , V , Q , and approximately of c . It cannot be drawn or determined

geometrically under ordinary practical conditions, but after algebraic determination it may be verified by diagram.

Table of relative Trapezoidal Sections of maximum discharge having a given area A , and given side slopes t to 1.

| Ratio of side slope t to 1 | Numerical factors to be multiplied by \sqrt{A} | | | Corresponding value of A in terms of d^2 |
|------------------------------|--|-------------------|--------------------------|--|
| | for d depth | for b bed width | for R hydraulic radius | |
| 0 to 1 | 0·7071 | 1·4142 | 0·3536 | $2 d^2$ |
| $\frac{1}{4}$ to 1 | 0·7430 | 1·1601 | 0·3715 | $1·8116 d^2$ |
| $\frac{1}{2}$ to 1 | 0·7590 | 0·9382 | 0·3795 | $1·7361 d^2$ |
| $\frac{3}{4}$ to 1 | 0·7587 | 0·8121 | 0·3794 | $1·7370 d^2$ |
| 1 to 1 | 0·7559 | 0·7559 | 0·3780 | $1·75 d^2$ |
| 1 to 1 | 0·7395 | 0·6126 | 0·3745 | $1·8284 d^2$ |
| $1\frac{1}{4}$ to 1 | 0·7158 | 0·5021 | 0·3579 | $1·9516 d^2$ |
| $1\frac{1}{2}$ to 1 | 0·7071 | 0·4714 | 0·3536 | $2 d^2$ |
| $1\frac{3}{4}$ to 1 | 0·6891 | 0·4174 | 0·3445 | $2·1056 d^2$ |
| $1\frac{1}{2}$ to 1 | 0·6621 | 0·3517 | 0·3310 | $2·2812 d^2$ |
| 2 to 1 | 0·6434 | 0·3003 | 0·3180 | $2·4722 d^2$ |
| $2\frac{1}{2}$ to 1 | 0·5887 | 0·2268 | 0·2944 | $2·8852 d^2$ |
| 3 to 1 | 0·5484 | 0·1780 | 0·2742 | $3·3246 d^2$ |
| 4 to 1 | 0·4853 | 0·1195 | 0·2426 | $4·2462 d^2$ |

This general trapezoid comprises also the rectangular and the square sections; these including most ordinary forms of canal and channel section. Sections with curved side-walls may be dealt with by an approximative corresponding process. The theory applied in the foregoing reduction is not complete nor rigidly correct, though nearly so; its application to deep sections in which the depth exceeds the width in moderation will be less accurate, and it probably would not hold at all for those in which the depth exceeds double the width.

The condition of equal-discharging canal sections.

In navigable canals, and canals of supply and of irrigation, high velocities and great fluctuation of draught under variation of supply are generally inadmissible, thus precluding the use of sections of absolute maximum discharge. An economic section will then not allow of any waste of sectional area, or of depth which is more expensive than width, but will have the highest maximum discharge that the limiting predetermined velocity and other fixed local circumstances admit. These circumstances are, the nature of the soil in the bed and banks, their liability to damage from erosion, and the side slope that can be practically maintained in it; the hydraulic slope and the inclination of bed that are locally practicable; and in some cases the navigable depth to be maintained during conditions of lowest supply. The mean width of section therefore generally remains the only important function of discharge that can be much varied in designing the section; hence, if a predetermined depth has to be approximately maintained, the usual practice is, to assume originally some fixed convenient ratio of mean width to depth, such as 10 to 1, 14 to 1, or 16 to 1, and after calculating the velocity due to this as well as the other predetermined conditions, to reduce or increase the assumed mean width by two or three feet at a time by repeated trial until a safe bottom velocity is attained in the form of section.

Such a final section being, then, safe as regards limiting velocity and sufficient for the required discharge, is then perhaps only one out of a number of equal-discharging sections that might be devised; and some other one of these might be preferable for any special

reason. It may therefore be necessary to know the relations between mean width and depth in such a series of sections, when the side slopes have been finally determined.

In order to discover the relation between mean width and depth, giving various sections that will discharge the same quantity of fluid, when the hydraulic slope is a constant quantity, we must use the condition that the areas of all such sections are inversely as the square roots of their hydraulic radii; that is,

$$A \sqrt{R} = a, \text{ a constant; and as } A = WD; R = \frac{WD}{W+2D};$$

$$\text{this becomes } \frac{W^3 D^3}{W+2D} = a^2,$$

which may be reduced to either of the following forms in terms of the modified section according as either d or w is the new quantity sought,

$$d^3 - \frac{2a^2}{w^3}d - \frac{a^2}{w^2} = 0; \text{ or } w^3 - \frac{a^2}{d^3}w - \frac{2a^2}{d^2} = 0.$$

In the first case, let $W=100, D=1, 2, 3, 4, 5, 6$, successively, then the values of a are thus in each of the six cases,

| | | | | | | | | |
|-----|---|---|-------|-------|-------|-------|------|------|
| D | . | . | 1 | 2 | 3 | 4 | 5 | 6 |
| a | . | . | 99.01 | 277.3 | 504.7 | 769.9 | 1066 | 1389 |

and for a fixed value $w=90$, the corresponding values of d are

| | | | | | | | | |
|-----|---|---|-------|-------|-------|-------|-------|-------|
| d | . | . | 1.074 | 2.151 | 3.232 | 4.312 | 5.391 | 6.483 |
|-----|---|---|-------|-------|-------|-------|-------|-------|

and for a fixed value of $d=2.5$, the corresponding values of w are

| | | | | | | | | |
|-----|---|---|-------|-------|-------|-------|-------|-------|
| w | . | . | 27.25 | 72.53 | 130.1 | 197.2 | 272.1 | 353.8 |
|-----|---|---|-------|-------|-------|-------|-------|-------|

The application of this principle is useful in designing canal sections, where the discharge, the hydraulic slope, and either the depth of water or the width are to be invariable throughout a long course.

The table of equal-discharging sections, arranged in several groups in Table V. answers most practical purposes of this sort, as it admits of interpolation ; but in any very special case it will be necessary to compute from the above formula and solve the cubic equation for each result. In such work the trigonometrical solution given at page 188 of 'Accented Four Figure Logarithms' (London, 1881, Allen) with the help of those tables¹ will give results correct in four figures with the minimum of labour.

Sections of flow in water-pipes.

The form of hydraulic section of a water-pipe admits of little or no variation. All small water-pipes and most large ones are generally cylindrical and kept constantly full during discharge, for the reason that a circular section perfectly fulfils the conditions of a maximum-discharging section, as it has the least wetted perimeter for a given sectional area. An open semicircle has the same geometrical property, and also has its hydraulic mean radius equal to half its middle depth, but this is not the case with a closed semicircle. The relative dimensions of such sections in terms of the square root of the area are thus :—

¹ Tables of sines to seconds for the first thirty minutes have been added to that book for this special purpose.

| Section | Numerical factors to be multiplied by \sqrt{A} | | | Corresponding values of A in terms of d^2 |
|-------------------------------|--|-----------------------|--------------------------|---|
| | for d depth | for w surface width | for R hydraulic radius | |
| Circle | 1.1284 | 0 | 0.2821 | 0.7854 d^2 |
| Open semicircle | 0.7979 | 1.5958 | 0.3990 | 1.5708 d^2 |
| Open segment of 90° | 0.3729 | 1.8002 | 0.5642 | 0.9155 d^2 |

In old water-pipes the section may be much diminished by incrustation and deposit; when this is the case, the reduced section should be employed in calculating its discharge; but in most cases of old pipes the cause of decreased velocity and discharge is not merely the diminution of section, but the higher friction due to foulness and roughness of the interior surface, so that the former mode of making allowance is grossly insufficient. The correct method is to use a modified coefficient of roughness (n), and the corresponding velocity coefficient (c) due to the conditions of the case in the general formula

$$Q = A . c . 100 \sqrt{RS}$$

See coefficients suited to old metal, and damaged materials in bad condition, in Working Table, No. XII.

A still better method is to keep the water-pipes free and clean, and apply some enamelling process as that of Dr. Angus Smith, so that the full discharge due to new material in good order may be always maintained.

Sections of Flow in Culverts and Drain-pipes.

The ordinary conditions and purposes of a culvert, sewer, or drain-pipe are, that it shall carry away the

maximum supply of liquid with a head of pressure amounting to zero, that is, when just full, that it shall be able to withstand pressures that may occur from a low head of a few feet when blocked or over-supplied, that the form of its perimeter shall not favour lodgment, that the lower part of its section shall be so formed as to secure comparatively high velocity under low discharge, and that the upper part of the section shall possess sufficient strength to withstand the weight of superincumbent earth, when buried in the soil.

The cylinder having a circular section does not completely meet these requirements, and hence is generally adopted for small culverts and for drain-pipes of half a foot in diameter and less, regard being had to convenience in manufacture and laying.

Larger culverts have been made of an endless variety of sectional form, from squares and deep rectangles with or without curved crowns and inverts, to geometrically constructed and artificially made-up ovals and ellipses. The nature of the material used and economy of construction may outweigh the need of suitability of sectional form in other respects; but when this is not the case the ovals and approximate ovals constitute the best type of culvert section from thoroughly answering the purposes first mentioned. Their crowns are nearly semicircular, thus possessing strength; their inverts are sharply curved, thus giving higher flush when very partially filled; their sides are of flat curvature or nearly straight, thus preventing lodgment. It may perhaps be urged that they have the defect of weakness at the sides under lateral pressure of earth; this would doubtless be a substantial objection in loose soil or under some special circumstances, but in firm soil and in ordinary

cases it is a comparatively trivial one, as the blows and shocks received in laying are far more destructive than ordinary lateral pressure; besides this, it must be noticed that the excavation for placing an ovoid culvert is rectangular, slightly exceeding the external width of the culvert, and necessitates packing or backing; thus a slight increase of strength is afforded by the additional concrete filling. The bulging-in of sides of culverts has thus to be provided against under many circumstances, and delicate refinement on this point is impracticable in ordinary cases, while the necessity for avoiding internal lodgment is peremptory. For the same reason straight sides sloping from the springing of the crown to the springing of the invert are generally both unobjectionable and advantageous.

The conditions of culverts and drain-pipes, as well as usual custom and practice, impose limits on their sizes and dimensions in section. Cylindrical culverts and drain-pipes are now seldom made with diameters exceeding 1·5 foot; when used in larger sizes it is in cases where they can be kept steadily well supplied, and not allowed to run very low, a condition that occurs infrequently with diameters exceeding 5 feet. Ovoidal sewers of various patterns are generally adopted in a series of regular sizes from 1 by 1·5 feet up to 6 by 9 feet. The two types of oval most commonly used are Hawksley's and the Metropolitan pattern, originally, it is believed, designed by Phillips; both of these, as well as the following type, are circular-headed. The tendency of engineers up till now having continually been to adopt culvert sections that allow of higher flushing with the same amount of supply, this principle is carried out to the full in the Pegtop form of culvert section designed by the author, where the in-

vert is made small to produce greater scour, and the sides, being straight, possess the great advantage of preventing the lodgment of sediment. These three types of ovoid, together with the cylinder, include all that is commonly necessary: their sectional data given in Table V. are arranged for cases where they are either quite full, two-thirds full—that is, filled to two-thirds of their vertical depths, or one-third full. For any other special depths of flow, which are not frequently wanted, the sectional data must be calculated with the help of a table of circular arcs and sectors (see Miscellaneous Tables); examples of such calculations will be hereafter given.

Culverts and drain-pipes are generally treated as falling in some one of three classes as regards size, the small, the intermediate, and the large; there are also usual practical limits to their inclinations. As regards material, they are made in plain earthenware and glazed stoneware up to dimensions of 2 by 3 feet, rarely above that, and brickwork and concrete, either plain or coated with cement, is used in larger dimensions. Iron of all sorts, either plain, painted, or enamelled, may of course be used in any dimensions, the adoption of wrought iron beginning where cast iron becomes inapplicable from the size of the casting being inconvenient in transport, or from other reasons.

Proceeding to the calculation of hydraulic data for culvert sections.

The calculation of hydraulic radii and sectional areas of partly-filled culverts.

The determination of values of R , the hydraulic radius, and A , the sectional area for culverts when partly

filled, being sometimes rather troublesome, a few examples of such cases may be of use as a guide; the cases selected being those of various sections, filled to one-third and two-thirds their depth adopted in Table V. In such cases fractions of areas and of perimeters of circles are frequently used; and for such purposes the table of arcs and sectors in the Miscellaneous Tables has been specially constructed.

Taking the Pegtop section, the geometrical construction of which is as follows:—

Taking the transverse diameter=2; the long diameter, or total vertical depth=3; the radius of the upper circle is 1.0, the radius of the invert is one-eighth the total depth=0.375; and the straight sides, which are tangential to both upper and lower circles, are each equal to one-half the total depth=1.5. For the complete section of the culvert, the sector of the upper circle extends beyond the semicircle to nearly 20° on each side; while the sector of the lower circle extends correspondingly to 20° less than the semicircle on each side; *i.e.* these two sectors are 220° and 140° respectively.

The full sectional area—

$$A_1 = \text{Sector of } 220^\circ \text{ to radius } 1 + \text{Sector of } 140^\circ \text{ to radius } 0.375 \\ + \text{twice half depth} \times \text{mean radius};$$

(Using the table of arcs and sectors),

$$= 1.91987 \times 1^2 + 1.22173 \times (0.375)^2 + 3 \times 0.6875 = 4.15418.$$

And the complete perimeter—

$$P_1 = \text{Arc of } 220^\circ \text{ to diameter } 2 + \text{arc of } 140^\circ \text{ to diameter } 0.75 + \\ \text{twice half depth.}$$

$$= 1.91987 \times 2 + 1.22173 \times 0.75 + 3.0 = 7.75604.$$

And R_1 the hydraulic radius of the full section=0.536.

The values of R_1 for any other diameter are proportional.

For the same culvert-section when filled to two-thirds its depth.

$$A_2 = 4.15418 - \text{area of semicircle to radius 1} \\ = 4.15418 - 1.57080 \times 1^2 = 2.58338$$

$$P_2 = 7.75604 - \text{arc of semicircle to diameter 2} \\ = 7.75604 - 1.57080 \times 2 = 4.61444$$

And R_2 , the required hydraulic radius = 0.560

The values of R_2 for any other diameter are proportional.

For the same culvert-section when filled to one-third the depth.

$$A_3 = \text{sector of } 140^\circ \text{ to radius } 0.375 + \frac{3}{4} \text{ depth} \times \frac{R+3r}{4} \\ = 1.22173 \times (0.375)^2 + 0.75 \times \frac{1+1.125}{2} = 0.96868$$

$$P_3 = \text{arc of } 140^\circ \text{ to diameter } 0.75 + \frac{3}{4} \text{ of the total depth} \\ = 1.22173 \times 0.75 + \frac{1}{2} \times 3 = 2.54130$$

And R_3 , the required hydraulic radius = 0.381

The values of R_3 for any other diameter are proportional.

Checking the above by calculating for the middle portion of the section.

$$\text{Area} = 2 \text{ sectors of } 20^\circ \text{ to radius } 1 + \frac{3}{4} \text{ depth} \times \frac{3R+r}{4} = \\ 0.34907 + 0.75 \times \frac{3.375}{2} = 1.61470$$

and above, $2.58338 - 0.96868 = 1.61470$

Perimeter = 2 arcs of 20° to diameter $2 + \frac{1}{2}$ total depth.

$$= 0.34907 \times 2 + \frac{1}{2} - 3 = 2.07314$$

and above, $4.61444 - 2.54130 = 2.07314$

Dealing in the same manner with Hawksley's Ovoid Section, the geometrical construction of which is thus,—

Taking the transverse diameter = 2, and the radius of the top semicircle = 1 ; the radius of each curved side of 45° is = 2, the radius of the invert of 90° is = 0·5858, and the total vertical depth is 2·5858. The sectors cut off by the trisection of the depth are 164° 12' and 21°.

The respective areas are—

$$A_1 = 1\cdot5708 \times 1^2 + 0\cdot7854 \times 2^2 - \frac{1}{2}2 + 0\cdot7854 \times (0\cdot5858)^2 = 3\cdot9820$$

$$A_2 = 0\cdot138 \times 1\cdot99 + 0\cdot7854 \times 2^2 - \frac{1}{2}2 + 0\cdot7854 \times \cdot3432 = 2\cdot6858.$$

The middle area being more convenient to calculate, this is
 $0\ 138 \times 1\cdot99 + \cdot36652 \times 2^2 - \cdot38386 \times \frac{2}{3} + \cdot34 \times \cdot88578 = 1\cdot6580$
 and A_3 the area of bottom portion = $2\cdot6858 - 1\cdot6580 = 1\cdot0278$

The corresponding perimeters are—

$$P_1 = 1\cdot57080 \times 2 + 0\cdot7854 \times 4 + 0\cdot7854 \times 1\cdot1716 = 7\cdot20337$$

$$P_2 = \cdot13788 \times 2 + 0\cdot7824 \times 4 + 0\cdot7854 \times 1\cdot1716 = 4\cdot33753$$

and the perimeter of the middle third is

$$= \cdot13788 \times 2 + \cdot36652 \times 4 \qquad \qquad \qquad - 1\cdot74184$$

$$P_3 = 4\cdot33753 - 1\cdot74184 \qquad \qquad \qquad = 2\cdot59569$$

Hence the three corresponding hydraulic radii are

$$R_1 = 0\cdot553, R_2 = 0\cdot620, R_3 = 0\cdot396.$$

Checking the above by the top area and perimeter to two-thirds the depth,

$$\text{area} = 1\cdot5708 \times 1^2 + \cdot36652 \times 2^2 - \cdot38386 + \cdot34 \times \cdot88578 = 2\cdot9542$$

$$\text{and } 3\cdot9820 - 1\cdot0278 \qquad \qquad \qquad = 2\cdot9542$$

$$\text{perimeter} = 1\cdot57080 \times 2 + \cdot36652 \times 4 \qquad \qquad \qquad = 4\cdot60768$$

$$\text{and } 7\ 20337 - 2\cdot59569 \qquad \qquad \qquad = 4\cdot60768$$

In the same way with Phillips' Metropolitan Ovoid, of which the geometrical construction is thus:—

Taking the transverse diameter = 2, and the radius of the top semicircle = 1, the extreme vertical depth is = 3; the radius of the curved side = 3; the radius of the invert is (one-sixth the depth, or) 0.5; and the depth from springing to bottom = 2; the curved side has an arc of $36^{\circ} 52' 14''$, and the invert an arc of $106^{\circ} 16'$. A trisection of the depth cuts off $19^{\circ} 28'$ of the side arc in the middle portion.

The respective areas, when full, two-thirds full, and one-third full, are

$$A_1 = 1.5708 \times 1^2 + .64352 \times 3^2 + .92735 \times (0.5)^2 - 2 \times 1.5 = 4.594$$

$$A_2 = 4.5942 - 1.5708 = 3.023$$

and the area of the middle portion is

$$.33975 \times 3^2 - 2 \times \frac{1}{2} \times 2 \times .70693 + .29307 \times .82914 = 1.887$$

$$A_3 = 3.0234 - 1.8868 = 1.136$$

The respective perimeters are

$$P_1 = 1.57080 \times 2 + .64352 \times 6 + .92735 \times 1 = 7.930$$

$$P_2 = .64352 \times 6 + .92735 = 4.788$$

$$\text{Mid-portion perimeter} = .33975 \times 6 = 2.038$$

$$P_3, \text{ of lower third} = 2.75$$

Hence the hydraulic radii corresponding are

$$R_1 = 0.579, R_2 = 0.631, \text{ and } R_3 = 0.413.$$

For similar culverts of other dimensions the areas can be reduced in the ratios of the squares of these diameters and the hydraulic radii in direct proportion to the diameters themselves.

The above cases show the utility of the Table of Arcs

and Sectors given in the additional Tables, which can be applied to all similar purposes.

These three types of culvert-section, as well as the cylinder, are illustrated in the Frontispiece of Canal and Culvert Tables by figures of equal sectional area; whose relative diameters are thus,

| | |
|-----------------------------|--------------------|
| Cylindrical Section | 1.1286 |
| Hawksley's Ovoid | 1.0002 and 1.293 |
| Metropolitan Ovoid | 0.9331 and 1.3996 |
| Pegtop Section | 0.9813 and 1.4720. |

They are divided to thirds of their actual longer diameters, and the dotted line on the Pegtop Section shows the gain in height of flushing that this has in comparison with the Metropolitan pattern, of equal full sectional area. Its form is effective in preventing lodgment, and very convenient in calculations for intermediate depths.

For the converse process of finding the height to which a certain quantity of liquid, or a fixed sectional area will fill a cylindrical culvert, there are two practical modes:—

First. Let a be the area of the wet segment,
 l its perimeter, or arc of the wet segment,
 r the radius of the circle,
 n the angle of the sector,
 h the required height or depth,

$$\text{Then } h = r - k = r \left(1 - \cos \frac{n}{2} \right); \dots \dots \dots \text{(I)}$$

For example.—Let $a = 0.229$; $r = \frac{1}{2}$; $l = 1.231$;

Then by Table of Arcs and Sectors, $n = 141^\circ 0' 22''$
 and $h = \frac{1}{2} (1 - 0.3337) = 0.333$.

Second method. Without using cosines

$$k \cdot \sqrt{r^2 - k^2} = l \times \frac{r}{2} - a;$$

or,
$$k^2 = \frac{r^2}{2} + \sqrt{\frac{r^4}{4} - \left(\frac{l \cdot r}{2} - a\right)^2} \dots \dots \text{(II.)}$$

Applying this to the same example,

$$k^2 = 0.125 + \sqrt{0.15625 - (1.231 \times \frac{1}{4} - 0.229)^2} = 0.02793,$$

$$k = 0.1671; \text{ and the required depth } h = r - k = 0.333.$$

It will be noticed that in either case the length of the arc is assumed; should this not have been previously determined, the height can only be obtained from values of a and r through the tedious process of solving an equation of a high degree. Thus, the formula for the approximate area of a segment is

$$a = \frac{4h^{\frac{3}{2}}}{15} (2\sqrt{4d-3h} + \sqrt{d}); \text{ where } d \text{ is the diameter}$$

Putting $x = \frac{h}{d}$; this becomes $x^{\frac{3}{2}}(2\sqrt{4-3x} + 1) = \frac{15a}{4d^2}$.

And putting $y = x^{\frac{1}{2}} = \left(\frac{h}{d}\right)^{\frac{1}{2}}$; $y^8 - \frac{5}{4}y^6 - \frac{5a}{8d^2}y^3 + \frac{75a^2}{64d^4} = 0$

Numerical examples can be solved with this formula by Horner's method, or more readily by the aid of the dual-logarithms of Mr. Oliver Byrne; modes not very well suited to the daily wants of professional men; nor is there any necessity for adopting this method, as the length of the arc must be obtained to calculate the hydraulic radius; and in that case either of the two more practical methods above exemplified affords a more rapid solution.

7. THE HYDRAULIC SLOPE.

The hydraulic slope, inclination, or declivity, sometimes termed the gradient, is an important function in velocities and discharges in open channels and unfilled culverts, even including those just filled. When applied to liquid flowing under gravity free from pressure, the hydraulic slope in any unit of length is the ratio of the difference of level of the water surface in that length to that length, or is the sine of the slope of the water surface. Thus, if the difference of level in 1 000 feet along the central fillet of the water surface be 2 feet, then,

$$S = \frac{H}{L} = \frac{2}{1000} = 0.002; \text{ and it is in this form that the}$$

inclination is most conveniently introduced in equations and calculations of flow in open channels.

It should be noted that the fall of the bed of a river or canal is not necessarily any function of the velocity, expressed by the value S . The bed may perchance be uniform in regular fall, and also exactly parallel to the water surface for some distance, or it may be otherwise, or highly irregular. When parallel, the fall of the bed happens to be represented by S ; when otherwise, the longitudinal irregularity is comprised in the term n , the combined coefficient for roughness and irregularity.

The slightest variation in S having so important an effect on the mean velocity, its value in cases of channels and rivers of slight inclination should be determined by exact levelling operations on both banks between accurate gauge-levels and carefully verified.

In canals and culverts.

In designs of canals for irrigation, water supply, or drainage, the hydraulic slope is generally also the inclination of the bed, and this is determined to suit the limiting velocities allowed in the canal, the maximum being that nearly producing erosion, the minimum one that just deposits sediment. When such canals exist not only in design, but in operation, the actual hydraulic slope must be obtained by observation.

In navigable canals the conditions are sometimes similar, though more often, as the canal may consist of several still-water reaches, a hydraulic slope does not exist or is exceedingly slight.

In culverts and drain-pipes in their ordinary state not under pressure, the hydraulic slope exists as in open canals; the inclination of the bed or invert, arranged in accordance with local conditions and available outfall, being generally nearly parallel to it.

When a culvert is blocked, a low head of pressure may accumulate; the case then becomes one of discharge under pressure, corresponding to that of water-pipes.

In water-pipes.

In pipes under considerable pressure, such as water-pipes under a statical head of 50 feet or more, the term hydraulic slope is not strictly applicable to any actual or theoretical inclination, but is used for the theoretic inclination from the point where the pressure is zero to any point of discharge under consideration.

The discharge and also the velocity at any point in a continuous series of pipes under pressure are those

due to the statical head, or difference of level between water surface in the reservoir, or top of the stand-pipe as the case may be, and the point under consideration ; the section at the point of actual severance and discharge may be treated as an orifice under direct head, and the velocity calculated as that due to the head and section less all allowances for friction, bends, and contractions along the whole course of the water from its highest point. All such causes of loss of velocity are represented by the effects that would be produced by corresponding loss of head of pressure. The length of the line of pipes and the sources of friction and retardation are here the important factors in the calculation. Table IX. is given to assist in obtaining such losses.

Water-pipes are irregular in their courses and inclinations ; they are usually placed two or three feet below ground, sometimes following its sinuosities, to protect them from frost and damage, and are rarely allowed to rise above their mean inclination : should they do so, a great loss of head results, unless air vessels are applied at those points, from which the air is allowed to escape through cocks every two or three days. Under such irregular conditions, it becomes difficult to estimate the loss of head due to friction with much accuracy.

The other mode of calculating velocities and discharges of water in pipes under pressure is to treat them in accordance with imaginary hydraulic slopes or inclinations from the highest water surface to the point under consideration ; and to apply the ordinary formula for flow given at page 32. This method presupposes that the pipes have a single inclination throughout from the highest point of supply, and, even after making allowance, can only yield an approximate value of the

discharge, even if it arrives at that. It is, however, very commonly adopted.

As it is comparatively rare that a single pipe is laid to any very great distance with a uniform fall, being more generally cut up into lengths having different falls, it becomes necessary to proportion the diameter of the pipe in these different lengths, so that the discharge may everywhere be that due to the smallest diameter. When with such a series of pipes of different diameters the total head is given, and the discharge is required, the case does not admit of direct solution, as each pipe must have its own proper head ; in this case it is best to assume a discharge, and obtain separate heads due to it for each pipe in the series ; the true heads, both total and separate, may be then obtained by proportion, and the inclinations of each pipe, as well as the mean inclination for the whole series (which is the inclination that would be adopted for a single uniform pipe throughout) marked on the section of the design. The final discharge can then be calculated from any one of the pipes. An example of this is attached to Working Table, No. VIII.

8. THE DISTRIBUTION OF VELOCITY IN SECTIONS OF PIPES AND CHANNELS.

The laws of distribution of velocity in the section of an open channel, canal, or river, are still incomplete. The most valuable information on this subject, quoted in the remainder of this section, is that deduced by d'Arcy and Bazin, by Captain Allan Cunningham and by Humphreys and Abbot, from the results of their extensive experiments and investigations.

A certain amount of knowledge has been deduced from observation of the variation of velocity in open channels in the vertical planes, but as regards that in the horizontal planes at a section, nothing has absolutely—and very little relatively—yet been determined. In full cylindrical pipes, on the contrary, the conditions of velocity are comparatively simple.

In full pipes.

The experiments of d'Arcy, in 1851, established the law of velocity in full pipes expressed in the following equation suited to metric measures—

$$\frac{V-v}{\sqrt{RS}} = 11.3 \left(\frac{r}{R} \right)^{\frac{3}{2}}$$

$$\text{or } R(V-v) = 11.3 \cdot r \cdot \sqrt{rS},$$

where V = central velocity.

v = the velocity anywhere at a distance r from the centre.

R = the radius of the pipe.

S = the loss of head per linear metre or hydraulic slope.

This formula was deduced by d'Arcy from observations taken at from one-third to two-thirds of the radii of various pipes from the centre; beyond $\frac{2}{3}$ of the radius, it is probable that the law does not hold good, and that the decrement of velocity should be more rapid than that indicated by the formula. Under any circumstances, however, it is clearly established that the velocities in a full cylindrical pipe are equal at all points equidistant from the centre, and that the above law of decrement holds good for the central $\frac{2}{3}$ of the diameter taken in any direction. In a pipe of rectangular section, the velocities are equal at any four points, taken sym-

metrically with reference to the centre of figure in a corresponding manner.

In small artificial channels.

In open channels, however, this almost mathematical symmetry is entirely absent, and the perturbation produced near the surface of the water does not allow any hope that a formula can be arrived at, which would give the actual velocity at any point in terms of the mean velocity and the co-ordinates determining the position of that point. These perturbations appear to be more considerable in proportion to the diminution of velocity and the increase of depth of channel, and are coincident with a depression of the locus of maximum velocity ; in extreme cases, the curves of equal velocity in the section cut the surface of the water very obliquely.

The following are the conclusions drawn by Bazin on this subject :—

1st. For a very wide rectangular channel—

$$\frac{V_s - v}{\sqrt{HS}} = K \left(\frac{h}{H} \right)^2,$$

where V_s = central velocity at the surface.

v = velocity at a point at a depth h below it.

H = total depth of water.

S = hydraulic slope of the water surface.

This law of velocity is proved to hold good for very wide channels ; the cases under experiment give a practically constant value of $K = 20.0$, the extremes varying between 15.2 and 24.9 ;—it would also appear that for a rectangular canal of infinite width, in which the influence of the sides is made to disappear entirely, K would = 24.0 ;—the units are metric as before.

When, however, the depth of a rectangular channel is great enough in proportion to the breadth to make the influence of the lateral walls show itself in the middle of the current, this law does not hold, nor does any law of decrement of velocity seem possible, and incomplete generalisations, in terms of the mean velocity, can alone be arrived at.

If, then, V_m = the mean velocity in a canal, the section of which is very great in proportion to its depth—and V_s = central velocity at the surface, the other symbols being used as before,

$$\begin{aligned} V_m &= \frac{1}{H} \int_0^h \left[V_s - K \left(\frac{h}{H} \right)^2 \sqrt{RS} \right] dh \\ &= V_s - \frac{K}{3} \sqrt{RS} \end{aligned}$$

and the depth h below the surface is determined by the expression $\left(\frac{h}{H} \right)^2 = \frac{1}{3}$; whence $h = 0.577 H$, which is, in fact, saying that the mean velocity is found at about $\frac{3}{5}$ of the total depth. This, however, assumes the before-mentioned parabolic law of the decrease of velocity in each vertical plane, an hypothesis only admissible in a very large and perfectly regular canal.

In fact, however, and from experiments quoted, it appears that the locus of mean velocity is often below $\frac{3}{5}$ of the depth, and more often below $\frac{3}{4}$ of it; and that when the depth of the canal is great, and the velocity feeble, the curve of mean velocity approaches still nearer the bottom, and goes as low as $\frac{4}{5}$ of the depth.

Taking the above relation $V_m = V_s - \frac{1}{3} K \sqrt{RS}$, where $\sqrt{RS} = V_m \sqrt{A}$, and $K = 24.0$, for a channel of infinite width; in this case also we get $V_s = V_m(1 + 8 \sqrt{A})$ as a

result applicable to this special case, which supposes the parabolic law applicable throughout the whole breadth of the channel ; and this differs greatly from the results of the experiments on such channels, which give $V_s = V_m (1 + 14 \sqrt{A})$.

The locus of maximum velocity is, however, not always at the centre of the surface, but is at a greater depth in proportion as the depth of the canal is greater and the mean velocity is less, being sometimes as low as $\frac{2}{5}$ the total depth.

The determination of bottom (V) velocity can, in rectangular canals, be alone made in the special case of one supposed to be of infinite breadth ; for this case, putting $h = H$ in the original formula, we obtain the velocity $V_b = V_s - K \sqrt{RS}$; but in all other cases no law can be given. The greatest of bottom velocities is in the middle and the least at the sides.

The velocity along the vertical sides of a rectangular canal is generally greater in the middle than at the top or at the bottom ; but beyond this fact, the determination of the exact velocity at any point of the side remains a very difficult problem yet unsolved.

The laws of velocity in canals of semicircular section are far less complicated than those of rectangular section :—the law of decrement of velocity is expressed in the following formula :—

$$\frac{V_s - v}{\sqrt{RS}} = 21 \left(\frac{r}{R} \right)^3,$$

the extreme values of the coefficient deduced from experiment being 18.2 and 23.2 ; and the terms of the expression being similar to those in the equation for decrement of velocity in sections of pipes before men-

tioned:—If in this we make $r=R$, we obtain, as for rectangular channels, the bottom velocity,

$$V_b = V - 21 \sqrt{RS}.$$

And the mean velocity will be deduced thus:—

$$\begin{aligned} V_m &= \frac{1}{\pi R^2} \int_0^R \left[V - K \sqrt{RS} \left(\frac{r}{R} \right)^3 \right] 2\pi r \cdot dr \\ &= V_s - \frac{2}{5} K \sqrt{RS}; \text{ where } \sqrt{RS} = V_m \sqrt{2A}; \end{aligned}$$

hence $\frac{V_m}{V_s} = 1 + \frac{2}{5} K \sqrt{2A}$; where $K=21$

$$= 1 + 11.9 \sqrt{A}:$$

an equation differing but little from that deduced from the experiments on such semicircular canals.

The radius r_s of the circle of mean velocity of the section $= R \cdot \sqrt[3]{\frac{2}{5}} = 0.737R$;—which is saying that this is at about three-quarters of the radius from the centre, whereas in fact it is farther.

Taking finally the two expressions for decrement of velocity in canals of rectangular and semicircular section,

$$\frac{V_s - v}{\sqrt{HS}} = K \left(\frac{h}{H} \right)^2; \text{ and } \frac{V_s - v}{\sqrt{RS}} = K \left(\frac{r}{R} \right)^3;$$

a general expression may be deduced from them,

$$V_s - v = \phi \sqrt{RS};$$

and as under these circumstances absolute velocities cannot be dealt with, it is better to make use of relative velocities, and by dividing each side of the general equation by V_m to transform it into the form

$$\frac{V_s - v}{V_m} = \phi \sqrt{A}; \text{ which is therefore true for all canals}$$

where ϕ is a function of the relative (not of the absolute) co-ordinates determining the position of the point

whose velocity is under consideration, their values being taken in proportion to the dimensions of the section.

With regard to velocities in artificial channels generally, by far the most important result arrived at by D'Arcy and Bazin is the relation between the maximum velocity and the mean velocity of discharge, represented by this equation, suitable to mètres :

$$\frac{V_x}{V_m} = 1 + 14 \sqrt{A'}; \text{ and since } A = \frac{RS}{V^2}; V_x - V_m = 14 \sqrt{RS}$$

these equations reduced to English measures become

$$\frac{V_x}{V_m} = 1 + \frac{25.34}{c \times 100}; \text{ and } V_x - V_m = 25.34 \sqrt{RS}.$$

The advantage in gauging derived from the application of this principle is very great; but the coefficients of reduction are doubtful in exactitude, as shown by Captain Cunningham's recent experiments on a large scale, and are certainly not suited to general application.

In large natural channels.

The laws of variation of velocity in horizontal planes with reference to different forms of section have not yet been satisfactorily deduced, such velocities have therefore to be determined locally when required; the horizontal curves of velocity again vary much in different stages of the river or stream under consideration; the records therefore of such velocities involve much labour, and have not yet shown themselves of sufficient practical importance to repay the labour and trouble of their observation.

As to the variation of velocity in vertical planes, the following is the deduction of Bazin ('Annales des Ponts et Chaussées,' Sept. 1875, pages 309 to 351):—

The velocities of a current at different points on the same vertical line vary as the ordinates of a parabola ; thus, if D be the total depth,

u the velocity at any depth d below the surface,

U the maximum velocity at any depth d' ,

$$u = U - M \left(\frac{d - d'}{D} \right)^2,$$

where M is a quantity dependent on d' .

And if u_m = the mean velocity on the vertical line

$$\begin{aligned} u_m &= \int_0^1 \left\{ U - M \left(\frac{d}{D} - \frac{d'}{D} \right)^2 \right\} dx ; \text{ where } x = \frac{d}{D} \\ &= U - M \left[\frac{1}{3} - \frac{d'}{D} + \left(\frac{d'}{D} \right)^2 \right] ; \text{ where } M = 20 \sqrt{DI} \\ &= U - 20 \sqrt{DI} \left(\frac{d}{D} \right)^2 ; \text{ when } d' = 0, \text{ or the maxi-} \\ &\quad \text{mum velocity is at the surface.} \end{aligned}$$

Or in this case, the parabola has the equation $y = 20x^2$

where y the ordinate $\frac{U - u}{\sqrt{DI}}$, and $x = \frac{d}{D}$.

But when the maximum velocity is below the surface a different value is given to M , and the equation then becomes

$$u = U - 20 \sqrt{DI} \left(\frac{x - a}{1 - a} \right)^2$$

$$\text{where } x = \frac{d}{D}, \text{ and } a = \frac{d'}{D}$$

$$\text{and } \frac{u}{u_m} = \frac{U}{u_m} - 20 \sqrt{A} \left(\frac{x - a}{1 - a} \right)^2$$

where U is the mean velocity (V_m) of the whole section.

If then this new value of M is introduced into the general equation above given,

$$u_m = U - M \left(\frac{1}{3} - a + a^2 \right)$$

it becomes
$$\frac{V_m}{u_m} = 1 + 20 \sqrt{A} \left(\frac{\frac{1}{3} - a + a^2}{(1-a)^2} \right)$$

In experiments on regular conduits 6·5 feet wide the value of $\frac{V_m}{u_m}$ varied between 1·09 and 1·19 ; and in others on the Saône, Seine, Garonne, and Rhine, the value varied between 1·1 and 1·3 : the experiments of Humphreys and Abbot on the Mississippi correspondingly give a value of 1·02.

These results are hence both theoretically and practically correct and useful, and generally applicable even on a large scale.

In very large natural channels.

The laws of variation of velocity in vertical planes of very large natural channels have been also fully investigated by Captains Humphreys and Abbot on the great Mississippi Survey.

From their experimental data it has been deduced that the velocities at different depths below the surface in a vertical plane, vary as the abscissæ of a parabola, whose axis is parallel to the water-surface, and may be considerably below it, thus proving the maximum velocity to be generally below the surface ; the equation of this curve with reference to its axis, taking the depths, relatively to the total depth, as ordinates, was obtained in the form—

$$y^2 = 1\cdot2621 D^2 x$$

where D = total depth of bed below the surface, and x and y are the co-ordinates to the axis.

They also deduced that if d_1 is the depth of the axis of the parabola, or locus of maximum velocity from the surface, then

$$d_1 = (0.317 + 0.06 f) R$$

where R = hydraulic mean radius, and f = force of wind either positive or negative, and taken = 1 when the velocity of the wind and current are equal, and = 0 for a cross wind or calm.

The following are other important equations, with regard to velocity in vertical planes, deduced by Captains Humphreys and Abbot.

(For symbols refer to page 12, Chapter I.)

Formulae for velocity in any vertical plane :

$$(1) \quad b = \frac{1.69}{(D + 1.5)^{\frac{1}{2}}} = 0.1856 ; \text{ only when } D > 30 \text{ feet,}$$

$$(2) \quad d_1 = (0.317 + 0.06 f) D ; \text{ very nearly,}$$

$$(3) \quad V = V d_1 - (bv)^{\frac{1}{2}} \left(\frac{d - d_1}{D} \right)^2,$$

$$(4) \quad V_o = V d_1 - (bv)^{\frac{1}{2}} \left(\frac{d_1}{D} \right)^2,$$

$$(5) \quad V_D = V d_1 - (bv)^{\frac{1}{2}} \left(1 - \frac{d_1}{D} \right),$$

$$(6) \quad V_m = \frac{2}{3} V d_1 + \frac{1}{3} V_D + \frac{d_1}{D} \left(\frac{1}{3} V_o - \frac{1}{3} V_D \right),$$

$$(7) \quad V_{1D} = V_m + \frac{1}{1.2} (bv)^{\frac{1}{2}},$$

$$(8) \quad V_{d_1} = V_m + (bv)^{\frac{1}{2}} \left(\frac{1}{3} + \frac{d_1 (d_1 - D)}{D^2} \right),$$

$$(9) \quad V = V_m + (bv) \left(\frac{D \left(\frac{1}{3} D - d_1 \right) + (2d_1 - d)}{D^2} \right),$$

in which equation (9) is a mere combination of equations (3) and (8).

For velocity in the mean of all vertical planes the following have been deduced :

$$(1) \quad b = \frac{1.69}{(r + 1.5)^{\frac{1}{2}}}.$$

$$(2) \quad d_1 = (0.317 + 0.06 f)r.$$

$$(3) \quad U_m = 0.93v.$$

$$(4) \quad U = 0.93v + \left(\frac{dr(0.634 + 0.12f) - d^2}{r^2} - 0.06f + 0.016 \right) (bv)^{\frac{1}{2}}$$

$$(5) \quad U_o = 0.93v + (0.016 - 0.06f) (bv)^{\frac{1}{2}}.$$

$$(6) \quad U_r = 0.93v(0.06f - 0.35) (bv)^{\frac{1}{2}}.$$

$$(7) \quad U_{d_1} = 0.93v + \{ [0.317 + 0.06f]^2 - 0.06f + 0.016 \} (bv)^{\frac{1}{2}}.$$

$$(8) \quad v = ([1.08 U_{ir} + 0.002b]^{\frac{1}{2}} - 0.045b^{\frac{1}{2}})^2.$$

The most important result of all these data and deductions is the following, a fact of great practical use in gauging rivers, that the ratio of the mid-depth to the mean velocity in any vertical plane is independent of the width and depth of the stream (except for an almost inappreciably small effect) absolutely independent of the depth of the axis of the curve before referred to, and nearly independent of the mean velocity. The formula expressing this is

$$(7) \quad V_{id} = V_m + \frac{(bv)^{\frac{1}{2}}}{12};$$

where V_m is the mean velocity on any curve in the vertical plane.

V_{id} is the mid-depth velocity.

v is the mean velocity of the river.

D is the depth of the river at the spot.

$$b = \frac{1.69}{(D + 1.5)^{\frac{1}{2}}} \text{ generally ; and } = 0.1856, \text{ when } D \geq 30 \text{ feet}$$

The application of this result to gauging is shown in Chapter II. on Field Operations.

Verticalic Velocity generally.

The following are Captain Cunningham's deductions resulting from a thorough investigation of the subject in connection with his observations on large canals.

Parabolic Formulæ.—It seems natural to inquire, first whether the mean velocity past a vertical cannot be found from velocity-measurements at only two or three points on that vertical. And here considerable aid may be derived from study of the velocity-parabola. Whether the vertical velocity-curve be really a common parabola or not matters little: it must be admitted that it does certainly approximate to a parabola. This approximation is quite sufficient to admit of its use in determining an approximate value of mean velocity.

And first, it is clear that, as three data suffice to determine the velocity-parabola completely, velocity-measurements at three distinct points on the same vertical will of course suffice to determine the mean velocity.

[The three points must of course be suitably situated to give a tolerably accurate determination.]

The first step is to find an expression for the mean velocity. Adopting the well-known property—

Area of parabola between tangent and diameter = $\frac{1}{3} \times$ circumscribing rectangle, (1), it follows that, the lamina of discharge D , passing by a vertical axis or depth H , is equal to the inclusive rectangle less the sum of the parabolic areas above and below the axis,

$$\text{or } D = VH - \frac{1}{3}(V - v_o) \cdot Z - \frac{1}{3}(V - v_H) \cdot (H - Z). \quad (2)$$

where V is the maximum vertical velocity, v_0 is the surface velocity, v_H the bed-velocity, Z is the depth at which V exists, z that of v .

Writing the equation of the curve in the form

$$V - v = m(z - Z)^2, \text{ where } m = \frac{1}{p}, \text{ and } p = \text{parameter.} \quad (3)$$

and writing $z = 0$, $z = H$ in succession therein (so that v becomes v_0 and v_H)

$$V - v_0 = mZ^2, \text{ and } V - v_H = m(H - Z)^2, \quad (4).$$

Substituting these into the expression (2)

$$\begin{aligned} D &= VH - \frac{1}{3} \{mZ^3 + m(H - Z)^3\} \\ &= VH - \frac{1}{3} mH^3 + mH^2Z - mHZ^2, \quad (5). \end{aligned}$$

$$\begin{aligned} \therefore \text{mean velocity } U &= \frac{D}{H} = (V - mZ^2) + mHZ - \frac{1}{3}mH^2, \\ &= v_0 + mHZ - \frac{1}{3}mH^2, \quad (6), \end{aligned}$$

by substituting from (4). This is the working expression for U , with which other values obtained in terms of observed velocities are to be compared.

Three-velocity Formulæ.—Now let three velocity-measurements $v_{\lambda H}$, $v_{\mu H}$, $v_{\nu H}$ be taken at *any* depths λH , μH , νH , (where λ , μ , ν are proper fractions,) and let it be proposed to find an expression for the mean velocity in terms of these; let this be—

$$U = \alpha \cdot v_{\lambda H} + \beta \cdot v_{\mu H} + \gamma \cdot v_{\nu H}, \quad (7),$$

where α , β , γ are numerical coefficients to be determined.

Subtracting (3) from (4), there results the following general expression for v :—

$$v = v_0 + 2mZz - mz^2, \quad (8)$$

Writing $z = \lambda H, \mu H, \nu H$ in succession, this gives—

$$v_{\lambda H} = v_0 + 2mZ \cdot \lambda H - m\lambda^2 H^2, v_{\mu H} = v_0 + 2mZ \cdot \mu H - m\mu^2 H^2, \\ \text{and} \quad v_{\nu H} = v_0 + 2mZ \cdot \nu H - m\nu^2 H^2, \quad (9).$$

Multiplying by a, β, γ in succession, and adding it follows from (7) that—

$$U = (a + \beta + \gamma) \cdot v_0 + 2mHZ (a\lambda + \beta\mu + \gamma\nu) - mH^2 (a\lambda^2 + \beta\mu^2 + \gamma\nu^2) \quad (10).$$

This expression becomes identical with (6) by making—

$$a + \beta + \gamma = 1; a\lambda + \beta\mu + \gamma\nu = \frac{1}{2}; a\lambda^2 + \beta\mu^2 + \gamma\nu^2 = \frac{1}{3}; \quad (11).$$

These being simple equations in a, β, γ suffice to determine a, β, γ in terms of λ, μ, ν whatever values these may have. The general solution is not of much practical use: the most useful particular solutions appear to be when the three velocity-measurements are made at mid-depth ($\mu H = \frac{1}{2}H$) and at two points equidistant from mid-depth (in which case $\lambda H + \nu H = H$), so that—

$$\mu = \frac{1}{2}; \lambda + \nu = 1, \quad (12).$$

which reduce (11) to—

$$a + \beta + \gamma = 1; a\lambda + \frac{1}{2}\beta + \gamma\nu = \frac{1}{2}; a\lambda^2 + \frac{1}{4}\beta + \gamma\nu^2 = \frac{1}{3}; \quad (13).$$

Multiplying the last two by 2 and by 4 respectively, and subtracting in turn from the first,—



$$a(1-2\lambda) + \gamma(1-2\nu) = 0; \quad a(1-4\lambda^2) + \gamma(1-4\nu^2) = -\frac{1}{3}; \quad (14).$$

Substituting $\lambda + \nu$ for 1 into the former,—

$$(a-\gamma)(\nu-\lambda) = 0; \quad \text{whence } a = \gamma \text{ (as } \nu, \lambda \text{ are supposed unequal),} \quad (15).$$

And from the latter, $2a \cdot \{1 - 2(\lambda^2 + \nu^2)\}$, or $2a \{(\lambda + \nu)^2 - 2(\lambda^2 + \nu^2)\} = -\frac{1}{3}$

$$\text{whence, } a = \gamma = \frac{1}{6(\lambda - \nu)^2}, \quad \text{or} = \frac{1}{6(2\lambda - 1)^2}. \quad (16a).$$

$$\beta = 1 - 2a = 1 - \frac{1}{3(\lambda - \nu)^2}, \quad \text{or} = 1 - \frac{1}{3(2\lambda - 1)^2}. \quad (16b).$$

Hence by assigning simple values $0, \frac{1}{6}, \frac{1}{4}, \frac{1}{3}$ to λ , the following simple cases result,

$$U = \frac{1}{6}(v_0 + 4v_{\frac{1}{2}H} + v_H), \quad \text{or} = \frac{1}{8}(3v_{\frac{1}{6}H} + 2v_{\frac{1}{2}H} + 3v_{\frac{5}{6}H}), \quad (17a).$$

$$= \frac{1}{3}(2v_{\frac{1}{4}H} - v_{\frac{1}{2}H} + 2v_{\frac{3}{4}H}), \quad \text{or} = \frac{1}{2}(3v_{\frac{1}{3}H} - 4v_{\frac{1}{2}H} + 3v_{\frac{2}{3}H}), \quad (17b).$$

The first will be recognised as Simson's well-known formula, that is of no use for practical determination of U , as it involves the bed-velocity which does not admit of direct measurement. The other three give simple values, easily applicable to practical velocity-measurement.

Two-velocity Formulæ.—There being only three equations (11) connecting the six quantities $a, \beta, \gamma, \lambda, \mu, \nu$, it seems worth while to inquire whether an expression could be found for the mean velocity involving velocity-measurements at only two (instead of three) distinct points, as this would materially reduce the field-work necessary to find the mean velocity.

It is sought then to determine a, β, λ, μ , so as to determine U by the simpler formula—

$$U = \alpha v_{\lambda H} + \beta v^{\mu H}, \quad . \quad . \quad . \quad (18).$$

Either by a similar investigation to the preceding, or by simply writing $\gamma = 0$ in the previous Result (11), the equations connecting α, β, γ are seen to be

$$\alpha + \beta = 1, \quad \alpha\lambda + \beta\mu = \frac{1}{2}, \quad \alpha\lambda^2 + \beta\mu^2 = \frac{1}{3}, \quad . \quad (19).$$

from which it is clear that λ, μ are no longer independent; for, solving for α, β in the two first,

$$\alpha = \frac{\mu - \frac{1}{2}}{\mu - \lambda}, \quad \beta = \frac{\frac{1}{2} - \lambda}{\mu - \lambda}, \quad . \quad . \quad (20),$$

And from the third, $\frac{1}{2}\lambda^2 - \mu\lambda^2 + \mu^2\lambda - \frac{1}{2}\mu^2 = \frac{1}{3}(\lambda - \mu)$, the following equation is obtained by substitution, and dividing by $(\lambda - \mu)$, (which is always possible, since λ, μ must be unequal)—

$$\lambda\mu - \frac{1}{2}(\lambda + \mu) + \frac{1}{3} = 0, \quad . \quad . \quad (21),$$

which is the equation connecting λ, μ , from which in fact

$$\lambda = \frac{\frac{1}{2}\mu - \frac{1}{3}}{\mu - \frac{1}{2}}, \quad \text{or} \quad \mu = \frac{\frac{1}{3} - \frac{1}{2}\lambda}{\frac{1}{2} - \lambda}, \quad . \quad . \quad (22),$$

so that either is determined in terms of the other.

Thus the mean velocity (U) may be found from velocity-measurements at *only two* distinct depths $\lambda H, \mu H$ —whereof one is arbitrary, and the other is determined by (22)—by the simple formula (18), wherein α, β are given by (20).

Hence by making $\lambda = 0, \frac{1}{6}, \frac{1}{4}, \frac{1}{3}$, the following simple cases result,

$$U = \frac{1}{4}(v_0 + 3v_{\frac{2}{3}H}), \quad \text{or} = \frac{1}{7}(3v_{\frac{1}{6}H} + 4v_{\frac{1}{3}H}), \quad . \quad (23a).$$

$$U = \frac{1}{7}(4v_{\frac{1}{4}H} + 3v_{\frac{2}{3}H}), \quad \text{or} = \frac{1}{4}(3v_{\frac{1}{3}H} + v_H), \quad . \quad (23b).$$

These are the simplest formulæ by which the mean

velocity past a vertical can be determined from velocity-measurements at only two distinct points.

The first of the formulæ (23a), above¹ is by far the best for general purposes, because it involves only one sub-surface velocity ($v_{\frac{2}{3}H}$), and that at the highest possible level ($\frac{2}{3}H$), and therefore admitting of more accuracy in its determination than those at lower levels involved in the other formulæ. The last is of no practical use, as it involves v_H , a quantity which cannot be practically measured.

[It is not difficult to show that the two velocity-measurements must always lie one in the upper third, and one in the lower third of the depth, *i.e.*, λ lies between 0, $\frac{1}{3}$, and μ between $\frac{2}{3}$ and 1.]

Test of Formula.—Denoting for distinctness' sake the value of mean velocity derived from the above simple formula (first of 23a), by u_m , it is written thus,

$$u_m = \frac{1}{4} (v_0 + 3v_{\frac{2}{3}H}), \quad . \quad . \quad (23a, bis).$$

The value of this quantity has been calculated for all the 46 average vertical curves of the Roorkee Experiments, and is shown there in the sub-column headed u_m in Abstr. Tab. 3, 4 for comparison with the fundamental value $U = D \div H$. To facilitate this, the discrepancy ($u_m - U$) is also shown. These discrepancies will be seen to be always small (nowhere exceeding 0.07) as might be expected, and usually negative, showing that $u_m < U$ usually.

The closeness of the values of v_m , U is involved, of course, in the general approximation of the observation-curves to parabolæ.

¹ Published for the first time, it is believed, by Capt. Cunningham.

Depth of Mean Velocity-Line.—By the term ‘Line of mean velocity’ is here meant the stream-line in which the average forward velocity is equal to the average mean velocity past the vertical. To find the depth (h_0) of that line, the equation of the curve (18) gives (writing $z=h_0$, and $v=U$)—

$$U = v_0 + 2mZh_0 - mh_0^2, \quad . \quad . \quad . \quad (24a).$$

$$= v_0 + mZH - \frac{1}{3}mH^2, \text{ by Result (16), } (24b).$$

$$\text{Hence } h_0^2 - 2Zh_0 = \frac{1}{3}H^2 - ZH,$$

$$\text{whence } h_0 = Z \pm \sqrt{\frac{1}{3}H^2 - ZH + Z^2}, \quad . \quad . \quad . \quad (25),$$

$$\text{and } \frac{h_0}{H} = \frac{Z}{H} \pm \sqrt{\frac{1}{3} - \frac{Z}{H} + \left(\frac{Z}{H}\right)^2}, \quad . \quad . \quad . \quad (25a),$$

The quadratic in h_0 has of course two roots: but it is easily seen by writing (25) in form—

$$h_0 = Z \pm \sqrt{H\left(\frac{1}{3}H - Z\right) + Z^2}, \quad . \quad . \quad . \quad (25b),$$

that one root is always negative when $Z < \frac{1}{3}H$, and is therefore of no ¹ interest; when $Z > \frac{1}{3}H$, both roots are +, which shows that there are in this case two lines of mean velocity equidistant from the axis (as is evident from the symmetry of the parabola). It may be shown also that the larger root is always greater than $\frac{1}{2}H$, for writing the larger root of (25) in form—

$$h_0 = Z + \sqrt{\left(\frac{1}{2}H - Z\right)^2 + \frac{1}{12}H^2}, \quad . \quad . \quad . \quad (25c),$$

so that

$$h_0 = Z + \text{a quantity} > \frac{1}{2}(H - Z), \text{ whence } h_0 > \frac{1}{2}H, \quad (25a),$$

which shows that—

¹ As this would correspond to a line *above the surface*.

‘The mean velocity Line is always below the mid-depth,’ (26).

In the illustration of this by diagrams of observed velocities, it is seen that the vertical line drawn through the tip of the mean velocity ordinate (U) cuts the observation-curves below the mid-depth in almost all cases.

It is evident that the depth of the mean velocity-line (defined by h_0) depends on the position of the maximum velocity line (defined by Z), and varies therefore with the variation of the latter; also from (25a) it follows that :—

‘The relative depth of the mean velocity line ($h_0 \div H$) depends solely on the relative depth of the maximum velocity line ($Z \div H$),’ (27a).

The range of the maximum velocity line appears in the same diagrams to be from a little above the surface down to about mid-depth. The values of h_0 corresponding to various values of Z within this range are shown below.

| | | | | | | | | |
|-------------------------|------------------|------------------|-------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Value of $Z \div H$, | $-\frac{1}{4}$, | $-\frac{1}{8}$, | 0, | $\frac{1}{8}$, | $\frac{1}{6}$, | $\frac{1}{4}$, | $\frac{1}{3}$, | $\frac{1}{2}$, |
| Value of $h_0 \div H$, | .554, | .560, | .577, | .598, | .607, | .632, | 0 & .667, | .211 & .789. |

whence it follows that—

‘The mean velocity past a vertical cannot be directly measured in practice by any single velocity-measurement,’ (27b),

as the single measurement would be required in the mean velocity line, a line whose position is not known *a priori*.

Again, taking the larger root of (25) (which is the one of most interest), viz.,

$$h_o = Z + \sqrt{\left(\frac{1}{3}H - Z\right)H + Z^2}, \quad . \quad . \quad (25 \text{ bis}),$$

it is clear that the surd is $> = < Z$ when $\frac{1}{3}H > = < Z$,

$$\therefore h_o > = < 2Z \text{ when } Z < = > \frac{1}{3}H, \quad . \quad . \quad (28).$$

Now from the symmetry of the curve it is clear that the velocity (v_{2z}) at depth $z=2Z$ is the same as the surface velocity, *i.e.*, $v_z = v_o$.

Hence—

The mean velocity (U) $> = <$ the surface velocity (v_o) when $Z > = < \frac{1}{3}H$, (29).

Single-velocity Approximations.—Writing down the general values of U, v from Eq. (6), (8),

$$v = v_o + 2mZz - mz^2, \quad U = v_o + mZH - \frac{1}{3}mH^2, \quad . \quad (30),$$

it is manifest that there is no value of z (taken as a function of the depth H only) which will make the general value of v either equal to U , or even proportional to U , in consequence of the presence of the variable and unknown Z . The flatness of the velocity-parabolæ is, however, in all cases so great that an approximation is possible. The closeness of this approximation depends on a prior rough knowledge of the range of $Z \div H$. Now a glance down the column (Tab. 3, 4) showing the values of $Z \div H$ in the 45 curves of the Roorkee Experiments will show that the range of this quantity is—except for verticals quite close to the vertical walls of the rectangular channel (*i.e.* for all verticals more than 5' off the walls)—only from about 0 to $\frac{1}{3}$, and for this range of $Z \div H$, the value of $h_o \div H$ has been

already shown to range from '577 to '667; with a mean value of about $0\cdot625 = \frac{5}{8}$.

Now the velocity corresponding to the value $z = \frac{5}{8}H$ is from (30),—

$$v_{\frac{5}{8}H} = v_0 + m \left(\frac{5}{4}ZH - \frac{25}{64}H^2 \right), \quad . \quad . \quad (31)$$

and the difference between this and the mean velocity is—

$$v_{\frac{5}{8}H} - U = \frac{1}{4}m \left(ZH - \frac{11}{48}H^2 \right),$$

which ranges from $-\frac{11}{192}mH^2$, when $Z=0$,

$$\text{to } +\frac{5}{192}mH^2, \text{ when } Z = \frac{1}{3}H. \quad . \quad (31a).$$

In the other case. Near the margin of the rectangular channel the limiting values of the quantity $Z \div H$ are $\frac{1}{3}$ and $\frac{1}{2}$, and the table of values of $h_0 \div H$ already given shows that there are two sets of values of $h_0 \div H$ corresponding, viz., one between 0 and '211, and one between '667 and '789, with mean values of about '105 and '728. The former is the better for practical velocity-measurements on account of the greater accuracy of work near the surface.

Now the velocity corresponding to the value $z = \frac{1}{10}H$ is—

$$v_{\frac{1}{10}H} = v_0 + m \left(\frac{1}{5}ZH - \frac{1}{100}H^2 \right) \quad . \quad . \quad (32),$$

and the difference between this and the mean velocity is—

$$v_{\frac{1}{10}H} - U = \frac{1}{5}m \left\{ -4ZH + \frac{97}{60}H^2 \right\},$$

which ranges from $+\frac{17}{300}mH^2$, when $Z = \frac{1}{3}H$,

$$\text{to } -\frac{23}{300}mH^2, \text{ when } Z = \frac{1}{2}H, \quad . \quad . \quad (32a).$$

Now, in consequence of the flatness of all the curves the quantity m (=reciprocal of parameter) is always a very small quantity ; so that ‘the several discrepancies

$$-\frac{11}{192}mH^2, \frac{5}{192}mH^2, \frac{17}{300}mH^2, -\frac{23}{300}mH^2,$$

just shown are always very small quantities,’ . (33),
and :

‘The two velocities $v_{\frac{5}{8}H}$ (*i.e.*, at $\frac{5}{8}$ depth) in general, and $v_{\frac{1}{10}H}$ (*i.e.*, at $\frac{1}{10}$ depth) near margin of a rectangular channel are probably the best approximations obtainable from velocity-measurement at a single point,’ . (34).

Mid-depth-velocity, (v_H).—Writing $z = \frac{1}{2}H$ in the general expression (8) for v , the mid-depth-velocity is seen to be,—

$$v_{\frac{1}{2}H} = v_0 + mZH - \frac{1}{4}mH^2, \quad \text{whilst } U = v_0 + mZH - \frac{1}{3}mH^2, \text{ (by (6)),} \quad (35),$$

so that the difference $v_{\frac{1}{2}H} - U = \frac{1}{12}mH^2$ is always a positive quantity . (36).

Thus in the velocity-parabola—

‘The mid-depth-velocity is always $>$ the mean velocity by a small quantity, *viz.*, $\frac{1}{12}mH^2$, not depending on the position of the axis,’ . (36a)

It will be seen also that the discrepancy $\frac{1}{12}mH^2$ is always $>$ the greatest possible discrepancies with the two approximations last proposed.

[The property just proved, *viz.*, that the ‘mid-depth ordinate exceeds the mean ordinate by a small quantity’ is a property in no way peculiar to the parabola. All experiment agrees in showing that as a rule—

‘The average vertical velocity-curves are every-

where convex down-stream; and are always very flat curves.'

These two properties involve the property in question; for in any convex curve whatever the tangent at the point M where the middle ordinate mM meets the curve lies wholly without the curve, so that the curve falls wholly within the circumscribing trapezoid; also the middle ordinate = area of circumscribed trapezoid \div depth; and the mean ordinate = area of curve \div depth (by definition); so that the middle ordinate always $>$ the mean ordinate; also, when the curve is very flat, it is clear that the excess of the former over the latter must be a small quantity.]

This is fully borne out by the Roorkee Experiments: the value of the quantity $(v_{\frac{1}{2}H} - U)$ is given for every series in Abstr. Tab. 3, 4, Col. 9, and it will be seen from them that its value is positive in 40 out of the 46 Series, and zero in 2 more. The only cases in which $v_{\frac{1}{2}H} < U$ are shown in following table:—

| Serial Number | Number of Sets | Value of $(v_{\frac{1}{2}H} - U)$ | Remarks |
|---------------|----------------|-----------------------------------|---|
| 9 | 14 | -07 | Several very low velocities about the mid-depth (i.e., at 4' and 5' depth). |
| 21 | 16 | -01 | |
| 44 | 5 | -11 | These two curves on the exceptional vertical close to the 4' drop-wall are of exceptional shape (not wholly convex), so that the property (47) of a convex curve could not be expected. |
| 45 | 6 | -06 | |

It may hence be concluded that 'the difference $(v_{\frac{1}{2}H} - U)$ is always a small quantity, and usually +, so that $v_{\frac{1}{2}H}$ usually exceeds U ,' (37).

Ratio $U \div v_{\frac{1}{2}H}$.—This ratio has acquired quite exceptional importance of late years from the assertion, at p. 294 of the Mississippi Report, of its approximate constancy under all circumstances at the same site, and the proposal therein to utilise this supposed property in discharge-measurement.

From the result $v_{\frac{1}{2}H} = U + \frac{1}{12}mH^2$, Eq. (36), it is clear that the ratio $U \div v_{\frac{1}{2}H}$ is—in the velocity-parabola at any rate—not a constant quantity (unless mH^2 be proportional to U), nor a function of U only (unless indeed mH^2 be a function of U). The value of the ratio is in fact—

$$\frac{U}{v_{\frac{1}{2}H}} = \frac{U}{U + \frac{1}{12}mH^2} = \frac{1}{1 + \frac{1}{12} \cdot \frac{mH^2}{U}}. \quad (38).$$

Now from the admitted smallness of the quantity $\frac{1}{12}mH^2$ (the same as $v_{\frac{1}{2}H} - U$) it is clear that this ratio will be tolerably constant (< 1 , of course) at any rate as a rough approximation.

The conclusion advanced by the Mississippi Report is that this ratio depends chiefly on the mean velocity (V) of the whole channel, at any rate in a deep channel.

But the argument is based (see Mississippi Report) upon the assumed value for the parameter $\frac{1}{m}$ or $p = H^2 \div \sqrt{\beta V}$, and upon a further *assumed* relation that $U = .93V$ approximately (*i.e.*, with sufficient approximation for the purpose of proving the dependence of the ratio $U \div v_{\frac{1}{2}H}$ on V). Applying these two Results, the ratio $v_{\frac{1}{2}H} \div U$ indeed becomes—

$$v_{\frac{1}{2}H} \div U = 1 + \frac{1}{12 \times .93} \sqrt{\frac{\beta}{V}}, \text{ where } \beta = \frac{1.69}{\sqrt{H + 1.5}}, \quad (39),$$

which depends in deep channels at any rate (in which β varies very little) chiefly on V ; and this result is proposed, at p. 293 of the Mississippi Report, as 'the absolute numerical value of the ratio for any curve of actual observations.'

But the argument is inconclusive on account of the uncertainty (and probable incorrectness as general truths) of the two assumptions $p = H^2 \div \sqrt{\beta V}$ and $U = .93V$ approximately. The assumption $U = .93V$ approximately is obviously not true at all parts of a channel, for it is equivalent to assuming that—

'The mean velocity past a vertical (U) is approximately the same right across a channel,' which is true enough throughout great part of the width, but very far from true regarding velocities near the banks. Thus result (39) is not a general truth, but is at the utmost limited in application to those parts of a cross-section, the mean velocity past the verticals of which is nearly the same.

In fact the real evidence of the proposed law for this ratio must be held to depend, not on the argument which led to it, but, on the numerical comparisons exhibited (Mississippi Report, p. 294) showing—

1st, the values of the ratio $U \div v_{\frac{1}{2}H}$ (computed direct from the velocity-data).

2nd, the values of its proposed equivalent, viz., of

$$1 \div \left(1 + \frac{1}{11.16} \sqrt{\frac{\beta}{V}} \right).$$

3rd, the discrepancies between the above values.

These are shown in the Mississippi Report for 15 cases, viz., 8 Mississippi curves, 2 of Capt. Boileau's curves from small canals, and 5 curves on the Rhine. The

discrepancies shown are certainly surprisingly small in the 8 Mississippi curves, in which they do not exceed $\frac{2}{10}$ per cent. ; whilst in 4 of the European curves they rise to 2 to 3 per cent.

Upon this evidence the important conclusion is drawn (*ib.*) that—

‘The ratio of the mid-depth velocity to the mean velocity in any vertical plane is practically independent of the depth and the width of the stream, of the mean velocity of the river, of the mean velocity of the vertical curve, and of the locus of its maximum velocity. In other words, it is a sensibly constant quantity for practical purposes.’

And upon this conclusion it is proposed that the field-work for computing the total discharge of a large channel should in future be limited to mid-depth velocity-measurements.

The practical value of this conclusion depends chiefly on the amount of error likely to be made in its application. Now the value of the ratio (39) proposed involves unfortunately the unknown quantity V (= mean velocity of the whole channel). If an approximate value of this were known *a priori*, it would give the value of the ratio in question with sufficient approximation.

It was apparently supposed (Mississippi Report) that the ratio in question varied within such small limits *under all circumstances whatever* (even in different channels) that it might be assumed sensibly constant for all practical purposes of discharge-measurement of large channels. The additional evidence now available by no means confirms this hypothesis: the ranges of average values of the ratio in question—*i.e.* of the

average experimental values of $U \div v_{\frac{1}{2}H}$ —are given below from all the known published cases.

| Experiments | Reference to Original | Number of Curves | Range of Average Values of the ratio $U \div v_{\frac{1}{2}H}$ |
|-----------------------------------|---------------------------|------------------|--|
| Mississippi . . . | Miss. Report, p. 294 . | 8 | ·9868 to ·9624 |
| Rhine . . . | ” ” . | 5 | ·9569 to ·9322 ¹ |
| Small Canals, Capt. Boileau . . . | ” ” . | 2 | ·9640 to ·9417 |
| Bazin . . . | Bazin Experiments . | ? | not given |
| Lake Survey . . . | Reports of 1868-70 . | ? | not given |
| Irrawaddi . . . | Report of 1875, Appx. C. | 14? | 1·092 to ·976 |
| Connecticut . . . | Report of 1878, p. 350 | 27? | ·961 to ·918 |
| Roorkee . . . | Roorkee Expts., Tab. 3, 4 | 16 | 1·045 to ·961 |

Thus it appears that—

‘The ratio $U \div v_{\frac{1}{2}H}$ is liable to range from about 1·082 to ·918, *i.e.*, about 16 per cent.’ . . . (40), an amount not fairly negligible even in the rough process of discharge-measurement of large channels.

9. DISCHARGES OF RIVERS.

To determine with accuracy the discharge of any ordinary or large river, independently of velocity-observation, is at present impossible. To this general truth there is only one exception, the case of a long straight and uniform reach of river, whether canalised artificially or naturally; then it may be treated nearly as a canal.

If it be required to determine approximately the discharge of a river from its section, slope, and condition as regards roughness of bed surface and irregularity; the section may be sounded, and the hydraulic slope ascertained by levelling, but the required coefficient (n) of

¹ Printed ·0322 in Mississippi Report.

roughness and irregularity must be guessed by an experienced hydraulician from comparison with other rivers and their coefficients. (See Kutter's local values of n for natural channels in Table XII.) This being done, the value of c may be calculated by the formula or obtained from Table XII., and the calculation of discharge can be effected through the general formula

$$Q = A . V = A . c . 100 \sqrt{RS}.$$

It is obvious that it is preferable to take at least a few velocity-observations. (See Gauging, Chapter II.)

There are also two other theories of flow, or modes of approximating to river-discharges without velocity-observation, that are of some practical value under certain conditions; besides a large number of formulæ whose merits are demonstrated by comparison (in Chapter III., Hydrodynamic Formulæ) to be very inferior.

Of the two former the first is that of Dupuit; it neglects friction on the sides of the section of flow, thus considering motion in all vertical planes to be the same, and dealing with horizontal laminæ only; the surface lamina is considered to be in the condition of a solid gliding over an inclined plane, and each lamina below, except the bottom one, is urged on by its own weight and its cohesion to the upper lamina; the bottom fillet is retarded by its adhesion to the bed. Putting this in the form of an equation, summing, rejecting certain terms, integrating and applying three numerical coefficients, Dupuit obtains a result, which for English feet is—

$$v = \frac{S . R A}{0.08 W} - 0.082 + (0.0067 + 0.9114 RS)^{\frac{1}{2}}.$$

It is this formula that has produced more correct practical results generally than any one of the formulæ having fixed coefficients; next to it, in order of correctness, coming the Chezy formula, with a fixed coefficient $c=1$. This theory assumes that the uppermost lamina moves invariably with the maximum velocity, which is not the case; the neglect of the friction of the banks might not vitiate results if applied to large rivers or shallow channels; it is probable, therefore, that a modification of this formula in accordance with correct data of the relations between maximum and mean velocity, might render it very useful and practical. Hitherto the formula has been generally treated as a pipe-discharge formula, and as a modification of the Chezy type; the theory, however, is one pre-eminently adapted to wide rivers, and the results (see Article in Chapter III., Hydrodynamic Formulæ) are undeniably correct as good approximations. For more information, refer to Dupuit's 'Etude Théorique et Pratique sur le Mouvement des Eaux courantes' (Paris, 1848), and Claudel's Tables, which contain extracts therefrom.

The second theory is that of the Mississippi Survey, mentioned in the Mississippi Report, Philadelphia, 1861, which deduces the new formula, mentioned as giving the most correct results of all yet known; it is, however, unfortunate in its formulæ being rather inconvenient in some respects. While, therefore, the investigation and deduction of the formula is valuable on account of the experimental data applied to it, the result is not practically useful; as the formula was virtually set aside by the Mississippi Survey, whenever careful river-gauging was carried out, in favour of other equations deduced from velocity-observation.

In a work of this scope, it is impossible to go beyond the mere outlines of the demonstration adopted. Adopting the notation of the Mississippi Survey given at pages 11 and 12, it may be stated as follows.

The theory accepts uniform motion and the usually accepted application of the laws of uniform motion, but, in retarding force, denies the stability of position of maximum velocity, and makes allowance for the resistance of the air on the water surface, as well as for the effect of wind.

The process of reasoning pursues the following equations obtained for the forces:—

$$(1.) \quad lGgAS = l(p+w) \phi \frac{U_o W + U_r p}{W+p}$$

dividing both sides by Ggl ,

$$\text{putting } U_o = 0.93v + (0.016 - 0.06f) (bv)^{\frac{1}{2}}$$

$$U_r = 0.93v + (0.06f + 0.35) (bv)^{\frac{1}{2}}$$

(2.)

$$\frac{AS}{W+p} = \phi \left\{ .93v + \frac{(bv)^{\frac{1}{2}} \left(W \left(.333 - \frac{d_1}{r} \right) + p \left(\frac{d_1}{r} - .667 \right) \right)}{W+p} \right\}$$

putting $W = qp$, where q practically = 1 for large rivers.

$$(3.) \quad \frac{AS}{W+p} = \phi (0.93v + .0167 (bv)^{\frac{1}{2}}) = \phi (z) = C z^2.$$

$$(4.) \quad C = \frac{AS}{(p+W) z^2};$$

by practical observation $C = \frac{S}{195}$, hence

$$(5.) \quad z = \left(\frac{195 AS}{p+W} \right)^{\frac{1}{2}}$$

In this equation there are practically only four variables, A , $p + W$, S and z , and for ordinary natural channels p nearly $= 1.015 W$; hence if the values of any three are given, the fourth may be obtained, the transpositions of the equation being—

$$(6.) \quad S = \left(\frac{(p + W)z^2}{195A} \right)^2$$

$$(7.) \quad A = \frac{(p + W)}{195S^{\frac{1}{2}}}$$

$$(8.) \quad p + W = \frac{195 AS}{z^2}.$$

Now z is a variable, of which only two absolute values are known, viz., that for a rectangular cross section, and that for an ordinary river section, which are—

$$z = v + 0.167 b^{\frac{1}{2}} v^{\frac{1}{2}}$$

$$z = 0.93v + 0.167 b^{\frac{1}{2}} v^{\frac{1}{2}}.$$

Substituting these in (5) and solving, we get for rectangular channels

$$(9.) \quad v = \sqrt{0.0064b + (195R_1 S^{\frac{1}{2}} - 0.08b^{\frac{1}{2}})^2}.$$

For ordinary river channels,

$$(10.) \quad v = (\sqrt{0.0081b + (225R_1 S^{\frac{1}{2}} - 0.09b^{\frac{1}{2}})^2});$$

For large rivers, where $R > 12$ feet, and where $b = \frac{1.69}{(R + 1.5)^{\frac{1}{2}}} = 0.1856$, the first term may be neglected, and this latter equation becomes—

$$(11.) \quad v = ([225R_1 S^{\frac{1}{2}}] - 0.388)^2;$$

If the discharge is known, and also two of the four variables in equation (5), provided they are not A and v ,

the other two variables may be computed by eliminating the unknown variable in the second member of that one of the transpositions of equation (11) whose first member is the variable sought, by substituting for it its value deduced from the equation (12).

$$v = \frac{Q}{A}.$$

No difficulty will be found in performing the calculation, except when S and $p + W$ are the known variables, in which case an equation of a higher degree than the second cannot be avoided, and successive approximation must be adopted as follows :—

Assume a value of A , and find two values of v , one from equation (12), the other from (10) or (9), as the case may require ; these values of v will not agree, hence continue assuming new values for A , until the resulting values of v are identical.

The above-mentioned Mississippi formulæ apply only to the discharges of very large rivers ; their adoption is not to be recommended in any other cases.

10. BENDS AND OBSTRUCTIONS.

The irregularities of a river materially affect its velocity ; the following remarks on this subject, by Captains Humphreys and Abbot, are instructive on this point.

‘ Even on a perfectly calm day, there is a strong resistance to the motion of the water at the surface, independent of, and not mainly caused by the friction of the air ; the principal cause being the loss of force, arising

‘from the upward currents or transmitted motion caused by the irregularities at the bottom. There is also an almost constant change of velocity at various depths, resulting from the wind in a great measure; and eddies changing their position and magnitude cause variations in the velocity of the river at a given point, and these again are influenced in intensity by the wind.’

Such irregularities are of course beyond calculation; others again may, in some instances, have their results approximated to, and allowances made for them, by considering a certain portion of the head on the stream as neutralised by them; and these are known as bends or obstructions whose effects are within the range of calculation. Generally the disturbing effects of lateral bends and curves, and of shoals and obstructions, constituting vertical bends, as well as alterations of section, cannot be calculated with any practical accuracy. It is, therefore, best entirely to avoid such difficulties; but when this cannot be done, the following formulæ may be used in preference to neglecting the allowance.

The old general formula for loss of head, h_1 due to a bend in a canal, river, or water-pipe, is of very doubtful value; it is

$$h_1 = \frac{c \cdot \sin^2 a \cdot V^2}{\sqrt{R}}$$

where c is an experimental coefficient generally taken at the fixed value 0.5184;

a = the arc of any bend, not exceeding 90° ;

h_1 and R the radius of bend are in feet, and V is in feet per second.

The total loss of head, due to the bends for which allowance is to be made throughout a course, is then the sum of all such values h_1 obtained.

River bends.—A more modern formula suited to rivers is that adopted by the Mississippi Survey, it is—

$$h_f = \frac{V^2 \sin^2 a}{134};$$

where a = angle of incidence of the water in passing round the bend:—it is, however, always assumed that each angle is one of 30° , and the effect is estimated as due to the number N whether integral or fractional of such bends or deflections of 30° ; and this enables the formula to be put into the simpler form—

$$h_f = \frac{N V^2}{536} = N V^2 \times 0.001865.$$

The values of this formula, for various velocities and bends, are given in Part 2 of Table IX., and an explanatory example is attached.

Pipe-bends.—A formula more suited to bends of pipes is that of Weisbach; it is for cylindrical pipes—

$$h_f = \frac{a}{180} \frac{V^2}{2g} \times \left\{ 0.131 + 1.847 \left(\frac{r}{R} \right)^{\frac{1}{2}} \right\}$$

and for rectangular tubes—

$$h_f = \frac{a}{180} \frac{V^2}{2g} \times \left\{ 0.124 + 3.104 \left(\frac{d}{2R} \right)^{\frac{1}{2}} \right\}$$

but as the bends of pipes, known as quarter bends, are generally taken as 90° ; the value of the factor in either case

$$\frac{a V^2}{180^\circ \times 2g} \text{ then becomes } = \frac{V^2}{128.8} = 0.007764 V^2.$$

In this formula r and R are the radii of the pipe and of the bend, and the other terms are as before. The loss of

head due to bends in pipes is, however, generally required in relation with discharges, not with mean velocities of discharge. The values approximately given by this formula have, therefore, been tabulated in this form, and are given in Part 1 of Table IX. ; an explanatory example is also attached to it.

Obstructions.

While the above formulæ may be thus employed for the present, it must be noticed that they are merely approximately correct, and that extensive and numerous careful experiments are yet required before an accurate determination of the head, representing the loss of effect caused by a bend of every sort and condition, will be arrived at.

The ordinary formula for calculating the rise in feet resulting from an obstruction in the section of a river channel is that of Dubuat ; it is—

$$h_{//} = \left(\frac{V^2}{o^2 \cdot 2g} + S \right) \left\{ \left(\frac{A}{a} \right)^2 - 1 \right\}$$

where A , a , are the normal and the reduced sectional areas of flow.

S is the sine of the hydraulic slope of the river, and o is the experimental coefficient for discharge through the bridge opening taken as a sluice or orifice.

Now, as in most cases S is less than 0·001, that term may be neglected, and taking $o = 0\cdot96$, $o^2 = 0\cdot92$, and the formula becomes—

$$h_{//} = 0\cdot0169 V^2 \left\{ \left(\frac{A}{a} \right)^2 - 1 \right\}$$

For other values of o , suitable to any special case, the corresponding value of o^2 must be applied in the original formula.

The values of this are given in Part 3 of Table IX., and an explanatory example accompanies it.

II. DISCHARGES FROM ORIFICES AND OVERFALLS.

The discharge from orifices and overfalls, which to the practical man generally resolve themselves into sluices, weirs, and water-cocks, is a subject that was fully entered into by hydraulicians of past times, and to which very little information has been added by recent experimentalists. Nor is it by any means likely that further contributions will be soon made to this branch of hydraulic science, as there have recently been to that of channel-discharge; the practical interest attaching itself to the exact determination of discharge of a sluice or a weir not being in excess of the amount of exactitude already attained. As all accepted information on this subject is to be found, with but little variation, in the older books, the author had little choice left to him, in compiling from them; much of the following was reduced from Bennett's translation of d'Aubuisson's hydraulics, for want of a copy of the original.

Setting aside the experiments of the more ancient philosophers, it may be assumed that the discharge from any orifice under theoretically constant pressure is

$$Q = A V = A.o \sqrt{2gH}$$

where H = the head of pressure of the orifice,

o = the coefficient of reduction obtained by experiment on such orifice,

V = the mean velocity of discharge.

The first of the more modern hydraulicians to obtain experimental values of o , on a scale larger than the previous very petty experiments, was Michelotti: his experiments conducted at Turin in 1767, under heads of pressure up to 22 feet, determined coefficients of reduction varying from 0·615 to 0·619, for circular orifices, up to $6\frac{1}{2}$ inches in diameter, and coefficients varying from 0·602 to 0·619 for square orifices, up to 3 inches in length of side. The next important experiments did not so much include increase of head as increased dimension of opening. Messrs. Lespinasse and Pin, Engineers of the Languedoc Canal, 1782 to 1792, made experiments on rectangular openings, or sluices 4·265 feet broad, and having heights varying from 1·575 to 1·805 feet, under heads on their centres of from 6·2 to 14·5 feet; the coefficients deduced varied from 0·594 to 0·647, the mean being 0·625; they also observed that the discharge from two sluices opened at one time side by side was not double that from one sluice. In 1826 at Metz, MM Poncelet and Lesbros deduced a law for the determination of coefficients of discharge of rectangular orifices under various proportions of head of pressure and depth of opening to width; these coefficients, ranging from 0·572 to 0·709, are given in Table XII. The next important experiments recorded were those conducted by M. George Bidone, at Turin, in 1836, on orifices on parts of which the contraction was suppressed, the extreme of suppression being a case in which the whole of the contraction was suppressed by fitting an interior short tube to the mouth of the orifice: his resulting formula of discharge was for rectangular orifices—

$$Q = o \cdot A \sqrt{2gH} \left(1 + 0\cdot152 \frac{p}{P} \right)$$

and for circular orifices,

$$Q = o \cdot A \sqrt{2gH} \left(1 + 0.128 \frac{p}{P} \right)$$

where p is the portion of the perimeter P whose contraction is suppressed.

About this time also some further experiments were made by Castel and d'Aubuisson; and some by Borda on orifices in sides not plane, but of compound formation.

In small orifices generally.

The results of all these experiments show that the extreme limits of the value of o are 0.50 and 1.00 for orifices in all sorts of sides, and under all conditions, and are 0.60 and 0.70 for orifices in plane sides; also that the general mean value of o for orifices in a thin plate is 0.62; this, however, is perhaps more true for small circular orifices than for any other class of them. In this case therefore

$$V = 0.62 \times 8.025 \sqrt{H} = 4.975 \sqrt{H},$$

and for rectangular orifices of a similar class, the special values of o , ranging from 0.572 to 0.709, given in Table XII., must be applied to the general formula

$$V = o \times \sqrt{2gH}$$

in order to determine the mean velocity of discharge, which when multiplied by the sectional area gives the quantity discharged per second.

Effect of initial velocity.—In the special case in which the reservoir of supply, still being kept at a constant level, is seriously affected by the velocity of the water

supplying it, the discharge of the orifice will be augmented on this account, and then

$$V = o \sqrt{2g \left(H + \frac{W^2}{2g} \right)} = o \sqrt{2gH + W^2}$$

where W = the initial velocity of entrance.

Attached channel.—When an open channel is attached to the orifice at its exit, in such a manner that the sides and bottom of the channel are continuations of those of the orifice, the coefficient of contraction remains the same, except when the head on the orifice is less than $2\frac{1}{2}$ times the height of the orifice; in this latter case the coefficient may have to be materially reduced. An extreme case given by Poncelet and Lesbros, being one of a discharge through an orifice 0·164 feet high, under a head of 0·118, gave a value of $o = 0·452$, while without an attached channel the value of o was $= 0·612$; further, when the level of the attached channel was exactly at the same level as the floor of the reservoir of supply, the value of o was reduced to 0·443. The law of reduction of coefficient necessary for these cases is not yet given in a definite form. The inclination of the attached channel when less than one in 100 did not affect the coefficients in any way, but when increased to one in 10 had the effect of increasing the coefficient from 3 to 4 per cent.

Orifices with mouthpieces attached were even in the time of the Romans known to have a greater discharge than those without them. In order to effect this increase it is, however, necessary that the length of the attached or additional tube should be twice or three times the diameter of the orifice, otherwise the fluid vein does not entirely fill the mouth of the passage. The experiments of Michelotti and Castel determined a mean coefficient

of discharge for cylindrical mouthpieces of 0·82, the extremes being 0·803 and 0·830; the singular effects produced under some circumstances by the application of cylindrical mouthpieces are more curious than useful. Conical converging mouthpieces increase the discharge more highly: the experiments on them of Castel, engineer of the waterworks of Toulouse, are exceedingly interesting; they demonstrated that under varied heads the coefficients of discharge and of velocity were practically constant for the same mouthpiece, and that for the same orifice of exit the coefficient of discharge increased from 0·83 for a cylindrical mouthpiece in proportion to the increase of the angle of convergence of the mouthpiece employed up to 0·95 for an angle of $13\frac{1}{2}^{\circ}$; and that beyond this angle the coefficient of discharge diminishes to 0·93 for 20° , and afterwards decreases more rapidly. The length of mouthpiece employed in these cases as well as in the former was $2\frac{1}{2}$ times the diameter of the orifice. Some experiments by Lespinasse on the canal of Languedoc showed the enormous increase of discharge effected by using converging mouthpieces: his mouthpieces were truncated rectangular pyramids 9·59 feet long, the dimensions at one end $2\cdot4 \times 3\cdot2$ feet, at the other $\cdot44 \times \cdot62$ feet, and were used in mills to throw the water on to water-wheels; their opposite faces were inclined at angles of $11^{\circ} 38'$ and $15^{\circ} 18'$, and the head employed was 9·59 feet; the experiments resulted in determining a coefficient of discharge varying from 0·976 to 0·987.

Conical diverging and trumpet-shaped mouthpieces still further increase the discharge from an orifice: the experiments of Bernouilli, Venturi, and Eytelwein have thrown much light on this subject, and showed the co-

efficient to lie between 0.91 and 1.35. Venturi concluded that the mouthpiece of maximum discharge should have a length nine times the diameter of the smaller base, and a flare of $5^{\circ} 6'$, and that it would, if properly proportioned to the head of pressure, give a discharge 1.46 times the theoretic unreduced discharge through an orifice in a thin side.

Sluice gates, large openings, &c.

It may be observed, however, that although the minutiae of discharges under certain experimental conditions have been sedulously preserved, there is yet considerable doubt what coefficients should be used for large sluices and wide openings of different sorts. It may be unfortunate that experimentalists should differ, but at the same time the circumstances, under which the amount of discharge from a sluice is an important consideration, only occur generally to those who are capable and have the opportunity of determining it accurately by experiment themselves.

The ordinary coefficient for a sluice of moderate size, for small lock or dock-gates, or mill-gates, is generally taken at 0.62; that for a narrow bridge-opening, which may be considered as a large sluice, at 0.82; and that for very large well-built sluices, very wide openings out of reservoirs level with the bottom of the reservoir, and large bridge-openings of the modern type, at 0.92.

The term H , representing the effective head of pressure, is differently estimated in various cases: in ordinary cases of sluices, supplied from a reservoir above them, the head is the difference of level between the surface of the water in the reservoir and the centre of figure of the

sluice ; but when the sluice is drowned, that is, has a perceptible depth of water in the tail race standing above the sluice itself, the head is the difference of level of the water above and of that below it ; in bridge-openings also, the head is the difference of water level on the up-stream and down-stream sides of the bridge.

The most recent experimental determination of coefficients of discharge for head-sluices supplying small channels is that of d'Arcy and Bazin ; the results of these operations will be given, with the account of the mode of gauging adopted by them, in Chapter II.

The above includes all the general deductions about orifices that are likely to be of any use to the engineer ; a more practical collection of coefficients of discharge for orifices is given in Part 4 of Table XII. ; and the value of the expression $V = o \sqrt{2gH}$ is given in Table X., for various heads, and for all the values of o that are commonly used ; some explanatory examples also follow that table.

The discharge of pipes under pressure.

This subject may be treated as one closely allied to the discharge of orifices in one respect. If at any point in a pipe or series of pipes under pressure the continuity of the pipe be cut off, the discharge at that point will obviously be that of an orifice under pressure, provided the necessary free fall be allowed ; the dimensions of the orifice will be those of the section of the pipe at the exit, and the head will be the statical pressure, less a reduction of head representing the friction throughout the whole course of the series of pipes of supply, and another for contraction at entry and at exit.

In actual practice, this method could alone be conveniently applied at the extremity of a series of pipes for direct determination of discharge; but having obtained by this or any other method the discharge at any one point in a line of pipes, the discharge at any other point along the same line may be relatively determined by making allowance for the friction developed in the intermediate length by a representative head.

A more common mode of making calculations of discharge, pressure, and diameter of pipes under pressure has been in accordance with mean inclinations of the various general lines of pipes in a series, and by applying the ordinary formula for flow (transformed for diameters of cylinders) as before given

$$Q = c \times 39.27 \sqrt{Sd^5}$$

$$\text{or } H = \frac{0.0648 Q^2}{c^2 d^5}$$

It is, however, evident that this method of assuming a mean hydraulic slope taken from a point where the pressure is zero to the point of contemplated discharge, and treating the discharge according to the principles of flow, from a summit due to that hydraulic slope, is an inexact method; for it is very evident that the same data as bases of calculation might apply to two very different conditions of length of pipe, thus neglecting considerable amounts of friction.

Overfalls and Weirs.

An overfall may be treated as a wide rectangular orifice in an ultimate position, where the head on the upper edge is zero; and its discharge may be therefore computed in the same manner as that of an orifice.

The discharge of an orifice is according to the parabolic theory—

$$Q = o \times \frac{2}{3} \sqrt{2g} \times w(h_1 \sqrt{h_1} - h \sqrt{h})$$

where h and h_1 are the heads on the top and bottom edge, and w is the width of the orifice; but if H = mean head on the centre of the orifice, and d is its depth when the orifice becomes an overfall, this formula becomes

$$Q = o \times \frac{2}{3} \sqrt{2g} \times w \left\{ \left(H + \frac{d}{2} \right)^{\frac{3}{2}} - \left(H - \frac{d}{2} \right)^{\frac{3}{2}} \right\}$$

developing this, and putting $wd = A$, the sectional area,

$$Q = oA \frac{2}{3} \sqrt{2gH} \left(1 - \frac{d^2}{96h^2} \right)$$

and as d is comparatively small, the last term may be neglected, hence

$$Q = oA \frac{2}{3} \sqrt{2gH}; \text{ and } V = o \frac{2}{3} \sqrt{2gH}$$

where H is the head on the sill of the overfall.

The value of the coefficient, o , varies according to the conditions of the overfall. It was determined by M. Castel, at Toulouse, by a large series of experiments; and also by Francis, in the Lowell experiments referred to in Chapter II. on Gauging. (For obstructed overfalls see also a paragraph following.)

The experiments of M. Castel showed that, for the accurate employment of a general coefficient the dimensions and conditions of an overfall should fall within one of the three following classes.

1st. When the length of the overfall sill extends to the entire breadth of the channel, and the head on the sill is less than one-third the height of the dam or barrier, the coefficients remain remarkably constant,

varying only from 0·664 to 0·666. Hence generally for this case, $o = 0·666$.

2nd. When the length of the overfall sill is less than the entire breadth of the channel of supply, but is greater than a quarter its breadth, the coefficient lies between the two extremes of 0·666 and 0·598, and is strictly dependent on the ratio of the length of sill to breadth of channel; hence it is for the following relative lengths of sill:—

| Relative lengths of sill | Coefficient | Relative lengths of sill | Coefficient |
|--------------------------|-------------|--------------------------|-------------|
| 1·00 | 0·666 | 0·50 | 0·613 |
| 0·90 | 0·658 | 0·40 | 0·609 |
| 0·80 | 0·647 | 0·30 | 0·600 |
| 0·70 | 0·635 | 0·25 | 0·598 |
| 0·60 | 0·624 | | |

3rd. If the length of the overfall sill be equal, or even only nearly equal, to one-third the breadth of the channel, the coefficient remains very constant, varying only between 0·59 and 0·61. Hence generally for this case, which is particularly favourable for gauging small streams, $o = 0·60$.

In other cases, that is, when the length of the sill is less than a quarter the breadth of the channel of supply, the coefficient depends on the absolute length of sill, and requires determining specially; it increases from 0·61 to 0·67 in direct proportion to the diminution of absolute length of sill.

Velocity of approach.—With reference to the three cases suitable for practical purposes, the experiments of M. Castel showed that when the sectional area of the overfall was less than one-fifth of that of the normal section of the channel of supply, the effect of velocity of approach in the channel did not modify the value of the coefficient; for other conditions, the modification

necessary was not determined in a very satisfactory form:—the new equation for mean velocity of discharge being changed from

$$V = o \frac{2}{3} \sqrt{2gH}$$

into

$$V = o \frac{2}{3} \sqrt{2g(H + 0.035W^2)},$$

where W = the surface velocity of approach, not determined from observation, but from its assumed ratio to the mean velocity. Perhaps therefore it is preferable to modify the coefficient, o , into a new coefficient o_1 , comprising the allowance, thus

$$o_1 = o \left\{ \left(1 + \frac{h}{H} \right)^{\frac{3}{2}} - \left(\frac{h}{H} \right)^{\frac{3}{2}} \right\}$$

where h is the head due to the velocity of approach, and H is the head on the weir sill.

Attached channels.—For the special cases in which channels are attached in continuation of the sides of the overfall, the coefficients in the experiments of Poncelet and Lesbros were reduced by 18 to 33 per cent. If, however, the fall to the channel is more than 3 feet, no reduction is generally made in the coefficients.

It may be noticed that the head on the sill used in the above expression is that in the centre of the overfall, which is independent of the rising of the water at the wings, a phenomenon to be observed in almost all cases of weir discharges.

In all the above cases, it is supposed that thin edges as of metal sheets, or one-inch waste-boards, are used; for broad or round-lipped crests, the coefficients will require reduction. See the coefficients given in Part 5 of Table XII.

Obstructed Overfalls.—When obstacles occur on the

sill of an overfall, as dwarf pillars or blocks, a deduction in the discharge over the sill is made not only on account of the reduction of section, but on account of the contractions resulting. Francis's formula is applicable to these circumstances in cases where the length of weir sill equals or exceeds the head ;—it is

$$Q = \frac{2}{3} o \sqrt{2g} \cdot (l - 0.1 n H) H^{\frac{3}{2}},$$

where n = the number of end contractions,
(note that $n = 2$, when there is no central obstruction,)

l = length of weir sill,

$lH = A$ the sectional area of discharge,

and $o = 0.6228$.

In case the weir sill has the same breadth as the channel of supply, $n = 0$; and in that case

$$Q = 3.332 l H^{\frac{3}{2}}.$$

This, it will be observed, varies from that of Castel, which, under the same conditions, when $o = 0.666$, gives

$$Q = 3.563 l H^{\frac{3}{2}}.$$

Partly Drowned Overfalls.—When a weir has its tail water above the edge of the sill, it may be treated as a combination of an overfall with an orifice ; the upper portion down to the level of the lower water as an overfall, and the lower portion from that down to the sill level as a rectangular orifice, and the discharges calculated separately for each. The same value of H is used in both cases, H being the head due to the overfall, that is, down to the level of the tail-race.

Some further values of coefficients of weir discharge are given in the accounts of gauging in Chapter II. To aid in the computation of discharges from overfalls, the

velocities of discharge due to various heads and various coefficients may be obtained from those given in Table X., by reducing the velocities there given by one-third ; the results multiplied by the section of overfall are then the required discharges. The method thus adopted enables the same table to be used in computing the discharges of both orifices and overfalls. A table of weir coefficients is given in Table XII., and some explanatory examples accompany Table X.

12. EFFLUX OR DISCHARGE FROM PRISMATIC VESSELS, LOCKS, BASINS, RESERVOIRS, OR TANKS.

The following formulæ given by d'Aubuisson may be considered useful for reference in the cases in which they are required in engineering practice :—

First Case.

Simple discharge from a reservoir.

(1st.) When the reservoir empties itself through an orifice or sluice with free exit.

Velocities.—The ratio between the velocity at the orifice of discharge and that of the water in the reservoir is in the inverse ratio of their sectional areas.

Head.—If H = actual height of water in the reservoir ; h = the height due to and generating the velocity of discharge, and A and a are the sectional areas of the reservoir and the orifice respectively.

Then
$$h = \frac{HA^2}{A^2 - a^2}$$

Discharge.—A reservoir emptying itself through an orifice in a given time would discharge a volume equal to half that due to the head at the commencement, kept constant during the same time. For such examples applied to locks, see Table X.

Time.—The time in which a prismatic reservoir empties itself is double that in which the same volume would be discharged if the initial head had remained constant.

The time of descent, t , to a given depth, $d = H - h$,

$$t = \frac{2A}{oa\sqrt{2g}}(\sqrt{H} - \sqrt{h});$$

and the quantity discharged in a given time, t ,

$$\text{is } Q = A(H - h) = \frac{t \cdot o \cdot a \sqrt{2g}}{A} \left(\sqrt{H} - \frac{toa\sqrt{2g}}{4A} \right),$$

and the mean hydraulic head, H_1 under which the same quantity would be discharged in the same time is—

$$H_1 = \left(\frac{\sqrt{H} + \sqrt{h}}{2} \right)^2$$

where H and h are the heads at the beginning and end of the time of discharge, the reservoir receiving no supply during that time.

(2nd.) When the basin or reservoir receives a constant supply during the time of discharge.

If q = quantity supplied per second,

t = time in which the surface will descend the depth, $x = H - h$.

$$t = \frac{2A}{(oa\sqrt{2g})^2} \left\{ oa\sqrt{2g}(\sqrt{H} - \sqrt{h}) + q \cdot \text{hyplog} \frac{oa\sqrt{2gH} - q}{oa\sqrt{2gh} - q} \right\}$$

when there is no supply, or $q=0$, this equation resolves itself into that previously given.

(3rd.) In the case of there being no supply, but the discharge instead of being effected through an orifice is conducted over an overfall, having a length of sill $=L$,

$$t = \frac{3A}{oL \sqrt{2g}} \left\{ \frac{1}{\sqrt{h}} - \frac{1}{\sqrt{H}} \right\}$$

Non-prismatic reservoirs are extremely difficult to deal with, and the investigation of any special case here would be comparatively useless.

Second case.

When one reservoir empties itself into a partly filled reservoir.

(1st.) When each of the two reservoirs being exceedingly large practically preserves its own level, the communicating sluice being below the lower surface of water; then if H, h , are the heads; a the sectional area of the sluice,

$$\text{the discharge } Q = oa \sqrt{2g(H-h)}.$$

(2nd.) When the upper reservoir being exceedingly large preserves its own level, and the lower reservoir having a definite area (A), receives the supply through a sluice of a section (a), required the time t in which the surface of the lower basin will rise to a certain height.

If H, h , be the heads on the lower surface at the beginning and end of the time, t ,

$$\text{then } t = \frac{2A}{oa \sqrt{2g}} (\sqrt{H} - \sqrt{h}),$$

this formula, like that previously given, is useful for determining the time necessary to fill a lock chamber; when $h=0$, or the levels become the same, the case is that of canal locks, and the sectional area of the sluice may be determined from this equation.

(3rd.) When neither reservoir receives any supply, and both are limited in size, if the surfaces are originally at different levels, and the communication sluice is opened, the surface of one will rise and the other fall.

If A, B , are the sections of the two vessels,

H, x , the heads at the beginning and end in A ,

h, y , the heads at the beginning and end in B ,

a =the sectional area of the pipe or sluice,

t =time during which the sluice is open,

then

$$t = \frac{2A\sqrt{B}}{oa.(A+B)\sqrt{2g}} \left\{ \sqrt{B(H-h)} - \sqrt{(A+B)x - AH - Bh} \right\}$$

and if it be required to know the time t' in which the two surfaces will be level; in that case, $x=y = \frac{AH+Bh}{A+B}$,

and then

$$t' = \frac{2AB\sqrt{H-h}}{oa(A+B)\sqrt{2g}}.$$

This formula is convenient for determining the time occupied in bringing the water in the two chambers of a double lock to the same level, by means of a sluice of known dimensions.



CHAPTER II.

ON FIELD OPERATIONS AND GAUGING.

1. Direct measurement of discharge. 2. Gauging by rectangular overfalls.
3. Appliances and instruments for the measurement of velocities.
4. Baldwin and Whistler's gauging by means of surface velocities.
5. Francis's gauging canals and streams with loaded tubes. 6. The Mississippi field operations for gauging very large rivers. 7. Field operations in gauging crevasses: and computation of coefficients.
8. Captain Humphreys' improved system of gauging rivers and canals, and General Abbot's mode of determining a discharge on any given day. 9. The experiments of d'Arcy and Bazin on the Rigoles de Chazilly et Grosbois. 10. Velocity observations on great rivers in South America, by J. J. Révy. 11. Captain Cunningham's experiments on the Ganges Canal. 12. General remarks on systems of gauging, and conclusions therefrom.

I. DIRECT MEASUREMENT OF DISCHARGE.

THE direct measurement of the discharge of a channel or stream can be obtained by means of gauge-wheels. The channel is widened until the water flows at a moderate depth, less than five feet, over a horizontal and carefully constructed apron which is divided by piers into a number of equal openings. At each of these openings a gauge-wheel is placed, which fits the opening every way within a quarter of an inch. Sheet piling is driven across the head of the apron and along the banks approaching it for some little distance, so as to force the whole of the water of the stream to pass between the piers and drive the wheels. The measurement of the water is determined by the number of revo-

lutions of the wheels, which should be all coupled on to one shaft and be made self-recording on a dial-face, and by the dimensions of the wheels, or spaces between their blades, as well as by the depth of water passing over the apron, which is observed at intervals of about five minutes on gauges erected for the purpose. This method of obtaining a discharge is expensive, interferes with navigation as well as the passage of the water, and is therefore very rarely adopted.

2. GAUGING BY RECTANGULAR OVERFALLS.

The water of a canal or stream is made to discharge itself over a single horizontal dam, or over a series of small overfalls specially constructed for the purpose. The discharge over overfalls of certain dimensions, and under certain circumstances, is known by many series of experiments to be correctly expressed by a formula, containing the required data and dimensions, known as Francis's formula; it is

$$Q = o \times \frac{2}{3} \sqrt{2g} [l - 0.1nH] H^{\frac{3}{2}}$$

where

l = length of weir-sill.

H = head on the weir from still water.

n = number of end contractions.

If the weir-sill is of the same length as the breadth of the channel of approach, $n = 0$; if less than it, and there is no central pier or obstacle, $n = 2$; each pier or obstacle involving two additional end contractions.

Taking $\sqrt{2g} = 8.025$ and $o = 0.6228$,

$$Q = 3.33198 [l - 0.1nH] H^{\frac{3}{2}}.$$

This gives results within one per cent. of absolute exactitude. The dimensions in this formula being taken in feet, the discharges will be in cubic feet per second.

The following conditions should be observed in gauging by rectangular overfalls.

As regards form of construction :—

1. The dam in which the overfall or series of overfalls is placed should have the sills truly horizontal, and the sides of the overfalls truly vertical : the dam itself should be vertical all along on the up-stream side, but the sills should all be sloped off on the down-stream side at an angle of 45° or more with the horizon ; all the edges of discharge should be sharp and true, after passing which the water should discharge itself unobstructed.

2. In order to obviate the necessity of allowing for the velocity of approach in the channel, the area of the overfall—*i.e.*, the quantity $l \times H$, must not exceed one-fifth the area of the channel ; otherwise an allowance must be made on this account, as given in the paragraph on Weirs, Chapter I., Section II.

3. Should the velocity in the channel of supply not be uniform in all parts of its section, arrangements must be made to make it so ; this can be done by placing gratings, having unequally distributed apertures, all across the channel, and as far from the overfall as possible, and letting the water pass through them under a small head.

4. In addition to the above it is absolutely necessary that the air under the falling sheet of water should have free communication with the external air.

With regard to dimensions :—

5. Should the overfall not extend to the entire width

of the channel of supply, there should be at least a difference at each end equal to the depth on the overfall, so as to produce complete end contraction.

6. When the breadth of the overfall is equal to that of the stream, and even under all circumstances, the depth on the weir should be less than one-third the height of the barrier.

7. The depth on the weir must be always less than one-third of the length of the sill.

8. The head on the overfall, H , should never be less than 0·2 feet; it is better, also, to make it more than 0·5 feet and less than 2 feet.

9. The fall from sill to tail-water should not be less than half the depth on the weir, in order to ensure a free fall.

The following practical directions suitable to streams and moderate rivers are given as examples, where ordinary care and accuracy is required.

First case.—When the discharge is supposed to be less than 40 cubic feet per second :—

First, according to the above rules, make H greater than 2 feet; and $H \times l$ less than one-fifth of the channel section; let l be greater than 3 feet, but less than one-third the width of the channel; and, to ensure a free fall, arrange so that the lower edge of the sill may not be less than half a foot above the tail-race. Under these conditions the coefficient of discharge to be used will be $c = 0\cdot623$, and any error should not be more than one per cent.

Before constructing the weir, observe the surface velocity in the channel (V_s) and the transverse section (A); the approximate discharge will then be $Q_s = V_s \times A$, and assuming a value for l as before mentioned, obtain

a value for H by means of the ordinary formula, making use of the approximate discharge for this purpose. H should be from 1 to 3 feet, and should such a value not result, from the application of the previous conditions, use another value for l , so as to secure this condition, as well as to retain the other conditions before mentioned. When this is gained, the opening may be cut of the required dimensions in one-inch plank, and the dam well puddled ; and as, in practice, the dimensions are not likely to be very closely adhered to, they should be measured again when the orifice is completed, and applied in the formula before given.

Second case.—When the supposed discharge is more than 40 cubic feet per second, but is manageable :—

Find the approximate discharge at the spot from the section and velocity, when the surface of the stream is level with a fixed mark on a post or stone, at from 100 to 200 feet below the intended site of the weir. Having previously selected a place where the stream is regular in width and inclination, construct the dam so that the weir-sill may be equal to the full breadth of the channel, and square the ends of the opening with planking. Put a gauge at each end, with the zero at the level of the upper edge of the sill of the overfall, which should be from 1 to 5 feet above the fixed benchmark.

When the water is up to the mark, read the height on either scale ; take their mean, and use it as a value for H in the weir formula before given to obtain the velocity and amount of discharge. If necessary, obtain the surface velocity of approach W , and make suitable allowance for it as before mentioned under the head of weir discharges in Chapter I. In this case $\phi=0.666$.

3 APPLIANCES AND INSTRUMENTS FOR VELOCITY MEASUREMENT.

There are many cases when it is not advisable to construct a dam or gauge by overfalls, and also cases where the simple calculation of discharge due to the hydraulic slope, and the terms of its cross-section, does not give sufficiently accurate results. Under these circumstances velocity observations must be made, and other data correctly obtained, so as to obtain from them the required discharge, which, when divided by the sectional area, gives the mean velocity of discharge.

In all cases where velocity must be observed it is advisable to choose a straight reach of channel having a tolerably uniform section; it is also advantageous that the bank should admit of the measurement of a straight line parallel to the general direction of the channel, and at right angles to the line of intended river section of observation, to serve as a base for triangulation, and location of courses, and sections.

To obtain perfect uniformity of channel, a flume or timber lining to the reach of well-joined plank may be constructed, giving about two hundred feet of perfectly uniform section; this gives the means of accurately measuring the dimensions of the stream, the whole of the water of which is forced to pass through it by means of sheet piling at its upper entrance. It should not produce any sensible disturbance in the flow of the water, and not interfere with the navigation or passage of water. Velocity observations are then made either at the middle section or on a measured length along the flume, at such intervals that the variation of

observed velocity in section shall never be very marked. The summation of the products of these representative velocities by their corresponding portions of sectional area gives the required discharge. A long and accurately constructed open aqueduct in perfect order answers all the purposes of a flume.

Failing all such opportunities, the channel itself must be employed in its natural state ; in this case the effect of various velocities on the bed and banks should be noted from time to time during the observations. Should any exact determination of the water section be impossible it becomes necessary to resort to soundings. These may either be taken by means of a surveyor's 100-foot chain, with a suitably heavy leaden weight attached to one of the handles, or with a sounding line. The determination of the position of each sounding can in narrow reaches be best made by stretching a rope across, and measuring the distances of the sounding points from one bank along the cord. In wide reaches where this is impracticable, the sounding points have to be fixed by angular observation and connected with the base line of triangulation at the moment of sounding either by an observer with a theodolite on the shore, or with a pocket sextant in a moored boat.

The fall of the water surface at all states of the channel is one of the data generally required. To determine this, a gauge-post is erected, driven into the ground at each sounding section, and the heights of the water shown on them continually recorded so as to show all variations of depth ; the connection of level between the two or more gauge-posts is made by levelling either from one post to the other, or from both to a fixed bench-mark. In many cases the fall of the water

surface is so slight that the ordinary level and staves cannot give sufficiently exact results; instruments of greater precision must then be used.

An ordinary gauge-post may also be too coarse for indicating the slight variation of the water surface during the period of gauging; in that case a superior appliance, a hook-gauge or a tube-gauge, is necessary.

Boyden's hook-gauge.—It is well known that the capillary attraction of water about any simple rod-gauge for determining water level will falsify readings. To obviate that defect this gauge has a hook at its lower end, which can be raised or lowered by turning a screw; when the point of the hook is even a thousandth part of a foot above the water surface, the water around it is sensibly elevated by the capillary attraction, and obviously distorts the reflection of light from the surface; when the hook is lowered just sufficiently to cause this distortion to disappear, the point of the hook must coincide with the water surface; a true reading, exact within 0.001 of a foot, can then be read, by means of a vernier attached to the rod of this gauge which is graduated to hundredths of a foot. As this instrument can only be effectively used in still water, it is held in a box, the inclosed water communicating with the external water only by means of a hole; or, if the depth at some distance off is the object, by a pipe leading from that place to the hole in the box; any oscillation of the water surface in the box may then be diminished or nearly removed by partially obstructing the hole or communication at will. Should perfect rest not be attainable, a good mean position of the point of the hook may be obtained by adjusting it to a height at which it will be visible above the water surface for half the time. It is convenient to have also a

hook made with a small semispherical knob on it, so that a level-staff can then be held on it for taking a sight with an instrument.

Bazin's tube-gauge is, unfortunately, not described in sufficient detail, nor are drawings of it given in his 'Recherches Hydrauliques.' It seems, however, to have been a glass tube having a mouthpiece of only a millimètre in diameter, and that it enabled variations of water level of one millimètre to be easily read; it is hence extremely probable that it resembled in some respects the velocity gauge-tube of d'Arcy, used for taking velocity measurements, hereafter described. It is, in fact, evident that an instrument on this latter principle, capable of indicating variations of velocity with precision, would also indicate with exactness the moment of the withdrawal from, or submersion of its mouthpiece in, the water, and that this motion could be easily manipulated with a clamping and a tangent screw.

The following are the different instruments and appliances for measuring velocity; but most if not all of these involve the application of a special coefficient of reduction due to the particular appliance, in order to obtain the *actual* velocity of the water in feet per second.

1. *Surface floats*.—Surface velocity may be very simply measured by observing the time of transit over a known distance or length of a reach of a river, of any light floating body, a wafer, a ball of wood or cork, or a partly filled bottle. This method is coarse, and fallacious; a later float may outrun an earlier one, when there is much local variation of velocity.

2. *Loaded rods and tubes*.—Mean verticalic velocity, being the mean velocity past any vertical axis, or the

mean of all the velocities from water surface to the bottom under any point in a vertical plane, is measured by a loaded wooden rod or hollow tube placed vertically, having a length nearly equal to the depth of the channel. The time of transit of such a rod will then give approximately the mean velocity of the vertical plane of the water in which it moves. These tubes are generally weighted inside and capped, as the painted metal tubes of the Lowell experiments hereafter mentioned, thus obviating the necessity of attaching weights.

The loaded tubes and rods used in the velocity observations on the Ganges Canal by Captain Cunningham will be described hereafter in Section II of this chapter, which is devoted to those experiments.

Another recognised mode of observing mean vertical velocity consists in lowering from the surface to the bottom, and raising again to the surface any accumulative self-recording current meter. This is an operation requiring extreme care; the meter must be sufficiently weighted, and, if necessary, also managed by a cord from an additional boat moored up stream so as to ensure its moving vertically up and down; the lowering and raising of the meter must also be evenly and steadily managed, so that the results may not be falsified.

3. *Floated frames.*—Mean sectional velocity can be approximately obtained in small streams and canals at one operation only by making a light covered framework nearly the size of the whole cross-section of the stream, and so arranging it by floats and weights that it will assume a vertical position at right angles to the thread of the current; its time of transit can then be noted, and this will be the approximate mean velocity of the section.

4. *Double floats*.—These are used for sub-surface velocities.

A weighted float, consisting of ball, or cube of wood, or hollow tin weighted with lead, is sunk to the required depth, being attached by a cord or thread to a small upper float on the surface of the water; the upper float being made of cork, light wood, or hollow tin, carrying a vertical stick, or wire, for convenience of observation, and the length of cord being so adjusted as to prevent the weighted float from sinking lower than the depth at which the current velocity is required. The time of transit of this double float, over a measured or a calculated distance, is observed, and is supposed to represent the velocity of the stream at that depth, independently of any coefficient of reduction.

Another form of double float is a pair of equal hollow balls connected or linked together, the upper one on the surface, and the lower one weighted sufficiently to keep it at the certain depth; the velocity of this double float, as observed on a measured distance, is supposed to be that of the current at half the depth of the lower ball.

The double-floats invariably used in the Mississippi Survey were kegs without top or bottom, ballasted with strips of lead, so as to sink and remain upright; they were 9 inches in height, and 6 inches in diameter; the surface floats, when of light pine, $5\cdot5 \times 5\cdot5 \times \cdot5$ inches, when of tin, ellipsoids, axes $5\cdot5$ and $1\cdot5$ inches, the cord one-tenth of an inch in diameter; for observations more than 5 feet below the surface, the kegs were 12 inches high by 8 inches in diameter, and the cord nearly two-tenths of an inch. It was believed that neither the weight of the surface float nor the force of the

wind directly affected their velocities to any appreciable amount.

5. *Instruments of angular measurement.*—A quadrant having a graduated arc has a string attached to its centre, and a ball attached to the string, which is immersed in the stream. The current moving the ball produces an angular change from verticality in the position of the string; the velocity is then equal to the square root of the tangent of this angle multiplied by a coefficient, which is constant for the same ball only.

6. *The tension balance.*—A ball is immersed in the stream and attached by a wire to a balance, which registers the amount of pull. Another very similar method requires a small plate instead of a ball, which is connected with the balance, and which is directly opposed to the current.

The tachometer of Brünings is the best known instrument of this type. It consists of a plate fixed at one end of a horizontal stem, which moves in the socket of a vertical bar, by means of which the instrument either rests on the bottom of the channel or is suspended from above. A cord of fixed length is fastened to the other end of the stem, and, passing under a pulley, is attached to the short arm of a balance, on whose other arm a weight is suspended, being placed in such a position that the equilibrium is established with regard to the force of the current under observation. The position of the weight on the graduated arm of the balance indicates the velocity observed.

7. *The rotary screw.*—A light metal screw, similar to that of a ship's patent log, will, when submerged in a current, rotate at a velocity approximate to that of the water in which it is placed. If on the axle

of the screw a thread is set turning one or more worm-wheels, the number of revolutions of the worm-wheel will indicate the approximate velocity of the water, from which, by applying a coefficient of reduction applicable to the particular instrument, thus including all allowances for friction and other causes, the true velocity of the current may be obtained. There are several current meters of this type : Saxton's, Brewster's, and Révy's, hereafter described, are all modifications of this form. Some of these instruments are not suited to great depths and high velocities ; others are made self-recording in such a way as to make allowance in the indicated number of revolutions for the loss of velocity by friction ; the latter is a great disadvantage, as it is always practically necessary to test each particular instrument, and make use of a coefficient, however small it may be, in order to obtain accurate results.

The earliest now known instrument of this type is the hydrometric mill of Woltmann, used by him in 1790. The wings on its axle resembled those of a windmill, and were square copper plates, set at an angle of 45° , having their sides $\cdot 082$ feet and their centres at $\cdot 164$ feet from the axis of rotation ; for small velocities the size and distance of the wings was doubled. In great depths this instrument was attached to a bar and lowered from a platform between two boats, and the instrument put in gear or out of gear by means of a cord at any depth. This type of current meter, from its convenience of use in observing velocity at any depth, has been re-invented many times.

On the gauging of the Paraná and La Plata, by Mr. Révy, the screw current meter, with some alterations and improvements made by him, was invariably adopted.

For ordinary currents the screw used by Mr. Révy consisted of two long thin blades of German silver, having a diameter of 6 inches, and a pitch of 9 inches; the thread of its axis worked on two worm-wheels of 3 inches in diameter, one wheel having 200, and the other 201 teeth; each revolution of the screw moved the first wheel one tooth onwards, the second wheel moving one tooth onwards for each complete revolution of the first wheel; this allowed of the continuous reading of 40,000 revolutions; the two worm-wheels had graduated divisions around their circumferences, corresponding to the teeth in number and position, which were read off at an index through a glass plate covering them. A nut was also used for clearing the worm-wheels from the thread of the axle of the screw, by means of which the instrument was either put in gear or out of gear by hand; a wire attached also enabled this to be done from above when the instrument was at any depth.

For strong currents, the screw-blades were shorter and stronger, and made of steel. Some of the screws used were only 4 inches in diameter. The divisions on the circumferences of the wheels were found to be too near for convenient reading; 100 and 101 divisions would have been preferred to the existing arrangement of 200 and 201.

These meters were generally used for observing velocities of more than 10 feet per minute, their corrected results being absolutely correct within 1 inch per minute of velocity. They required extreme care and continual watching: the slightest bend or damage to a screw-blade, or any clogging or accidental tightening of a screw being liable to vitiate results.

When in good order, exposure to a gentle breeze is

sufficient to keep the instrument revolving;—failing this, cleaning and oiling, or readjusting carefully, is absolutely necessary. In order to keep a check on the observations, a second current meter should always be at hand.

The principal advantage of current meters of this description is the convenience with which they can be worked, and their unvarying utility in observations at any depth of water.

8. *The differential tube.*—Pitot's tube is a glass tube bent at the lower end; it is sunk to the required depth, and its lower orifice directed against the current: the velocity is deduced from the difference of water-level in this tube and that in another free from the effect of the current. The first improvement of this instrument is that of Dubuat, who gave the orifice of the tube a funnel shape, and closed it by a plate pierced with a small hole, thus considerably reducing the objectionable oscillations of the water in the tube. The next is by Mallet, who terminated the horizontal branch of the tube by a cone, having an opening of 2 millimètres, and made the tube itself of iron with a diameter of 4 centimètres; he also introduced a float and stem which, elevated by the force of the current, indicated heights on a graduated scale. The last improvement was that of d'Arcy, hereafter described.

In the experiments of d'Arcy and Bazin, on the Rigoles of Chazilly and Grosbois, the gauge-tube of d'Arcy, a development of the tube of Pitot, was generally used for taking velocity observations.

Pitot's tube, used in 1732, demonstrated the principle that the difference of water-level, h , shown by the two tubes, one vertical and the other curved, and directed

against the current, was that due to the velocity, and that the latter could be obtained from the former, by making use of the formula $V^2 = 2gh$.

The error in this was caused by the fact that the water in a vertical tube immersed in a current stands lower than the water surface outside; the difference being a quantity dependent on the square of the velocity immediately below the orifice. In addition to this Pitot's tubes had a serious disadvantage in that the oscillation of the water within the tubes, whose orifices were of the same diameter as the tubes themselves, did not allow the difference of level to be correctly observed.

These objections are entirely removed in the improved tube of d'Arcy, which has an orifice 1·5 millimètres in diameter for a tube one centimètre in diameter; in addition to this the lower portions of the tube to which the orifices are attached have a small diameter, and are made of copper: besides this, two cocks are introduced which add greatly to convenience of manipulation. The lower cock, which can be worked by a wire and lever, enables the orifices to be opened or closed at any moment from above, and thus allows the difference of water-levels of the tubes to be read off at leisure, after withdrawing the instrument from the water. The upper cock, after the water in the tubes is drawn up by the breath at an upper orifice, shuts off the air, and enables the difference of water-level in the tubes, which is not affected by dilatation or compression of the atmosphere, to be read off above against a scale.

This gauge-tube is described in 'Les Fontaines Publiques de la Ville de Dijon, 1856,' and drawings of it are given in the 'Recherches Hydrauliques' of d'Arcy and Bazin, 1865.

In the latter the vertical glass tubes are 1·25 m. long, the two small copper tubes below them being inclosed in a copper casing, 0·77 m. long, 0·06 m. broad, and 0·011 m. thick, terminating in a sharp wedge-shaped point to reduce the effect of the perturbation of the current. The tubes themselves are affixed to an upright of light boxwood, which is graduated and supplied with a vernier; the whole instrument being attached to an iron standard on which it slides, and to which it can be fixed by screws at any height; a handle turning the instrument directs the orifices in any required direction; and an additional movable wooden arm is used to enable the instrument to rest by means of it on any crossbeam or timber from which the observations are being taken.

In taking an observation with the instrument it is usual to take a mean of three maxima and minima.

The following is the theory of the determination of the coefficient of reduction μ in the formula $V = \mu \sqrt{2gh}$ for any instrument.

If a single curved Pitot tube be placed in a current, first, with its orifice directed against it, and recording a height h' , above the natural water surface; secondly, when directed with it, and recording a loss of level, h'' , below that of the natural water surface; and thirdly, when directed at right angles to the current, recording a loss of level h''' , then—

$$\frac{V^2}{2g} = m'h' ; \quad \frac{V^2}{2g} = m''h'' ; \quad \frac{V^2}{2g} = m'''h''' ;$$

and hence—

$$V = \sqrt{\frac{m'm''}{m'+m''}} \sqrt{2g(h'+h'')} = \mu \sqrt{2g(h'+h'')} ;$$

$$V = \sqrt{\frac{m'm'''}{m'+m'''}} \sqrt{2g(h'+h''')} = \mu' \sqrt{2g(h'+h''')} ;$$

and finding from tables the values of velocities V' and V'' corresponding to the heights $h' + h''$ and $h' + h'''$; the above equations become—

$$V = \mu V' ; \text{ and } V = \mu' V'' ;$$

hence there is a constant relation between the theoretic height $\frac{V^2}{g}$ due to the velocity of the fillet under consideration, and the quantities h' , h'' , h''' ; and the coefficient of reduction can therefore be obtained for any sort or form of orifice by means of a few experiments; also, when once the coefficient of reduction for the instrument is determined, it is unnecessary while observing velocities to make further use of the level of the water, in which the instrument is plunged.

9. *Grandi's Box*.—A box, having a small hole in the side towards the current, is sunk to a certain depth and withdrawn after a certain time; the amount of water in the box indicates the velocity at that depth.

10. *Boileau's Air-Float*.—A glass tube of fixed length is immersed in a position parallel to the current; the upper end of the tube has a conical mouthpiece fitted to it of any convenient size; the velocity of passage of a globule of air through the tube indicates the velocity of the current.

11. *Jackson's Current-meter*.—This instrument, designed by the author in Berar in 1870, is a spring indicator, or an adaptation of the principle of the spring-balance or weighing machine to measuring a sub-surface velocity at any point excepting at the exact surface or at the perimeter: it admits of convenient testing and verification by direct application of weights.

12. *De Perrodil's Torsion Current-meter*.—The principle of this instrument is the estimation of current

effect on the twisting of a wire : it reads to minute fractions of a foot per second.

Some of these modes of measuring velocity have for the present practically fallen into disuse, on account of the very limited range of their applicability ; others, on the contrary, have been severally adopted by various hydraulicians in modern times, to the entire exclusion of the rest. It may be noticed more especially that some of them merely afford a mean of a velocity varying throughout an extended time, and from this cause falsify any deduced velocity for any special moment of time ; others are inconvenient to manipulate, and a few yield inaccurate results whatever coefficient of reduction may be applied to the special instrument. The accounts of gauging operations given in the following sections of this chapter illustrate the use of some of these appliances.

4. GAUGING CHANNELS BY MEANS OF SURFACE VELOCITIES ONLY.

The experiments of Messrs. Baldwin and Whistler on discharges of canals of rectangular section are worthy of notice. They obtained discharges on the canals by means of surface velocities and flume measurement, and simultaneously gauged the actual discharges by gauge wheels, with the view of determining practically the relation between surface velocity and mean velocity, for channels of a certain size conveying water at certain velocities.

In one case the flume was 27·22 feet wide, with depths of water from 7·52 to 8·14 feet, having surface velocities from 3·07 to 3·34 feet per second ; the observations deduced a mean coefficient of velocity ·857, the extremes being ·838 and ·856. In the other case, the flume was 29·94 feet wide, with depths of water from 7·67 to 8·85

feet, having surface velocities from 1·91 to 2·77 feet per second ; the observations deduced a mean coefficient for the surface velocity of ·814, the extremes being ·797 and ·846.

In other cases, the data of which are not forthcoming, the coefficients of surface velocity were ·835, ·830, ·810 ; and taking ·829 as the mean of the five results, it can be favourably compared with De Prony's coefficient ·816, obtained from experiments on wooden troughs 18 inches wide, having depths of water from 2 to 10 inches, and velocities varying from 5 to 4·25 feet per second. Another point which Messrs. Baldwin and De Prony agreed in determining was that their coefficients should be slightly reduced for lower velocities and increased for higher. The result is that the proportion between the surface velocity and the mean velocity of discharge for rectangular channels in plank, and within certain limits of velocity and proportions of cross-section, may be said for practical purposes to lie between ·8 and ·85. Under similar local conditions, therefore, the discharge of a canal of rectangular section can be rapidly obtained by a few surface velocity observations, the inclination of the water surface, and the measurement of its section. Recent experiments, however, show that the above law of velocity does not hold generally ; hence this mode of gauging does not admit of extensive application.

5. GAUGING CANALS WITH LOADED TUBES ; BY FRANCIS.

Under the then existing arrangements at Lowell, a daily account was usually kept of the excess of water, if any, drawn by each manufacturing company over and

above the quantity it was entitled to under its lease. In ordinary times, occasional measurements were sufficient; but when water was deficient, frequent measurements were made. In the latter case, the following was the usual course of proceeding:—

A gauging party, consisting of one or more engineers with assistants, was assigned to each flume where measurement is necessary; and arrangements were so made that the observations for a single gauging occupied about an hour, the intervals during the day being occupied in working out the results, which were immediately communicated to the manufacturers, so that the machinery might be adjusted to the amount of water they were entitled to draw.

The following are the dimensions of the measuring flumes used, and the quantities of water usually gauged in them; the depth of water in the flume generally varying from 6 feet to 10 feet.

| | | | | |
|---------------|------|-------------------|------|-------------------|
| Merrimac | 100' | long by 50' wide, | 1500 | cub. ft. per sec. |
| Appleton | 150 | 50 | 1800 | „ „ |
| Lowell, M. C. | 150 | 30 | 500 | „ „ |
| Middlesex | 150 | 20 | 200 | „ „ |
| Prescott | 180 | 66 | 2000 | „ „ |
| Boott | 100 | 42 | 800 | „ „ |

The loaded tubes used were cylinders 2 inches in diameter made of tinned plates soldered together, with a piece of lead of the same diameter soldered to the lower end, having sufficient weight to sink the tube nearly to the required depth, thus leaving generally about 4 inches above the water surface. A red-paint mark was made to show the amount of immersion required, leaving a space between the bottom of the tube and the bottom of the canal of 1 foot. The tubes were of thirty-three different lengths, varying from 6 to 10 feet; six of each length were provided for this purpose.

In order to adjust the tube precisely, it was placed in a tank made for the purpose, and small pieces of lead were dropped into the top of the tube, and rested on the mass of soldered lead, and more were added until the tube was sunk to the required depth, when the orifice at the top was closed by a cork. The tubes were allowed to remain floating for some time in the tank in order to discover any leak. If they leaked, they were taken out and filled with water to discover the position of the leak, when the leak was soldered and the tube adjusted again. The centres of gravity of the tubes adjusted were 1.78 to 1.90 feet from their bottom ends; and thus being low, the tubes had a strong tendency to remain vertical.

The tubes were put into the water by an assistant standing on a bridge below the upper end of the flume, a thing requiring a little practice to do well; he stood with his face up-stream, with the tube in hand, the loaded end directed downwards, but slightly up-stream, holding it at an angle with the horizon, greater or less, depending upon the velocity of the current. At a signal he pushed the tube rapidly into the water at the angle at which he previously held it, until the painted mark near the upper end of the tube reached the surface of the water; he retained his hold of the upper end of the tube until the current brought it to a vertical position, when he abandoned it to the current.

There were three transit timbers placed across the flume, the middle one equidistant from the other two, their up-stream edges vertical, and distinctly graduated in feet from left to right. An assistant stood at each transit timber to note the transits, and the assistant at the middle transit timber observed the depth of water in the flume at each transit in a box close to him between

the lining planks and the wall of the canal, which communicated with the flume by a pipe about 4 feet above the bottom. The box contained a graduated scale, divided to hundredths of a foot, the zero point being at the mean elevation of the bottom part of the flume between the upper and lower transit timbers. The bottom of the flume was very nearly horizontal; the elevations to obtain the mean were taken at 32 points, giving an extreme difference observed of $\cdot 027$ feet in one case. The course of the tube, denoted by the distance in feet from the left side of the flume when the tube passes the transit timbers, was also observed and called out by the assistants; the mean course being obtained by adding the distances at the upper and lower transit timbers to twice that at the middle, and dividing the result by four for a mean distance.

The usual method of observing the transits was by means of an assistant carrying a stop-watch beating quarter-seconds, who walked down and recorded every transit himself; but when greater exactness was required, an electric telegraph made for the purpose was used, by which the transit observers communicated transits to a seated observer from their stations, the times of signals being noted by him to tenths of seconds according to a marine chronometer placed before him beating half-seconds:—an assistant was also required to carry back the tubes to the up-stream station. In the usual method before stated, a party of five was sufficient for all purposes. The observations were made at distances apart about 1.5 feet in the cross-section, as may be seen in the following gauge record for one set of observations; the mean velocities of the tubes for these mean distances were calculated and plotted on a diagram of

Gauge record of the quantity of water passing the Boott measuring flume, May 17, 1860, between 10:30 and 11:30 A.M. Length between transit timbers, 70 feet; breadth of flume, 41.76 feet; length of immersed part of tube, 8.4 feet.

| Position of tube at starting | Mean velocity of transit | Position of tube at upper transit | Position of tube at lower transit | Mean position | Depth of water in flume | Products of mean velocity and widths |
|------------------------------|--------------------------|-----------------------------------|-----------------------------------|---------------|----------------------------------|--------------------------------------|
| 0.0 | 2.102 | .3 | .8 | .55 | 8.510 | $2.073 \times 1 = 2.073$ |
| 1.5 | 2.258 | 1.8 | 1.6 | 1.70 | 8.481 | $2.193 \times 1 = 2.193$ |
| 3 | 2.318 | 3.2 | 2.1 | 2.65 | 8.450 | &c. 2.284 |
| 4.5 | 2.473 | 4.4 | 4.5 | 4.45 | 8.470 | 2.359 |
| 3 | 2.373 | 6.2 | 5.4 | 5.80 | 8.445 | 2.422 |
| 7.5 | 2.593 | 8.2 | 10.1 | 9.15 | 8.438 | 2.478 |
| 9 | 2.672 | 9.7 | 10.4 | 10.05 | 8.440 | 2.529 |
| 10.5 | 2.800 | 10.5 | 8.8 | 9.65 | 8.470 | 2.577 |
| 12 | 2.713 | 12.3 | 10.9 | 11.60 | 8.483 | 2.623 |
| 13.5 | 2.778 | 13.8 | 15.5 | 14.65 | 8.490 | 2.666 |
| 15 | 2.800 | 15.2 | 18.0 | 16.60 | 8.500 | 2.705 |
| 6.5 | 2.373 | 17.0 | 20.4 | 18.70 | 8.498 | 2.744 |
| 18 | 2.593 | 18.0 | 17.8 | 17.90 | 8.505 | 2.776 |
| 19.5 | 2.431 | 19.7 | 19.0 | 19.35 | 8.505 | 2.801 |
| 21 | 2.280 | 21.1 | 20.9 | 21.00 | 8.522 | 2.811 |
| 22.5 | 2.201 | 23.4 | 29.3 | 26.35 | 8.533 | 2.798 |
| 24 | 2.077 | 23.7 | 22.1 | 22.90 | 8.510 | 2.747 |
| 25.5 | 2.071 | 26.5 | 29.7 | 28.10 | 8.495 | 2.648 |
| 27 | 2.258 | 27.0 | 25.2 | 26.10 | 8.483 | 2.514 |
| 28.5 | 2.258 | 28.6 | 26.5 | 27.55 | 8.495 | 2.363 |
| 30 | 2.414 | 31.0 | 34.3 | 32.65 | 8.550 | 2.242 |
| 31.5 | 2.500 | 32.1 | 30.0 | 31.05 | 8.630 | 2.174 |
| 33 | 2.258 | 32.5 | 28.1 | 30.30 | 8.610 | 2.129 |
| 34.5 | 2.672 | 34.6 | 36.7 | 35.65 | 8.625 | 2.090 |
| 36 | 2.431 | 36.5 | 35.0 | 35.75 | 8.632 | 2.108 |
| 37.5 | 2.456 | 37.5 | 35.5 | 36.50 | 8.612 | 2.135 |
| 39 | 2.500 | 40.1 | 40.5 | 40.30 | 8.578 | 2.160 |
| 40 | 2.500 | 39.0 | 39.6 | 39.30 | 8.578 | 2.023 |
| 41 | 2.397 | 41.2 | 40.6 | 40.90 | 8.560 | 2.243 |
| 41.76 | — | — | — | — | — | 2.286 |
| | | | | | | 2.339 |
| 0.0 | 2.047 | .5 | .4 | .45 | 8.471 | 2.371 |
| 10 | 2.642 | 9.8 | 8.7 | 9.25 | 8.580 | 2.413 |
| 20 | 2.174 | 20.9 | 19.9 | 20.40 | 8.605 | 2.453 |
| 30 | 2.273 | 31.5 | 33.8 | 33.65 | 8.635 | 2.483 |
| 41 | 2.295 | 41.4 | 40.6 | 41.00 | 8.610 | 2.513 |
| 41.76 | — | — | — | — | — | 2.530 |
| | | | | Mean | 8.5204 | 2.541 |
| | | | | | | &c. 2.544 |
| | | | | | | $2.504 \times 1 = 2.500$ |
| | | | | | | $2.417 \times 1 = 2.417$ |
| | | | | | | $2.264 \times .76 = 1.721$ |
| | | | | | | Sum 101.523 |
| | | | | Mean | $\frac{101.523}{41.76} = 2.4311$ | |

Products of each foot of breadth

section paper having the mean widths in feet of the flume scaled on one side, and the other calculated velocities for those widths scaled on the other: a curve joining these points was then drawn on the diagram, from which the mean velocity for each foot in width of the flume was scaled off and entered in the record; from these the mean velocity due to the total width was obtained. In this case it was 2.4311 feet per second; and since the mean section of waterway between the upper and lower transit timbers was $= 41.76 \times 8.5294 = 356.188$ square feet, the approximate discharge $= 2.4311 \times 356.188 = 865.929$ cubic feet per second.

To obtain the true discharge from this approximate result, an empirical factor, depending on the difference (d) between the depth of water in the flume, and the depth to which the tube was immersed, divided by the depth of water in the flume, was applied: the expression of correction being $1 - 0.116 (d^{\frac{1}{2}} - 0.1)$. The value of this expression for various values of d is given in the table following at p. 148.

In this case d , the quantity before mentioned,

$$= \frac{8.5294 - 8.4000}{8.5294} = 0.0152;$$

and hence the true discharge

$$Q = 865.929 \times \left\{ 1 - 0.116 (\sqrt{0.0152} - 0.1) \right\} = 863.59.$$

Remarks.—These observations were made in a flume placed below a quarter bend in the canal, which caused the velocity to be much greater on one side than the other. To obviate this, an oblique obstruction was placed near the lower end of the bend, which removed

Table of correction for Discharges obtained from Tube Velocity observations, being values of the expression $1 - 0.116(d^{\frac{1}{2}} - 0.1)$ for different Values of d (from the Lowell Experiments).

| d | Correc- tion | d | Correc- tion | d | Correc- tion | d | Correc- tion | d | Correc- tion |
|------|-----------------|------|-----------------|------|-----------------|------|-----------------|------|-----------------|
| ·000 | 1·01160 | ·020 | ·99520 | ·040 | ·98840 | ·060 | ·98319 | ·080 | ·97879 |
| ·001 | 1·00793 | ·021 | ·99479 | ·041 | ·98811 | ·061 | ·98295 | ·081 | ·97859 |
| ·002 | 1·00641 | ·022 | ·99439 | ·042 | ·98783 | ·062 | ·98272 | ·082 | ·97838 |
| ·003 | 1·00525 | ·023 | ·99401 | ·043 | ·98755 | ·063 | ·98248 | ·083 | ·97818 |
| ·004 | 1·00426 | ·024 | ·99363 | ·044 | ·98727 | ·064 | ·98225 | ·084 | ·97798 |
| ·005 | 1·00340 | ·025 | ·99326 | ·045 | ·98699 | ·065 | ·98203 | ·085 | ·97778 |
| ·006 | 1·00261 | ·026 | ·99290 | ·046 | ·98672 | ·066 | ·98180 | ·086 | ·97758 |
| ·007 | 1·00189 | ·027 | ·99254 | ·047 | ·98645 | ·067 | ·98157 | ·087 | ·97738 |
| ·008 | 1·00122 | ·028 | ·99219 | ·048 | ·98619 | ·068 | ·98135 | ·088 | ·97719 |
| ·009 | 1·00060 | ·029 | ·99185 | ·049 | ·98592 | ·069 | ·98113 | ·089 | ·97699 |
| ·010 | 1·00000 | ·030 | ·99151 | ·050 | ·98566 | ·070 | ·98091 | ·090 | ·97680 |
| ·011 | ·99943 | ·031 | ·99118 | ·051 | ·98540 | ·071 | ·98069 | ·091 | ·97661 |
| ·012 | ·99889 | ·032 | ·99085 | ·052 | ·98515 | ·072 | ·98047 | ·092 | ·97641 |
| ·013 | ·99837 | ·033 | ·99053 | ·053 | ·98489 | ·073 | ·98026 | ·093 | ·97622 |
| ·014 | ·99787 | ·034 | ·99021 | ·054 | ·98464 | ·074 | ·98004 | ·094 | ·97604 |
| ·015 | ·99739 | ·035 | ·98990 | ·055 | ·98440 | ·075 | ·97983 | ·095 | ·97585 |
| ·016 | ·99693 | ·036 | ·98959 | ·056 | ·98415 | ·076 | ·97962 | ·096 | ·97566 |
| ·017 | ·99648 | ·037 | ·98929 | ·057 | ·98391 | ·077 | ·97941 | ·097 | ·97547 |
| ·018 | ·99604 | ·038 | ·98899 | ·058 | ·98366 | ·078 | ·97920 | ·098 | ·97529 |
| ·019 | ·99561 | ·039 | ·98869 | ·059 | ·98342 | ·079 | ·97900 | ·099 | ·97510 |
| | | | | | | | | ·100 | ·97492 |

all the trouble in measurement due to the original irregularity ; the other remaining irregularities may be seen by plotting a diagram of the velocities. It is hence advisable in all cases to equalise the velocities on each side of the axis, should they require it.

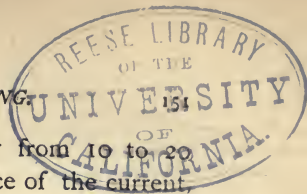
In gauging a branch canal it is best to put the flume in it near its off-take from the main canal, with its axis nearly parallel to that of the branch canal. Its section may be determined by roughly calculating the expected discharge, and making it so as to suit a velocity of from 1 to 3 feet per second ; its length should not be less than 50 feet, allowing 20 feet above the upper transit timber to enable tubes to attain the same velocity as the water, and 5 feet below the lower timber, the transit course of 25 feet, run over in $7\frac{1}{2}$ or 10 seconds, can be then noticed by a practised observer with a quarter-second stop-watch.

In gauging rivers by means of loaded tubes, flumes are dispensed with, and marked cords may be substituted for the graduated transit timbers, being supported from the bottom if necessary, so as to be always visible ; in large rivers triangulation observations are necessary. The reach should be 50 to 100 feet long, and the bottom irregularities may be removed or filled in to a certain extent beforehand, so as not to interfere with the poles, which should, when immersed, reach to about six inches from the bottom. Boats will be required to convey the poles. As the cross-section may be irregular, it will be necessary to divide it into several parts, finding the area and mean velocity for each division, and calculating the corrected discharge for each division separately ; the sums of these corrected discharges will then be the true discharge for the river at that spot.

6. FIELD OPERATIONS FOR GAUGING THE MISSISSIPPI RIVER AND TRIBUTARIES, BY CAPTAINS HUMPHREYS AND ABBOT IN 1858.

Soundings.—The strength of the current, the depth and width of the river, and the floating driftwood, all combined to render an accurate measurement of the dimensions and area of cross-sections a difficult operation on the Mississippi. After various experiments, the following system was adopted, by which accurate work was done even in the highest stages of the river. The middle stages were usually selected for this purpose, being preferable to the low stages, during which there would have been exposure to oppressive heat and disease, and more favourable than the high stages, when the difficulties attending accurate measurement were greatest.

Preparatory to making a cross-section of the river, whether for general purposes of comparison or for determining a discharge, a base line, varying in length from 400 to 1 000 feet, was measured along the bank near the water's edge; an observer with a theodolite was stationed at each extremity of this line. The one directed the telescope of his instrument across the river, so as to command the line on which the soundings were to be made; the other prepared to follow the boat with his telescope, in order to measure its angular distance from the base line when each sounding was taken. The boat, a light six-oared skiff, contained a man provided with a sounding chain, a recorder with a flag, and three oarsmen. The strongest kind of welded jack-chain was employed, to which bits of buckskin were attached at intervals of 5 feet, smaller divisions being measured with



a rod in the boat. The sinker, varying from 10 to 20 pounds in weight according to the force of the current, was a leaden bar whose bottom was hollowed out and armed with grease, in order to bring up specimens of the bed of the river. The patent lead was also used for the latter purpose. The boat was rowed some little distance above the proposed section line, and allowed to drift down with the current, the sounding lead being lowered nearly to the bottom. By this precaution, the deflection of the line by the force of the current was prevented. When the first observer, stationed opposite the proposed section line, saw that the boat had nearly reached it, he waved a flag as a signal to take a sounding, and then carefully turned his instrument so as to keep the vertical hair of his telescope upon the point where the chain crossed the gunwale of the boat. The recorder in the boat, seeing the signal, waved his flag to the second engineer to follow the boat carefully with his telescope. The man with the sounding chain allowed it to slip rapidly through his hands until the lead struck the bottom, when he grasped the chain at the water surface, and instantly rose to a standing position. This motion was the signal for arresting the movement of each telescope, and recording the angles. The recorder in the boat noted the depth of the water, and the nature of the bottom soil adhering to the lead. By the angles measured at the base line, the exact position of the sounding, which was never more than a few feet above or below the proposed section line, was ascertained. The process was repeated until soundings enough had been taken to give an accurate cross-section of the river. Careful lines of level were then run up each bank from the water surface to points above the level of the highest

floods, when such points existed, or to other convenient bench-marks. Generally, the triangles were computed, and the work plotted before leaving the place, in order to fill by additional soundings any gaps which might appear on the diagram.

At places where a series of daily velocity observations was to be made, additional precautions were taken, and two independent sections, 200 feet apart, were sounded with the greatest care. Soundings, repeated from time to time upon these lines, uniformly showed that no sensible changes took place in the bed of the river. The mean of all such sections, when reduced to the same stage of the river, was accordingly always taken for the true cross-section at the locality. The change in area produced by any change of level in water surface could then be readily computed from the plotted section. To determine the daily changes of this level, a gauge-rod, graduated to feet and tenths, was observed daily, its correctness of adjustment being frequently tested by comparison with secure bench-marks. An accurate knowledge of the area of the cross-section on any given day was thus obtained. The tables of soundings for each cross-section, which were all numbered, also denoted the distance of the sounding from the base line, the depth of high water during that year, and the nature of the bottom.

Velocity Measurements.—Narrow and straight portions of the river, where the form of its cross-section approximated most nearly to that of a canal, where the waters of the highest floods were confined to the channel by natural banks or by levées, and where the river at all stages was free from eddies, were selected for the permanent velocity stations.

The depth and violence of the river rendered the measurement of its velocity, especially below the surface, exceedingly difficult. Of all the methods known for determining this quantity, that by double floats was found to give the best results. The method of conducting these observations was as follows:—Two parallel cross-sections of the river having been made as already explained, 200 feet apart, a base line of the same length was laid off upon the bank from one to the other, being of course at right angles to both. This length was sufficient to ensure accuracy without being too great either for observing many floats in a day, or for avoiding local changes in velocity. An observer with a theodolite was stationed at each extremity of the base line. It is evident that, when the telescopes were directed upon the river, with their axes set at right angles to the base line the vertical cross hairs marked out the lines of sounding upon the water surface, and that the time of passage of a float between these lines was that consumed in passing 200 feet. Also, that if the angular distance of a float from the base line when crossing each line of sounding was measured, its distance in feet from the former could readily be computed, and its path fixed. Upon these principles the observations were conducted. Two skiffs were stationed on the river, one considerably above the upper, and the other below the lower section line, the former being provided with several keg floats. At a signal from the engineer at the upper station, whose telescope was set upon the upper section line, a float was placed in the river. The keg immediately sank to the depth allowed by its cord, and the whole float moved down toward the lower line. The observer at the lower station followed its motion, keeping the cross hair of his telescope

directed constantly upon the flag. At the word 'mark' uttered by his companion, when the float crossed the upper line, he recorded the angle shown by his instrument, and then, setting his telescope upon the lower line, watched for the arrival of the float. In the meantime, the observer at the upper station, whose theodolite supported a watch with a large seconds hand, recorded the time of transit of the float across the upper line, and then followed the flag with his telescope. At the word 'mark' given by his assistant, when the flag crossed the lower line, he recorded the line and angular distance from the base line. The float was picked up by the lower boat. By this method, the exact point of crossing each section line, and the time of transit, were ascertained. When the velocity was not too great, the time was noted by the engineer at the lower station also, to guard against error. A stop-watch was sometimes used. As it was evidently impossible to observe floats daily in all parts of the cross-section, the best practical method found was to adopt a uniform depth of 5 feet for all the floats, distribute them equally across the entire river, and afterwards divide the resulting velocities into groups or divisions within which the variation of velocity was but slight; a mean relative velocity, and a mean relative discharge, for each division was then computed, the sum of the latter being an approximate mean discharge of the river, which, when divided by the area of the whole river section, gave a mean relative velocity for the whole river. The resulting discharge, when multiplied by the ratio of the velocity at the assumed depth (in this case 5 feet) to the mean velocity for the whole vertical curve, gave an accurate mean discharge of the river for that place and day.

Computation of Discharge.—A separate plot of each day's velocity measurements was made in the following manner:—Lines were drawn upon section paper to represent the section lines, the base line, and the water edges. The distances from the base line to the points where each float crossed the section lines were then computed by a table of natural tangents, and the points laid down on the plot. Straight lines connecting the two corresponding points indicated the paths of the floats, which were of course nearly perpendicular to the section lines. The time of transit in seconds and the depth of the float were inscribed upon these plotted paths.

The diagram resulting showed that the velocities in different parts of the section increased gradually and quite uniformly with the distance from the banks until the thread of the current was reached, and, since these velocities were found to vary but very slightly for distances of 200 feet apart except in the immediate vicinity of the banks, the diagram of the daily velocity floats was divided by parallel lines 200 feet apart, the first being the base line, and the mean of all the velocities of floats in each division taken as the mean relative velocity for that division and recorded. For the shore divisions, unless the floats happened to be well distributed through them, the mean relative velocity was assumed to be eight-tenths of that in the outer edge; a rule deduced from a subdivision and study of the velocity when thoroughly measured in these divisions.

For checking and making interpolations among defective observations of any day in a division, the day's work was also plotted in a curve whose ordinates were the mean velocities of the different divisions, and whose

abscissæ were the distances of their middle points from the base line.

The river channel being of a natural form, the sectional areas of all the divisions were unequal, and again the ratios of these areas were not constant for different stages of the river. Each divisional area was therefore multiplied by its mean relative velocity, and the sum of the products was then the mean relative or approximate discharge of the whole section; dividing this discharge by the total area of the whole section, the approximate mean velocity of the river was determined. This computation was made by logarithms, and simplified by the use of a table constructed for the purpose. In order to correct these discharges, which were those due to the velocities five feet below the surface, it was necessary to determine the value of the ratio,¹

$$\frac{U_m}{\bar{U}_5} = \frac{U_m}{U_m + \left[\frac{1}{3} + \frac{(.317 + .06f)(10r - r^2) - 25}{r^2} \right] \sqrt{bv}}$$

and multiply them by it, thus getting the true discharges, which, when divided by their corresponding areas of cross-section, gave the final and correct mean velocity. The numerical values of the above expression or ratio were obtained in the following way, and put into the form of the table given.

The days on which observations were made were grouped according to even feet of the computed approximate mean velocities, it being assumed that the effect upon the desired ratio, produced by changes in mean velocity of less than one foot, might be neglected. Each group was then examined in connection with the

¹ See Mississippi velocity notation, page 12, Chapter I.

wind record, and days were rejected until only calm days or those on which the wind blew directly across stream, or those on which when combined the wind effects balanced each other, were left. The resulting mean day in each group was then equivalent to a calm day, so far as wind effect was concerned. The following mean quantities were then deduced for each mean day by dividing the sum of the quantities by the number of days going to make up the mean day, viz., an approximate mean velocity of the river (v), a gauge reading, and hence a mean radius (r), and a mean velocity five feet below the surface (U), found by taking a mean of the tabulated velocities of all the different divisions.

These values being substituted in the equation,

$$U = U_{a'} - (0.1856 v)^{\frac{1}{2}} \left(\frac{d - d'}{r} \right)^2$$

putting also $d = 5$, making $d' = 0.317r$, and $b = \frac{1.69}{(D + 1.5)^{\frac{1}{2}}}$
 $= 0.1856$ when $D = 730$; the value of $U_{a'}$ was computed and obtained.¹

Next this value of $U_{a'}$ was introduced into the same equation again to obtain new values of U , first for a value $d = 0$, secondly for a value of $d = r$, thus getting the surface and bottom velocities denoted by U^o and U_r . Substituting for these their values in the following equation, together with those computed for $U_{a'}$, d' , and r , the value of U_m was obtained

$$U_m = \frac{2}{3} U_{a'} + \frac{1}{3} U_r + \frac{d'}{r} \left(\frac{1}{3} U - \frac{1}{3} U_r \right)$$

¹ N. B.—The general value of b is $\frac{1.69}{(R + 1.5)^{\frac{1}{2}}}$.

Table of Ratios for correcting the approximate discharges of the Mississippi.

| LOCALITY | Approximate mean velocity of river | Wind down 4 | Wind down 3 | Wind down 2 | Wind down 1 |
|-----------|------------------------------------|-------------|-------------|-------------|-------------|
| | Feet | | | | |
| Columbus | 1.6826 | .90759 | .92250 | .93791 | .95390 |
| | 2.4440 | .92202 | .93519 | .94874 | .96273 |
| | 3.6548 | .93719 | .94826 | .95917 | .97118 |
| | 4.5097 | .94400 | .95407 | .96428 | .97463 |
| | 4.3426 | .94908 | .95829 | .96809 | .97741 |
| | 6.6496 | .95406 | .96261 | .97131 | .98016 |
| | 7.4282 | .95751 | .96550 | .97365 | .98193 |
| | 8.3162 | .95983 | .96747 | .97523 | .98311 |
| Vicksburg | 3.6038 | .93881 | .94854 | .95846 | .96863 |
| | 4.4110 | .94544 | .95458 | .96423 | .97340 |
| | 5.5571 | .95161 | .96017 | .96895 | .97783 |
| | 6.7363 | .95631 | .96440 | .97264 | .98103 |
| | 7.0529 | | | | |
| Natchez | 4.6901 | .94566 | .95501 | .96454 | .97428 |

| LOCALITY | Approximate mean velocity of river | Calm | Wind up 1 | Wind up 2 | Wind up 3 | Wind up 4 |
|-----------|------------------------------------|--------|-----------|-----------|-----------|-----------|
| | Feet | | | | | |
| Columbus | 1.6826 | .97040 | .98750 | 1.00521 | 1.02357 | 1.04262 |
| | 2.4440 | .97737 | .99192 | 1.00721 | 1.02294 | 1.03923 |
| | 3.6548 | .98302 | .99521 | 1.00767 | 1.02048 | 1.03359 |
| | 4.5097 | .98546 | .99641 | 1.00760 | 1.01903 | 1.03058 |
| | 4.3426 | .98723 | .99727 | 1.00689 | 1.01793 | 1.02858 |
| | 6.6496 | .98918 | .99837 | 1.00773 | 1.01727 | 1.02697 |
| | 7.4282 | .99035 | .99891 | 1.00762 | 1.01648 | 1.02551 |
| | 8.3162 | .99112 | .99927 | 1.00756 | 1.01598 | 1.02453 |
| Vicksburg | 3.6038 | .97895 | .98956 | 1.00037 | 1.01142 | 1.02271 |
| | 4.4110 | .98310 | .99300 | 1.00307 | 1.01337 | 1.02389 |
| | 5.5571 | .98693 | .99613 | 1.00557 | 1.01518 | 1.02494 |
| | 6.7363 | .98952 | .99823 | 1.00706 | 1.01604 | 1.02519 |
| | 7.0529 | .99006 | | | | |
| Natchez | 4.6901 | .98420 | .99433 | 1.00466 | 1.01522 | 1.02602 |

A calm or wind at right angles to the current = 0 ; a hurricane = 10.

Using the resulting value of U_m , also the values already deduced for v and r and b , and giving f its value successively for each of the various forces and direction of the wind, in the following equation :—

$$\frac{U_m}{U_5} = \frac{U_m}{U_m + \left[\frac{1}{3} + \frac{(0.317 + 0.06f)(10r - r^2) - 25}{r^2} \right] (bv)^{\frac{1}{2}}}$$

the table of ratios for the stations was computed.

The approximate discharge for each day at each station was multiplied by the ratio in the table most nearly corresponding to its approximate mean velocity to obtain the true discharge, from which the true mean velocity was then obtained.

7. FIELD OPERATIONS IN GAUGING CREVASSES BY CAPTAINS HUMPHREYS AND ABBOT.

The phenomena observed in the discharge of water, through crevasses, or breaks in levées at seasons of high water, were—

1. That the effect of every crevasse, even though as large as 327 feet wide and 15 feet deep, along the line of levée, extends only for a short distance from the bank ; in the above instance, it did not affect the line of motion of floating bodies passing 200 feet from the natural bank, or 300 feet from the break in the levée.

2. Between the crevasse and the outer limit of its influence there is always a movement of the water towards the break from all points below and above, which increases towards the break, and rapidly diminishes on

reaching the ground in rear of the levée, where it spreads in every direction, but mostly towards the swamps.

3. There is a sensible slope along the course of this movement.

4. In passing the break, whether by a cascade or not, the water is higher in the middle of the opening than at either side.

The following was the ordinary method of computing a discharge. Knowing, from measurements made after the cessation of the flow, the high-water depth of the given crevasse, which was estimated on the line of levée, if no material excavation was made there, and on the batture in front of the levée, if holes were dug on the line of the break; the depth on the given day was found by subtracting from this high-water depth the stand of the river below high-water mark—a quantity which was always known either from local information or from a comparison of the nearest river gauges. Taking D to represent this depth, and w the maximum width of the crevasse after cessation of flow; and knowing from exact information the date of breaking of the levée, and that of the cessation of flow, the width of crevasse of any desired day could be computed; and the required discharge per second was then assumed to be equal to the continued product of this width w , the depth D , and the velocity (v); or $Q = w \times D \times v$; the velocity when D was less than 4 feet was taken $= 2.818 \sqrt{D}$ (Castel's weir formula); and when D was greater than 3 feet, v was taken $= 10 - \frac{17}{D}$; the general formulæ for discharge corresponding to each case being

$$Q = (100 + n - 4) \left(\frac{w - 100}{N - 5} \right) D (2.818 \sqrt{D})$$

$$Q = (100 + \overline{n-4}) \left(\frac{w_i - 100}{N-5} \right) D \left(10 - \frac{17}{D} \right)$$

where n = numbers of days of discharge which have preceded the given day, and N = total number of days of discharge.

Coefficient of correction for special cases of crevasses.

There are cases in which the conditions of the flow of water were considerably modified ; such as when the levée was so far distant from the river that the depth at the edge of the natural bank was much less than that at the base of the levée ; or when trees, a growth of saplings, or other obstacles existed in front or in rear of the break, both of these causing a diminution of discharge. So when the reported depth of crevasse included that of previously existing excavations on the line of levée, in these cases the resulting calculated discharge would be too high, and it then became necessary to apply in each case a special coefficient of correction. The coefficient for crevasses flowing into the Yazoo bottom was thus determined. The areas of these bottom lands and their watersheds were as follows, in square miles :—

| | | |
|---|------|----------|
| Yazoo bottom | 7110 | Total. |
| Yazoo watershed | 6740 | |
| St. Francis' bottom | 6900 | } 34 600 |
| St. Francis' watershed | 3600 | |
| Tennessee and Kentucky bottom | 750 | |
| Tennessee and Kentucky watershed. | 9500 | |

The yearly rainfall in feet was—

| | |
|---|------|
| At New Harmony, Indiana | 3'92 |
| At West Salem, Illinois | 4'02 |
| At St. Louis, Missouri | 5'18 |
| Mean downfall at head of region | 4'38 |
| At Memphis, downfall for middle of region | 4'42 |
| At Jackson, downfall for foot of region | 4'99 |

Mean for whole region feet
 4.60

Giving total yearly downfall,

$$= 34\,600 \times 4.6 \times (5280)^2 = 4\,437\,126\,144\,000 \text{ cubic feet.}$$

To obtain the total yearly drainage, the discharge at Columbus, together with that of the Arkansas and White Rivers, was deducted from the discharge at Vicksburg; and from this also a deduction was made an account of the river during the year between Columbus and Vicksburg being lower by a mean difference of 6.8 feet throughout a mean width of 3 300 feet for 589 miles in length; thus getting the drainage

| | |
|-----------------------|-----------------------------|
| | 4 372 572 757 200 |
| Channel drainage | <u>69 786 604 800</u> |
| Total yearly drainage | 4 302 786 152 400 cubic ft. |

And ratio of drainage to downfall is hence

$$= \frac{4\,302\,786\,152\,400}{4\,437\,126\,144\,000} = 0.96 \text{ nearly.}$$

Next, the total rainfall for the Yazoo basin, area 13 850 square miles, for from December 1, 1857, to July 15, 1858 = 3.64 feet $\times 13\,850 \times (5\,280)^2 = 1\,405\,461\,657\,600$ cubic feet; the mean rainfall 3.64 during that time being determined from the mean of the registered falls at Memphis, and at Jackson, 3.19 and 4.08 feet; applying to this rainfall the coefficient of drainage before determined, the drainage from the Yazoo basin = 1 349 243 191 300 cubic feet.

The area of the Yazoo bottom was dry on December 1, 1857, but at high water July 15, 1858, it had a mean depth of water of 3.08 feet over an area of 6 800 square miles; having received between those dates $6800 \times$

$(5.280)^2 \times 3.08 = 583\ 885\ 209\ 600$ cubic feet, and the discharge of the channel of the Yazoo, the sole outlet, was measured during this time = $1\ 408\ 665\ 600\ 000$ cubic feet. Hence, $1\ 992\ 550\ 809\ 600$ cubic feet represented the total quantity which, entering the Yazoo basin between those dates, eventually drained off into the Mississippi; and the total amount of overflow from the Mississippi basin into the Yazoo basin was $1\ 992\ 550\ 809\ 600 - 1\ 349\ 243\ 191\ 300 = 643\ 307\ 618\ 300$ cubic feet; this quantity as computed by the uncorrected crevasse formula was—

$$1\ 758\ 153\ 600\ 000 ;$$

hence the required coefficient of correction for the formula equals the former divided by the latter = nearly $\frac{1}{3}$. This, therefore, holds good for the crevasses in the district for which it is obtained, and the same principle may be applied to other districts.

8. SYSTEM OF GAUGING BY MID-DEPTH VELOCITIES, PROPOSED BY HUMPHREYS AND ABBOT.

The details of field operation to be adopted differ according to the size of the river. 1st. If the river be small and considerable exactness be required, the boat should be anchored at various equidistant stations, the banks being considered two of them, and the station actual mid-depth velocities measured by any of the known methods; the number of stations being sufficient to prevent the velocity of the water between any two of them from varying materially. 2nd. In the case of a large river, if the depth is uniform, sufficient accuracy

may be obtained by observing the times of transit of a large number of double floats well distributed across the river section, the kegs being uniformly sunk beneath the surface to a depth equal to half the hydraulic mean radius of the river. Should it happen that the cross-section is not sufficiently uniform and symmetrical to admit of this, the site or reach is ill chosen for the purpose. The results should then be plotted and grouped into divisions of equal width, and the mean result for each division calculated, including, of course, interpolated velocities should any be missing.

The depth of water in the river should be noted on a permanent gauge-post during the observations, or before and after. By this method the results obtained will be in the first case absolutely, and in the second case nearly, unaffected by the wind, no matter what its direction or force may be.

The method of computing the discharge from these observations will vary according to the accuracy required.

First method.—A close approximate result may be obtained by taking a mean of all the different station or division mid-depth velocities, and applying a coefficient of 0.95 for large, and 0.93 for ordinary rivers, to obtain the mean velocity of the river. In this method there are two causes of error which very nearly balance each other, namely, the inequality in area of the different divisions, and the difference between the mid-depth and mean velocities in any vertical plane, and the above coefficients meet those errors. For a rectangular cross-section, no coefficient is required.

Second method.—If greater precision be required, a more accurate mean velocity of discharge of the river

(V) may be computed by substituting the grand mean of all the station mid-depth or division velocities for $U_{\frac{1}{2}r}$ in the following formula,

$$v = [(1.08 U_{\frac{1}{2}r} + 0.002b)^{\frac{1}{2}} - 0.045b^{\frac{1}{2}}]^2$$

This formula is deduced by substituting for U_m its value $0.93v$ in the general expression,

$$U_{\frac{1}{2}r} = U_m + \frac{1}{12}(bv)^{\frac{1}{2}}$$

and reducing the resulting equation.

As has been already stated, when the mean radius exceeds 12 feet, $b = 0.1856$, and under any circumstances

$b = \frac{1.69}{(r + 1.5)^{\frac{1}{2}}}$. The formula therefore gives at once v

the mean velocity of the river; and this simple method is quite exact in ordinary river sections, though not applicable to rectangular sections.

Third method.—Should, however, a very high degree of accuracy be required for testing formulæ, or constant coefficients, an amount of exactitude affected only by instrumental errors of observation may be secured by substituting the different observed division mid-depth velocities successively for $V_{\frac{1}{2}D}$ in the formula

$$V_m = V_{\frac{1}{2}D} - \frac{1}{12}(bv)^{\frac{1}{2}}$$

and the results will be true values of the mean velocities of the different divisions in terms of $v^{\frac{1}{2}}$ and known quantities. The sum of the products of these expressions, by the corresponding division areas, should be placed equal to the product of v by the total area of the cross-section;

and this equation, involving v and $v^{\frac{1}{2}}$ and known quantities, will give two positive values of v ; the less of which, corresponding to the actual case when the velocity is greater at the axis, is the value of the true mean velocity of the river. This method, though accurate in principle, is probably not so good for ordinary purposes as the previous more simple one, which neglects the latter attempt at extreme accuracy and involves less observation, and consequently less instrumental error, as well as less labour.

General Abbot's Method of determining on any given day the discharge of a large river that has been previously surveyed and gauged.

The previous field operations consist of a survey and numerous soundings of a straight and regular portion of the channel between two bench-marks, A and B, fixed permanently near the water, whose relative levels are accurately known. An accurate plan of the river between these points is necessary, the mean cross-section derived from the soundings, and a series of careful gaugings of the river on permanent gauge-posts. It is desirable that the course of the river between A and B should be as straight and regular as possible, in order to eliminate to the utmost the effect of bends, although allowances almost invariably must be made on that account. The points A and B should be well chosen, as far apart as practicable, and distant from any eddy, and be placed where the current on the bank flows with equal velocities. The latter condition is necessary, because water in motion exerts less pressure than when at rest, and if it moves rapidly past one bench-mark, and is nearly

stationary at the other, a difference of level independent of the motive power of the stream would vitiate the observations.

On the required day the water surface at each end of the reach, A and B, has to be simultaneously referred by accurate levels to the bench-marks, to obtain the difference of level of water surface and the gauge depths. Nothing more is required. A calm day should be selected.

The formula to be used is that given in the paragraph on velocities :

$$v = [\sqrt{0.0081b + (225 r, \sqrt{s})^2} - 0.09b^{\frac{1}{2}}]^{\frac{1}{2}}$$

the terms of which have been already explained, excepting s ; in this case s is the sine of the slope of the water surface corrected for bends, and is obtained numerically by subtracting the value of h'' , due to effect of bends (*vide* Paragraph on Bends) from the total fall between the level stations, and dividing the difference by the total distance between them, measured on the middle line of the channel.

The method of successive approximation must be adopted to find the value of v in this formula. The following formulæ give the value of each variable in the above equation in terms of the others and known quantities. Taking $Z = 0.93v + 0.167 \sqrt{bv}$, and assuming $p = 1.015 W$, should it not have been measured,—then

$$s = \left(\frac{Z^2}{195r} \right)^2 ; \quad a = \frac{(p+W) Z^2}{195 \sqrt{s}} ; \quad r, = \frac{a}{p+W} ;$$

$$\text{and } p + W = \frac{195 a \sqrt{s}}{Z^2}.$$

For small streams.—General Abbot modifies the above formula into the following, where v' is the value of the first term in the expression for v —

$$v = \left\{ \sqrt{0.0081b + (225r, \sqrt{s})^{\frac{1}{2}} - 0.09 \sqrt{b}} \right\}^2 - \frac{2.4 \sqrt{v'}}{1+p}$$

or putting $M = 0.0081b$ and $M' = \frac{2.4}{1+p}$

$$v = \left\{ \sqrt{M + 225 r, \sqrt{s} - \sqrt{M}} \right\}^2 - M' \sqrt{v'}$$

in which the term involving M' may be neglected, for streams larger than 50 or 100 feet in cross-section; and for large rivers exceeding 12 or 20 feet in mean radius, M but not \sqrt{M} may be neglected. The following table facilitate the application of the formula.

| r | M | \sqrt{M} | p | M' | Log. M' |
|-----|--------|------------|-----|-------|-----------|
| 1 | 0.0037 | 0.0930 | 5 | 0.400 | 9.602060 |
| 2 | 0.0073 | 0.0855 | 6 | 0.343 | 9.535294 |
| 3 | 0.0065 | 0.0803 | 7 | 0.300 | 9.477121 |
| 4 | 0.0058 | 0.0764 | 8 | 0.267 | 9.426511 |
| 5 | 0.0054 | 0.0733 | 9 | 0.240 | 9.380211 |
| 6 | 0.0050 | 0.0707 | 10 | 0.218 | 9.338456 |
| 7 | 0.0047 | 0.0685 | 12 | 0.185 | 9.267172 |
| 8 | 0.0044 | 0.0666 | 14 | 0.160 | 9.204120 |
| 9 | 0.0042 | 0.0649 | 16 | 0.141 | 9.149219 |
| 10 | 0.0040 | 0.0634 | 18 | 0.126 | 9.100371 |
| 12 | 0.0037 | 0.0610 | 20 | 0.114 | 9.056905 |
| 14 | 0.0035 | 0.0590 | 22 | 0.104 | 9.017033 |
| 16 | 0.0033 | 0.0573 | 24 | 0.096 | 8.982271 |
| 18 | 0.0031 | 0.0558 | 26 | 0.089 | 8.949390 |
| 20 | 0.0029 | 0.0544 | 28 | 0.083 | 8.919078 |
| 30 | 0.0024 | 0.0494 | 30 | 0.078 | 8.892095 |
| 50 | 0.0019 | 0.0437 | 50 | 0.047 | 8.672098 |
| 100 | 0.0013 | 0.0369 | 100 | 0.024 | 8.380211 |

9. THE EXPERIMENTS OF D'ARCY AND BAZIN ON THE RIGOLEs DE CHAZILLY AND GROSBOIS IN 1865.

These experiments, in small channels under various conditions, were made with the principal object of obtaining coefficients of reduction due to various surfaces of bed and banks; their details cannot fail to be interesting to those intending to gauge channels of any description.

The canal of supply was Bief No. 57, of the Canal de Bourgogne, from which the water was taken into a receiving chamber through four iron sluices, 1^m wide, and being capable of being raised 0.40^m, having their sills 0.60^m below ordinary water level of the canal. This chamber was 5.40^m wide by 14.00^m long, having its bottom 0.80^m below the entrance sills; the gauge-sluices opening from it into the channel of experiment were of brass, twelve in number, each having a section of passage when opened of 0.20 × 0.20^m, and having their sills 0.40^m above the bottom of the chamber, and 0.40^m below the sills of the entrance sluices before mentioned. These orifices resemble those of the type employed by Poncelet and Lesbros, and would, according to them, require a coefficient of reduction of discharge of 0.604, provided that the effect of the velocity of approach be neglected; in this case, however, it augmented the discharge, and an allowance had to be made on that account. The water in the chamber was constantly kept at a level of 0.80^m above the centre of the gauge-sluices; an appliance for showing the slightest variation of its level being continually watched by a sluice-keeper.

The channel of experiment was 450^m long before it commenced to bend towards the river Ouche ; it was water-tight, and was lined with planks of poplar : its fall for the first 200^m was 0·0049 per mètre, and for the next 250^m was 0·002 per mètre up to the bend, after which its fall to the river for the remaining 146^m was 0·0084 per mètre. The different provisional constructions for employing various inclinations, and sections of different forms, were made in plank within this channel, the spaces being filled with rammed stiff earth. Nails were driven into the bottom of the channel at various points to serve as bench-marks, from which every variation in depth of water could be obtained with exactitude. Most of the experiments were made by successively opening the twelve gauge-slucices, having one fixed section and amount of supply in each case, and thus twelve results were obtained for comparison in every experiment conducted.

The velocities were principally observed with d'Arcy's current-meter, but in some cases also with floats. The latter were sometimes simple wafers, and sometimes pieces of wood or cork weighted with lead, 2½ inches in diameter, and 1 inch thick ; their times of transit over distances of from 40 to 50 mètres were noted with chronometers indicating fifths of seconds, and the mean of five or more observations, in which the float following the course of the axis of the channel was adopted as finally correct.

The following was the mode of determining the measurement of discharge at the off-take.

The coefficient of discharge at the four entrance sluices was determined by closing the lower sluices and

noting the time in which the former filled the chamber to a certain height ; in this way the following coefficients were obtained for a head on the sill of from 0·55^m to 0·70^m, when one single sluice was opened at a time.

| Sluice raised. | Coefficient. |
|-----------------------------|--------------|
| 0·10 ^m | 0·645 |
| 0·20 ^m | 0·639 |
| 0·30 ^m | 0·631 |
| 0·40 ^m | 0·621 |

When the four sluices were opened at once to the full height 0·40^m, the coefficient was 0·637, instead of 0·621.

It was hence evident that, in order to obtain a sufficiently constant discharge, the use of the second set of twelve sluices became absolutely necessary. The conditions of construction of the latter did not, however, render the contraction complete, and hence the coefficients of Poncelet and Lesbros were not applicable to them. In order to have effected this, a chamber large enough to entirely annihilate all velocity would have been necessary, the sluices should have been farther apart, and their sills should have been at least 0·60^m above the bottom of the chamber. It was hence necessary also to determine the coefficients of discharge for these sluices by direct observation.

In June 1857, experiments were made with this object ; a portion of the channel was closed up, and filled by opening one, two, three, &c., up to twelve sluices at a time, and the volumes thus discharged in a certain time carefully measured. The discharges per second were in these cases from 0·103 to 1·242 c.m. ; and when each sluice was opened separately the discharges varied between 0·1022 and 0·1057 c.m., giving coefficients varying from 0·645 to 0·658. The irregularity of the latter was considered due to the irregularity of form of the

bottom of the portion of channel filled not allowing the exact volume to be calculated: hence a mean coefficient of 0.650 was adopted provisionally for any number of sluices open at one time. In 1860, it was determined to obtain this coefficient with greater exactitude, and further experiments were made: all the practical details were carefully reinvestigated: the influence of the variations in depth of the bief or canal of supply was eventually found to exercise no effect on the irregularities; the gauge used was supplanted by a glass tube having a mouthpiece of 1 millimètre in diameter, by means of which variations in depth of water as small as 1 millimètre could be easily read. The results under these conditions were thus:—

| | |
|--|-------|
| For a discharge from 1 sluice, the coefficient was | 0.633 |
| 2 sluices, „ | 0.642 |
| 3 „ „ | 0.646 |
| 4 „ „ | 0.649 |
| 5 „ and upwards to 12 | 0.650 |

For a sluice raised only 0.10^m instead of being fully opened, the coefficient was found to depend on the number of other sluices open, thus:—

| | |
|---|-------|
| When 1 other is opened full, the coefficient for the partly | |
| opened one is | 0.650 |
| 2 | 0.657 |
| 3 | 0.660 |
| 4 | 0.662 |
| 5 and upwards | 0.663 |

The determination of the coefficient for reduction for the current-tube.

This was effected by three methods—

1st.—By comparing the velocities obtained by means of the tube with the surface velocities shown by floats.

The data according to the floats were obtained in channels two mètres wide, having a discharge furnished by five sluices open at a time: the results gave a coefficient varying from 0·981 to 1·039 as extremes, and 1·006 as the mean of all.

2nd.—By moving the instrument at a known velocity in a mass of still water. The floats and the current-tube were drawn by men for a distance of 450 mètres, each 50 mètres furnishing a set of observations; the obliquities of the course of traction furnished the principal obstacle to arriving at a very exact result. The velocities employed varied from 0·609 to 2·034 mètres, giving coefficients of reduction varying from 1·015 to 1·053 as extremes, the general mean of all being 1·034: this was considered far too high, and the results of this set of observations were therefore entirely discarded.

3rd.—By measuring by means of the current-tube the velocities at a great number of points in the transverse section of the channel, and comparing the discharge calculated from these velocities with that determined by the experiments previously described; the points referred to were distributed rectangularly in vertical and horizontal lines; the discharge of each rectangle was calculated, and the sum of these discharges was employed to obtain an approximate discharge of the canal. These comparisons gave results varying from 0·968 to 1·029 as extremes, the general mean of all being 0·993.

The mean of the means obtained by the first and third methods gave a coefficient of nearly unity, which was therefore adopted for the instrument under trial.

Having thus securely determined the amount of discharge passing down the canal of experiment at any time, the levels of the water surface and its inclination

being attainable also at any time with exactitude, the sectional area at any point being also known, and the coefficient of reduction for the current-tube being determined so exactly that any velocity observed by means of it was absolutely correct, the experiments for obtaining coefficients of discharge under different conditions, and for obtaining the ratio of the maximum velocity in a section to that of the mean velocity of discharge in open channels were undertaken.

The principal results of these experiments.

The first was the determination of the coefficient A in the formula $A = \frac{RS}{V^2}$ where R is the mean hydraulic radius, S the inclination of the water surface, or sine of its slope in one mètre, and U is the mean velocity of discharge.

The coefficient was considered to vary in four categories of channel.

1st.—When the bed and banks of the channel are made of well-planned plank, or of cement :

$$c_1 = 0.00015 \left(1 + \frac{0.03}{R} \right)$$

the data on which this was based are those of series No. 2 of Bazin's experiments, those of the Aqueduc des fontaines de Dijon of d'Arcy, and those of Baumgarten on the Canal Roquefavour.

2nd.—For bed and sides of ordinary plank, brickwork, or ashlar :

$$c_2 = 0.00019 \left(1 + \frac{0.07}{R} \right)$$

the data on which this was based were, for plank twelve

series of experiments of Bazin, and twenty-nine of Dubuat ; for brickwork, the series of experiments No. 3 of Bazin ; for ashlar, those of the Rigole Marée de Tillot, the Aqueduct of Cran, and the series No. 3 of experiments of Bazin.

3rd.—For channels of rubble :

$$c_i = 0.00024 \left(1 + \frac{0.25}{R} \right)$$

this was based on Bazin's experiments on the Rigoles de Grosbois, and the Marseilles Canal.

4th.—For earthen channels :

$$c_i = 0.00028 \left(1 + \frac{1.25}{R} \right)$$

the experiments on which this was based were those of d'Arcy and Bazin on the Rigoles of Chazilly and Grosbois, on the Marseilles Canal, the Canal du Jard, those of Dubuat on the Hayne, of Funk on the Weser, and those of various engineers of the French Ponts et Chaussées on the Seine and Saône.

The second result was the following formula for velocity :

V = the mean velocity of discharge.

V_x = the maximum velocity observed in the section.

$$\frac{V_x}{V} = 1 + 14 \sqrt{c_i} ; \text{ or } V_x - V = 14 \sqrt{R S} ;$$

or in the form most useful in the cases in which maximum velocities are observed as data for gauging,

$$V = V_x - 14 \sqrt{R S}.$$

Using values of c_i from 0.00015 to 0.003 the corresponding values of $\frac{V_x}{V}$ become thus :—

| $c,$ | $\frac{v}{\bar{v}}$ |
|-------------------|---------------------|
| 0.00015 | 0.854 |
| 0.0005 | 0.762 |
| 0.001 | 0.693 |
| 0.002 | 0.615 |
| 0.003 | 0.566 |

The above expression, involving terms not included in that of De Prony for the ratio of maximum to mean velocity of discharge, does not admit of comparison with it; but is evidently calculated to supersede it entirely.

The reduction of both of these results to English measures is given in Chapter I.

10. THE GAUGING OF GREAT RIVERS IN SOUTH AMERICA, BY J. J. RÉVY.

The account of the most recent operations in gauging very large rivers conducted by J. J. Révy, given in Révy's 'Hydraulics of Great Rivers' (London, 1874), includes a description of the method he adopted in current observations on the Paraná, La Plata, Paraná de las Palmas, and the Uruguay, from which the following brief *résumé* of operations is taken.

It seems to have been a work of some time and difficulty to find a reach of the Paraná sufficiently straight for conducting gauging operations and velocity measurements; a hundred miles of the river were searched unsuccessfully, but at last a reach straight for many miles was found. Here the river was about a mile in breadth, and the soundings showed from 5 to 71 feet of water; a gauge fixed in the stream did not show a variation of

level in the water surface of as much as a quarter of an inch in twenty-four hours ; and the inclination of the water surface in one mile was very nearly nothing. The fall observed by levelling for one mile with a 14-inch level, on equidistant staves placed 300 feet apart, was less than $0\cdot01$ of a foot ; it was therefore practically impossible under the existing state of the river bank, which was not adapted for levelling, and with the instruments at hand, to carry out levelling operations with any effective result ; as it would have involved ten miles of levelling on passable ground, and probably required also the use of superior instruments.

It was found that for the surveying and triangulation work, either calm weather or clear weather with a gentle breeze was absolutely necessary ;—for current observations calm days only allowed of operations being carried on.

A base line of 3 000 feet was measured on the low-lying left bank of the river, with a steel tape of 300 feet ; and lines were set out at right angles at each end of it, to give the direction of a river-section-line for soundings ; the prominent points in the neighbourhood and on the river bank were triangulated and tied into this base line.

Soundings.—Those on the lines of section were taken with the lead and cord ; the length of cord was measured with a tape at each sounding, each of these measurements taking one minute ; the position of each sounding was fixed by angular observation, with a 3-inch pocket-sextant giving readings to one minute, on the two flags one at each end of the base line. The angles were observed in from three to ten seconds each. The number of soundings taken in the section varied with the necessity for them : it was necessary to show, and hence also

to find the points in the river bed where there was a change of lateral slope, however many they might be, but in places where this slope was regular and gradual, the soundings were not considered necessary at closer distances than from one-twentieth to one-tenth of the breadth of the river. The section of the Paraná, where its breadth was more than 4800 feet, was sounded in two hours and sixteen minutes, after all the preliminary arrangements, drilling of the men, &c., had been properly carried out. In plotting the section, the position of each sounding was fixed both by means of the complements of the angles observed at those points, and the calculated distances from the base.

Velocity measurements.—These were made with the screw current-meters previously described. As the velocities had sometimes to be observed at great depths, the ordinary method of lowering the meter to its position by sliding it on an iron standard was utterly impracticable, and the following mode was adopted. The current meter was attached to one end of a horizontal iron bar, 9 feet long, 2 inches wide, and half an inch thick, which was suspended by chains passing through rings attached to it from a boat moored over the required spot; in order also to prevent the current from moving the bar from its proper position, cords from the rings of the bar were also attached to other two boats, one moored 100 yards up stream, the other 100 yards down stream. By these means the current-meter could be used with good effect in water up to 100 feet in depth, and in currents up to 5 miles an hour. Four sailors were necessary in taking current observations in this way. The observations of velocity were generally taken by an immersion of the current-meter for about five minutes, the time observed

by the watch being generally a few seconds more or less, which were allowed for in the resulting calculated velocity per minute; a second checking observation was also generally made by an immersion of one minute. The instrument was put in or thrown out of gear by means of a wire leading from it up to the boat, thus allowing or preventing the revolutions of the screw from recording themselves on the dial faces at any moment.

In the gaugings carried out, observations of mean vertical velocity, giving the mean velocity in any plane from the surface of the water to the bottom, seem to have been preferred wherever practicable. For these cases, in which it was necessary that the current-meter should be steadily and evenly lowered to near the bottom and raised again to the surface, it was found advisable always to work it from a platform between two boats, placed 12 feet apart, moored by four anchors, and to have the two suspending cords marked at every 3 feet with alternately red and white marks, as guides to those lowering and raising them; the cord attached to the down-stream boat was not, however, considered necessary in this operation, the up-stream cord preventing the instrument from going far out of the vertical direction. In these operations the instrument was put in gear by hand by tightening a nut on immersion, and put out of gear again in a corresponding manner on withdrawal from the water. In taking surface velocity observations, the current-meter was screwed on to a wooden staff, 3 inches wide and half an inch thick; the revolutions of the screw continuing after withdrawal from the water being at once stopped by hand so as not to vitiate the record on the dial-face.

The determination of the equation of correction for each

current-meter was conducted in the following way. It was tested at a low velocity by drawing it through a distance of 189' 6'' in the still water of a reservoir in a time of 2' 30'' giving a velocity of 75.9 feet per minute; the average of these trials gave a recorded number of revolutions of 172, or 68.8 per minute: in the same way also it was tested at a high velocity, and showed 176.13 revolutions per minute for a speed of 183.64 feet per minute. The equation of correction being that of a straight line, two points alone are necessary to determine it: on referring these to rectangular co-ordinates on a diagram, and joining them, the true velocity corresponding to any number of revolutions of the instrument could be scaled off from the rectangular co-ordinates to the resulting straight line. Or taking it algebraically, if x and y , x_1 and y_1 , be the corresponding pairs of co-ordinates for low and for high velocity,

$$\text{then } y = ax + b, \text{ and } y_1 = ax_1 + b;$$

$$\text{where } a = \frac{y_1 - y}{x_1 - x} = 0.9962,$$

$$\text{and } b = \frac{1}{2}(y_1 + y - ax_1 + x) = -6.811;$$

$$\text{hence } y = 0.9962 x - 6.811,$$

or in the form more useful for obtaining the true velocity, x , from the number of revolutions, y ,

$$x = 1.00381y + 6.837.$$

On applying to this equation a value of $y=0$, we obtain as a result that this particular instrument would cease to record revolutions for a velocity of less than 6.137 feet per minute.

Hourly Observations.—In consequence of the rivers

observed being tidal, and having a variable current, it was necessary to moor a permanent observatory at a convenient point in the deep part of the river on the line of section, and make hourly observations of the current from it throughout the day and night. The tidal rise and fall was also registered at every quarter of an hour; barometric, thermometric, and wind observations were also recorded.

The current observations, both surface, mean, and sub-surface, were taken with Révy's current-meter from a small boat moored temporarily fore and aft on the line of section already sounded, its position in each case being determined by angular measurement with a pocket sextant on the extremities of the base line, which fixed it within a few inches. For this work two sailors, two anchors, and several hundred yards of line were necessary. The current observations were taken at the surface, and at depths of 4, 7, 10, 16, and 23 feet, the latter being one foot above the bottom. The mean current observations were made three times in each case, and were found to check each other within 1'6 foot per minute in observations giving 80 feet per minute. The time of day of the current observations was always noted, and check observations were also taken from a fixed level, so that the observed tidal variation might be applied, and the effect of the tidal wave—a disturbing cause far greater than that due to the inclination of the water surface in the cases of these rivers—thoroughly investigated.

A convenient mode was adopted for testing the straightness of the reach of the river at the section in which the velocities were observed. The centre of gravity of the river section was found and marked on the drawing, and also the centre of gravity of a section

whose depths represented the surface currents in any convenient mode, either feet per minute or per second ; the horizontal distance apart of these two centres of gravity indicated the amount of effect of a bend in the reach at that section. In the Rosario section of the Paraná this was $\frac{1}{2\frac{1}{7}\frac{1}{3}}$ of the width of the river, and the section was considered favourable ; in the Palmas section it was as much as $\frac{1}{2\frac{1}{8}}$ the width of the river, and this was not considered favourable. In cases where a very straight reach is not to be obtained, the position of a section of observation is recommended to be taken at the point of contrary flexure of two reaches curving in opposite directions.

Conclusions.—The conclusions arrived at by M. Révy from his study of the current observations on the La Plata, Paraná, Paraná de las Palmas, and Uruguay, were—

1st. That at a given inclination surface currents are governed by depths alone, and are proportional to the latter. 2nd. That the current at the bottom of a river increases more rapidly than at the surface. 3rd. That for the same surface current the bottom current will be greater with the greater depth. 4th. That the mean current is the actual arithmetic mean between that at the surface and that at the bottom. 5th. That the greatest current is always at the surface, and the smallest at the bottom ; and that as the depth increases, or the surface current becomes greater, they become more equal, until in great depths and strong currents they practically become substantially alike.

Remarks.—The consideration of the foregoing *résumé*, as well as the study of the original books, leads to the further conclusions—that these observations and experiments on tidal rivers have yet thrown no light whatever

on the laws of velocity in ordinary rivers unaffected by tidal currents, the two matters being distinct and separate ; that a more complete account of the tidal action on these South American rivers might have rendered the records valuable and useful ; and that the further perfection of the Woltmann meter or water-mill by M. Révy proves its suitability to gauging operations on a large scale.

II. CAPTAIN CUNNINGHAM'S EXPERIMENTS ON LARGE CANALS.

The sites at which the experiments were made were those mentioned in the Table on the next page, this Table also describing generally their conditions, and mentioning the period over which the experiments were conducted at each.

An examination of the longitudinal sections at these reaches shows extreme irregularity of bed, deep scouring and high silting in various places, and considerable departure from the original bed slopes ; in this respect the conditions were extremely unfavourable. The cross sections, however, were moderately regular in form, and portions of reaches in which no general depression occurred were invariably selected. The supply of the canals was very variable ; the requisite control over the water was effected at the falls at the tail of each reach by raising or lowering the crest with balks of timber. Gauges, either permanent or temporary, were set up at each site, and soundings taken at each cross-section of observation. The sections in earth were mostly rough trapezoids, or coarsely formed sections ; those in the aqueduct were either simple or stepped approximate

Table of Sites of Observation on the Ganges Canal and its branches.

| Site | Bed Width | Maximum Depth | Maximum Discharge | Channel | | Season of Experiment |
|-------------------------|------------|---------------|------------------------------|-------------------|------------------|--|
| | | | | Bed | Banks | |
| Fifteenth mile . . | ft. 160 | ft. 12 | cub. ft. per sec. 7000 | Earth | Earth | March to May, 1878; November and December, 1878; April, 1879 |
| Solani embankment sites | 150 | 12½ | 7000 | Clay and boulders | Masonry steps | August, 1876, to December, 1878; April, 1879 |
| | 150 | 11¾ | 7000 | " | " | December, 1874, to January, 1875 |
| Solani twin aqueducts | 85 | 10 | 3500 | Masonry | Masonry vertical | December, 1874, to April, 1875; February, 1877, to December, 1878; April, 1879 |
| | 85 | 10 | 3500 | " | " | February, 1875; December, 1875, to December, 1878; April, 1879 |
| Bebra . . | 180 | 11½ | 6500 | Earth | Masonry slope | January to March, 1879 |
| Jaoli . . | 185 | 10½ | 6500 | " | " | " |
| Kamhera . | 55 | 6 | 980 | " | Earth | " |
| Right Jaoli | 16 | 4½ | 190 | " | " | March, 1879 |
| Mansurpur | 10 | 4 | 80 | " | " | " |
| Miranpur . | 11 | 3½ | 80 | " | " | " |
| Punora . . | 9 | 5 | 85 | " | " | " |

rectangles, the steps of 14-inch tread and 12-inch rise not continuing down to the bed, but terminating vertically.

The range of external conditions under which the observations were carried out at the two principal sites,

the main Solani embankment and the Solani right aqueduct, was extremely great—with high and low surface gradients, high and low water, and through great range of regulation at both the head and the tail of each reach; this rendered the results in these two cases highly valuable. The experiments on channels in earth were not carried out under such an extensive range of conditions, and afforded far less valuable results: extended experiment on them is yet a desideratum.

Proceeding to details and remarks on the velocity measurements: the terms adopted for velocities of various sorts by Captain Cunningham have the merit of great clearness. Taking x, y, z as co-ordinates of length along current, across it, and in depth respectively, h for depth, b for breadth, A for area, and t for time, the velocities of different sorts are thus distinguished:

1. Average velocity at any point:

$$v \text{ or } \int_0^t v dt \div t.$$

2. Float velocity, the mean of forward velocities or resolved parts of velocities parallel to the current axis through any point in a cross-section:

$$V \text{ or } \int_0^x v dx \div x.$$

3. Mean velocity past a vertical:

$$U \text{ or } \int_0^h v dz \div h.$$

4. Mean velocity past a transversal:

$$U \text{ or } \int_0^b v dy \div b.$$

5. Mean sectional velocity:

$$V \text{ or } \int_0^b \int_0^h v dy dz \div A.$$

In discussing the subject of instruments for measuring velocity, the obliquity and crookedness of the course of a float is not considered objectionable, as its actual motion gives a representative forward velocity ; but while the opinion that all floats and many velocity-meters afford a correct average of velocities during the time of actual observation may be correct, the objection that the result is not true for any single instant of time is not noticed. Among the enumerated advantages of floats are that they afford direct measurement of velocity, interfere little with the current, are not liable to injury, may be easily repaired, are cheap, and may be used in streams of any size. The nearest approach to the edge of a bank possible with floats was found to be about 7 inches. The sites of the experiments being very favourable to the use of floats, they were *exclusively used* in all the systematic work.

At each site of observation an upper and a lower rope were strained across the channel, to mark the extremities of the reach under experiments, and cord pendants were attached to these wire ropes at fixed distances suited to the intended paths of the floats ; the float velocities obtained were treated as actual velocities at the middle point of the float course. The deviation admissible from the float course was, in channels 150 ft. wide and upwards, 2 ft. ; in those of 70 ft. wide and upwards, 1 ft. ; and in those of 25 ft., $\frac{1}{2}$ ft. ; the utmost deviation being allowed only about the middle of the course ; near edges and banks a less deviation was allowed, about a third of the above. The dead run of the floats above the upper rope to allow of relative equilibrium being established before timing was generally 100 ft. ; but in narrow channels 50 ft. Moored boats were necessary

for casting and catching the floats, the number of men in each field-party with the boats and floats varied from thirteen to nine men.

The timing was managed by two thoroughly trained observers, a caller who watched the floats, and called as each float passed the upper rope, then ran to the lower rope, and called again just when each float passed the lower rope; the observer sat with a field-book and a loud half-seconds chronometer at a midway place, and recorded the times by ear alone. The maximum error admissible was half a second. In this respect there was a great improvement on the timing by watch adopted in the International Rhine observations. The usual length of run adopted was 50 ft.; in exceptional cases, where the tendency to deviation of the floats from their courses was greater, a 25 ft. run was preferred. Three timings were made and recorded, and the mean taken; all defective observations were rejected instantly in the field; the force of wind and the gauge-reading were invariably recorded with each set, as well as the distance of the float paths to right and left from the middle of the stream, the breadth of water surface, and the sizes of the floats or tinned tubes used. The speed of these timing observations was much affected by the number of float courses that turned out bad; as several floats were often used unsuccessfully in one set on one float course. The deduced velocities were taken out to hundredths of a foot per second, the hundredths being treated as approximately correct. The velocity of 5 ft. per second was considered unusually high; the maximum error in such high velocities, due to half a second in observation, was therefore one-twentieth or 5 per cent, and in low velocities of 1 ft. per second one per cent.

As to gauges, both still- and free-water gauges were adopted at various sites, and these were either permanent or temporary. In the permanent still-water gauges a pool with fine passages of communication afforded a good place for the gauge ; for temporary still-water gauges, a 3 in. stand-pipe was erected in the bank, and made to communicate with the water by a $\frac{1}{4}$ in. lead pipe with a contracted nozzle ; float sticks of 3 ft., 6 ft., and 10 ft., were used with indicators for convenience in reading. The oscillations of the water in free-water gauges were troublesome, especially in high wind ; the practice was to observe the maximum and minimum reading in half a minute, and to use the mean ; with temporary free-water gauges the difficulty was higher, the plan adopted was to make firm bench-marks less than a foot below the temporary water surface, and scale depth to surface with a brass rule having its thin edge directed up-stream. Free-water levels were proved to be slightly above still-water levels. The average of water-level at both banks of a section was invariably determined and used ; the differences of level frequently being very marked and much affected by the wind. Gauge-readings were made at the beginning and end of each set of observations and the mean adopted.

Soundings were taken both along the cross-section and along the courses, and at distances 50 ft. apart in wide channels, and at 25 ft. apart in small channels ; these had to be repeated after any presumed change in the bed and banks, and the average depths were made dependent on the mean water-level. The sounding rods were wooden rods $1\frac{1}{2}$ in. square, and from 11 ft. to 15 ft. long, protected by iron shoes and having rings above for convenience in withdrawal. The readings were seen by

an observer on the bank and read to a tenth of a foot, occasionally even this could not be done with certainty.

Both the direction and the force of the wind was recorded at the beginning and end of each set of observations ; but the anemometers did not compare favourably ; and the wind data obtained can only be looked on as a rough estimate of the wind. The reduced levels were referred to the datum of mean sea level at Karachi ; all special levelling was done twice over with an excellent 20 in. level, and no discrepancies exceeding 0.01 ft. were allowed. The computation of the final hydraulic elements from the observed data was exceedingly laborious ; but that, as well as all work admitting of check, was verified by two persons independently.

Unsteadiness of motion producing variation in velocity was investigated, and a large series of experiments tabulated to demonstrate the effect ; the conclusion being that the amount of velocity variation at one and the same point is liable to be at least 25 per cent. of the mean value. Under such circumstances single or detached velocity observations are nearly valueless ; but the assumption that synchronous measurement cannot possibly be secured in actual practice is perhaps overstated ; it would certainly be very expensive. Falling back then on average velocities, the conclusion is applied that averages should be formed from about fifty values ; the course of the four years' experiments was accordingly entirely regulated on that basis, and the measurements done in groups.

The systematic float velocity-measurements were also made in as rapid a succession as possible on either a vertical or on a transverse axis, in groups of three at each point, thus :

| On a vertical. | On a transversal. |
|-------------------------------|------------------------------|
| At surface. | At the point near left bank. |
| At a depth of 1 ft. | At next point. |
| At a depth of 2 ft. | &c. |
| &c. | At point nearest right bank. |
| At the point near to the bed. | |

Also six rod velocities, the whole forming a set. The only other systematic velocity work was central surface velocity measurements, which were done in groups of 48 in as rapid a succession as possible, thus forming a set of another sort. Sets were then taken up in succession under nearly similar external conditions, so long as the water-level remained nearly constant and the wind moderate, up to a limit of about sixteen sets. But if the water-level changed more than 0.1 ft., or the wind exceeded 15 ft. per second, the field work was usually closed.

Such sets as were executed in sequence were then combined into one series by tabulation on the same sheet, each series admitting a maximum range of water-level of 0.3 ft., irrespective of the state of the wind, and only to some extent irrespective of the surface slope at the site. This careful mode of combination is a great advancement on the method often adopted elsewhere of combining sets on different verticals in all depths of water, and sometimes even at different sites.

A conclusion drawn from the plotting of these sets is valuable. Notwithstanding unsteady motion, the average velocity at a point is probably constant under similar external conditions, any departures from this law shown in the velocity curves being due to insufficiency of velocity observations, to irregularity of contour of bed and banks at the site, or to irregularity of the channel above and below the site. The recognition,

however, of unsteady motion being the ordinary normal condition of flow, and of the vertical interlacing of stream lines, is strongly insisted on.

With regard to longitudinal slopes. First, as the bed slopes were very irregular, an average bed slope, equal to the fall between two adjacent permanent floorings divided by the distance between them, became the only representatively useful quantity. Both the average surface slope of the water for a long distance above and below any site, and the local surface slope at the site, were always determined with great precision, the surface slope per 1 000 never exceeding 0·48; it was a matter of extreme delicacy, in which the reference to water-level was more important. This was done simultaneously by two observers in calm weather on each bank, in some cases only. The condition that the real surface slopes at opposite banks are not generally equal was not fully recognised till a late period. The amount of surface fall deduced from gauge readings above and below site, supplemented the slopes deduced by levelling, but was in many cases imperfect from the conditions of control of the reach. The conclusions derived from the diagrams of surface gradients are that the local surface slope depends jointly on the surface falls both above and below, but that the latter by no means suffice to indicate the former. It is also observed that the mean velocity and discharge at any site was more dependent on the value of the surface slope than any other element.

Surface convexity received the attention of Captain Cunningham. Noticing the theory that the pressure in a fluid in motion is always less than the mere hydrostatic pressure, and comparatively less with more velocity, and the opinion that lateral motion would sectionally enforce

a convexity in the middle, and thus form an accumulative layer above the *locus* of maximum velocity of the section, he remarks that the above is true, excepting the sectional convexity, which is almost wholly wanting. The observations for convexity were exceedingly delicate and tedious; yet from a series of them, made at the Solani embankment main site, the conclusion was drawn 'that the surface of water in motion in a long straight reach with tolerably uniform bank is, on the average, nearly level across.'

Such a general law seems almost unaccountable by abstract reasoning, and may be true only for special conditions and circumstances, probably under peculiar irregularities of bed above and at the site; but the deduction is one that cannot be set aside, although it undoubtedly requires the light of further and extended special experiment under higher velocities, and with strictly uniform conditions of bed and of section.

While concluding this notice of the preliminary conditions under which the experiments were made—conditions sufficiently involved and irregular to deter the most arduous of hydraulic enthusiasts—we may notice that it seems surprising that the Government did not make some grant for largely improving and rendering regular the beds of the canal in the vicinity of the sites before experiment; also that a bolder comprehensive method of meeting the expenditure would have been conducive to continuous work. The straining against difficulties, as well as the labours of the undertaking, had to be met by the unsparing energies of the experimentalist; and though under such circumstances results redound more greatly to credit, it is much to be deplored that his efforts were thus fettered.

Continuing to verticalic velocity curves, or observations of velocity past a vertical, it may be noticed that all sub-surface velocities were obtained by timing double floats. These were of two patterns, one a ball of acacia wood, 3 in. in diameter, boiled in oil and loaded with lead; to this a surface cork disc, 2 in. in diameter and $\frac{3}{8}$ in. thick, was attached by a brass wire 0.012 in. thick; the other a shell of copper 0.02 in. thick, $1\frac{5}{8}$ in. in diameter, loaded with lead; to this a cork surface disc, 1 in. in diameter, $\frac{1}{4}$ in. thick, was attached by an oiled silk thread $\frac{1}{80}$ in. thick. Velocities being observed at every foot of depth, as many as ninety floats were used in a set, and three observations were made at every point; defective courses were made up by subsequent courses, and the mode of timing was that already described with surface floats and rods. The velocities were plotted to vertical axes, mostly central verticals, on a scale exaggerated ten times for the velocity ordinates; the curves formed were approximate parabolas, having general features agreeing closely with the similar cases of Bazin on a smaller scale; the errors due to the employment of floats are such as to produce curves flatter than they should be. From these were computed the mid-depth velocities $v_{\frac{1}{2}H}$, the bed velocities v_H and the mean velocities V .

The mid-depth velocity at every vertical was found to be subject to great and rapid variation; thus disproving the assumption of constancy asserted in the Mississippi Report, for which no proof was afforded by observations; but its variability was proved to be less than that of either the surface velocity or the bed velocity. It was also discovered that any marked increase or decrease of either the surface, the maximum, or the mean velocity was ac-

complicated on the whole by increase or decrease of the whole of the velocities on the same vertical.

The calculation of the parabolic elements of the velocity parabolæ was thus effected :

Taking the two general formulæ, $Z^2 = p (V - v_0)$, $p(v_0 - v) = z^2 - 2Zz$, where Z is the depth of maximum velocity, p is the parameter, z the depth to any point, the known values being $v_0, v_{\frac{1}{2}l}, v_l$ corresponding to $0, \frac{1}{2}l, l$; these were substituted for v and for z in the above and the equations solved for p, Z , and V . Thence

$$p = \frac{l^2}{2 \cdot 2v_{\frac{1}{2}l} - (v_0 + v_l)},$$

$$Z = \frac{1}{2}l + (v_{\frac{1}{2}l} - v_0) \cdot \frac{p}{l},$$

$$V = v_0 + \frac{Z^2}{p}.$$

The parabolæ determined by each group of three data being usually different, the most probable parabola was determined by the method of least squares, a mode laborious but correct. An investigation of parameter variation showed that the data did not admit of sufficient accuracy in the determination of the value of p to enable its dependence on the external conditions to be traced. The depression of the line of maximum velocity is shown to be not sensibly affected by the wind but largely due to air resistance, and dependent on the surface slope near the site, but the quantitative connection cannot yet be traced.

The summation of velocity past a vertical was effected through various combinations of the trapezoidal, Simson's, cubic, and Weddle's rules, suited to the number (n) of equal spaces (k); of which the following are the general expressions.

$$\frac{1}{2}k\{v_0 + v_n + 2(v_1 + \&c. + v_{n-1})\}$$

$$\frac{1}{3}k\{(v_0 + v_n) + 4(v_1 + \dots + v_{n-1}) + 2(v_2 + \dots + v_{n-2})\}$$

$$\frac{2}{3}k\{(v_0 + v_n) + 2(v_3 + v_{n-3}) + 3(v_1 + v_2 + v_4 + v_5 + \&c. + v_{n-2} + v_{n-1})\}$$

$$\frac{3}{10}k\{(v_0 + v_2 + \dots + v_{n-2} + v_n) + (v_3 + \dots + v_{n-3}) + 5(v_1 + v_3 + \dots + v_{n-3} + v_{n-1})\}$$

The deductions with regard to mean velocity (U) past a vertical are that its line is always below mid-depth, but that it cannot be directly measured in practice by any single velocity observation ; that the mean velocity past a central vertical is dependent on the surface fall in the upper sub-reach, but cannot be deduced from it better than from any primary velocity. It may be deduced from two velocities by the following formulæ :

$$U = \frac{1}{4}(v_0 + 3v_{\frac{3}{8}}) ; \text{ or } U = \frac{1}{7}(3v_{\frac{1}{8}H} + 4v_{\frac{3}{4}H}) ;$$

$$\text{or } U = \frac{1}{7}(4v_{\frac{1}{4}H} + 3v_{\frac{5}{8}H}),$$

of which the first is considered the most convenient.

The value of U may also be obtained from a single observation with a loaded rod in depths not more than 15 ft.

The rods preferred and mostly used were 1-in. tin tubes painted and marked for immersion, loaded with fixed iron, and adjusted with shot ; they were made in sets of fixed length, but wooden rods were also used in shallow water. The bed and banks had sometimes to be dressed to admit of tube observation. The tube velocities were compared with double-float velocities for purposes of experimental test. An investigation of the theory of rod motion results in a conclusion that a proper rod length is from 0.945 to 0.927 of the full depth, when the maximum velocity is at within one-third depth from the surface, and from 0.927 to 0.950 of it when that is at between one-third depth and one-half depth.

Proceeding to transverse velocity curves, or curves whose ordinates are the forward velocities at all points of a transverse base line in a transverse section, the following is an abstract of the observations effected, which were made under varying conditions of water-level at each site.

| | | | | |
|----------------------------|----------------------|---------------------|-------|------|
| Surface velocities . . . | 10 series comprising | 109 sets at 4 sites | | |
| Mid-depth velocities . . . | 2 | „ | 17 „ | 1 „ |
| Bed velocities . . . | 2 | „ | 7 „ | 1 „ |
| Mean velocities . . . | 100 | „ | 581 „ | 11 „ |

The surface velocities were observed with pine discs 3 in. by $\frac{1}{4}$ in. ; the mid-depth and bed velocities with 1 $\frac{1}{8}$ in. double floats ; the mean velocities with 1-in. tin tube rods generally, and with 1-in. wood rods in depths less than 1 ft. As the ordinate spacing required closer ordinates where the change of velocity was more rapid, the transversals were divided into lengths or spaces, within each of which the sub-spacing was equal ; the arrangement being symmetrical to the centre line of the bed in every case. The mode and order of the field work and timing were similar to those already described, so also the arrangement in sets and series. The average velocity observations were finally plotted as rough curves to each transversal, as also the resulting means of the primary velocities, at surface, mid-depth and bed, and the sectional. The notation here used is : h =any depth ; b =surface breadth ; R =hydraulic radius ; H =central depth ; B =wet border ; S =surface slope ; and the values of these are given with the transverse velocity curves for each site. The causes and conditions accompanying local peculiarities in these curves are fully entered into ; but the principal deductions made from the whole set of curves are the following :



1. That like curves are similar under similar external conditions. 2 That like curves with equal mean velocity are, *cæteris paribus*, equally flat on the whole. 3. Curves of low velocity are flatter than those of like kind of high velocity. 4. The flatness of a curve depends more on the mean velocity than on the general depth, as shown by comparing low-water and high-water curves. 5. Wide sides give flatter curves throughout. 6. Sloping or stepped banks give rise to sharp curvature. 7. Vertical banks give rise to curvature also, but this is less than with the former. 8. In comparing unlike curves; of unlike curves under the same external conditions at the same site of rectangular section, the mid-depth curve is usually the outer, the mean velocity curve intermediate, and the bed curve the inner. The mean velocity curve is one of the flattest and the surface curve the most rounded, so much so, that near the banks the surface curve becomes one of the innermost. 9. The figure of a transverse velocity curve can be determined with equal precision at all parts excepting near the edge. 10. Edge velocity is assumed to be zero, but not plotted.

The attempt to arrive at a geometric figure for a transverse velocity curve generally was eventually given up as hopeless; but the sort of curve most nearly possessing the required properties is the elliptic curve of the type represented by the equation

$$\left(\frac{v}{v_0}\right)^n + m\left(\frac{y}{b}\right)^n = 1$$

The following were also general conclusions:

1. The figure of the transverse velocity curves is for given external conditions determined by the figure of the bed.

2. The velocity (v) should be expressed not only as

a function of the abscissa (y) but also of the depth (z); so that the equation should be of the form $v + V = f(y, z, \&c.)$; it may also be a function of the average effective distance from the wet border.

In the calculation of discharges, the mode and notation adopted were as follows. The data used were :

A system of depth ordinates H_y in the cross-section.

A system of velocity ordinates u_y in the velocity curves.

A system of curve areas $D_y = H_y u_y$ with the same abscissæ $\pm y$; *i.e.*, at the same points of the transversal.

The quantities $D_y = H_y u_y$ were prepared by multiplying separately every rod velocity u_y by the average depth H_y along the float course. These so-called superficial discharges D_y past the several verticals whose abscissæ are y are then equally spaced quantities used in ordinary approximation formulæ, of which the prismoidal formula is one, to obtain the total or cubic discharge. The following were the four formulæ used; the quantities a, a_1, a_2 , at equal spacing b to right or left of the centre line being distinctively dashed thus— a', a'', a_1', a_1'' , &c.

1. Simson's

$$\frac{1}{3} b \{ (a_2'' + a_2') + 4 (a_1'' + a_1') + 2a_0 \}$$

2. Cubic.

$$\frac{3}{8} b \{ (a_3'' + a_3') + 3 (a_1'' + a_1') \}$$

3. Weddle's.

$$\frac{3}{10} b \{ (a_3'' + a_1'' + a_0 + a_1' + a_3') + 5(a_2'' + a_0 + a_2') \}$$

4. Simson's modified.

$$\frac{1}{3} b \{ (3E + e) + 4(Q + M),$$

where q a missing quantity = $\frac{1}{2}(M + E)$ is between two

adjacent quantities ME , these and e being all alike at equal spacing. This last was convenient for such cases.

With a rectangular cross-section the total discharge $=D_m H$; D_m being the superficial discharge past the mean velocity transversal, or area of mean velocity curve.

The conclusions arrived at with regard to total or cubic discharge were: That it is sensibly constant from instant to instant, but that at any site it increases and decreases rapidly with the rise and fall of water-level. It is liable to increase or deficiency from a cross wind blowing towards or from the gauge. Moseley's discharge formula meets with very strong condemnation, and its faultiness is clearly proved in a most lucid manner. For comparison of discharges at successive sites, the field work should be either simultaneous or in the same body of water at all the sites; and for those from successive observation at the same site, immediate succession is desirable. The discordance between successive comparable results under similar favourable conditions may be expected to be seldom over 3 per cent.

With regard to mean velocity, the following also are the conclusions of Captain Cunningham.

1. That the arithmetic mean of velocities past neighbouring points on a transversal is not the mid-distance velocity, but errs in defect.

2. The mean velocity past a transversal and the mean sectional velocity are less variable from instant to instant than most of the individual velocities, but the former varies sensibly.

2. The mean sectional velocity is constant from instant to instant, and more so than the discharge.

4. The chief source of variability in successive mean

velocity-measurements is that each single result is imperfect, and this is due to unsteady motion.

5. The mean surface and central surface velocities U_o , v_o , and also the mean sectional, central mean, and central surface velocities (V_o , U , v_o), and the quantity \sqrt{RS} increase and decrease with either R or S .

6. In high up or down-stream wind, surface velocity observations are liable to be under or over-estimated, and are quite unsuitable for computation of discharge; but mean-velocity observation is but little affected by wind of any sort, and error is then attributable to an abnormal gauge reading.

7. The ratio $c = V \div U_o$ generally increases with increase of depth, and probably with decrease of velocity or surface slope; but its variation is obscure, perhaps owing to the effect of wind on U_o .

8. For rapid approximation to mean velocity a good average central mean velocity observation is at present the most reliable mode.

9. The ratio $c = V \div 100 \sqrt{RS}$ increases and decreases generally with increase and decrease of R , depends in some complex manner on S , and also on the nature of the bed and banks at the site.

This last conclusion is obviously of the highest importance in its bearing on *calculated velocity formulæ*.

In a careful examination of these latter, Captain Cunningham states that these are all, with the sole exception of that of Herr Kutter, quite untrustworthy, and that Bazin's relation $c_b = 100 C \div (100 C + 25 \cdot 34)$ is fundamentally incorrect as a relation between $c = V \div v_o$ and C .

The rejected formulæ among the really old ones are those of Dubuat, 1786; Girard, 1803; De Prony, 1804;

Young, 1808; Dupuit, 1848; St. Venant, 1851; Ellet, 1851; and among newer ones, those of Bornemann, Hagen, Gauckler, Mississippi, and Gordon.

The only two formulæ of sufficient value to merit extended discussion were those of Bazin and Kutter. The results of their examination are:

1. That the form of the value of C in the Bazin formula is defective.

$$C_b = \left(a + \frac{\beta}{R} \right)^{-\frac{1}{2}}.$$

This was also Herr Kutter's conclusion.

2. That making K a constant in the expression:

$$V_x - V_m = K \sqrt{RS}$$

is not just, and K varies from 22.4 to 9.9 in 61 cases, and from 17.0 to 10.7 in 43 selected cases given by Bazin.

3. The effect of applying Bazin's coefficient c_b to central surface velocities v_o is to produce too low values of mean velocity.

4. Bazin's ratio c_b increases with R , whereas the experimental values of c_b show no signs of this.

5. For earthen channels Bazin's ratio c is so low as to be of little use.

Next, regarding Kutter's coefficients (C_k);

1. The formula, though complex and laborious, is the best empirical formula yet proposed for calculated mean velocity (and hence for discharge).

2. When the surface slope measurement is a good average, done in calm air on both banks on a canal in good train, C_k will give results whose error will probably seldom exceed $7\frac{1}{2}$ per cent. in large canals.

3. The coefficient of rugosity must be experimentally determined for each site.

It may be here noticed that the books of the author were employed by Captain Cunningham to obtain values on the Kutter system suited to English purposes, and are referred to repeatedly; and that with reference to the liability to error of $7\frac{1}{2}$ per cent. in these quantities, it is clear that as discharges under favourable circumstances of experiment are allowed to be liable to 3 per cent. of error, the former being about double, this proves a high degree of exactitude for a mere calculated velocity formula, and practically justifies the claim advanced in those books to an accuracy within about 5 per cent.

The above constitute the principal results of Captain Cunningham's experiments.

In addition, much care and experiment were devoted to fan current meters, Moore's and Révy's and to improving them by separating the recording portions from the fans; but from uncertainty of oriculation, of depth, of gearing, and of non-measurement of forward velocity, their employment was eventually considered simply useless. A series of observations on the effect of silt resulted in the following conclusions, that, 1. There is no obvious connection between the velocity and the silt density of different parts of a site; the silt density varies from instant to instant at one and the same point. 2. The silt density and silt discharge do not appear to depend sensibly either on the depth or the velocity at a site, but in the Ganges Canal they depend chiefly on the silt admitted with the supply.

The observations on evaporation produced the following conclusions: 1. The evaporation from a floating evaporimeter on a large still-water surface or river is

far less than from a small vessel on land. 2. The evaporation from the Ganges Canal at Rurkhi averages about $\frac{1}{10}$ inch daily out of the rainy season; and the loss by evaporation is about $\frac{1}{150}$ th part of the full supply of the canal, or about ten minutes' full supply daily.

The main result of the whole may be expressed in a few words, 'That most of such hydraulic results as were previously accepted by only the few have now been so verified on a large scale as to command their acceptance by the many.'

12.—GENERAL REMARKS ON SYSTEMS OF GAUGING.

The foregoing brief accounts of the modes adopted by various hydraulicians in carrying out field operations form a far better guide to the engineer about to undertake the execution of gauging operations than any arbitrary advice, or set of rules, could possibly be; the author may, however, be permitted to make a few remarks in conclusion. It is, of course, assumed that the most advisable mode of proceeding in one case might not be applicable to another, and that the method of gauging should be suited to the general object, the place, and the circumstances. When the object is of an experimental nature, having scientific results in view, the experimentalist himself is the best judge of the mode most suited to his object. Most gauging operations, however, have for their purpose the determination of the discharge of a river, or of a canal, with as little labour and expense and in as short a time, as anything approaching to accuracy of result will admit; in these

cases the amount of predetermined accuracy greatly affects the choice among modes to be adopted.

1. The most rapid and least accurate mode of determining the discharge of a river or canal at a certain place and time is that which dispenses with velocity observations, and makes use of a calculated velocity formula as a substitute. The dimensions of two parallel sections of a straight reach of the channel are measured, the inclination of the water surface between the two is levelled, and the nature and quality of the bed and banks are noted; these data enable the discharge to be calculated by the aid of the most modern and most correct formula with a certain amount of approximate truth. The point now to be considered is what amount of exactness may be reasonably expected from the practical application of this method.

The general formulæ for mean velocity of discharge and for discharge in open channels,

$$V = c \times 100 \sqrt{RS}; \quad \text{and } Q = AV;$$

where

$$c = \frac{\sqrt{R}}{100n} \left(\frac{m + 1.811}{m + \sqrt{R}} \right); \quad \text{and } m = n \left(41.6 + \frac{0.00281}{S} \right);$$

seem theoretically to leave nothing more to be desired, except perhaps a simplification of form not attainable in the present state of hydraulic science. It is applicable to channels of all dimensions, from the smallest distributary or rigole to that of the Mississippi; and can be applied to channels of any material, from weed-covered earthen beds to cut stone and carefully planed plank, the data on which it is most carefully based being those of numerous experimentalists. The functions or terms involved are only three, R , S , and n ,

of which the two former can in most cases be readily and sufficiently exactly observed in practice ; the great difficulty, however, lies in the determination of the third function. An examination of the general and the local values of n , given in Working Table No. XII., will explain this. Among the general values suitable to beds of special construction, from well-planed plank to rubble, the value of n ranges from 0'009 to 0'017 ; and the gradations of roughness or quality of surface are clearly marked by the corresponding values of n , the greatest gap being the difference between 0'013 for ashlar and 0'017 for rubble, a difference that can be easily worked up to in practice without any likelihood of important error. It would hence appear that there would be no difficulty in practice of determining discharges with fair accuracy by means of the above calculated velocity formula for channels constructed in such artificial materials. It is, however, in the cases more usual in practice, namely, in those of canals having earthen beds and banks, and in natural river channels, that the values of n offer so wide a range of choice, that the calculated discharge might involve serious error as the result of the adoption of an unsuitable coefficient. For earthen canals the values of n range from 0'020 to 0'035, the gradations of which are far from being yet sufficiently definitely marked ; and for local values the range is about the same. It would seem, therefore, that in these cases it would be necessary to determine by velocity measurement the discharge of the river or canal at the site under consideration, and thence deduce a value of n suitable to it before the above method could be applied for obtaining its discharge at any time or place with sufficient accuracy ; or, in other words, a

small amount of actual gauging must be done before this mode of procedure can be adopted. In the future we shall probably have the values of this function more definitely laid down, and we shall then be able to make use of this method more readily, and with greater confidence in the results; now we have only the present amount of information to guide us, and are hence unavoidably forced into a certain amount of velocity measurement as a means of correctly gauging canals and river channels in earth.

2. Assuming, therefore, that velocity measurement is absolutely unavoidable, the question next arises, what is the least amount of it necessary in determining a discharge? The results of Bazin, determining the relation between the maximum velocity in a section and its mean velocity of discharge, give the readiest solution of this problem for small canals. His formula is,

$$V_x - V_m = 25.34 \sqrt{RS},$$

where V_x = the maximum velocity, and V_m = the mean velocity of discharge; and it is evident that by combining with this formula the more modern coefficients of Kutter, we can, with the aid of only a few observations of maximum velocity, arrive at a mean discharge with rapidity and a fair amount of accuracy, and may be afterwards able to determine a discharge at any time under the same local conditions by means of the ordinary calculated velocity formula and the Kutter coefficient already mentioned, without the need of more velocity observation. The reduction of these equations from French measures is given at page 38, Chapter I.

It is extremely probable that this mode of gauging will be more universally adopted in future, and that a

large series of observations will throw more light on the relation of the maximum velocity to the mean velocity of discharge, and enable it to be determined with greater accuracy than is at present possible. Observers are therefore recommended to keep in view in all gaugings conducted on this principle, not only the sectional position of the maximum velocity in a section (which may be confined to a single point either in the middle of the channel at the surface, or at a few feet below it, around which the velocities may diminish in section rather suddenly, or may extend with but little diminution over an important portion of the section), but also the locus of maximum velocity, or its depth below the water surface, which may vary sensibly in a long reach of river. This inclination of the locus, as well as the amount of section of very high velocity, are data that will probably aid eventually in determining the ratio of maximum to mean velocity of discharge with greater precision than Bazin's formula now affords.

3. The next mode of gauging that seems most applicable to ordinary rivers is one of the modes recommended by Captains Humphreys and Abbot. This, however, involves a greater amount of velocity observation, and at the same time requires the velocities to be observed at a greater depth, for which all descriptions of current-meters are not applicable.

The velocities are all observed at a uniform depth equal to half the hydraulic radius of the section, and at equal distances judiciously chosen across the line of section ; and the mean of these velocities $U_{\frac{r}{2}}$ is taken ; —the mean velocity of discharge, V_m , is then obtained in the formula,

$$V_m = \left[\left(1.08 U_{\frac{r}{2}} + 0.002b \right)^{\frac{1}{2}} - 0.045 \sqrt{b} \right]^2$$

where $b = \frac{1.69}{(r+1.5)^{\frac{1}{2}}}$; and r is the hydraulic radius.

This mode should, however, be limited to very large rivers; in fact, the application of any of the Mississippi data or formulæ to artificial channels or small streams cannot be recommended.

The defect of the above method in assuming the relation $U = 0.93 V_m$ is sufficiently evident, so also is that of assuming the parameter of the parabolic curvature of mean verticalic velocity; but when these quantities are predetermined for any case under consideration, the same principles may be applied in gauging small streams or canals with quite as much success as in gauging the Mississippi.

4. If we accept the conclusions of Captain Cunningham, given at pp. 91 to 93, Section 8, Chapter I.; we may gauge any rectangular or approximately rectangular section of flow by single velocities taken at equal distances on a transversal; the depth of observation being $\frac{5}{8}$ the total depth generally, and $\frac{1}{10}$ the total depth at the points near the margins; these velocities will then be representative elementary mean velocities in their own portions of channel, from which the mean velocity for the whole section may be deduced with some degree of general correctness. Further correctness may be obtained by taking two velocity-observations on each vertical from which to deduce each mean verticalic velocity; the formula recommended for this is (see p. 87),

$$U = \frac{1}{4} (v_o + 3v_{\frac{3}{4}H});$$

that is to say, the surface-velocity and the velocity at $\frac{2}{3}$ the depth, are sufficient.

The defect in these methods is evident ; it consists in making the parabolic curvature dependent on one point or on two points, whereas three points are the least necessary. If, however, we apply the three-point method (see p. 86) and obtain values of U on each vertical through three synchronous observations on it, and make

$$U = \frac{1}{3} (2v_{\frac{1}{2}H} - v_{\frac{1}{4}H} + 2v_{\frac{3}{4}H}),$$

we may deduce a mean sectional velocity that is theoretically almost unimpeachable, though based on a very moderate amount of velocity-observation.

5. The next further attempt at accuracy in river gauging involves a complete investigation of the whole of the velocities in the channel section ; the velocity at every point in the cross-section should be known and plotted on a diagram, they can then be grouped into divisions of the section by vertical and horizontal lines within which the variation of velocity is not important : a mean velocity for each division is calculated and multiplied by the area of that division to obtain its discharge ; the sum of these discharges is the discharge of the whole section. There are, however, two or three methods of treating and observing the velocities. When these fluctuate locally to a very small degree within a short space of time, any velocities observed at the same site within a day or even within a week may be grouped together to serve as a basis of calculation ; similarly also when there is very little local variation of velocity in a reach, mean velocities observed over a portion of reach of from 50 to 200 feet in length will represent

mean velocities at the middle of that length. When both such advantages happen to be combined, the whole of the observation is much simplified, as the velocities must not then be necessarily confined to an exact sectional site, and need not be perfectly synchronous.

Preliminary observation is therefore necessary to determine the conditions under which the velocity-observations will yield correct results.

When the local variation of velocity along a reach is important, either a sufficiently favourable reach must be found, or the method of using loaded tubes and floats must be discarded in favour of other appliances that actually afford velocities at points of observation, or on vertical lines, at a single transverse section.

When velocities vary much at the same spot within a short time, synchronous or exactly simultaneous velocity observations at the given transverse section are absolutely necessary, and appliances must be used that will obtain these. Among them may be mentioned the d'Arcy gauge tube, and the author's current-meter.

Such detailed observations when carried out on an extended scale involve a large amount of labour, care, and skilled personal superintendence, but at the same time afford results not only valuable as regards the determination of the discharges of the river specially under consideration, but also as records of hydraulic experiment aiding in the progress of science.

CHAPTER III.

PARAGRAPHS ON VARIOUS HYDRAULIC
SUBJECTS.

1. On Modules. 2. The Control of Floods. 3. Towage. 4. On Various Hydrodynamic Formulæ. 5. The Watering of Land. 6. Canal Falls. 7. The Thickness of Pipes. 8. Field Drainage. 9. The Ruin of Canals. 10. On water-meters.

I. ON MODULES OR WATER-REGULATORS.

HYDRAULIC engineers not having yet arrived at a perfect module for regulating the amount of water drawn off in an open channel for irrigation or town-supply from an open canal or reservoir under a varying head of pressure, it is a matter of some interest to examine the older types of design of modules that have been used at various times, and in various countries, before going on to those of more modern form. Such designs being necessarily simple, they will be found perfectly comprehensible by means of description without the aid of drawings or diagrams.

Piedmont appears to have been the birthplace of modules, for although irrigation is essentially Oriental in origin, owing to its extreme reproductive power in hot climates, and though it was introduced into Europe by the Moors, we do not find, either in India or in Spain, where portions of these works still exist, anything

approaching to a module. The systems employed in carrying out irrigation almost prove that they had not such a thing at all. In India the practice seems to have been to turn water on to a field until either the landowner or the turner-on of water was satisfied, or perhaps rather until the landowner was satisfied that he could get no more. No doubt this was the best plan to start with, as the object of irrigation was to water the fields sufficiently ; and the landowner being the best judge as regards how much water was required for his crop, this mode insured the observation of the proper persons. This plan was, however, open to one very serious objection ; when the landowners discovered that an extra amount of water beyond that strictly necessary for the crop was in some cases capable of increasing the amount of produce to a small degree, they would take more water, either by stealth or otherwise ; the amount of perpetual squabbling on this subject would then have been very large, had it not been for the fact that in Oriental countries irrigation works were made by rajahs, emperors, or chiefs, whose despotic rule and despotic institutions supplied a very practical limit in such matters—moral or physical force.

In Spain, under Moorish rule, it is probable that this useful substitute for modules was also in vogue ; but in the huertas or irrigated lands of Spain, in more modern times and under Christian rule, the water being the joint property of several villages that combined to keep the works in order, and legislated for themselves about the distribution of the water, the first great step, the just division of the water on a large scale among the several villages, had to be regularly carried out. The canals being comparatively small, a proportional division

was effected by equalising the size of a certain small number of outlets from the main canal into the subsidiary channels, one village thus taking a fourth or a sixth of the total volume of water passing down the canal.

In Piedmont the conditions were different; the country being hilly, and the water taken from streams and torrents having a considerable fall, water power was extensively used for driving corn mills. It is probable that there were a few water-driven corn mills both in India and in Spain, but there such mills would be public institutions, the miller being a servant of the community, generally living on a fixed income, or yearly pay, given either in kind or in money by all the neighbouring villages using the mill. In Piedmont the mills were the private property of individuals, as they are at the present day in Europe; hence it was there that the first unit of water measurement was arrived at—the amount of water enough to drive a corn mill, which was probably then and there of about the same size and requirements. This amount of water then assumed a technical name, the *ruote d'acqua*; the same thing in Lombardy being called a *rodigine*, in Modena a *macina*, and in the Pyrenees a *moulán*—the same circumstances in various places leading to the adoption of a similar unit of measurement, which was naturally rather variable. In Piedmont the amount was generally about 12 cubic feet per second, and was supplied by an outlet about 1·60 feet square, the water issuing free from pressure at the surface level. The next step was the introduction of a smaller unit of measurement for purposes of irrigation for discharges under pressure, the Piedmontese *oncia*; which was a rectangular outlet 0·42 ft. broad, 0·56 ft.

high, having a head of water 0·28 ft. above the upper edge of the outlet ; its discharge was 0·85 cubic feet per second, and this was the immediate parent of the Piedmontese module, and, as far as we know, the ancestor of all modules.

Piedmontese Modules.—These, the most perfect type of which is that of the Sardinian code, were designed or intended to fulfil the following conditions: that the water should issue from the outlet by simple pressure, that this pressure should be maintained practically constant, that the outlet should be made square in a thin plate having vertical sides, that the issuing water should have a free fall, unimpeded by any back-water, and that the water of the canal of supply should rest with its surface free against the thin wall or stone slab in which the outlet was formed. The following is a description of the general type. The water is admitted through a sluice of masonry, having a wooden shutter working vertically, into a chamber in which the water is supposed to lose all its velocity and is kept to a fixed level mark by raising or lowering the shutter ; the chamber is of masonry and has its pavement on the same level as the sill of the sluice, the regulating outlet from this chamber being an orifice 0·65 feet square, having its upper edge fixed at 0·65 feet below the fixed water-level mark of the chamber. Its discharge is 2·04 cubic feet per second. If a larger discharge at one spot be required, the breadth of the outlet is doubled or trebled, the other dimensions remaining unaltered. Such are the sole unalterable conditions or data of this module ; all its others seem to have varied very greatly ; its sill is sometimes at the level of the bed of the canal of supply, sometimes above

it, and sometimes below it, in which case a slight masonry incline was made from the bed down to it ; the length and breadth of the chamber vary greatly, the former from 15 ft. to 35 ft., its form being circular, oval, or pear-shaped ; the side walls splaying outwards sometimes close up to the sluice, sometimes not till near the regulating outlet, the object being to destroy the velocity of the water within the chamber. The lower edge of the regulating outlet is generally, but not always, placed at 0·82 feet above the floor of the chamber. The paved floor of the chamber is in many cases, but not in all, continued at the same level beyond the outlet.

The practical advantages of this type of module consist, therefore, in having a chamber in which the water can be kept to a constant level, and from which the water can issue under a constant head of pressure through a regulating orifice of fixed dimensions.

Milanese Modules.—The *modulo magistrale* of Milan is the most improved type of Lombard modules, the *modulo* of Cremona and the *quadretto* of Brescia being very inferior to it in design, its principal advantage over the Piedmontese module being the fixity of dimension of almost all its parts ; in other respects it resembles it very much, the principal differences being that the water chamber is always rectangular and covered with slabs, and is hence called the covered chamber, that its flooring has a reverse slope in order to deaden velocity, and that the masonry channel beyond the regulating outlet has fixed dimensions also, a portion of it being called the outer chamber. In its general arrangement, the sluice of supply has its sill invariably on a level with the bottom of the main canal, which is

paved with slabs near it ; the breadth of the sluice is the same as that of the regulating or measuring outlet ; the sluice gate is worked by lock and level, being fixed and locked at any required height by catch lock and key. As to dimensions, the covered chamber is 20 ft. long, its flooring having a rise of 0·15 feet in that length, and its breadth is 1·64 ft. more than that of the sluice of supply, that is, 82 ft. more on each side ; the lower surface of its covering of slabs or planks is fixed at 0·33 feet above the upper edge of the regulating outlet, which is the height to which the water must be kept to secure the fixed discharge. In order to gauge the water in the chamber, a groove is made in the masonry so as to allow a gauge rod to be introduced within at the sill of the sluice, which will read 2·29 feet of water above the sill, when the proper head of pressure exists ; should it read more or less, the sluice gate must be raised or lowered. The outer chamber is 0·66 feet wider than the measuring or regulating outlet, its total length 17·79 ft. ; its side walls, which like those of the covered chamber are vertical, have a splay outwards, so that the width at the farther end is 0·98 feet greater than at the outlet end, that is to say, it is there equal in width to the covered chamber. To insure a free fall, the flooring of the outer chamber is 0·15 feet below the lower edge of the outlet, and has besides a fall of 0·15 feet in its length of 17·72 ft.

The total length of the module is nearly 37·75 ft., but its breadth is variable, according to the amount of discharge required. If intended to discharge a Milanese *oncia magistrale*, the Milanese unit, which varies from 1·21 to 1·64 cubic feet per second according to different computations, averaging 1·5 cubic feet per second, the

measuring outlet is 0·66 feet high and 0·33 feet broad, under a constant head of pressure of 0·33 feet; the breadth of the covered chamber being 2·13 feet and the breadths of the open chamber 1·15 feet and 2·13 feet.

It is essential to the effective operation of the regulating sluice that the difference of level between the water in the canal and that in the module be at least 0·65 feet; and as the height of water in the latter must be 2·29 feet, the depth of water in the canal must never be less than about 3 feet, in order to allow the module to work properly. The following are the relative levels of the parts of the module referred to the bottom of the main canal as a datum :

| | Feet |
|---|------|
| Water surface in the interior of the module . . . | 2·29 |
| Upper edge of the measuring outlet . . . | 1·96 |
| Upper end of flooring of open chamber . . . | 1·14 |
| Lower end of the same | 0·98 |

Such is the type of the Milanese modules, the dimensions being suitable for a discharge of 1·5 cubic feet per second; unfortunately, in point of fact, the type has been rarely adhered to rigidly, and thus its advantages as a universal, or even as a local water standard have been comparatively thrown away in practice. Its use, however, established a discovery that was at that time very important, viz., that larger outlets gave a greater discharge than that due to the proportion of their section for small ones; it was therefore determined that no single outlet of a module should be made for a discharge of more than eight oncia or 12 cubic feet per second; and when a greater discharge was required, two or more separate outlets were to be used side by side. A gauge post was also found to be

necessary in order to enable the water guardians to adjust the sluice accurately.

The principal defect of the Milanese modules is that, owing to the rush of water from the canal, it is nearly impracticable to keep a constant head of pressure on the measuring outlet ; besides this, sand and fine silt vitiate the accuracy of amount of discharge.

Such are the comparatively ancient modules, the Milanese *modulo magistrale* being the most improved one of them. Their type has been very much adhered to in modern times ; that of Messrs. Higgin and Higginson on the Henares Canal may be considered as the greatest improvement that can be made on them, without departing from that type. In this module, the entrance by a sluice into a chamber for destroying velocity has been preserved, but the exit is an overfall, and hence more susceptible of exact measurement of discharge ; the means applied to deaden the velocity of entrance are again different.

The entrance into the channel through a wall is a passage 1·96 feet (·6 mètre) square, regulated by a well-fitting cast-iron door raised by a screw ; the chamber is rectangular, 10·37 ft. long, by 7·20 ft. wide below, 9·20 ft. above, the side walls having a batter of 1 in 6. The bottom of the chamber is horizontal and at a level 72 feet below the sill of the entrance sluice. To deaden the action of the water, a partition of masonry grating is built across the chamber at a distance of 4 ft. from the wall, and 5 ft. from the overfall wall of exit, it is 1·37 ft. broad, and has eight slits or vertical passages not cross-barred, each slit being 0·45 feet wide. The water having been deprived of all action by passing through this arrangement, enters the second portion of the chamber,

and then passes over a weir having an iron edge 6.56 ft. (2 mètres) long, fixed nearly on a level with the top of the entrance sluice, or 2 ft. above its sill. The discharge required for irrigation being never to exceed 176 litres or 6.22 cubic feet per second, the depth on the weir sill will therefore never exceed 0.5 feet, the sluice opening being 1.97 ft. square.

There are two small side walls having a batter from above on either side of the sluice entrance, these walls projecting into the main canal, in order to protect the entrance and prevent silt from accumulating there, which otherwise, and perhaps even in any case, would have to be dug out occasionally. In order to keep the chamber in proper working order, a keeper must be employed, and a gauge post erected in the canal, by reference to which he lowers or raises the sluice, and keeps the water in the chamber always at a fixed level.

It is evident that the changes may be rung on this species of module to a great extent without effecting great improvement, by increasing the number and altering the positions of the sluices and overfalls, and modifying the arrangement for deadening the action of the water. This has been done in many cases without much result; it is hence not worth while to bring forward other examples of this type.

Although some of these are complicated in form, as well as much varied in detail, the types are exceedingly simple; they all require the occasional attendance of a keeper for adjusting them according to the variation of pressure; they are made of brickwork and masonry, and consist of a series of open passages and covered chambers connecting orifices and overfalls. It is quite evident that, except under special circumstances, such

modules are far behind the wants of an age that economises labour, attendance, and supervision wherever possible.

Self-acting Modules.—A module to be of much use now must in the first place be self-acting. Nor, indeed, is this all. A large number of self-acting apparatus for regulating the supply or flow of water have been designed and used, but three-quarters of them do not answer all the purposes required of them at present. Some are large, some expensive, others involve a large expenditure in protective or additional large chambers, others are complicated and liable to get out of order, and others involve a great loss of head, which, in the case of their application to irrigation canals of small fall, is an insurmountable objection. The worst of them may be said to be those that fail in their main object in producing practical invariability of discharge. With all these objections to deal with, it will not be necessary to do more than make passing comments on the greater number of them, and the principles involved in their design and construction.

We will, however, first mention the requirements of a good module. The first consideration is that under all ordinary circumstances the discharge may be practically constant and correct, that is, should not be liable to vary more than 5 per cent.; secondly, that it should be very simple in construction and application; thirdly, that it should not be liable to derangement; fourthly, that it be portable, easily applied and removed from any portion of the canal without involving much waste or loss; fifthly, that it should not involve much loss of head, and that it should be able to drain the

main canal or basin of supply, down to a level of one foot above its bed, and deliver water if need be as high as within one foot of full level in the canal; sixthly, that it be inexpensive, not costing in England more than about 10*l.*, and more than 5*l.* additional for its attachments, slabs, cisterns, or chambers, and setting it in place in working order.

There are perhaps only three modules yet designed that may be said to fulfil these conditions; these we will for the present term portable modules, and defer dealing with them until after commenting on the others, or ordinary self-acting modules, some of which have advantages or disadvantages worthy of notice, or have attracted special attention in any way.

Until recently, the power of flotation was the sole means adopted in self-acting modules for obtaining an equal discharge under varying heads in the canal or basin of supply. The simplest manner of applying this is perhaps in attaching or fixing the pipe or pipes of supply to the float itself, thus insuring a fixed head of pressure on their entrance, however much the surface level in the supplying basin may vary. So far as this, the modules depending on this principle appear excellent, but unfortunately all of these seem defective on account of other considerations. For instance, in '*the suspended opening*,' where the water enters through two horizontal pipes into the body of the float itself (which is kept submerged to a sufficient depth by weights) and passes out of it through a vertical pipe fixed on to the lower side of it, the vertical pipe has to slide up and down in a species of stuffing-box in a masonry platform below, so as to discharge itself clear of the water in the main canal, and prevent the latter from leaking through

into the well below the platform, from which the moduled water alone should be drawn off. This is plainly a contrivance that would be defective for purposes of irrigation ; should the vertical pipe not slide easily into the stuffing-box, the power of flotation may be entirely neutralised ; should it be too easy, there will be leakage, and perhaps to a serious amount ; the loss of level is seriously great, the delivery level never being higher than 1 ft. above the bed level of the canal. Modifications of this contrivance, having in view the abolition of the loss of head, have been made by using syphons either erect or inverted, instead of the sliding vertical pipe. They certainly attain that object, but introduce new defects sufficient to render them less useful for purposes of irrigation than the original suspended opening ; they are expensive, and difficult to manage, the action of the syphons is liable to be stopped by accumulation of air, and their discharge is not only practically low in comparison with their theoretical calculated discharge, but also is variable, as they are very liable to foul ; their adjuncts, chambers around and attached, are expensive. The vertical pipe arrangement of the suspended opening is the principle on which many so-called water-meters, used by water companies for discharging water in large quantities, have been constructed.

The same principle has been adapted to purposes of irrigation in the module of M. Monricher, on the Marseilles Canal, constructed between 1839 and 1850 ; it is intended to supply irrigation channels having discharges of from 1.06 to 4.24 cubic ft. (30 to 120 litres) per second as a constant supply. The details of construction are as follows : A masonry reservoir 11.15 ft. by 14.76 ft., having its bottom at a level approximately 3 ft. below

the bottom of the canal, is connected with it by a rectangular masonry passage having a horizontal masonry covering at the level of low-water surface in the canal; a transverse masonry wall stops the action of the water, which enters the reservoir afterwards by two passages, one on either side, the wall and passages taking up a portion of the reservoir space. Beyond two pairs of grooves for putting in stop-planks for shutting off the water entirely during repair, there is no other sluice or check to the free flow of the water. In the centre of the rectangular reservoir is a cylinder of masonry, having an internal diameter of 2'30 ft., being 1'00 ft. thick, the bottom of it being approximately 2'00 ft. below the bottom of the reservoir, and its top edge about 2'00 ft. below low-water canal surface. An iron cylinder is made to fit the internal masonry closely, and to slide up and down it, and to hang by a rod and adjusting screw to a wooden bar supported by two wooden floats placed clear of the masonry, each of which is 1'64 ft. deep, 1'31 ft. broad, and 5'24 ft. long. There are also two vertical bars in the reservoir outside the floats, up and down which the bar slides on rings. The adjusting screw enables the iron cylinder, which is about 5'8 ft. long, to be placed so that its upper edge may be set at any depth below the water surface, so as to produce any required discharge. This, when once fixed and checked, is never altered. The whole is inclosed in a locked building.

The water of the reservoir therefore enters the iron cylinder above, and flows out below; the lower water being divided from the rest of the reservoir above by masonry partitions, it rises through the masonry passage thus made into the masonry water-course or irrigation

channel, the bottom of which is not more than 75 ft. below that of the bed of the main canal; the channel section is 2'00 ft. by 1'31 ft., having a small enlargement 3'28 ft. square at the commencement of the channel. Plans and details of the module here described are given in Moncrieff's 'Irrigation in Southern Europe.'

In this module, therefore, the section of outlet, viz., that of the iron cylinder, is constant; the edge of the cylinder rises and falls by flotation; the loss of level is as small as can be conveniently obtained in modules of this principle of design, and if the cylinder could, without much care or superintendence, be made to work well in the masonry without leakage or friction to any detrimental extent, as stated by the engineers of the Marseilles canal, the amount of inaccuracy of discharge cannot be great. It would doubtless be an improvement were some arrangement applied to this module for preventing silt from entering the reservoir, which must be liable to interfere with the working of the cylinder, and produce a greater deteriorating effect in this module than in many others. The masonry portion of the module would require good workmanship, and the putting together of the whole in good working order considerable care. It is, therefore, rather expensive, and certainly has not the element of portability.

The suspended plug is, like the suspended opening, a principle that has been adopted for modules and applied in a very large variety of ways, some of which involve complexity of parts and details. Its main principle is probably slightly more modern than that of the latter: both are decidedly old, but as these old contrivances are perpetually being re-invented, a brief description of

their principles may be of use to some, while comments on them may deter others from wasting their energies on an idea that appears to have been fully worked out.

The simplest case of the suspended plug is this. A circular orifice is fixed in a floor at the level of the bed or bottom of the canal or reservoir, and a plug of varying section is suspended in it, being attached to a float that rises and falls with the surface of the water; the annular water passage thus left open is made to discharge equal quantities under varying heads by proportioning the section of the plug throughout its length; the area of the annular opening being in inverse proportion to the velocity of discharge. To insure a free fall there is a well below the floor into which the water falls to a depth equal to that of the depth of the floor from high-water level of the canal. The depth of the float and its attachment to the plug prevent its acting at a depth of water of less than one foot in the canal. These two points, which are serious objections to the adoption of this module on irrigation canals, have been much modified in the more complicated modules constructed on this principle, which will hereafter be mentioned. As to the plug itself, it is either a conoid hung in a circular orifice, or a flat-sided conoid of equal thickness in one direction hung in an orifice which is rectangular laterally and of circular curvature transversely; in the latter case a fixed area is left open on the flat sides of the plug which has to be allowed for in the calculations for the section of the plug. The diameter of the plug in the case of the conoid is obtained by calculating the areas required to pass the required discharge for various heads of water, as, from 1 to 10 ft. for every three inches, and deducting these from the

fixed area of the orifice, the remainders are then the areas of the circular sections of the plug for those depths from which the diameters are obtained. The flat conoid can be made of the same lateral section for all discharges, the thickness of the flat sides being increased in direct proportion.

The following is an example of a module designed on the suspended plug principle, and is perhaps the simplest application of it in actual practice. It was designed by Don Juan de Ribera, projector of the Lozoya canal, or canal of Isabella Segunda, and is used on that canal with good effect.

It is so arranged that the size of the outlet diminishes when the head of water increases. The module itself is a long tapering bronze plug, 0.524 ft. in diameter at its lower end, and is attached to a circular brass float above, which floats freely in the water of a masonry well 3.38 ft. by 3.94 ft. square and 4.16 ft. deep; at the bottom of this well, which is on a level with the bottom of the main canal and the rectangular masonry passage connecting them, is a circular orifice 1.56 ft. in diameter, within which the lower end of the module is made to work vertically, the plug and plate being of bronze to prevent rust. Below this well again is a second one, into which the water falls after having passed through the ring between the orifice and the plug. The entrance of the rectangular passage leading from the canal, which is only about 3 ft. long, is protected from silt by an iron grating, and is covered in at the top by slabs to the full level in the canal; the well is also covered in by a locked iron trap-door. In this module friction is reduced to a minimum; the module hangs freely from the centre of the float, and can be slightly raised or lowered in order

to diminish or increase the discharge passing through the ring or space between the edge of the orifice and the plug; but when a constant discharge is required it is finally properly adjusted, and then entirely left alone. The float is about 2 ft. in diameter, having a thickness in the middle of about 0.9 ft., and at the edges of 0.6 ft.

This module discharges one cubic mètre (35.3166 cubic feet) per hour, and is hence styled an horamètre, the discharge being .2777 litres, or .0098 cubic feet per second. The curve of the module or bronze plug is such, that the roots of the vertical abscissæ vary inversely as the differences between the squares of the radius of the orifice and of the horizontal co-ordinate. Hence, if the required discharge is given with a head of water of one mètre, when the diameters of the orifice and plug are respectively .20 and .1653 mètres, then, if the head of water be reduced to .81 mètres, the diameter of the plug at the level of the orifice must be .1610 mètres, as

$$\sqrt{1} : \sqrt{.81} :: (.20)^2 - (.1610)^2 : (.20)^2 - (.1653)^2.$$

The lengths corresponding to the different diameters of the taper of the plug will, for a constant diameter of orifice of .20, be as follows:—

| | | | | | |
|---------------------------|-------|-------|-------|-------|-------|
| Depths from water surface | .10 | .12 | .16 | .41 | .77 |
| Diameters of plug | .00 | .0585 | .0912 | .1211 | .1374 |
| Depths from water surface | 1.26 | 1.90 | 2.71 | 3.71 | |
| Diameters of plug | .1480 | .1554 | .1610 | .1653 | |

The principle being that the velocity of discharge through an orifice varies with the square root of the head of water; thus, taking R r to represent the radii of the orifice and plug respectively, the discharge per second

$$Q = 0. \pi (R^2 - r^2) \sqrt{2gH},$$

Q 2

H being the head of water, the value of the experimental coefficient, o , being for this case deduced, from a series of experiments of Don Juan de Ribera, to be $\cdot 63$, in accordance with similar results obtained in ordinary practice in parallel cases. This is probably the module in most perfect accordance with theory yet designed; it is, however, of small dimensions, and hence likely to be much affected by even the very small proportion of silt that would pass through the grating. Its principal defect is, that the loss of level necessarily involved in it in order to obtain a free fall would render it inapplicable in a very great number of cases, where even a few inches of fall are of extreme importance.

The modifications of this type of module consist in putting the float in a separate chamber, which thus becomes a silt trap, and relieves the orifice from being affected by silt, the connection between the float and the cone being either a chain passing over two runners or a lever: in these cases the plug is reversed, having its broader end upwards; the friction involved affects the working of the module and its accuracy of discharge, and, in the case of levers, the lengths of the arms modify the quantities employed in the calculations of sections of discharge. In some cases the form of the lower well assumes various forms, having for their object the reduction of the loss of level existing in the more simple type. It is extremely doubtful whether any of these modifications can be considered advantageous on the whole.

Rising and Falling Shutters.—Contrivances of this type are generally suited for large quantities of water where great accuracy is not required. The falling shutter,

as used on canals in England or Scotland, is an oblique shutter hinged below, and raised or lowered in front of an opening in the side of the canal by two floats in recesses, the water passing over the upper edge of the shutter in a tolerably uniform volume. The rising shutter is a vertical shutter in front of an opening in the side of and down to the bottom of the canal; it is raised or lowered by means of a float attached to it by a chain passing over a runner, the float being in a separate chamber, and having trunnions and friction rollers running in curved grooves or recesses on each side of the chamber; these curves require very accurate construction in order that the discharges may not vary under different heads. Shutters of this description having pressure on one side only are very liable to stick, and get out of order; they are hence very inferior in practice, although new ones under favourable conditions can be made to work very accurately.

The above three types comprise the whole of the non-portable self-acting modules that have been much used in practice to good effect.

Portable Self-acting Modules.—In this class we comprise such modules as could be removed or replaced without much difficulty or loss. There are three such modules that have attracted attention, though there are probably others not so well known.

Carroll's Module.—The first is that of Lieutenant Carroll, of the Royal Engineers; its principle is exactly that of the well-known draught regulator: the pressure of the water is made to regulate the opening in the one case in the same way as an increased draught of air is

made to partially close the opening in the other ; and the application of the principle is excellent for the intended purpose—it can be made almost entirely of iron, is simple, effective, and admits of removal without causing much loss or expense. Drawings of this module are given in the Rurkhi Professional Papers.

Anderson's Module.—The second is a modification of the hydraulic lift regulator, invented by the late Mr. Appold, used to regulate the descent of hydraulic passenger-lifts under a variable load ; it has been applied to its new object by Mr. W. Anderson, of the firm of Eastons and Anderson, and in some respects resembles the module of Lieutenant Carroll : the velocity through the pipe of discharge is, however, in this case made to move a suspended plate of curved form, in front of an opening also fixed inside the pipe, and the opening is therefore reduced by increase of velocity.

In December 1866 some experiments were made with a 6-inch Appold regulator at the request of Col. Smith, consulting engineer to the Madras Irrigation Company, and of Mr. Clark, hydraulic engineer to the Municipality of Calcutta.

In one experiment, in which the regulator was used to discharge water from a tank 7' 7" square internally during 13 minutes, the surface of the water in the tank sank as follows, in one-minute intervals : $3''\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{5}{16}$, $3\frac{1}{4}$, 3, $3\frac{3}{16}$, $3\frac{1}{8}$, 3, $3\frac{1}{16}$, 3, $3\frac{1}{16}$, $3\frac{1}{8}$, $3\frac{3}{8}$ " ;—the total quantity discharged in 13 minutes was

$$= 7' 7'' \times 7' 7'' \times 3' 5\frac{1}{2}'' = 197.22 \text{ cubic feet,}$$

or about 15 cubic feet per minute.

In the second experiment, the surface of the water in

the tank sank as follows, in one-minute intervals: $3''\frac{5}{16}$, $3\frac{5}{16}$, $3\frac{3}{8}$, $3\frac{1}{2}$, $3\frac{5}{16}$, $3\frac{1}{2}$, $3\frac{3}{8}$, $3\frac{3}{8}$, $3\frac{3}{8}$, 3 , $3\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{3}{16}$, $3\frac{1}{4}$, $3\frac{3}{8}$, $3\frac{5}{16}$, $3\frac{9}{16}$, $3\frac{1}{16}$, $3\frac{5}{8}$, $3''\frac{5}{8}$; the total quantity discharged in 20 minutes was

$$= 7' 7'' \times 7' 77'' \times 5' 8'' = 323 \text{ cubic feet,}$$

or about 16.13 cubic feet per minute.

In the latter case the heads at the beginning and the end of the discharge over the centre of the pipe were 22.8 feet and 12.24 feet.

In each case the same regulator or module was used; its square aperture on the delivery side was $5''\frac{17}{32}$ high, and $3''\frac{1}{16}$ broad, or a section of $20''\cdot 35$; the swinger was $3''\frac{7}{8}$ wide, nearly touching at top and bottom; the case $5\frac{1}{4}$ wide, and the area for water passage $8\frac{9}{16}'' \times 1\frac{3}{8}'' = 11''\cdot 77$ in section.

Two of these Appold's modules are it is believed in use on the Tumbaddra canals of the Madras Irrigation Company. From the convenience of form that this module possesses, being self-contained, and externally a simple iron tube, with an enlargement like a box in the middle of it, that admits of being attached or detached from an orifice very rapidly, it would appear to be preferable to that of Lieut. Carroll, and less liable to damage in transit.

The equilibrium module.—The third portable self-acting module is the design of the author of this work, and is named the Equilibrium Module. It consists in the first place of a box or chamber, having an entrance and an exit orifice, and one or two air-holes above; within this box is the pipe leading horizontally from the entrance orifice for a short distance and then turning

vertically upwards; this is terminated by a dead end, but has two or four slits or narrow vertical openings in the sides, through which the water passes when the module is open and working. There is at all times enough water within the chamber to rise above the level of these openings, and to work a float above them; this float, working vertically, raises or lowers the cap that slides over the head of the pipe, and gradually opens or closes the slits in accordance with the variation of the level of water in the chamber; which is below the low-water surface of the canal or tank of supply. The form of construction adopted reduces to a minimum the depth from the water-level within the chamber to the openings, which discharge above the sliding collar, and thus causes the loss of head to be unimportant.

This is also a small module, possibly only a quarter larger than the Appold module before mentioned, and equally convenient as regards portability; it is simple in design, being actually little more than one of the old types of equilibrium steam valve applied as a module in a chamber under pressure: it could, however, be made of any size, the adjustment of the sizes of the orifices of entrance, of exit, and of the slit-openings being the only important points of variation. It might also, for rough purposes, be made generally of stone-ware, and the pipe would then be square in section and have only two slits, the other two sides forming part of the box. This module slightly resembles the old cylinder sluice, which is also a modification of a double beat steam valve; the latter, however, is not so simple, being far more liable to choke or get out of order, one of its valves working within the pipe, and it is therefore not so effective in constant use as any of the three already mentioned.

Modules have been here treated as principally intended for regulating irrigation ; the reason of this is that the requirements are then more stringent in many particulars. A module for water supply of other kinds, (frequently termed a water-meter, although possessing regulating power) generally acts under greater head and freedom from silt, and may hence be of coarser design.

2. THE CONTROL OF FLOODS.

The prevention of the submergence of land by inundations from overcharged rivers, and the drainage from marshes and submerged land of the water that has been allowed to accumulate over it, are kindred engineering problems that appear at first sight to present but little difficulty. Their theoretical solution, when merely on a small scale, is ready and simple ; on a larger one, however, the practical details brought into these problems affect them to such a degree, that, although the principles involved cannot be said to be subverted, their carrying out is forced into a comparatively new form.

Land liable to submergence from a river is lower than the extreme flood-level, and in open communication with it ; the remedies consist, therefore, either in lowering the extreme flood-level in the channel by providing other passages for the water, partially diverting it, or dredging out a deeper channel, or by warping up the land liable to submergence, or by cutting off possible communication in flood stages between the river and the land by means of embankments. Submerged land, again, remains in that condition for want of sufficient natural outfall ; an outfall has, therefore, to be cut, tunnelled

dredged, or enlarged to a sufficient extent to allow gravity alone to do the work, should that be possible or economically sufficient ; in other cases pumps are indispensable.

Imagining, then, the case to be one of an area of a few hundred acres, liable to inundation from a river with a moderate declivity, the application of these principles involves generally but little difficulty as regards engineering, and becomes a local economic question, rather than an engineering practical problem. Putting the case again on a large scale, a vast tract submerged by the floods of a river having a very small declivity—the usual condition when large areas are submerged—the dimensions entering into the works that would be necessary in adhering rigidly to the above principles become so large, that their complete execution is positively impossible in most cases. Let us adduce the embankments of the Ganges, the Mahanaddi, the Po, and the levées of the Mississippi, which are not and never can be complete and sufficiently developed to insure, by means of themselves alone, the absolute protection of all the lands on their banks from the devastating effects of extreme floods.

To this it might, though perhaps rather thoughtlessly, be replied, that very extensive works may be so costly as to be impossible, but that the application of the principles need not vary. It is, however, in point of fact also a matter of modification of the application of principle.

The case of a comparatively small river supplying the flood, very nearly, and in most cases totally, limits the consideration of the flood to its principal point, the extreme flood-level ; the catchment area of a small river

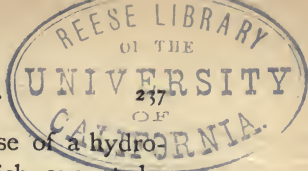
being tolerably uniform supplied throughout the rainfall, its upper portions do not require very special consideration ; the declivity of the small river being tolerably rapid, the condition of the lower ranges of the river does not affect the matter to any very important degree. Remote local conditions being comparatively disregarded, and it being possible to cope with the flood at the required point both successfully and economically, the works involved are necessarily small.

On a large scale, on the contrary, the extreme flood level, the nature, causes, and duration of the flood may be greatly affected by any of the physical conditions of the entire catchment area of the region watered by the river and its tributaries, from the loftiest hill on the watershed down to the currents of the ocean, miles beyond the river's mouth ; and as these physical and meteorological conditions vary greatly throughout large countries, a perfect knowledge of them as regards the country under consideration is absolutely necessary in order to arrive at sufficient information to enable one to propose measures for the mitigation of the effects of the flood. In other words, the natural drainage of the whole region under any state or circumstances, as well as everything that practically affects it in any way, must be thoroughly known in detail.

It will be unnecessary to dilate on the physical laws and conditions of our sphere, matters best understood from studying the larger works on physical geography to be found in any good library : and a knowledge of these will hence be assumed. The detailed knowledge, however, of the special physical conditions and rainfall of the region under consideration, may possibly not be obtainable from any book whatever. It is not sufficient

to possess meteorological statistics of observations taken at a few towns in the valley of the river, and at one or two points or villages on the hills; it is needful to know definitely what is the greatest amount of rain that ever falls in the region, the greatest area in it over which rain falls at any one time, and which portions of the area they are likely to be at any time; or generally how much water, when, and where, so that it may be practically accounted for. Detailed observations taken for many years at a very large number of meteorological stations are therefore requisite, and it is almost painful to reflect in how very few instances are even a moderately small number forthcoming. As a notable exception to this apparent apathy, may be noticed the large number of meteorological stations in the United States of America, and the large sum annually spent by their Government in obtaining such information. Besides the meteorological data, a correct detailed topographical and hydrographical knowledge of the whole of the catchment of the river, based on engineering surveys and velocity observations, is necessary in order to determine the discharge and the flood level of the river at any time, and under any possible meteorological condition. Having all this information we are enabled at any time to state what will be the results in rise and amount of discharge of the river, corresponding to and resulting from any special rainfall lasting for any usual or unusual time over an area, or detached portions of area within the catchment basin, and the evils to be contended with are then fully known before commencing to deal with them and attempting to mitigate their ill effects by means of engineering works of any sort.

To this it may be replied, that the expense of ob-



taining all these data, and especially those of a hydrographical and topographical nature, which cannot be done except by skilled hydraulic engineers, must necessarily be very large ; and if after all this it should be discovered that under any circumstances no engineering works could remove the evils, or even moderate them to an important extent, the expense would have been uselessly incurred.

Not entirely so. Even should no works be attempted, the information can be made use of in the protection of human life, and in thus mitigating the fearful effects produced by sudden and devastating floods. The extent of land liable to submergence under certain conditions of rainfall in any part of the country being known to a practical certainty, the telegraph can be employed to warn the inhabitants of an impending flood, and allow them to save at least their own lives, and perhaps also that of their cattle and movable valuables. It may be urged that the terrible catastrophes resulting in large loss of life generally commence with the bursting of an embankment, which happens before the flood overtops it ; doubtless it is so, but it would be an important part of the topographical knowledge to ascertain to what height of flood these embankments, which, when in sound condition, are in most cases only sufficient protection against very moderate floods, are practically safe. Timely warning could, therefore, be afforded in any case, and the inhabitants would be spared the terrible infliction, in case of flood, of watching the waters rising, and not knowing either how much higher they might rise, or to what height of flood their dams might be safe.

But to proceed to the main object, the protection of

the land, as well as its inhabitants, when the matter is one of large extent and importance.

The usual practice hitherto, notably in the case of several districts in Holland, seems to have been, to construct continuous lines of embankment along all the existing edges of the various channels of the river, and to discharge the waters within them on the flooded land into the rivers by means of pumps. This caused, no doubt, a certain amount of mitigation of evil up to certain height of flood level only ; beyond that, it is sufficiently evident in theory, and has been fully established in practice, that the means employed cease to be a remedy, and become a decided aggravation of the cause of disaster, effecting an excess of external pressure on the embankments. Besides this, as the channels of the river are under these circumstances allowed to silt themselves up, not only the bed level, but also the flood level corresponding to the same amount of discharge, is allowed to rise also ; a second aggravation of the evil. Beyond this again, the immense length of these circuitous embankments causes them to be exceedingly costly. These three reasons will, it is hoped, have sufficiently demonstrated the fallacy of employing the means that are occasionally appropriate on smaller works to those of large extent.

Before entering into the subject of works based on better principles, let us first examine the conditions of a flood under circumstances that admit of observation.

Let us imagine ourselves to be standing on the bank of an Indian river, as wide as the Thames at Hammer-smith, in a mansun season of unusually high rainfall, the maximum annual rainfall being 74 inches, the day maximum 7 inches. The mansun, or periodic rainy

season, has set in tolerably mildly ; the river swells, increases in depth and velocity, and is discoloured at first ; this afterwards passes away, and the water then runs steadily, tolerably clear. The rain increases in the plains, and the sky gives prospects of a heavy storm in the direction of the uplands of the river. Let us watch the effect. The rainfall of the plains, in fact the down-pour all around us, increases the depth and the velocity of the river, but its colour is unchanged, in fact it seems nearly pure. Suddenly a roaring of waters, like that below an overtopped mill weir, is heard, and up stream we notice a white line of foam approaching ; three or four minutes, and a flood sweeps by on the surface of the river, like a wall of water 3 or 4 feet in height ; all this water is muddy and dark with detritus. The waters after this again rise still higher for twenty-four hours, but are yet muddy ; the low-lying lands near the river are submerged. We learn afterwards that a considerable fall of rain has taken place in the uplands of the river, and that towns and villages in the plains have been inundated.

Such is the flood, its subsidence is a matter of less moment ; and such is the type of flood to which those causing serious catastrophes generally belong. In this case we fully satisfy ourselves of the rationale of the flood ; the lowland water rises steadily and clear, going perhaps one mile an hour ; the upland water comes down with a velocity of nearly six miles an hour and charged with silt—for where else is this velocity and this silt to come from except from its course in the hills?—and tops the lowland water ; the combination of waters gradually decreasing in speed spread themselves out over the land in the first locality, where the form of

channel and banks admit of it, and perhaps in more than one, extending even for miles beyond the natural bed of the river.

How is such a flood to be controlled? Apart from the Dutch principle, already shown to be fallacious on a large scale, there are only two methods, either or both of which can be adopted. The first, the improvement of the whole of the natural drainage lines of the country to such an extent that the velocity of the waters may under such circumstances be increased throughout the whole course of the river, and a little beyond it, into the sea or next large river, and so that the natural bed, thus improved, may be sufficiently large to carry off any previously known flood, without being exceeded. The second, any means of separating the upland from the lowland waters, holding or retarding either the one or the other, or portions of either one or the other, and providing for their discharge either separately in different courses, or at different times in the same watercourse.

Let us first indicate the nature of the works requiring execution, when the former principle alone is adopted: the perfecting of the natural lines of drainage.

The ultimate free delivery of the water into the sea, or any way entirely free of the river, is perhaps the most important point of all, the low-lying lands on the lower ranges of the river being there more extensive than elsewhere; to insure a free delivery, the main outlet of the river should be carried out to deep water, protected on both sides by banks or jetties, against the shore currents, and so directed as to avoid as much as possible the retarding influence of sea storms; through the delta, also, a single direct channel of properly determined dimensions should be made and protected

by embankments; by these means the mass of water will, in forcing its way in this course to the sea, scour for itself a deeper bed at the outfall and throughout the lower ranges of the river, and carry off floods more rapidly, improving the river continually. A further advantage from confining the river to one channel is that of the reclamation of a large amount of land previously occupied by marshes, as well as by the numerous old channels of the delta.

In the middle ranges of the river the works to be adopted are all such as will promote a more rapid discharge: the enlargement of the bed wherever it is contracted or narrowed; the removal of obstacles, rocks, small islands, silt deposits, shoals, or anything that impedes velocity; the straightening of the course wherever it can be done to good effect; the prevention of the deposit of silt in such places as would be objectionable; the deepening or dredging of the bed in the requisite places: the whole course to be put under a regimen that would remain constant generally, and besides continue to improve itself by scouring in contradistinction to its former habits of silting up and causing its flood levels to rise.

In the uplands, all the works which should be constructed are those that have for their object the control of the detritus washed down, and the prevention of its deposit at unfavourable spots. If the silt could by any means be entirely prevented from being carried down into the middle ranges of the river, or into the plains, it would be a great achievement; but this being hardly possible, palliative measures are perhaps all that can be adopted. Besides this, the hills might be covered with thick plantations, which, catching the rainfall, would

delay its departure, prolong the duration of the flood, and thus lessen the amount of flood water passing off at any one time, or mitigate the flood.

The necessary works dependent on the second of the principles previously mentioned, would be so greatly dependent on local circumstances that they can only be indicated generally. The separation and control of the water from the uplands can be attained by making storage reservoirs at certain places at the foot of the hills, and running all the water falling on them into these by means of catchwater drains skirting the bases of the hills; from these reservoirs the water can be allowed to escape under control into the main watercourse; or, if practicable, the upland waters may be discharged through very large catchwater drains, independently of any reservoir, into some other collateral watercourse that may be convenient, employing even, if necessary, a separate outlet for the discharge into the sea of the upland waters.

In the case, however, of the main river or watercourse being employed as the outlet for the upland waters, it becomes necessary to separate the lowland waters from them as long as possible. In order to do this, the arterial drainage lines of the plains on each side of the main river require rectifying and improving; their waters then have to be cut off from it, and carried by two canals down the valley of the main river as far as some point where it may be advisable to discharge them into it through regulating sluices, or, if preferable, into some artificial reservoirs or lakes. These latter works would insure the additional advantages of perfecting the entire drainage of the country, and of having a good water-supply for irrigation.

The adoption of the two principles thus described would insure a perfect remedy and an effective control of floods under any practicable circumstances. That such works would necessarily be expensive there is no doubt whatever, but they would still be less costly and more effective than the continuous lines of embankment designed on the fallacious principles before quoted ; the works again would improve the rivers instead of deteriorating with lapse of time, and the gain by reclamation and irrigation would, apart from other collateral advantages, yield a profitable return.

3. TOWAGE.

Recent experiments show that the pull on the tow-rope of a barge is, within practical limits, proportional to the square of the speed, and that it varies widely according the form of the barge ; assuming then a general formula,

$$R = b T V^2$$

where R is the resistance in lbs.,

T = the displacement of the barge in tons,

V = the velocity through the water in miles per hour,

and b is a coefficient depending on the form of the barge.

It has been found that for the small and bluff barges of about 70 tons employed on the Thames, and for limits of speed not exceeding 5 miles an hour, the coefficient

$b = \frac{1.5}{\sqrt[3]{T}}$ or generally about 0.369 ; and that for well-

formed barges of medium size,

$$b = \frac{0.75 \text{ to } 1.00}{\sqrt[3]{T}} \text{ or generally about } 0.170 ;$$

and for the best ship-shaped barges with good lines, as those employed on the Danube wire-rope system, which have a length about eight times their beam, and are about 287 tons' displacement,

$$b = \frac{0.5 \text{ to } 0.6}{\sqrt[3]{T}} \text{ or generally about } 0.109.$$

The limit of speed for ships will be about 10 miles an hour, and beyond these limits the resistance R would vary with the fourth power of V ; but within the assumed limits, calculations may be made on the above data.

The number of horses required to draw a train of barges may hence be readily deduced. The best performance of a draught-horse working 8 hours a day, is assumed to be at the speed of $2\frac{1}{2}$ miles per hour, when he will exert an average pull of about 120 lbs. ; substituting this value in the above formula, we obtain for the tonnage that one horse will pull at the speed of 2.5 miles an hour in still water,

$$T = \frac{R^3}{0.17 V^2} = \frac{120}{0.17 (2.5)^2} = 113 \text{ tons.}$$

In a current, the resistance or the pull upon the tow-line will increase as the square of the speed through the water, but the horse in this instance moving over the ground is going at a less speed than that of the boat through the water ; and this is an important distinction, which must not be overlooked in estimating the effect of a current. The mode in which the necessary correction must be effected will be best illustrated by an example.

Referring to the last example, let us assume that the barge of 113 tons' displacement encounters an adverse current of 1 mile an hour, and it is required to know the reduced speed at which the horse will then go, assuming him to be performing the same average work per hour.

In the last case, the said work in mile-pounds was $120 \times 2.5 = 300$ mile-pounds per hour; in the present case the pull upon the rope will be proportional to the square of the velocity through the water (V), and the pull the horse is capable of pulling will be inversely proportional to the velocity at which he is travelling (v); and the difference between these two velocities will be the speed of the current (v_1); we have therefore

$$V = v + v_1 \text{ where } v_1 = 1 \text{ mile per hour}$$

$$R = .17 T V^2$$

and $Rv = 300$ mile-pounds per hour

$$V^2 (V \pm v_1) = 15.4$$

whence $R = 19.4 V^2$, and $V^3 - V^2 = 15.4$.

Solving which we obtain $V = 2.86$ miles per hour, the speed of the boat through the water; and the speed past land, or rate at which the horse is going, will be $2.86 - 1 = 1.86$ miles an hour.

It will be observed from this example that the influence of the current is relatively less important when horses are employed, than when steam-tugs, either paddle or screw, are used, the reason being that in the latter case the reaction operates upon the moving current, whilst in the first case against the immovable tow-path. Thus in the present example, if the power, instead of being an animal moving on the tow-path, had been a steam horse in a tug, the speed through the water would be the same, whether the water was still, or ever so rapid

a current. In this instance 2·5 miles an hour the speed past the land, which is the useful result, would be reduced to 1·5 miles an hour in the case of the tug, instead of to 1·86 when horses are used.

The difference of conditions will be more strongly marked if we assume the current to be 2·5 miles an hour, because then it is obvious that the steam tug, capable of moving through still water at that rate, would simply simply maintain its position if it encountered such a current; and although the paddle-wheels or screw would be revolving at the same rate as before, the only result of their effects, namely, the maintenance of position of the boat, would be equally attained if she dropped anchor; in short, the whole power exerted would be thrown away. In the instance of the barge towed by horses, on the other hand, the whole power exerted would be utilised; and it may be shown by the same reasoning as in the last example, that the 113 ton barge would be towed by one horse against a current of 2·5 miles an hour, at the rate of $1\frac{1}{6}$ miles an hour.

Obviously the same reasoning would apply, whether the motive power on the tow-path were horses or a locomotive, or whether the tow-path were dispensed with, and a rope were laid down in the bed of the river, and coiled round a drum in a steam-barge in the manner now generally admitted to be the most economical mode of conducting heavy traffic at a slow speed in rivers of rapid current and on still-water canals.

From the above we may conclude that, in order to tabulate for the effect of a current on the diminution or increase of speed of a horse, we have to calculate the increased or diminished value of V , the velocity through the water, and apply it in the general formula—

$$R = bT V^2$$

inserting different values for the constant b , which lie between $\cdot 109$ and $\cdot 369$, according to the form of the barge.

In the above case $R = 120$ lbs. for a draught horse ; but for other animals corresponding values of R , with reference to their best continuous speed, can be applied.

Assuming a case of a current of 3 miles an hour, and that the ordinary limits for the speed of the horse in towing a load with and against stream, are 4 and 1 mile an hour respectively, the velocity through the water becomes 1 and 4 miles an hour, and the loads 706 and 44 tons, the horse performing the same average work, but executing the average pull of 75 lbs. with stream, and 300 against it.

The values required are given for the limits in the following form.

For barges having 113 tons' displacement, and a coefficient $b = 0\cdot 17$, the results are as follows:—

| With a current v_1 | | | In still water | Against a current v_1 | | |
|----------------------|-----------|----------|----------------|-------------------------|-----------|-----------|
| $v_1 = 3\cdot 0$ | 2\cdot 5 | 1\cdot 0 | | 0 | 1\cdot 0 | 2\cdot 5 |
| $V = 1\cdot 79$ | 1\cdot 88 | 2\cdot 2 | 2\cdot 5 | 2\cdot 86 | 3\cdot 66 | 3\cdot 97 |
| $v = 4\cdot 79$ | 4\cdot 38 | 3\cdot 2 | 2\cdot 5 | 1\cdot 86 | 1\cdot 16 | \cdot 97 |
| $V_1 =$ | 5\cdot 00 | 3\cdot 5 | 2\cdot 5 | 1\cdot 5 | 0 | -0\cdot 5 |

Here v_1 is the velocity of the current, whether favourable or adverse.

V is the velocity of the barge through the water.

v is the speed of the horse.

V_1 is the velocity through the water for the case in which a steam-barge is used, and is given to illustrate the comparison. The foregoing formulæ on towage

were denounced by a reviewer in 'The Engineer;' apparently the critic had confounded formulæ for resistance with those for horse-power; yet a reply forwarded to the denunciation was not published in the paper referred to. A more important paper would have been great enough to acknowledge a blunder: the attempt to shelve it has not succeeded.

4. ON VARIOUS HYDRODYNAMIC FORMULÆ.

The results of the various formulæ given for determining discharges, according to various authors, vary very greatly; and it is hence interesting to examine them in a tabulated form in comparison with measured discharges.

The following data of comparison are given by Mr. David Stevenson, and by Captains Humphreys and Abbot; they apply to four cases of river discharge, from a small stream up to the Mississippi; thus including all limits within which such formulæ are required.

1. For a small stream of 24 cubic feet per second. Mr. David Stevenson made careful measurements, and velocity observations, and compared the deduced results with the results of formulæ, thus:

| | |
|---|-------|
| 1. Deduced discharge | 24'22 |
| 2. By Dubuat's formula | 32'50 |
| 3. By Robinson's formula | 36'90 |
| 4. By Ellet's formula | 46'40 |
| 5. By Beardmore's tables | 38'92 |
| 6. By Downing's formula, coefficient 1'00 | 41'23 |
| 7. By Leslie's formula, coefficient 0'68 | 28'04 |

2. For a river of 2424 cubic feet per second. Mr.

David Stevenson and Dr. Anderson made velocity observations on the Tay, at Perth, and the comparisons are thus :

| | |
|---|------|
| 1. Deduced discharge | 2423 |
| 2. By Dubuat's formula | 2987 |
| 3. By Robinsōn's formula | 2560 |
| 4. By Ellet's formula | 2033 |
| 5. By Beardmore's tabular formula | 2609 |
| 6. By Downing's formula, coefficient 1'00 | 2769 |
| 7. By Leslie's formula, coefficient 0'68 | 2083 |

It is unfortunate that in these two cases the hydraulic data, which would enable us to extend the comparison to other formulæ, are not given.

3. For a large river of 31 864 cubic feet per second ; the data of the Great Nevka, measured by Mr. Destrem were as follows :

Area of section 15 554 sq. feet ; width 881 feet ;
 discharge 31 864 c. feet ; perimeter 893 „ ;
 mean velocity 2'0486 ft. per sec. ; max. depth 21 „ ;
 hydraulic slope 0'000 014 87 :

The following are the results due to these data calculated by various formulæ and compared with the actual discharge :

| | |
|---------------------------------------|--------|
| 1. Deduced discharge | 31 864 |
| 2. Young's coefficient | 21 102 |
| 3. Eytelwein's coefficient | 23 389 |
| 4. Downing's coefficient | 25 031 |
| 5. Dubuat's formula | 16 931 |
| 6. Girard's formula | 22 491 |
| 7. De Prony's canal formula | 22 357 |
| 8. Young's formula | 19 777 |
| 9. Dupuit's formula | 23 456 |

| | | | | |
|--------------------------|---|---|---|--------|
| 10. St. Venant's formula | . | . | . | 21 811 |
| 11. Ellet's formula | . | . | . | 13 807 |
| 12. Humphreys' formula | . | . | . | 39 938 |

4. For a very large river, the Mississippi at Carrollton, the measured data at high water in 1851, were,

Area of section 193 968 sq. ft. ; width 2653 feet ;
 discharge 1 149 948 c. ft. ; perimeter 2693 „ ;
 mean velocity 5'9288 ; maximum depth 136 „ ;
 hydraulic slope 0'000 020 51 ;

and the corresponding results, which are kept in terms of mean velocity to lessen the figures, were,

| | | |
|-------------------------------|---|------------------------|
| 1. Deduced mean velocity . | . | 5'9288 feet per second |
| 2. Young's coefficient . | . | 3'2400 „ „ |
| 3. Eytelwein's coefficient . | . | 3'5898 „ „ |
| 4. Downing's coefficient . | . | 3'8434 „ „ |
| 5. Dubuat's formula . | . | 2'7468 „ „ |
| 6. Girard's formula . | . | 4'8148 „ „ |
| 7. De Prony's Canal formula . | . | 3'7271 „ „ |
| 8. Young's formula . | . | 3'2741 „ „ |
| 9. Dupuit's formula . | . | 4'8752 „ „ |
| 10. St. Venant's formula . | . | 3'4907 „ „ |
| 11. Ellet's formula . | . | 3'0451 „ „ |
| 12. Humphreys' formula . | . | 5'8903 „ „ |

A careful examination of these results in four cases of rivers cannot fail to be instructive.

In the fourth case, a very large river, Humphreys' formula is by far the most correct, and then come in order of correctness, Dupuit, Girard, and Downing, while Ellet and Dubuat are again the worst. In the third case, Downing is most correct, then Dupuit, afterwards Humphreys' formula, and Ellet and Dubuat again the worst. In the second case Ellet and Dubuat

remain the worst, and the best are Robinson, Beardmore, and Downing. In the first case Leslie and Dubuat are best, and Downing worst.

It will be understood that the formula mentioned as Downing's, being more familiar to many under that name, is really that of d'Aubuisson, applied to English measures, without any modification.

Collecting the results, the formulæ may be thus compared :—

| | | Worst Formulæ | Best Formulæ |
|-----------------|-----------|------------------|---|
| 1. Small stream | 24 | Downing | Leslie and Dubuat |
| 2. Small river | 2 424 | Ellet and Dubuat | Robinson, Beardmore, and Downing |
| 3. Large river | 31 864 | Ellet and Dubuat | Downing, Dupuit, and Humphreys |
| 4. Very large | 1 149 948 | Ellet and Dubuat | Humphreys, Dupuit, Girard, and Downing. |

The inevitable conclusion from all these comparisons is that not one of these formulæ is correctly applicable to rivers of different sizes, nor holds its own equally as regards correctness throughout. For the few and special cases in which the discharge of an extremely large river is required, the Humphreys formula might be used advantageously, in spite of its form being rather unwieldy; and in the same way Dupuit's formula for a large river. But for ordinary general purposes the thing that the practical hydraulic engineer requires is a formula tolerably well suited to all cases and of a simple form, so as to admit of easy rapid calculation. The most simple formula having a fixed coefficient is that of Downing or d'Aubuisson, which gives for mean velocity of discharge

$$V = 100 (RS)^{\frac{1}{2}}$$

where R = mean hydraulic radius

and S = mean hydraulic slope;

and this, too, is the formula shown to have been generally the most correct throughout all the comparisons and discrepancies, failing only in the very smallest streams, and evidently worse according as the stream or discharge is less. This then is the best basic formula for general purposes, though it requires modification by experimental coefficients to answer ordinary requirements in canals or canalised rivers.

The formulæ of Young, Eytelwein, Beardmore, Stevenson, and Leslie, all belong to this type, merely using other fixed numerical coefficients instead of 100.

Putting the basic formula into the general form

$$V = c \times 100 (RS)^{\frac{1}{2}}$$

where $c = 1$ according to Downing,

the values of c , according to the other formulæ of the same type are thus :

| | c |
|---|-------|
| Young, for large streams | 0·843 |
| Neville, rivers, velocity < 1·5 feet | 0·923 |
| " " " > 1·5 feet | 0·933 |
| Eytelwein, generally | 0·934 |
| Beardmore, open channels | 0·942 |
| Stevenson, for rivers of 30 cubic feet | 0·690 |
| " " 2500 cubic feet | 0·960 |
| Leslie, small streams | 0·688 |
| " large streams | 1· |
| Downing, Taylor, d'Aubuisson, for open channels | 1· |

By comparing results through formulæ containing these coefficients, we may then tabulate a series of variable values of c that will be practically correct, when suitably applied into the general formula. The comparisons before mentioned show that Downing's coefficient 1·00

gives too small results in cases when the area exceeds 7000 square feet, with a mean velocity of 2·5 ft., or a discharge of 17 500 cubic feet per second, and too large results for cases of smaller data ; that the Eytelwein coefficient '934 in the same way is too small above and too large below discharges of about 2000 cubic feet per second ; and the Young coefficient '843 is incorrect for everything above 900 cubic feet per second ; also that for petty streams of 25 cubic feet per second, a coefficient of about '600 is tolerably correct.

It is evident then that with a very large number of cases of carefully measured discharge, this principle of determining practical coefficients in relation to approximate volume or velocity might be carried out to further exactness ; allowances for irregularities, lateral bends, and so forth, being either comprised in or made independent of this coefficient.

Kutter's coefficients comprise *all such* allowances, they introduce a subsidiary variable coefficient of rugosity, and are applied in the general formula, to canals and rivers of every sort.

The author's coefficients (*c*) are analogous to Kutter's, being dependent on fixed surface-rugosity coefficients (*n*) valuated differently, but do not comprise irregularities or bends ; they apply to canals and are not intended for rivers.

Since the above was written, the large hydraulic experiments of Captain Allan Cunningham on the Ganges Canal have also indisputably demonstrated that the whole of the old hydraulic formulæ, including the more recent formula of Bazin, utterly fail in general application. The variable coefficients, adopted with the applied modifications in the author's Canal Tables,

are declared to be the sole coefficients of general applicability, yielding results within $7\frac{1}{2}$ per cent. of quantities determined by experiment ; while these latter are admittedly liable to an error of 3 per cent. in the cases of the Ganges Canal. The errors due to the old formulæ above proved to amount to 50 per cent., and even more, will, it is hoped, not find now any supporters.

To apply the same method of comparison to discharges through pipes, taking the same general formula,

$$V = c \times 100 (RS)^{\frac{1}{2}}.$$

This formula being more convenient in practice in terms of the diameter of the pipe (d), it becomes for full cylindrical pipes, where $R = \frac{1}{4}d$; $V = c \times 50 (dS)^{\frac{1}{2}}$.

And again as the actual discharge is the quantity most often wanted, this is

$$Q = Av = c \times 0.7854 d^2 \times 50 (Sd)^{\frac{1}{2}} = c \times 39.27 (Sd^5)^{\frac{1}{2}};$$

and transposing this, we have $d = \frac{1}{c^{\frac{2}{5}}} 0.23 \left(\frac{Q^2}{S} \right)^{\frac{1}{5}}$.

Taking an example to compare the results of the various formulæ, let $Q = 18.57$ cubic feet per second, when $S = 1$ in 1276 ; the results then are for diameter :

| | |
|---|-------|
| 1. By Dubuat's formula | 33.74 |
| 2. By Neville coefficient .228 | 36.80 |
| 3. By the above formula, coefficient 0.23 | 37.12 |
| 4. Young's modification of Eytelwein | 37.17 |
| 5. Beardmore, coefficient .235 | 37.92 |
| 6. Hawksley (in Box's tables) | 39.59 |
| 7. De Prony and d'Arcy | 47.71 |
| 8. De Prony's modification of Dubuat | 48.16 |
| 9. Gerney | 48.84 |

Besides these, there are very many authors that would give results for diameter very much below that of Young ; it appears also that none of these formulæ apply equally well to both high and low velocities of discharge, although it is unfortunate that a sufficiently large number of data are not forthcoming to determine correctly the limits at which it would be advisable to change the coefficient.

The above comparisons, while showing the merits of the various formulæ in certain cases, also point to the very evident conclusion that a variable coefficient of discharge is necessary for rivers, canals, and pipes ; and that it must be suitable both to the dimensions, the surface, the fall, and conditions of irregularity of each particular case. The best *mode* now known of doing this in cases of canals, artificial channels, culverts, and pipes, is applied in Chapter I. of this Manual. With rivers, however, some velocity-observation is indispensable.

5. THE WATERING OF LAND.

The following is the usual mode of classifying crops with regard to their special treatment under irrigation.

1. Grass meadows, or natural meadows of gramineæ.
2. Dry grain crops or cereals.
3. Leguminous crops.
4. Root crops.
5. Those specially requiring more water : rice, indigo, tobacco, sugar, bamboo, water-nuts.
6. Garden or fruit crops.
7. New plantations, and trees.

Peculiarities of climate, soil, and water will generally affect the amount of water required for irrigation probably more than the species of crop. In England meadows of grass land, or Italian rye-grass, are those

that generally profit most from irrigation. The usual plan is to keep the land flooded to a depth of two inches during the months of October, November, December, and January, for twenty days at a time, and then to let the water drain off from it for five days, before putting it again under water. In frosty weather however, the field should always remain flooded. In February and March the fields are flooded for eight days at a time at night only ; at the end of March the land is left dry ; and in May the grass-crop is cut. Irrigating fields in England in the hot weather is liable to produce rot in sheep, but does not harm cattle.

There are two methods of laying out the courses or channels in English fields :

1. The bedwork system, applicable to flat land.
2. The catchwater system, applicable to steeper country.

According to the former, the land is made into a series of very flat ridges, having a general direction nearly at right angles to the channel of supply, and being never more than 70 yards long and about 40 feet wide, the inclination of the ridge itself having a fall of about 1 in 500, and the inclinations of the sides of the flat ridges varying with the retentive power of the soil, from 1 in 100 to 1 in 1000 ; the crown of the ridges is not necessarily, therefore, in the middle of the breadth of the base of the ridge. The feeding and drainage channels are generally from 20 inches wide at their junctions to 12 inches at their ends.

The catchwater system used in Devonshire and Somersetshire consists of a series of ridges made across the general course of the water, which hold the water

up, and retain it over successive long strips, the water passing slowly round the end of one ridge to the lower land above the next ridge, and so on. This is necessarily far cheaper than the other system—about half, and can be carried out at the cost of about five pounds an acre.

Throughout the world generally, there may be said to be only four methods of distributing water on or throughout surfaces, of which all others are mere modifications. In all cases it is best that the land should have one general slope throughout, the irrigation channel running along the head of this slope, the main catchment drain along the bottom.

The first method is that to which the English bedwork system belongs, the field being prepared in furrows and ridges alternately from the head to the foot of the slope, either in the direction of the fall or making an angle with it, according as the quality of the soil and the general slope of the land may require; these flat furrows, being from 10 feet to 50 feet wide and only a few inches in depth, receive the water from the irrigating channel, which will then cover the land nearly up to the crests of the ridges, or in fact entirely if need be.

The second method is very similar to the first, but the water, instead of flowing in the furrows, runs in little trenches cut along the crests of the ridges, overflows the sides, waters the slopes, and drains off in the furrows down to the main catchment drain. The ridges used in this system are generally wider than those of the first system, and have a greater lateral inclination.

The third or commonest method for applying water on a small scale is to distribute the water in little

trenches around small squares and rectangles of land, allowing it to permeate throughout the surface inclosed, which must be very nearly level with the water in the trenches.

The fourth method, most commonly adopted in Spain, Portugal, and India, in cases where it is required that a large quantity of water should remain on the land for some time (as on rice-crops, and several grain and other crops in their early stages, that could not thrive on hard baked soil), consists in levelling the land into a number of nearly flat squares and rectangles, divided from each other by small ridges or dwarf mud walls, to hold the water on them. The number of rectangles depends on the fall of the ground; the water is allowed to flow in at some corner or temporary break, and flow out in the same way on to the next rectangle when it has remained sufficiently long.

As to soil:—For the surface, the most permeable is best, being most easily warmed, and allowing the water to arrive at the roots of the grass most quickly; a retentive surface-soil causes evaporation, and cools the land, which is generally a disadvantage, though not so under some circumstances;—a subsoil of clay, being retentive, is an advantage in very dry climates, as it economises water. In hot climates the nature of the soil is of inferior importance to the quality of the silt transported and deposited.

As to the quantity of water required for irrigating a certain area:—In Piedmont and Lombardy one cubic foot per second waters 50 to 100 acres of marcite or grass-land, or only 40 acres of rice; in England the amount required is generally also 1 cubic foot per second per 50 to 100 acres; in the Madras Presidency and in

the North-West Provinces 1 cubic foot per second waters in ordinary seasons 100 acres of rice, or other very wet cultivation, but in very dry seasons the duty is as low as 50 acres. Taking all the crops watered throughout, counting single waterings in all, the duty per cubic foot per second is 200 acres both in Northern and in Central India;—the highest duty actually performed being about 270. In Northern India one cubic foot per second waters $4\frac{1}{2}$ to $5\frac{1}{2}$ acres for 24 hours. But details as to amount necessary in Spain, Italy, France, for Orissa, the Panjab, and India generally, will be found in the Hydraulic Statistics.

As to quality:—Pure water is bad for rice cultivation, and is always far inferior to that which brings fertilising particles with it. The best water for irrigating land may be said to be that which brings with it a fertilising matter most suitable to the improvement of the land under irrigation. As a rule, water containing much hydrous oxide of iron is very bad; so also the water that comes from forest or peat-moss is inferior. The water that comes from a granite formation, holding potash, is good; so also is water that comes from pure carbonate of lime; if the water is brackish, it is no objection; salt-water meadows are highly productive. A good method of foretelling the effects of the water is by observing the natural products of the irrigating water, such as the grasses and plants that grow on its borders.

With regard to the temperature of the water, very cold spring-water is not generally good, and crops require careful preservation from the effects of frost in winter. Warmed water is generally advantageous, and causes rapid growth; it is partly for this reason that water that has been long exposed to air, soil, and sun is

more fertilising than it was in its previous condition. Morning and evening are the best times for watering. The long exposure of the water is much affected by the inclination of the land ; the inclination of the main channels in Lombardy is about 1 in 3600, in Piedmont 1 in 1600, in Provence 1 in 1000, in Tyrol 1 in 500 to 1 in 300, in Northern India it is generally kept between 1 in 1000 and 1 in 2000. In India generally it is usual so to arrange the inclinations that the resulting mean velocity of current may never exceed three feet per second.

In connection with the watering of the land, the management of its drainage is a matter of the highest consequence. Modes and styles of drainage are necessarily varied, according to local circumstances ; but they all have one main object, to keep the circulation of the water and the air through the soil under perfect command, so that the periods of intermission may be so managed as to suit the soil, the crop, and the circumstances. Any want of good management on this point is liable to cause most deplorable results ; stagnation, causing decomposition and malarious effects in the neighbourhood, and even, in the case of sewage irrigation, making the very crops grown to be useless as food for man or beast.

For the healthy support of crops, a certain amount of water and of stimulant may be used advantageously (*see* Hydraulic Statistics : Watering of Crops in France) ; beyond this, any addition is worse than a loss—it is a positive source of injury—clogging the soil, and preventing it from fulfilling its necessary functions. With regard to the period of intermission advisable, it probably varies greatly ; recent experience in England

would, however, seem to show that equal intervals of watering, and of draining off, for twelve hours at a time, afford the most rapid way of utilising in irrigation as much sewage as possible: further experience, however, is perhaps likely to show that this is not by any means a rule to be followed generally in all soils and conditions.

Assessment of Water-rate.—There are three principles on which water-rate may be levied on land.

1. By fixed outlet, or by module.

The small channel of supply being constantly full and of a certain section, the rate may be charged at so much per square inch or square foot of section, independently of the amount of pressure, for a certain time, as by the hour or day of 24 hours. This has been adopted in Italy, but has not been found to act well.

A further development of this method is to regulate by module all the water when distributed; a mode more likely to be adopted at present, now that modules are less expensive and more effective than formerly.

2. By area of land irrigated, or by crop.

This has the following disadvantages; the land to be irrigated is always varying in amount, and this cannot be watched in detail continually, nor can the landowners be trusted to state truthfully the amount of acreage over which water has been distributed. The crop can also be varied, so as to use more or less water, and the payment by crop also would be useless against cheating. Again, in a good rainy season the cultivator might try under these circumstances to do without the canal water, thus causing the water-rate to be precarious.

3. Water distribution by rotation.

An irrigating channel of fixed dimension, giving a constant fixed discharge, passes through the lands of several proprietors ; a period of rotation is fixed for this channel, from 6 to 16 days according to the crops, the former for rice and the latter for meadow land, as, for instance, in Italy. Each landowner can then have the whole volume of the channel turned on to his land once in the total period of rotation for a certain number of hours, as from two to forty or fifty according to the amount of land he owns.

For example. Let ten days be the period of rotation, and let him require twelve hours' supply once in that period. His name is placed on the list, say sixth, and he gets his supply turned on at a fixed hour and turned off at a fixed hour also. If the channel gives twenty cubic feet per second, his amount of water is equivalent to a continuous discharge of $\frac{20 \times 12}{240} = 1$ cubic foot per second. In this way intermittent supplies admit of mutual comparison.

Last with regard to the cultivators themselves :— Whether on the Continent, or in England, the farmer is generally a grumbler under any state of affairs. In India the cultivator invariably complains, although his assessment is very small by comparison with the local circumstances ; if he grow two very moderately good crops in the year, it would only amount to about two and a half per cent. per annum on the value of the produce, and he can therefore well afford to pay high water-rates, especially since both the yield and the number of crops produced on irrigated land is doubled, and the highest water-rate is small in comparison with the expense of making wells and raising the same

amount of water by animal power throughout the year ; he enjoys also the advantage of living under a tenure that remits the land assessment, and distributes food gratis in years of famine, while not demanding more assessment in years of plenty. If the water-rate is in some just proportion to the increase of produce and saving of expense resulting from the irrigation, it matters not how high per acre the rate may appear to be. If the irrigation is applied to suitable land in such a way that the natural drainage of the country is not interfered with, there can be no detriment to the health of the cultivator ; this can, however, be rarely carried to perfection in actual fact. To this it can be replied, that the population will thrive on the whole and increase largely, which may be considered as a set-off on that account, and that landowners who prefer going away can always do so and part with their land at a premium ; land always commanding a ready sale. A compulsory water-rate on land that is under water command cannot be considered a hardship by any one that considers the subject in a fair, unprejudiced manner ; the privilege of being able to obtain water should be paid for, and since the same principle has always been applied to town supply of water, for which every inhabitant has to pay whether he uses it or not, there is no reason for leaving the payments of water-rate in the country to be optional. Whether both the landowner and the occupier should pay separately for the advantages they both receive is a point dependent on the local tenure of land ; under ordinary circumstances they doubtless should do so, the occupier being benefited by increase of produce, the landowner by increase of rent ; but in any case the whole of the advantages should be paid for.

6. CANAL FALLS.

That a fall of water at the headworks, or at any part of a canal, should be allowed to remain unutilised, appears, in these days of expensive fuel and costly motive power, to be a very painful waste of a valuable advantage. One's natural tendency is to devise means and ways of using everything, and to imagine that there could hardly exist circumstances under which it would be necessary to arrange for the destruction of the power and velocity generated by a fall of water. Grinding corn, pressing sugar, or extracting oil, are requirements even in semibarbarous countries, by which such motive power could be easily utilised, even if it were available for only four months in the year. In spite of this, however, it seems rather frequently to occur, that in distant countries the engineer has to devise means for destroying the effect of a fall of water ; this occurs, generally, either at the headworks of a canal, where the water entering the canal in flood seasons has a great head of pressure, or at certain points in a canal where, owing to the inclination of the country being steeper than that due to a convenient velocity of canal current, it has been found necessary to concentrate the superabundant fall : the Ganges Canal and the Bari Doab Canals have many such examples. In either case, as the fall is independent of navigation of any sort, which has to be conducted in a special channel of *détour*, the problem is one of economy. The natural means would be to break up the force of the water by both lateral and vertical breaks and angular obstacles, and to oppose the remains of the velocity by a pierced breakwater, beyond which

the water would issue with so small a current as not to be able to cause any damage to the bed and sides of the canal, or to cause any prejudicial effect to navigation.

The breakwater, involving an enlargement of the width of the channel, and, if a rock foundation be not available, requiring artificial and carefully made foundations carried to some depth, is necessarily expensive, and is hence generally dispensed with, except under favourable circumstances.

The fall itself is generally a modification of one of the four following types :—

1. A uniform, or a broken general incline.
2. A vertical fall with gratings.
3. A vertical fall with a water-cushion.
4. An incline or fall with a talus of boulders, &c.

The most primitive mode of managing such falls of water was to conduct it down an incline, made as gradual as possible, and break up the velocity by a series of steps. A long reach of rocky bed offers a convenient opportunity for such a construction, which could be hewn in the solid rock. In other cases, where it would require building on artificial foundations, the expense would be very great ; and, even if the incline were so made that the resulting velocity were not high, the edges of the treads of the steps, even in good stonework, would soon wear, and the maintenance of the fall would also become an important item of expense. Apart from these objections also, this type is unsatisfactory. Although the treads of the steps may be set with a correct reverse inclination, so as to oppose more directly the inclined direction of motion of the momentum of the

water ; and, although a further improvement may be made in giving a more considerable reverse inclination to the treads, and by allowing a large proportion of the water to run off laterally and wind down the steps ; yet under all circumstances the inherent defects remain ; the steps cannot accommodate themselves to the variation of the quantity of water passing down the fall ; if the steps are small, they fail to receive effectively the over-falling water when the amount increases, and become then comparatively valueless ; if the steps are very large, the rise and tread of each step causes the velocity acquired from each step (which, it must be remembered, increases in the ratio of the square of the height of the step) to be very much increased, and to become very destructive to the stonework.

The next improvement on the inclined type of fall is the ogival fall used on the canals of Northern India ; in this the general slope of descent from the head to the foot of the double curve is from one to six to one in nine ; the upper one-third of the slope being the chord of the upper or convex curve, which is tangential to the surface of the water in the upper reach ; and the lower two-thirds of the slope being the chord of the concave curve, which is tangential to the convex curve above, and tangential to the horizontal line at its lower extremity. The height and length of the fall applicable to any special case is determined by equating the discharge of the open channel above with the discharge over a weir. The principle which this form of construction asserts is that the water at the foot of the descent, being deprived of all vertical action and delivered horizontally, will not cause any damage to the bed of the channel in the lower reach.

In canals where it is required that the discharge should remain perfectly uniform and unaffected by its fall down the weir or incline, an ogival fall must necessarily have its sill raised above the level of the channel-bed of the upper reach ; as would also a fall of uniform slope.

Curves on more carefully eliminated principles have also been tried with the object of effecting some improvement, but the advantages resulting appear comparatively small. These curves generally effect, no doubt, some saving of masonry in comparison with that for a single uniform slope, and probably deliver the water with less destructive result than the latter ; they are, however, still expensive, and the action of the water delivered is rather concentrated, and hence destructive. An attempt at economy on such falls has been made by narrowing the fall, and thus diminishing the amount of masonry ; but the results, caused by the increase of action as well as irregularity of effect of the water, require greater expenditure in repair ; they present also the additional disadvantage that during repair the whole fall instead of a part has to be stopped.

In the above cases of inclined falls it is supposed that it has been found convenient to concentrate the fall in a comparatively short length ; in other cases, where it is spread over a long reach, it is usual to attempt to annihilate the velocity resulting at the foot of the incline by introducing a reach of canal having a reverse slope ; and in cases where a greater length still can be allowed for the incline, to break it up into portions of descent, each followed by a portion with a reverse slope and then a short horizontal length, thus opposing the accelerating effect in detail without allow-

ing its results to accumulate. In such work the bed of the channel must necessarily be paved ; if the velocity do not exceed 10 feet or 12 feet per second, large rough convex boulders, laid dry, form the most suitable paving ; and even up to 15 feet per second the same method may be adopted if very large boulders alone are used ; beyond that velocity the boulder work requires packing with shingle and pebbles, and grouting with good hydraulic mortar.

While the above arrangements may destroy a great deal of the velocity, there is perhaps almost always a certain amount of it still remaining at the foot of the incline, and should the channel at this place happen to be in soft soil, further arrangements, tail-walls, brush-wood spurs, or piles, are also necessary.

The Bari Doab Canal tail-walls offer an example illustrating such a case, the arrangement being generally as follows : At the foot of the incline the bed of the channel is made horizontal for some distance, and the banks are then splayed outwards in a curved form until the top width of the channel at water level is one-half wider than before : this, giving additional water-way, reduces the velocity ; the channel is then narrowed to nearly its normal width by walls of dry boulders on each side, which project into the stream at an inclination of 1 to 5, and slope longitudinally with a fall of 1 in 20 from their commencement, where their height is up to full supply-level, down to the level of the bed : these are, of course, totally submerged at full supply, and produce the effect of concentrating and directing the current to the middle of the channel. The objections raised to these tail-walls as employed on the Bari Doab Canal is that they do not appear to answer their pur-

poses sufficiently completely, and it is supposed that by giving the whole arrangement, both the enlargement and the reduction of section, a greater length, it would fully answer all purposes ; this, however, would add greatly to the expense.

Vertical falls with gratings.—This is one of the most economic and convenient modes of dealing with a canal-fall. The sill of the fall is not raised above the bed of the upper channel and the whole section of passage is hence unimpeded by reduction ; the grating, which may be placed at any slope from 1 in. 3 to 1 in 10, presents a large perforated surface to the action of the water, thus keeping the upper water up to its proper level, and distributing the effect of the falling water passing through it on a long portion of the bed, diminishes the action to such an extent as to render it harmless. The gratings are supported on cross bearers, which again rest on masonry piers or iron stanchions, erected at about 10 feet intervals along the edge of the fall or weir. The higher a fall of this description is, the more truly the water falls and the more manageable it is. These gratings require clearing occasionally, and hence necessitate the attendance of a man ; but as frequently there is a lockman to attend to the neighbouring lock, for the navigation passage near the fall, there is no additional expense incurred on this account, as one man can attend to both. This type of fall admits of comparatively little variation in design.

Vertical falls with water-cushions.—This is the form generally adopted by nature in discharging water down a fall ; the action of the water scours for itself a basin, which fills and forms a natural water-cushion, the scour continuing until an equilibrium is established between

the force of the descending water and the resistance offered by the depth of water in the basin. The fall itself has a tendency to approximate to the vertical, the force of wind and spray from the falling water making it slightly overhanging, and in some cases even causing a retrogression of fall, and coincidentally also a retrogression of water-cushion, thus giving it an elongated form ; the scoured silt, or debris, is deposited in the bed of the stream lower down.

The most natural mode of designing a vertical fall with water-cushion for a canal would perhaps depend on a consideration of what sort of fall nature would make for herself under the special circumstances and conditions of the case, and what improvements or modifications of that would be necessary. The objections to allowing nature to make her own fall and water-cushion are these :—first, it requires time, and this, in some, though not in all cases, is an objection in itself ; second, any want of homogeneity of the soil or rock would result in an irregular form of basin, which might become almost unmanageable ; third, the scour and silt deposited in the channel below would be a serious injury to it ; fourthly, the retrogression of the fall might eventually undermine the weir or dam, and cause its entire destruction. But this latter objection might be very easily counteracted by protective measures.

In cases, then, where these four objections can be removed or are unimportant in result, there is no reason why a natural or a slightly modified natural fall should not be adopted. When the soil is firm or of homogeneous rock, a great deal of the objection disappears, a certain amount of excavation and trimming can then

be so made as to aid in the natural action, and lateral encroachment may be easily provided against; a tolerably regular basin can then be economically made.

As to the form of basin best suited for a water-cushion, the breadth in plan should be rather wider than the extreme breadth of the falling water, as the wind may bear the latter considerably to one side; the length, again, will probably vary from $1\frac{1}{2}$ to 5 times the breadth, although it would hardly be advisable to make it quite rectangular in form, as the corners would be filled with useless water; the pear shape, therefore, is perhaps the best, and is certainly that most generally met with under natural conditions of homogeneity of soil. There would probably be no advantage, even if it were economic, to make the basin longer; the full or extreme depth may be terminated by a reverse slope at once, the deflected velocity thus obtained producing a greater degree of stillness than the passive effect of a longer continued full depth.

The main point, however, is to determine what depth of water is necessary in a water-cushion. The velocity of delivery is evidently dependent on the depth on the weir sill or fall above, and the height of fall down to the surface water in the basin: the resistance is the depth of water in the basin, and the quality of the material of which its bottom is composed. If, then, the depth be calculated by equating the forces for a depth producing equilibrium just clear of the bottom, we obtain an expression, involving also an assumption that the bottom is perfectly indestructible. It seems therefore, impossible at present to determine absolutely the actual depth necessary; and hence the practice is to assume an approximate calculated depth, and see how

this answers its purpose, altering or adding afterwards until it appears to be satisfactory.

The formula generally used for this purpose on the canals of Northern India is—

$$d = 1.5 \sqrt{h_1} \times \sqrt[3]{h_2}$$

d = the depth of water in the basin ;

h_1 = the total height of fall, including h_2 ;

h_2 = the depth or head on the weir sill.

This is probably very limited in its range of application ; for, in applying it to the well-known case of the projected Mahsur reservoir dam, designed by the engineers of the Madras Irrigation Company, it yields results very small in comparison to that allowed by the engineers : thus, for values of $h_1 = 43.5$ and $h_2 = 6$ feet, the calculated value of d , suitable to a brick bottom, is about 18 feet, while the engineers have allowed for a hard rock bottom a depth of water-cushion of 33 feet in this instance.

In a second instance of the same case, the formula gives for values of $h_1 = 16.81$, $h_2 = 8.56$, $d = 12.54$, which is very much less than that allowed, 16.19 feet ; this was also in hard rock.

Major Mullins, the Consulting Engineer to the Madras Irrigation Company, when commenting on these cases in the Proceedings of the P. W. D., for April 1868, refers also to a well-known natural fall as an illustration of the insufficiency of the above formula. The Rajah Fall at Gairsappa, with values of $h_1 = 8.29$ and $h_2 = 15$ feet, would, according to that formula, require a depth of water-cushion of only 10.8 feet for brickwork, or 7.2 feet for stone, a depth nearly a half less than the actual depth, 13.0 feet.

In a smaller natural case, in hills in Berar, coming under the observation of the author, for values $h_1=26$ and $h_2=1$, the depth, according to the above formulæ, would be for a brickwork bottom 7·65 feet, and for stone 5·6 feet; whereas, in the soundest of basalt, the actual depth was as much as 8 feet, or more than a quarter more than that calculated.

It would, therefore, appear that the above formula, apart from its varied coefficients for brickwork and stone, is generally defective, and that, until a very much wider range of experiments and observations is made, it would be more advisable to approximate to such depths as are obtained under natural conditions, than to follow any formula for determining the depth of a basin serving as a water-cushion.

In practice it would rarely be necessary to construct a water-cushion of very great depth, the fall, if over a weir, being generally easily broken into three or four portions, and it being advantageous to do so, as the catch channels are convenient for affording a supply at various levels; probably, therefore, the above-mentioned case of 43·5 feet of artificial fall may be considered as the extreme for which a water-cushion would be required. In the future, too, the waste of such a large amount of useful motive power will be deemed a barbarism, an additional reason that there is not much probability of the above case being exceeded.

Inclines and falls with a talus of large blocks.—Under some circumstances it is not advisable to terminate an incline with a long reach of ogival tail-walls, or a basin, nor to apply any of the foregoing methods to the foot of a vertical fall. The velocity of the water having to be counteracted, presuming that it cannot be utilised, an

alternative method is to allow the velocity to destroy itself by impinging on a large number of huge boulders and masses of stone of considerable weight. This mode was that adopted by Messrs. Fowler and Baker in the improvement of the Nile Barrage ; a most unfortunate dam constructed by the French at an immense expense, which failed to effect its purpose, otherwise than to serve as a bridge, until it was entirely remodelled by English engineers.

7. THE USUAL THICKNESS OF WATER-PIPES.

The thickness of a water-pipe is a matter depending on practical considerations, being comparatively little affected by the theoretical determination of what it should be in order to resist the pressure brought on it ; and is, like a very large number of the so-called calculations of the engineer, made almost entirely dependent on prescribed custom. The following notes on the formulæ in vogue are, hence, not given so much with the object of elucidating the principles as that the formulæ themselves, valueless as they seem, should be available for reference.

The largest scale on which a water-pipe to resist extreme internal pressure is made is that of the cylinders of hydraulic presses : in these the extreme working pressure is limited to 4 tons per square inch, the extreme permanent strain allowed in actual working being only one half of that ; and the thickness of the cylinder or pipe is determined by the formula of Barlow—

$$t = \frac{r.P}{C - P} ;$$

where t and r are the thickness and internal radius of the cylinder or pipe,

C is the cohesive strength of the material, and
 P is the internal pressure, both being in tons :

the general principle asserted in this mode of calculation being that the strain on the material is greatest at the internal surface, and less beyond, the extension varying with the square of the distance from the centre.

An example of the application of this formula, to a 10-inch cast-iron water-pipe, is given in Box's 'Hydraulics,' the results of which are as follows :—

Assuming the cohesive strength of cast iron to be 7 tons per square inch breaking weight ; the extension E , on the inside ring at the moment of rupture, for a length = 1,

$$E = \cdot 000\ 165\ W + \cdot 000\ 010\ 3\ W^2 \times L = \cdot 001\ 659\ 7 ;$$

and the extension at any distance from the centre is in the ratio of the square of that distance to that of the inside ring.

The strain, at any distance from the centre, is then obtained from the extension by the formula—

$$W = \sqrt{\left(\frac{E}{\cdot 000\ 010\ 3 \times L} + 64\cdot 16\right)} - 8\cdot 01$$

and the mean strain on each theoretical concentric ring of metal is the average between that at its external and its internal circumference ; the bursting pressure has then the same ratio to the mean strain as the thickness of the pipe has to its radius ; and tabulating these for a 10-inch cast-iron pipe, they are :—

| Thickness of Metal | Strain on the Metal | | | Bursting Pressure |
|--------------------|---------------------|------|-------|-------------------|
| | Max. | Min. | Mean | |
| 1" | 7'0 | 5'26 | 6'130 | 1'226 |
| 2 | 7'0 | 4'09 | 5'402 | 2'161 |
| 3 | 7'0 | 3'26 | 4'827 | 2'896 |
| 4 | 7'0 | 2'65 | 4'359 | 3'485 |
| 5 | 7'0 | 2'20 | 3'972 | 3'972 |
| 6 | 7'0 | 1'85 | 3'647 | 4'337 |
| 7 | 7'0 | 1'60 | 3'373 | 4'722 |
| 8 | 7'0 | 1'37 | 3'137 | 5'019 |
| 9 | 7'0 | 1'19 | 2'931 | 5'275 |
| 10 | 7'0 | 1'05 | 2'749 | 5'499 |

The practical empirical rule, however, that is usually given for the thickness of water-pipes is—

$$t = \left(\sqrt{\frac{d}{10}} + 0.15 \right) + \left(\frac{Hd}{25000} \right);$$

where H is the head of pressure, and d is the diameter of the pipe, and it is according to this that most tables are calculated.

The theoretical mode of arriving at the thickness of a water-pipe is, therefore, about the most unsatisfactory of processes; and it would probably be useless to enlarge on the topic. In English practice, the dimensions of cast-iron water-pipes are about those given by this formula, or have a thickness of one-fifth the square root of the diameter, and a little more to allow for defects in casting, and inexactitude of bore.

The dimensions of the pipes used at Glasgow by Mr. Bateman (see Appendix) have been treated as English standards for some time. In Continental practice thinner large pipes are used; those designed under restrictions by the author for Rio de Janeiro, when Hydraulic Engineer in charge of the waterworks, were partly in accordance with such practice. See Appendix.

While in the case of cast-iron pipes of all sorts, there has always been a tendency to theorise, and to base a thickness on the laws of pressure, and extension of material; in stoneware pipes, this has been almost entirely disregarded, and a thickness is generally given them that is established entirely on practice or usual custom, and often varies according to the caprice of the potter or manufacturer. This is generally accounted for by saying that earthenware or stoneware is a very variable material as regards strength, while cast iron is homogeneous, and is very much alike in substance: a little reflection, however, will show that this is hardly a sufficient reason. Carefully-made stoneware, after a very careful selection, may be, and often is, exceedingly equable, while the variety of qualities of cast iron—more especially since its high price has brought such a large amount of very inferior material into use—is now very marked; some cast iron being known occasionally to fall to pieces from its own weight. In spite of this, the manufacturers of stoneware pipes still consider them as unsuited to the discharge of water under pressure, or for drainage in cases where the outlet is liable to be stopped; and although they can make pipes that will easily bear a head of 40 feet, yet do not recommend them, alleging that the joints cannot be made to stand any pressure at all. There is, however, no reason to doubt that under skilled superintendence and management, stoneware and fire-clay pipes, as well as their joints, may be well enough made to serve most efficiently for the distribution and drainage of water under low heads, and that a considerable saving of expense may be effected by dispensing with iron in such cases.

8. FIELD DRAINAGE.

The drainage of the surface water of a field, forming part of the general drainage of the valley or catchment in which it is situated, is necessarily partly dependent on the conditions of that general drainage, the dimensions and fall of the watercourses, ditches, channels, and rivers, their straightness, and distribution of declivity, also on the position of the field with reference to higher land in the same catchment, the drainage from which may pass over or through it in various ways.

In the second place, the drainage of a single field is dependent on the geological formation at the place, the distribution and superposition of pervious and impervious strata, their undulations, configuration, and retentive qualities.

Any interference with the general drainage of the country by proposed works of improvement is a matter requiring the professional aid of the hydraulic engineer, while in the same way any intended alteration of the subterranean flow and conditions of moisture by such operations of marsh, bog, or spring drainage as tapping strata, boring, intercepting deep drains, small tunnels, &c., require that the hydraulic engineer should be also a hydro-geologist.

The drainage of any single field may be so entirely altered or modified by works or operations of these kinds, that any special drainage or series of drains on the field itself may be entirely unnecessary, as its soil may be thus rendered thoroughly fit for all the purposes of the agriculturist.

Treating for the present all engineering works and

hydro-geological operations as external matters, which might be either impracticable, not beneficial, or excessively costly, and supposing that the actual state of the general drainage and hydro-geological condition is moderately good, and incapable of much improvement, it may yet happen that a particular field may suffer from insufficient drainage, or may be improved by local drainage, or simple field-drainage.

The condition of good cultivable soil.—As the object of such drainage is to put the cultivable soil in the best possible condition, the first consideration is the quality of the soil. Should the soil be exceedingly porous and light, it may be deficient in retentive power and require consolidation, top-dressings of clay or marl and careful management; under such circumstances drainage would be hurtful, and deep-ploughing should be avoided, unless with the special object of subsoiling, or improving the soil by admixture with the subsoil turned up. Such soil benefits by irrigation, and the accompanying infiltration of clayey particles, and liquid manure in the soil. If on the contrary the soil should be exceedingly retentive and clayey, water or rain lodges in the soil, chills and binds it, rendering it unfertile and hard to cultivate. Such a soil would benefit greatly from field-drains and deep-ploughing, admixture of porous soil or burnt clay.

These are the two extremes of condition of cultivable soil, the one profiting least from drainage and most from irrigation, the other most from drainage. Apart from the composition of the soil itself, the climatic conditions, and the amount of rainfall, snow, dew, and atmospheric moisture affect the greater or less demand for drainage.

In a hot dry country, a retentive soil is favourable to the growth of rice and many wet crops that luxuriate in a semi-marshy state, and require very slow drainage ; in a moist chilly climate the same soil would require the most thorough drainage in order to grow cereals, roots, or pulses. Between the extremes both of quality of soil and of local moisture there is an infinite variety in degree, and the agriculturist has therefore to state his requirements as regards drainage in accordance with the conditions and the crops he wishes to grow. Absolute stagnation is invariably fatal to crops. Even with rice crops in India, rot will result ; a certain degree of circulation is necessary everywhere. In England there is a large amount of land that is, either naturally or through repeated deep-ploughing, sufficiently open to admit of full permeation of rain-water to a great depth, and thus capable of growing the ordinary crops of the country without special drainage ; the greater part of the land, however, is less favourable, allowing water to lodge in it within a few feet of the surface, and thus necessitating field-drains.

The condition of soil aimed at is an imitation of that which is naturally most fertile ; the retention of a moderate amount of moisture, a free permeation of irrigation-water or of rain-water downwards to a sufficient depth in wet weather, and a corresponding free capillary upward movement of moisture in dry weather or in the periods when irrigation is suspended ; the dispersion throughout the soil of air, moisture, volatile gas, and the soluble ingredients of accompanying fertilising manure, whether natural, chemical or artificial.

Depth of active soil and of humus.—Such being the

general condition requisite, the first and most natural question arises, how deep should such a soil be, and to what depth is drainage advantageous ?

The depth of active aërated *humus* that will support crops advantageously is a most variable unit ; it is generally believed that the greater the depth, the more fertile the land, that crops augment in yield by every additional inch and foot of humus. It may be so ; but, taking an extreme case coming under my personal observation in a province entrusted to my charge, a depth of from eighty to ninety feet of soil on the banks of the Purna in Berar did not yield markedly better crops than in other places where the depth was half of that. Also in other cases, frequently noticed by myself in the earlier days of my experience in irrigation as exceptional, but afterwards considered very commonplace—where cereals were grown under irrigation on pure sand, and on very nearly pure sand. A large extent of such land is irrigated, and at the end of the year, a thin surface crust of half-formed humus is formed ; the crop of that year is zero in one respect, usually consisting of grass seeds, &c., that on growing form a spongy layer of roots and verdure, useful in arresting and binding the humus. But in the second year, under the powerful sun of India, and by the aid of careful irrigation and good management, a very inferior first crop of cereals may be grown. In the third year a moderately bad crop is the result, and afterwards excellent crops of wheat and of other kinds of produce, that can exist without throwing very deep roots.

In such cases, the depth of humus and spongy crust together can hardly exceed three inches or perhaps four ; yet splendid crops are grown.

At Danzig on the sewage farm, excellent crops of vegetables were grown under rather similar conditions ; it is not necessary to mention many such well-known cases on English sewage farms, Aldershot, Edinburgh, &c. It may hence be considered that world-wide experience has disproved the old theory about depth of humus being the main source of fertility. It is really, therefore, only one of the sources, and its importance is frequently outweighed by other conditions, more especially by the depth of active soil.

In England moderate crops may be grown in six inches of soil on stiff land, but for really good crops, a depth of three times that, or eighteen inches, of active aërated soil may be considered a suitable minimum. The maximum may be determined by the extreme depth to which roots of grass and grain crops are found to penetrate, about seven feet in thoroughly-drained active soil.

Depth of field-drains.—Taking the two extremes of eighteen inches, and seven feet, as suitable to firm soil in England generally ; the minimum depth for field-drains, out of reach of the plough and not affecting the crop, by reducing the productive area, should be $2\frac{1}{2}$ feet, and in strong clay lands four feet. It may be noticed that water does not permeate truly horizontally, in a lateral direction from the bottom of the active soil to a field-drain ; but in perfect drainage should descend slightly in its lateral movement to the bottom of the field-drain ; hence the necessity for placing the drains lower than the bottom of the active soil. Local conditions, depth of soil and subsoil, and economic considerations form the guide to determining the greatest depth at which field-drains might be put ;

apart from them it would be difficult to say what would be the extreme depth that could not be advantageously exceeded under special circumstances.

Very strong clay-lands, with drains cut in the subsoil, would certainly be worse for having them very deep ; but, keeping in view future improvement of the sub-soil by disintegration—as well as economy of labour, it appears seldom necessary to drain beyond five or six feet in depth unless in boggy retentive land, and even then a few extra deep drains may be cut without interfering with the ordinary field-drains. The limits thus lie between $2\frac{1}{2}$ and six feet. Such general limits can, however, constitute merely a rough guide in connection with the special objects to be achieved, and the local circumstances. Drainage pure and simple has for its main object the removal of sub-surface water down to some or any practicable depth ; but another object is often blended with it, the further improvement of the subsoil, and the increase of depth of active soil, in the clayey and stiff lands to which drainage is most frequently applied. Some stiff subsoils are so impervious and hard as not to admit of improvement by drainage ; in such cases the field drains are perhaps best placed with their bottom just on the subsoil. Much good clay subsoil will, however, under drainage, alternately wash and contract, and gradually break up ; a most desirable change that may be much aided by extra deep trenching with steam-power ; in such cases the field-drain-soles may be sunk to a foot and a half in the subsoil, or even more when accompanied with subsoiling operations.

Distances between field-drains.—The closeness of the field-drains to each other must be determined so as to

afford sufficient active permeation of moisture throughout the whole of the intervening breadth of land ; this will depend on the qualities of the soil and subsoil down to the level of the sole of the field-drain, the drains being closer in stiff soil and under conditions of heavy local rainfall and further apart in more open soil, and a drier climate. In England the distances between the parallel lines of field drains usually adopted vary from fifteen to forty feet ; in any special case the distance should be based either on the evidence afforded by actual drainage in the neighbourhood under similar conditions, or on partial experiment on the spot. The size or dimensions of the field-drains may be determined in the same way, but this is naturally dependent to a certain extent on the sort of field-drain adopted.

The alignment and length of field-drains.—A field may consist of several planes, or several fields may lie in one general plane or nearly uniform slope ; but under all circumstances the field-drains, being set to some certain depth either below the surface, or below subsoil surface, lie in a plane or planes nearly parallel to those of the fields. Each plane has therefore to be treated separately as regards the alignment of the field-drains. The main drains, into which the field-drains run, are necessary at the bottoms or lower edges of these planes, and afterwards unite and run into some watercourse or general drainage-line of the country, at a point sufficiently low to secure sufficient outfall.

There are three modes of aligning field-drains, which under all circumstances are arranged in parallel lines in each separate plane, and besides at uniform or approximately uniform inclinations. The regularity of the fall

may in rather steep ground be attained by setting out the soles of the field-drains with the aid of boning staves, the A level, or some rough spirit-level ; but on slight inclines a small Gravatt level is absolutely necessary. The first and most common mode of alignment is to direct them on the lines of greatest slope from the top of a plane to the bottom ; such lines may be long even as much as 300 yards, while the distances apart may be from fifteen to forty feet as before mentioned in accordance with the soil and conditions : the drainage-action is then entirely lateral and works by permeation into the field-drains, which transport the filtered water into the main drains. The second mode is termed cross-drainage, the parallel field-drains running across the lines of greatest slope, that is being nearly horizontal, having a slight fall towards the main drains : in this case the permeation is aided by gravity, and may be more rapid ; the field-drains intercept the filtered water, and conduct it to the main drains at a comparatively slow velocity. The third mode, generally preferable to either, is the slightly oblique method ; the field-drains are only slightly inclined to the direction of greatest slope, that is from ten to twenty degrees, and are supplemented at long intervals, of about one hundred feet, by cross-drains that are nearly level. In this case both the preceding modes of drainage-action are employed ; gravity assists both in the lateral and in the transverse permeation, and interception is adopted to a small extent.

In comparing these three methods, it may be noticed that the first is that most usually adopted in England, and is generally far preferable to the second. The permeation is, no doubt, the least rapid part of drainage action ; the filtered water on arriving at the field-drain,

when in good order, rapidly runs into them through the joints, and still more rapidly is conveyed away. Keeping this in view, any check in the permeation due to any accidental circumstance or shortcoming will evidently produce a check in the drainage of a whole plot. For instance, the distance between the drains may be slightly too great, the depth may be slightly in excess, the soil may in certain places be less permeable than in others, a drain may become rather clogged. Now when the first method is adopted, the plots are very long narrow strips, half of the water from each strip going laterally into each field-drain, one on either side of it ; and should the permeation be accidentally retarded, a middle portion, perhaps the middle third, of the strip remains in an inactive condition. The length of the strip may be so long (200 or 300 yards) that permeation, aided by gravity in the direction of the main drain, is almost out of the question ; and here lies the defect in the first method.

The second method has no drains along the direction of greatest slope, but places the whole of the field-drains as interceptors, but putting them at the same distance apart as in the first method. It is true that with this method gravity aids the permeation, but as the permeation in each strip has to act over the whole of the breadth of each plot, instead of over half of it each way, nothing is gained ; in fact it is rather the reverse. The action of gravity is an aid, but not a very large one, as from many observations we may see permeation acting successfully against gravity, as in the lines of damp on sides of ditches, the rise of damp in walls based on damp foundations, &c.

In order to make this method as efficacious gene-

rally as the former, the distance between the field-drains should be reduced by about one-third, and this means having half as many drains again, and adding one half more to the cost of the drainage.

Experience has proved not only the truth of this deduction, but also that, even when the field-drains are placed still closer, the drainage effected has not always been thorough, and re-drainage on the first or longitudinal method had to be substituted in the end after the dearly-bought experience.

Cross-drainage on this generally unfortunate method is, however, specially applicable and advantageous when the upper strata contain much water and either crop out across the line of greatest slope, or discharge their water in natural furrows existing on the surface of the subsoil ; in that case the cross-field-drains act as interceptors to the fullest extent, and collect water readily as it comes forth, although not perhaps setting up a draining permeation in the strict sense, as their influence on permeation in the subsoil cannot be very large.

The slightly-oblique method preserves the advantages of the longitudinal method as regards lateral permeation, and remedies its defect in longitudinal permeation by the obliquity, which also aids in interception ; the occasional cross-drains at about 100 feet apart still further aid the longitudinal permeation, and assist in rendering the whole action complete and effective even under the incidental shortcomings that may occur anywhere and in anything.

The various sorts of field-drains.—The object, the disposition, and the depth of field-drains has been dealt with in the preceding paragraphs, independently of their actual form, sort, or construction, under the premise

that they are sufficiently large, porous, and well-constructed to carry off any effluent drainage, or filtered water, that may arrive and enter into them. The sort of drain adopted is necessarily in accordance with local circumstances and economy.

The oldest method was one of simple ridge and furrows, for carrying off surface-water, subsequently deepened to carry it off from a lower depth, and filled with porous soil or porous material. Such shallow drains interfered with ploughing, and reduced the effective cultivable area. Deeper sub-surface drains, covered with good soil, and leaving a flat surface equally productive everywhere, have long supplanted the old method. More latterly, porous cylindrical drain-pipes from 2 to 6 inches in diameter, with collars, have been usually adopted, in preference to other means ; and these, placed at the required depth, and covered to a sufficient height with porous soil, and finally with a good top soil, have been considered the most effective ordinary method. This may therefore be considered the typical English method for many years past, though not the most modern one. It is well suited to clayey lands in England, and to the condition that the pipes can be cheaply made or bought, and the clay dug out of the drains can be profitably burnt to form manure, or made useful locally.

Previous to the general adoption of cylindrical porous pipes, large drain-tiles, horse-shoe shaped in section, 4 inches high by 3 wide, with flanges, sometimes resting on separate tile-soles about 5 inches wide, and sometimes merely on the clayey bottom of the trench, were commonly used ; this arrangement developed into the flat-bottomed cylinders made in one piece, that are still used.

In some places, tiles of dried compressed peat may be made effective in field-drains, but the peat must be tough and fibrous to resist the action of water. In others, thorns and brushwood form a field-drain of an economical sort in fen-lands, where the material is cheap, and the flow of water is slow.

Stone drains, of rough stone, so arranged as to give large interstices below, and filled up above or covered with smaller stones above, are also economical in some localities ; but the method is inferior, and the damage to land by carting stone over it forms a strong objection. For slow drainage, cinders, gravel, or other porous materials are far preferable, from being more effective for a longer time and from being lighter to transport.

Many of these modes, though lacking permanence, are effective for a considerable time, and, being inexpensive, admit of renewal after a few years without prejudice to economy. One of the most important considerations is the extent to which they become deleterious or hurtful after becoming ineffective in lapse of time. Such inert matter as broken tiles, stones, &c., cannot be of any advantage in cultivable soil ; originally they are perhaps placed in the clayey or stiff subsoil ; but if effective drainage and deep ploughing and subsoiling be adopted, the subsoil becomes disintegrated, and the active soil may then reach down to near the level of the field drain ; the stones and inert matter are then out of place.

Stiff soils being those to which drainage and subsoil improvement is most applicable, the most modern mode of effecting drainage, by the deep drain-plough, is also best suited to them. The drain-plough cuts a mere gash in the surface of ground, but forms a cylindrical burrow or drain in the clay four feet below the surface. In less

stiff soil, drain-pipes can be laid in the passage to keep it permanently open ; the whole being effected by machinery in lengths of about 100 feet at a time.

The drain made, being parallel to the ground-surface, will not be on a regular incline in undulating ground ; the process is hence more adapted to level and evenly-inclined land. The advantages of this method are very great ; drainage becomes a more ordinary agricultural operation, the surface of the ground is not seriously interfered with, the process is inexpensive, and may be renewed every five or six years, and finally in stiff soil no inert matter, stones, or old pipes, are necessary, and hence are not allowed to accumulate.

The main-drains.—The system of field-drains, however constructed, constitutes the principal and effective portion of the drains ; they draw off sub-surface water, increase the depth of active aërated soil, put it into a condition for assimilating manure, and for supplying sustenance to the crops through their roots, at any moderate depth ; thus causing warmth in the soil and an intermittent hygrometric action beneficial both to the crop, shown by augmented produce, and to the husbandmen by diminution of heavy labour. The main-drains are mere collecting drains supplied from lower extremities of the field-drains and conveying the drained water into the arterial watercourses of the country.

There is generally but little choice as regards the alignment and length of the main-drains ; they run along the lowest lines in any field, or along water-course lines at the bottoms of the various planes making up the field, and through any hollows that may exist. They are made as straight as the lowest edges of the fields and of the planes, or as the directions of the watercourse

lines will conveniently admit. When several fields to be drained happen to be in one plane, and intervening hedges can be removed, one main-drain may be made to serve for all, though enlarged to do so efficiently. The removal of needless fences is very advantageous, not only for convenience in draining, but also from saving useful land ; irregular fences and crooked boundaries may be straightened with similar good effect. Main-drains are generally covered so as to protect the ends of the field-drains from injury ; their fall or inclinations need not necessarily be very regular, although these as well as the sections should be sufficient to convey away rapidly all water that may arrive under extreme conditions, as after heavy rainfall, when the watercourses of the country are in flood.

Utilisation of the effluent.—The various modes of utilising the water are necessarily dependent on its amount, the available fall, and the local circumstances ; it may be dammed, stored, and used either as a cattle pond, for irrigation, or as the motive power for preparing food for cattle, thrashing corn, or other operations connected with husbandry.

When sufficient ready outfall is not available, as in low fen-lands, or on the banks of watercourses and streams of small fall, a long channel may have to be made to conduct the effluent parallel to the watercourse until a sufficient fall is obtained ; and its discharge may also require tide-valves, to protect it from return-water during floods.

Time and expense.—The most favourable time for field-drainage is when the land is unoccupied and during dry weather ; in England during autumn and winter, after the cutting of a white crop, or a clover crop, or

when the land is in pasture or in stubble, and immediately before a summer fallow or a green crop. The work has necessarily to be suspended during severe frost ; but any intervals of slightly wet weather are advantageous opportunities for drain-ploughing or drain-cutting in stiff clay. The expenses of ordinary field-drainage in England vary from about 1*l.* to 20*l.* per acre or even more, 30*l.* to 40*l.* The justifiable cost will in any case be considered in its ratio to the eventual value of the yield per acre, or enhanced yield after thorough drainage is completed. The expenses will necessarily have to be borne by an additional rent-charge on the land for several years until the improvement effected is comparatively exhausted. In some cases the expenses are repaid in yield in two or three years, as the increase of *weight* of wheat grown per acre may amount to from half as much again up to nearly double, and the same for potato crops. Perfect draining, accompanied by good management and followed by good culture, is, however, generally necessary for such achievements.

Wet lands in England, that really require drainage, and will not repay the cost of thorough drainage, may generally be considered hardly worth the expenses of mere cultivation.

The drainage of irrigated fields is a matter most frequently distinct from ordinary field-drainage, and hence usually treated in connection with irrigation. The drainage of marshes and bogs and the diversion and control of springs is also a separate branch of draining requiring hydro-geological knowledge and special treatment, before ordinary field-drainage can be conveniently applied to the land afterwards available for cultivation.

9. THE RUIN AND DETERIORATION OF CANALS OF IRRIGATION.

IN canals purely intended for navigation, the velocity of the water has to be kept below a fixed maximum ; below that it may be anything down to still-water without causing serious harm ; but in irrigation canals, which are continually receiving fresh supplies of water, and distributing it over the land through minor channels, the velocity of the water must be regulated with extreme nicety and care, in order to avoid many evils ; the two extremes of which result either in making the canal utterly unremunerative from not carrying sufficient water for purposes of irrigation, or in the eventual ruin and destruction, from deterioration, of the canal itself. Such canals cannot be maintained like roads, by merely repairing and trimming worn places ; they also require that their suitable velocities should be perpetually watched and regulated, even in the case that the intended velocities were originally correctly determined, and the designs and works made in accordance with them.

One of the most important causes of ruin to works of irrigation is that the velocities were never originally well determined, but were faulty and unsuitable, if not throughout the whole of the works, then at least in portions of them, the result of which eventually affects the whole. This is the case with a great many Indian canals, and is likely to be so on many others, as the matter of hydraulic velocities is one on which knowledge has been very deficient.

The next cause in point of importance is faulty

engineering design and defective construction of the works themselves, but this admits of remedy, without going in most cases to such an enormous expense as the former class of error entails. Even under this head, the apportionment of the velocities at intakes, outlets, bridges, and such works, is of extreme importance.

Thirdly, even if we assume the comparatively unusual case of the original intended velocities and the works themselves having been correctly designed in the abstract, and of the works having been constructed to perfection, the canal itself may yet follow the steady course to ruin. For whenever rain falls on the canal, or freshets or floods occur in any of the streams, rivers, or sources of supply, which then increase the supply of the canal, the depth of water in the canal is increased at certain places; and besides, the hydraulic gradient is increased, thus causing a very large increase of velocity taken in proportion to the adjustable correct limits. Under the same circumstances, too, a certain amount of silt is washed into the canal from its banks, and silt-bearing water may also, from want of early precautions, enter from the streams of supply. A high wind may also increase these evils; while, again, the velocity of the canal water may again be increased by the augmented velocity of the water entering the canal.

The practical adjustment of the velocity, or its regulation, becomes, under such circumstances, a matter of extreme care and refinement, even with the aid of all the hydraulic science the world now affords, and the assistance of good instruments and appliances for determining velocities; while without both of these aids it is nearly impossible in most instances.

Setting aside the extreme cases in which the excess

of water admitted may be so large that it becomes necessary to let it out over the country by breaking down a bank, and assuming the very moderate one of the velocity being increased by only one-fifth, this alone is amply sufficient to cause scour and erosion of bed and banks to a very appreciable extent; and if this recurs at rainy seasons for years, it becomes positive ruin, not merely on account of the erosion itself, but because also the scoured matter is transported by the water in the form of silt and deposited at other parts of the canal. The whole regimen of the entire canal thus gets out of order, the velocities are redistributed unsuitably or in ill proportion; such errors augment very rapidly, and a partly worn and partly silted-up canal is the result. This is ruin, which cannot be set right except by extraordinary repairs costing half as much as the original cost of the canal; and this is the principal cause of ruin on works of the very best design.

Other causes of deterioration are the admission of silt-bearing water at intakes, neglect of petty repairs, and non-removal of such an average amount of sediment as may be deposited in the canal and channels from causes apart from the preceding. It may also be mentioned that neglect of repair in one year is not compensated for by double the amount in the next, under similar circumstances; but that all such results are cumulative, from increase of interference with the strict regimen of the canal, and its suitably apportioned velocities in various parts of its course.

The consideration of these causes, and more especially of the principal ones, leads to the inevitable conclusion that a careful adjustment, measurement, and regulation of the velocities of the water in canals and

works of irrigation is the basis of almost all measures for preventing or deferring eventual ruin.

That considerable refinement is necessary is evident from the fact that the maximum velocities permissible in canals are :—

| | | |
|------|-----------------|------------------------------|
| 2·5 | feet per second | for very sandy soil. |
| 2·75 | ” | ” sandy soil. |
| 3 | ” | ” loam. |
| 4 | ” | ” gravel and very firm soil. |

While with low velocities of 1·5 and 1·75 feet per second, any suspended silt may be deposited, and vegetation springs up—the other source of extreme damage. The interval between the extremes is comparatively small and very easily overstepped.

Our present knowledge of velocities, their calculation, determination, and measurement is extremely coarse at present (not long ago it was altogether erroneous), hence the necessity for more knowledge and greater refinement which should be based on extremely careful experiments, carried out under the most advantageous circumstances, with all the aid that improved instruments and appliances of every sort can give and civilised assistance can furnish. The results of greater refinement in dealing with velocities may therefore, if correctly made use of and applied, prevent the lamentable ruin to canals which is illustrated by so many nearly obliterated ancient works in several formerly well-irrigated countries.

The causes of deterioration, and the remedy for them, having been previously explained, the next point to be considered is whether it is worth while to go to the expense involved in applying a more refined

knowledge of hydraulic velocities, and in the methods of dealing with them. The amount actually invested in India in canals and works of irrigation, including distribution done at all times, is certainly not less than twenty millions of capital, clear of all working expenses. (For figures in detail, see 'Hydraulic Statistics,' Allen, 1875.)

Now in dealing with statistics of this description, for purposes of argument, it is absolutely necessary that no exceptional case, rates, or figures should be used ; this rule will therefore be rigidly adhered to, and instead of dealing with any special case of canal, a theoretical canal under conditions that average well among actual statistics will be dealt with. Let us suppose a completely developed irrigation canal to have cost one million pounds, the irrigated area to be half a million acres annually, and the net annual profit 10 per cent. on the capital. (The Eastern Jumna Canal yields 22, the Western Jumna Canal 31, and the Kalerun 24 per cent., and these are the three completely developed canals of India, while it is evident that half-developed canals do not afford a fair basis of calculation, any more than partly opened lines of railway.) Now although the duration of a canal, or its lifetime, cannot be actually rigidly estimated, it is perfectly fair to assume that a canal relieved from the wear and tear of excessive velocities and from large deposits of silt, retrogression of levels, and so forth, which are all solely due to the causes previously explained, will last for a duration exceeding by a quarter the period that a less carefully managed canal will last ; in other words, let us assume that if such a canal in one case will last fifty years, in the other it will only last forty years with the same

prosperity, full average irrigated and full returns ; while after that period they may steadily dwindle down from prosperity to ruin in a similar ratio. Taking the canal thus only at its climax, the total profits in either case will be in proportion to the number of years of duration ; for the actual time when the 10 per cent. annual profit dwindles down to below zero, or the canal is worked at what is called a loss, is a different corresponding period in each case. Thus the compared profits on a capital of one million pounds will be about as follows :—

1. In case of more gradual deterioration.

| | | | | £ |
|--------------------------------------|-----------|---------------|---|-----------|
| 10 | per cent. | for 50 years= | . | 5 000 000 |
| 8 | " | 10 " | . | 800 000 |
| 6 | " | 10 " | . | 600 000 |
| 4 | " | 10 " | . | 400 000 |
| 2 | " | 10 " | . | 200 000 |
| 1 | " | 10 " | . | 100 000 |
| Total profits during a century . . . | | | | 7 100 000 |

2. In case of more rapid ruin.

| | | | | £ |
|--------------------------------------|-----------|--------------------|---|-----------|
| 10 | per cent. | for 40 years . . . | . | 4 000 000 |
| 8 | " | 8 " | . | 640 000 |
| 6 | " | 8 " | . | 480 000 |
| 4 | " | 8 " | . | 320 000 |
| 2 | " | 8 " | . | 160 000 |
| 1 | " | 8 " | . | 80 000 |
| 0 | " | 8 " | . | nil. |
| Loss during 12 years to be deducted | | | | |
| at 1 per cent. | | | | 120 000 |
| Total profits during a century . . . | | | | 5 560 000 |

The difference of total profits, apart from either simple or compound interest on them, is about one million and

a half pounds sterling, or half as much again as the original capital expended on one canal. Taking twenty such completed canals to represent the capital invested in India of twenty millions sterling, the loss due to the more rapid deterioration becomes thirty millions sterling, or half as much again as the capital invested, if extended over a full century in each case. Over half a century the loss is simply equal to the value of the capital invested, and this seems a probable and fair estimate of the anticipated loss in that period, or damage done.

To this estimated loss, or to something very near to it, there is only one alternative, and that is, the expenditure of the same amount in extraordinary repairs; which might be set down in the returns either as added to the capital account, or as included in the ordinary repairs. But, however accounts may be managed, the amount estimated must either be lost, or spent in making head against the destruction occurring more rapidly in one case than in the other.

It is useless to ignore that there is a lifetime to everything; the principles of dilapidation cannot be controverted. It may, however, be asserted that under any circumstances instructions may be given that the canals shall be kept in perfect repair, that every care shall be taken, and so forth. This is the very point; the care cannot be taken to prevent such damage unless a higher knowledge of velocities enables a more refined care and a real prevention to be exercised. No doubt the damage, instead of being allowed to accumulate over so many years into absolute ruin, may be stopped by incurring more expense annually; but this is merely spreading the bill for damage over a number of years,

the expense is not prevented in that case, but merely divided ; and if this form of account be preferred, instead of dealing with a total loss of twenty millions in fifty years, it becomes a waste, loss, or combination of both, of 400,000*l.* yearly over the whole of the irrigation canals and works of distribution of India, which is simply due to the coarseness of our knowledge about velocities. Comparing this annual waste, or even merely a quarter, or a tenth of it, with the relatively small cost of a thoroughly well-conducted series of hydraulic experiments, we may easily see whether the latter are worth while from a financial point of view, as a just and remunerative investment or expenditure on public works.

The principle involved cannot be avoided by drawing any analogy between canals and railways. All improved modes and principles, and increased knowledge, experiments, and so forth, on railways, may have cost India nothing. As railways in their perfection were first required in England where they are still being improved at the expense of skill, money, and thought, all such ideas may be borrowed gratuitously. But there are no large irrigation canals in England, and India must necessarily work out its own improvements in that branch at its own expense, and effect permanent economies for itself, if at all ; although it may, and perhaps should, bring to bear on them the highest English skill available in every respect, and make use of it both at home and in India.

In following up, or copying in practice, any clearly defined thoroughly-worked-out principles, as those of roads, railways, and navigable canals, a routine system of the marionette type may be sufficient for the purpose :

but when practical improvement has to be gained by experience, experiment, and skill, such a system is inapplicable without further aid.

The method hitherto adopted of following up and using the hydraulic experience and formulæ devised in France and Germany, and of applying their errors as well as their principles on a very magnified scale, thus saving expense in experiments, has had the most disastrous effect on the irrigation works of India ; this point hardly requires exemplification. Latterly the large-scale experiments of Captain Allan Cunningham have demonstrated the immense amount of error involved in using the French and American formulæ and have pointed out the correct method. This, however, is not all that is required ; the correct principles must be applied in practice. Any dispensing with the application of improved knowledge in a branch of science that pre-eminently affects the permanent benefit of large and extensive works of irrigation seems therefore perfectly indefensible either on financial or on any other grounds.

10. ON WATER-METERS.

The term water-meter being frequently used with little discrimination, it becomes necessary to notice briefly the distinction between water-meters and modules or water-regulators. A module actually regulates the supply of water passing into a channel or into a pipe, or makes it practically constant, although both the amount of water and the pressure in the main canal, main pipe, or reservoir, supplying the branch canal or pipe, may be variable. A water-meter does not regulate supply it simply

measures or registers supply under corresponding circumstances. Such is the broad distinction ; yet water-companies frequently use modules for regulating their supplies, when in large quantities, and call them water-meters ; also real water-meters have sometimes auxiliary regulating appliances attached to them. In the former case there is an habitual blunder in language ; in the latter there is a constructive difficulty, apparently affecting the term used.

A module is undoubtedly the more perfect appliance as it both regulates and enables the amount of supply passing in any time to be arrived at by calculation, that is to say, it also answers the purpose of a water-meter. A registering or chronographic apparatus may be attached to a module, but it still remains a module. A simple water-meter or registering machine does not regulate supply with practical exactitude (or if it does so, it then is really a module) ; but, if it has an auxiliary regulator, this merely controls either pressure or quantity, or both, between two limits, convenient to the action of the mechanism, and the machine still remains a water-meter from the fact of its not possessing the complete qualities of a module.

The notion that all such appliances may be distinguished as regulators or meters, according as they are attached to reservoirs and canals or to pipes of supply, is erroneous.

For various types of module, see the paragraph devoted to that subject.

As to water-meters, nominally so-called, we may expect to find that some of them are really modules.

Trough-meters.—The earliest of the English water-meters dates from the time when iron pipes came

into use in England for conducting water, and was known as Crosley's water-meter. [It is said that Samuel Clegg, a mechanical engineer in charge of some pumps at Liverpool, in 1802, was the inventor of a gas-meter (See William Matthews's 'Hydraulia,' of April 1835), and of the stand-pipe, and that his ideas gave rise to the water-meter, but there is much doubt about this.] Samuel Crosley's first liquid-meter was a rotating drum inclosed in an air-tight vessel, and certainly was the converse of a gas-meter, as regards action. Crosley's second liquid-meter was a rotating trough, in pattern very like the first. (See p. 304, Matthews's 'Hydraulia.') This latter is the common one, and is well known to this day; it has been re-invented several times, and is sometimes known as Parkinson's, on account of some error (in the Minutes of Proceedings of the Institution of Civil Engineers, January 1851) having intentionally or undesignedly conveyed that this meter was his invention. But in this case neither favouritism, wealth, nor combination have sufficed to obscure the past. Crosley's liquid-meter is a good one, as regards exactitude of measurement; one of its defects is the loss of all pressure at points beyond it, or after the water has passed through it; hence, when applied to the supply of a single house, it must be placed at the top or at the highest level in that house where water is required. It has a ball-valve regulator for maintaining a constant level in the supply-trough

Piston-meters.—Brunton's meter (see copy of patent in 'Repertory of Arts,' &c., for July 1829) was a piston-meter; the water passed through a cylinder with packed piston and rod, nozzle, and valve, or cock; its principle consisted in applying the static fluid pressure on the

piston to move it with sufficient force to raise a weight on an inclined plane during the whole range of impulse ; the power generated is, at the termination of the impulse, capable of moving the valves or four-way cock, and reversing the pressure on the piston, by which the weight is again raised ; the motion is therefore continuous, and expresses the quantity of discharge, which is registered by wheelwork attached to the machine. This meter has been re-invented, with more or less improvement, by Kennedy (see 'Proc. Inst. C. E.' for 1856). The defects of meters of this type are, that the reversals of pressure cause shocks in the mains, and allow some water to pass unregistered ; also either the packed piston, the reversing cock, or the balance may be seriously affected by friction, so much so as to get jammed.

Frost's meter is also a piston meter, hardly preferable to the other two ; its reciprocating mechanism is not better, though it has a three-way valve moving an auxiliary piston and working another three-way exhaust valve ; its piston moves leather buckets within the cylinder, and the whole is liable to stick. (For drawings see 'Proc. Inst. C. E.' for 1857.)

Among the modern piston-meters is Galaffe's ; it has two cylinders and two slide-valves, working in cross action, thus neutralising much defect, or rather perhaps keeping it out of view. It is much used in Belgium, and is perhaps the best piston-meter now well known. The compensation of defect that it affords must not, however, make us lose sight of its inherent qualities. Richards' water-meter is the most recent piston-meter, and has some advantages in simplicity ; it seems to be a development of the gas-meter of the same inventor. All piston-meters appear to require supervision, and

to be generally unsuited to low speeds and small discharges.

Turbine-meters.—Water-meters on this principle are perhaps older than those of the preceding two classes, although it is impracticable to assign definite dates to their introduction. Their applied object is to register the velocity of supply through a fixed opening, but, as some friction must exist, they actually record a less velocity, and, when very defective from wear or rust, become utterly untrustworthy. There have been turbine-meters of several kinds, the modern form is the reaction turbine in common use; Siemens' turbine-meter is one of these. The peculiarity of this meter consists in the drag-boards attached to the rotating drum, which ensure that its velocity shall not exceed that of the water at any time, and thus within certain limits maintaining a constant speed of revolution under a supply that does not vary in amount; in other words, the effect of slight variation in the velocity of the water of supply is entirely annulled. This is a marked advantage, but the appliance suffers from the before-mentioned defects, inseparable from its class of water-meter.

Fan-meters.—These light fans, constructed with the object that the effect of all passing water shall be registered, are the water-meters of the most modern sort. They are much used in Germany, Russia, Italy, and France, but are not popular in England. Siemens' fan-meter has drag-plates to moderate velocity, as in his turbine-meter, and these constitute its chief advantage.

Tylor's fan-meter (described in a paper read before the Institution of Mechanical Engineers) has the same advantage as Siemens': its wheel is of indiarubber, its openings for entrance-water are well arranged, it is not

easily choked by sediment at the points of exit, and is generally a much-improved fan-meter. A special improvement in it is an appliance for regulating the speed of the fan by a counter-current of water, so arranged that it is adjustable from the outside of the case. This is of great convenience in testing, as any error in registration due to long use or accident can be remedied without taking the meter to pieces. On the whole, Mr. Tylor's fan-meter is perhaps the best of its kind ; it has been thoroughly tested by Mr. Anderson, who has a high opinion of it, and it is much used already in the Colonies.

The objections to fan-meters, or their defects, consist in allowing unregistered water to pass, in slowness in getting into motion at starting, and in spinning on after the supply has been cut off ; these defects do not compensate each other, but they may be much reduced by management and care.

General Remarks.—In order to arrive at a just and full comprehension of any particular meter or module, the thing itself should be inspected or examined during action under various conditions ; illustrations fail to convey the information that may be obtained in this manner.

It may be noticed that house-meters for registering small supplies of water must necessarily be more delicate in many respects than the large supply-meters of water-companies ; they should demand little or no supervision, and be so arranged as not to permit of being easily tampered with, either by the consumer or by the water-officials or agents. Probably some type of module, ensuring constant head during action, with a chronographic apparatus, admitting of independent check on

time, would best answer such purposes (see Modules, section I, Chapter III.).

For exact measurement of supply through pipes under variable pressure, a good pressure-gauge and a chronographic apparatus are necessary; besides this, the outlet must be free, and a considerable length of the pipe must be made of some exact diameter, less than the ordinary varying diameters above the point of observation: all the conditions require much precision and competent management.

HYDRAULIC WORKING TABLES.

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|--------------------------|-------------------------------|
| I. GRAVITY. | VII. CHANNELS AND CANALS. |
| II. CATCHMENT. | VIII. PIPES AND CULVERTS. |
| III. STORAGE AND SUPPLY. | IX. BENDS AND OBSTRUCTIONS. |
| IV. FLOOD DISCHARGE. | X. SLUICES AND WEIRS. |
| V. HYDRAULIC SECTIONS. | XI. MAXIMUM VELOCITIES. |
| VI. HYDRAULIC SLOPES. | XII. HYDRAULIC CO-EFFICIENTS. |

ADDITIONAL AND MISCELLANEOUS TABLES.

These tables can be used either with tradesmen's units or with the units of the English decimal scientific series.

TABLE I.—GRAVITY.

CALCULATED VALUES OF THE FORCE OF GRAVITY IN FEET AT
DIFFERENT LATITUDES AND ELEVATIONS, BEING A TABU-
LATED APPLICATION OF THE FORMULÆ

$$g = 32.1695 (1 - 0.00284 \cos 2l) \left(1 - \frac{2e}{r}\right).$$
$$r = 20887540 (1 + 0.00164 \cos 2l).$$

Values of the force of gravity in feet at different

| ELEVATION IN FEET | LATITUDE | | | |
|----------------------|----------|---------|---------|---------|
| | 0° | 5° | 10° | 15° |
| 0 | 32·0781 | 32·0795 | 32·0836 | 32·0904 |
| 100 | 32·0778 | 32·0792 | 32·0833 | 32·0901 |
| 200 | 32·0775 | 32·0789 | 32·0830 | 32·0898 |
| 300 | 32·0772 | 32·0786 | 32·0827 | 32·0895 |
| 400 | 32·0769 | 32·0783 | 32·0824 | 32·0892 |
| 500 | 32·0766 | 32·0780 | 32·0821 | 32·0889 |
| 600 | 32·0763 | 32·0777 | 32·0818 | 32·0886 |
| 700 | 32·0760 | 32·0774 | 32·0815 | 32·0883 |
| 800 | 32·0757 | 32·0771 | 32·0812 | 32·0880 |
| 900 | 32·0754 | 32·0768 | 32·0809 | 32·0877 |
| 1000 | 32·0751 | 32·0765 | 32·0806 | 32·0874 |
| 2000 | 32·0721 | 32·0735 | 32·0775 | 32·0843 |
| 3000 | 32·0690 | 32·0704 | 32·0745 | 32·0813 |
| 4000 | 32·0660 | 32·0674 | 32·0715 | 32·0783 |
| 5000 | 32·0630 | 32·0644 | 32·0685 | 32·0753 |

| ELEVATION IN FEET | LATITUDE | | | |
|----------------------|----------|---------|---------|---------|
| | 40° | 45° | 50° | 55° |
| 0 | 32·1536 | 32·1695 | 32·1854 | 32·2008 |
| 100 | 32·1533 | 32·1692 | 32·1851 | 32·2005 |
| 200 | 32·1530 | 32·1689 | 32·1848 | 32·2002 |
| 300 | 32·1528 | 32·1686 | 32·1845 | 32·1998 |
| 400 | 32·1524 | 32·1683 | 32·1842 | 32·1995 |
| 500 | 32·1521 | 32·1680 | 32·1839 | 32·1992 |
| 600 | 32·1518 | 32·1677 | 32·1835 | 32·1989 |
| 700 | 32·1515 | 32·1674 | 32·1832 | 32·1986 |
| 800 | 32·1512 | 32·1671 | 32·1829 | 32·1983 |
| 900 | 32·1509 | 32·1668 | 32·1826 | 32·1980 |
| 1000 | 32·1506 | 32·1665 | 32·1823 | 32·1977 |
| 2000 | 32·1473 | 32·1633 | 32·1793 | 32·1947 |
| 3000 | 32·1442 | 32·1603 | 32·1762 | 32·1916 |
| 4000 | 32·1411 | 32·1572 | 32·1731 | 32·1885 |
| 5000 | 32·1382 | 32·1541 | 32·1700 | 32·1854 |

TABLE I.]

GRAVITY.



latitudes and elevations above mean sea level.

| ELEVATION IN FEET | LATITUDE | | | |
|----------------------|----------|---------|---------|---------|
| | 20° | 25° | 30° | 35° |
| 0 | 32°0995 | 32°1108 | 32°1238 | 32°1383 |
| 100 | 32°0992 | 32°1105 | 32°1235 | 32°1380 |
| 200 | 32°0989 | 32°1102 | 32°1232 | 32°1377 |
| 300 | 32°0986 | 32°1099 | 32°1229 | 32°1374 |
| 400 | 32°0983 | 32°1096 | 32°1226 | 32°1371 |
| 500 | 32°0980 | 32°1093 | 32°1223 | 32°1368 |
| 600 | 32°0977 | 32°1090 | 32°1220 | 32°1364 |
| 700 | 32°0974 | 32°1087 | 32°1217 | 32°1361 |
| 800 | 32°0971 | 32°1084 | 32°1214 | 32°1358 |
| 900 | 32°0968 | 32°1081 | 32°1211 | 32°1355 |
| 1000 | 32°0965 | 32°1077 | 32°1208 | 32°1352 |
| 2000 | 32°0934 | 32°1047 | 32°1177 | 32°1322 |
| 3000 | 32°0904 | 32°1017 | 32°1146 | 32°1291 |
| 4000 | 32°0874 | 32°0986 | 32°1115 | 32°1260 |
| 5000 | 32°0843 | 32°0955 | 32°1084 | 32°1229 |

| ELEVATION IN FEET | LATITUDE | | | |
|----------------------|----------|---------|---------|---------|
| | 60° | 70° | 80° | 90° |
| 0 | 32°2152 | 32°2395 | 32°2554 | 32°2609 |
| 100 | 32°2149 | 32°2392 | 32°2551 | 32°2606 |
| 200 | 32°2146 | 32°2389 | 32°2548 | 32°2603 |
| 300 | 32°2143 | 32°2386 | 32°2545 | 32°2600 |
| 400 | 32°2140 | 32°2382 | 32°2541 | 32°2596 |
| 500 | 32°2136 | 32°2379 | 32°2538 | 32°2593 |
| 600 | 32°2133 | 32°2376 | 32°2535 | 32°2590 |
| 700 | 32°2130 | 32°2373 | 32°2532 | 32°2587 |
| 800 | 32°2127 | 32°2370 | 32°2529 | 32°2584 |
| 900 | 32°2124 | 32°2367 | 32°2526 | 32°2581 |
| 1000 | 32°2121 | 32°2364 | 32°2523 | 32°2578 |
| 2000 | 32°2090 | 32°2332 | 32°2491 | 32°2546 |
| 3000 | 32°2059 | 32°2301 | 32°2460 | 32°2515 |
| 4000 | 32°2028 | 32°2270 | 32°2429 | 32°2483 |
| 5000 | 32°1997 | 32°2239 | 32°2397 | 32°2452 |

TABLE II.—CATCHMENT.

- Part 1. Total quantities of water resulting from a given effective rainfall run off from any unit of catchment area.
- Part 2. Supply in cubic feet per second throughout the year, resulting from a given effective rainfall run off from one square statute mile of catchment area.
- Part 3. Supply in cubic feet per second, resulting from an effective daily rainfall for 24 hours over catchment areas.
- Part 4. Equivalent supply.

PART I.—Total quantities of water resulting from a given effective rainfall run off from any unit of catchment area.

| Rainfall in feet | Cubic feet per square chain | Cubic feet per century | Cubic rods per square league | Rainfall in inches | Cubic feet per acre | Cubic feet per square mile |
|------------------|-----------------------------|------------------------|------------------------------|--------------------|---------------------|----------------------------|
| 1 | 10 000 | 1 000 000 | 100 000 | 12" | 43 560 | 27 878 400 |
| 0·9 | 9 000 | 900 000 | 90 000 | 11" | 39 900 | 25 555 200 |
| 0·8 | 8 000 | 800 000 | 80 000 | 10" | 36 300 | 23 232 000 |
| 0·7 | 7 000 | 700 000 | 70 000 | 9" | 32 670 | 20 908 800 |
| 0·6 | 6 000 | 600 000 | 60 000 | 8" | 29 040 | 18 505 600 |
| 0·5 | 5 000 | 500 000 | 50 000 | 7" | 25 410 | 16 262 400 |
| 0·4 | 4 000 | 400 000 | 40 000 | 6" | 21 780 | 13 939 200 |
| 0·3 | 3 000 | 300 000 | 30 000 | 5" | 18 150 | 11 616 000 |
| 0·2 | 2 000 | 200 000 | 20 000 | 4" | 14 520 | 9 252 800 |
| 0·1 | 1 000 | 100 000 | 10 000 | 3" | 10 890 | 6 969 600 |
| | | | | 2" | 7 260 | 4 646 400 |
| | | | | 1" | 3 630 | 2 323 200 |
| | | | | " | | |
| 0·09 | 900 | 90 000 | 9 000 | 0·9 | 3 267 | 2 090 880 |
| 0·08 | 800 | 80 000 | 8 000 | 0·8 | 2 904 | 1 850 560 |
| 0·07 | 700 | 70 000 | 7 000 | 0·7 | 2 541 | 1 626 240 |
| 0·06 | 600 | 60 000 | 6 000 | 0·6 | 2 178 | 1 393 920 |
| 0·05 | 500 | 50 000 | 5 000 | 0·5 | 1 815 | 1 161 600 |
| 0·04 | 400 | 40 000 | 4 000 | 0·4 | 1 452 | 925 280 |
| 0·03 | 300 | 30 000 | 3 000 | 0·3 | 1 089 | 696 960 |
| 0·02 | 200 | 20 000 | 2 000 | 0·2 | 726 | 464 640 |
| 0·01 | 100 | 10 000 | 1 000 | 0·1 | 363 | 232 320 |

N.B.—1 square statute mile = 640 acres = 27 878 400 square feet.
 1 square league = 4 sq. London miles = 100 centuries = 10 000 sq. chains
 (Ramsden's).

1 square chain = 100 sq. rods = 10 000 square feet.

PART 2.—*Supply in cubic feet per second throughout the year, resulting from a given effective annual rainfall run off from one square statute mile of catchment area.*

| Annual rainfall in feet | Supply in cubic feet per second | Annual rainfall in feet | Supply in cubic feet per second | Annual rainfall in feet | Supply in cubic feet per second |
|-------------------------|---------------------------------|-------------------------|---------------------------------|-------------------------|---------------------------------|
| 0·1 | ·0883 | 2·1 | 1·8550 | 4·1 | 3·621 |
| 0·2 | ·1766 | 2·2 | 1·9433 | 4·2 | 3·7100 |
| 0·3 | 2650 | 2·3 | 2·0317 | 4·3 | 3·7983 |
| 0·4 | ·3533 | 2·4 | 2·1200 | 4·4 | 3·8866 |
| 0·5 | ·4417 | 2·5 | 2·2083 | 4·5 | 3·9750 |
| 0·6 | ·5300 | 2·6 | 2·2966 | 4·6 | 4·0633 |
| 0·7 | ·6183 | 2·7 | 2·3850 | 4·7 | 4·1517 |
| 0·8 | ·7066 | 2·8 | 2·4733 | 4·8 | 4·2400 |
| 0·9 | ·7950 | 2·9 | 2·5617 | 4·9 | 4·3283 |
| 1·0 | ·8833 | 3·0 | 2·6500 | 5·0 | 4·4166 |
| 1·1 | ·9717 | 3·1 | 2·7383 | 5·5 | 4·8583 |
| 1·2 | 1·0600 | 3·2 | 2·8266 | 6· | 5·3000 |
| 1·3 | 1·1483 | 3·3 | 2·9150 | 6·5 | 5·7417 |
| 1·4 | 1·2366 | 3·4 | 3·0033 | 7· | 6·1833 |
| 1·5 | 1·3250 | 3·5 | 3·0917 | 7·5 | 6·6250 |
| 1·6 | 1·4133 | 3·6 | 3·1800 | 8· | 7·0666 |
| 1·7 | 1·5017 | 3·7 | 3·2683 | 8·5 | 7·5083 |
| 1·8 | 1·5900 | 3·8 | 3·3566 | 9· | 7·9500 |
| 1·9 | 1·6783 | 3·9 | 3·4450 | 9·5 | 8·3917 |
| 2·0 | 1·7666 | 4·0 | 3·5333 | 10· | 8·8333 |

Similarly from 1 foot of effective annual rainfall, the supply per second is—

| | | |
|----------------------------|-------------|-----------------------|
| From 1 square league . . . | 3·170 979 2 | cubic feet per second |
| „ 1 century . . . | 0·031 709 8 | „ „ |
| „ 1 square chain . . . | 0·000 317 1 | „ „ |

PART 3.—Supply in cubic feet per second, resulting from an effective daily rainfall for 24 hours over catchment areas.

FOR CATCHMENT AREAS IN SQUARE STATUTE MILES.

| Catchment in sq. miles | For an effective daily rainfall in feet and decimals of | | | | | | | | | |
|---------------------------|---|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 0·1 | 0·09 | 0·08 | 0·07 | 0·06 | 0·05 | 0·04 | 0·03 | 0·02 | 0·01 |
| | Cubic feet per second | | | | | | | | | |
| 1 | 32·27 | 29·04 | 25·81 | 22·59 | 19·36 | 16·13 | 12·91 | 9·68 | 6·45 | 3·23 |
| 2 | 64·53 | 58·07 | 51·62 | 45·16 | 38·72 | 32·26 | 25·81 | 19·36 | 12·90 | 6·45 |
| 3 | 96·80 | 83·52 | 74·24 | 64·96 | 55·68 | 48·40 | 37·12 | 27·84 | 18·56 | 9·68 |
| 4 | 129·1 | 116·1 | 103·2 | 90·30 | 76·40 | 64·50 | 51·60 | 38·70 | 25·80 | 12·91 |
| 5 | 161·3 | 145·2 | 129·0 | 112·9 | 96·80 | 80·64 | 64·50 | 48·40 | 32·25 | 16·13 |
| 6 | 193·6 | 174·2 | 154·8 | 135·4 | 116·1 | 96·78 | 77·40 | 58·06 | 38·70 | 19·36 |
| 7 | 225·9 | 203·2 | 180·6 | 158·0 | 135·5 | 112·9 | 90·30 | 67·73 | 45·15 | 22·59 |
| 8 | 258·1 | 232·2 | 206·4 | 180·6 | 154·8 | 129·0 | 103·2 | 77·40 | 51·60 | 25·81 |
| 9 | 290·4 | 261·4 | 232·3 | 203·3 | 174·3 | 145·2 | 116·2 | 87·13 | 58·10 | 29·04 |
| 10 | 322·7 | 290·4 | 258·1 | 225·9 | 193·6 | 161·3 | 129·1 | 96·80 | 64·60 | 32·27 |

| Catchment in sq. miles | For an effective daily rainfall in inches and decimals of | | | | | | | | | |
|---------------------------|---|-------|-------|-------|-------|-------|--------|-------|-------|-------|
| | 1·0 | 0·9 | 0·8 | 0·7 | 0·6 | 0·5 | 0·4 | 0·3 | 0·2 | 0·1 |
| | Cubic feet per second | | | | | | | | | |
| 1 | 26·89 | 24·20 | 21·51 | 18·82 | 16·13 | 13·44 | 10·76 | 8·07 | 5·38 | 2·69 |
| 2 | 53·78 | 48·40 | 43·00 | 37·64 | 32·26 | 26·89 | 21·50 | 16·13 | 10·75 | 5·38 |
| 3 | 80·67 | 54·60 | 64·53 | 56·47 | 48·40 | 40·33 | 32·26 | 24·20 | 16·13 | 8·07 |
| 4 | 107·56 | 96·75 | 86·00 | 75·25 | 64·50 | 53·78 | 43·00 | 32·25 | 21·50 | 10·76 |
| 5 | 134·4 | 120·9 | 107·5 | 94·08 | 80·64 | 67·22 | 53·75 | 40·32 | 26·87 | 13·14 |
| 6 | 161·3 | 145·1 | 135·0 | 112·9 | 96·78 | 80·67 | 67·55 | 48·39 | 33·77 | 16·13 |
| 7 | 188·2 | 169·3 | 150·5 | 131·7 | 112·9 | 94·11 | 75·25 | 56·45 | 37·62 | 18·82 |
| 8 | 215·1 | 193·6 | 172·1 | 150·5 | 129·0 | 107·5 | 86·05 | 64·50 | 43·02 | 21·51 |
| 9 | 242·0 | 217·8 | 193·6 | 169·4 | 145·2 | 121·0 | 96·80 | 72·60 | 48·40 | 24·20 |
| 10 | 268·9 | 242·0 | 215·1 | 188·2 | 161·3 | 134·4 | 107·56 | 80·67 | 53·78 | 26·89 |

Similarly from 1 foot of day's rainfall, the supply is—

| | | | | | |
|----------------------|----|---|---|------------|-----------------------|
| From 1 square league | .. | . | . | 1157·40740 | cubic feet per second |
| „ 1 century | . | . | . | 1·15741 | „ „ |
| „ 1 square chain | . | . | . | 0·01157 | „ „ |

PART 3 (continued).—Supply in cubic feet per second, resulting from an effective daily rainfall for 24 hours over catchment areas.

FOR CATCHMENT AREAS IN ACRES.

| Catchment in acres | For an effective daily rainfall in feet and decimals of | | | | | | | | | |
|-----------------------|---|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 0·1 | 0·09 | 0·08 | 0·07 | 0·06 | 0·05 | 0·04 | 0·03 | 0·02 | 0·01 |
| | Cubic feet per second | | | | | | | | | |
| 25 | 1·26 | 1·13 | 1·01 | 0·88 | 0·76 | 0·63 | 0·50 | 0·378 | 0·252 | 0·126 |
| 50 | 2·52 | 2·27 | 2·02 | 1·77 | 1·51 | 1·26 | 1·01 | 0·756 | 0·504 | 0·252 |
| 75 | 3·78 | 3·40 | 3·03 | 2·65 | 2·27 | 1·89 | 1·51 | 1·134 | 0·756 | 0·378 |
| 100 | 5·04 | 4·54 | 4·03 | 3·53 | 3·03 | 2·52 | 2·02 | 1·513 | 1·008 | 0·504 |
| 200 | 10·08 | 9·08 | 8·07 | 7·06 | 6·05 | 5·04 | 4·03 | 3·025 | 2·017 | 1·008 |
| 300 | 15·13 | 13·61 | 12·10 | 10·59 | 9·08 | 7·56 | 6·05 | 4·538 | 3·025 | 1·513 |
| 400 | 20·17 | 18·15 | 16·13 | 14·12 | 12·10 | 10·08 | 8·06 | 6·050 | 4·033 | 2·017 |
| 500 | 25·21 | 22·69 | 20·17 | 17·65 | 15·13 | 12·61 | 10·08 | 7·563 | 5·042 | 2·521 |
| 600 | 30·25 | 27·22 | 24·20 | 21·17 | 18·15 | 15·13 | 12·10 | 9·075 | 6·050 | 3·025 |
| 640 | 32·27 | 29·04 | 25·81 | 22·59 | 19·36 | 16·13 | 12·91 | 9·680 | 6·453 | 3·227 |

| Catchment in acres | For an effective daily rainfall in inches and decimals of | | | | | | | | | |
|-----------------------|---|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 1·0 | 0·9 | 0·8 | 0·7 | 0·6 | 0·5 | 0·4 | 0·3 | 0·2 | 0·1 |
| | Cubic feet per second | | | | | | | | | |
| 25 | 1·05 | 0·95 | 0·84 | 0·74 | 0·64 | 0·53 | 0·42 | 0·315 | 0·210 | 0·105 |
| 50 | 2·10 | 1·89 | 1·68 | 1·47 | 1·28 | 1·05 | 0·84 | 0·630 | 0·420 | 0·210 |
| 75 | 3·15 | 2·83 | 2·52 | 2·20 | 1·92 | 1·58 | 1·26 | 0·945 | 0·630 | 0·315 |
| 100 | 4·20 | 3·78 | 3·36 | 2·94 | 2·56 | 2·10 | 1·68 | 1·260 | 0·840 | 0·420 |
| 200 | 8·40 | 7·56 | 6·72 | 5·88 | 5·12 | 4·20 | 3·36 | 2·521 | 1·681 | 0·840 |
| 300 | 12·60 | 11·34 | 10·08 | 8·82 | 7·68 | 6·30 | 5·04 | 3·781 | 2·521 | 1·260 |
| 400 | 16·81 | 15·12 | 13·44 | 11·76 | 10·24 | 8·40 | 6·62 | 5·042 | 3·361 | 1·681 |
| 500 | 21·01 | 18·91 | 16·81 | 14·71 | 12·80 | 10·50 | 8·40 | 6·302 | 4·202 | 2·101 |
| 600 | 25·21 | 22·69 | 20·17 | 17·65 | 15·36 | 12·60 | 10·08 | 7·562 | 5·042 | 2·521 |
| 640 | 26·89 | 24·20 | 21·51 | 18·82 | 16·13 | 13·44 | 10·76 | 8·067 | 5·378 | 2·689 |

The effective rainfall is the measured rainfall, after deduction for evaporation, absorption, and all losses.

PART 4.—*Equivalent supply.*

Cubic feet per second, per minute, and per day, into Gallons per second, per minute, and per day.

| Per second | | Per minute | | Per day of 24 hours | |
|------------|---------|------------|---------|---------------------|-----------|
| Cubic feet | Gallons | Cubic feet | Gallons | Cubic feet | Gallons |
| 0·01 | 0·06 | 0·6 | 3·74 | 864 | 5 384 |
| 0·02 | 0·12 | 1·2 | 7·47 | 1 728 | 10 768 |
| 0·03 | 0·19 | 1·8 | 11·21 | 2 592 | 16 152 |
| 0·04 | 0·25 | 2·4 | 14·95 | 3 436 | 21 536 |
| 0·05 | 0·31 | 3· | 18·69 | 4 320 | 26 920 |
| 0·06 | 0·37 | 3·6 | 22·43 | 5 184 | 32 304 |
| 0·07 | 0·44 | 4·2 | 26·17 | 6 048 | 37 688 |
| 0·08 | 0·5 | 4·8 | 29·90 | 6 912 | 43 072 |
| 0·09 | 0·56 | 5·4 | 33·64 | 7 776 | 48 456 |
| 0·1 | 0·62 | 6· | 37·39 | 8 640 | 53 844 |
| 0·16 | 1·04 | 10· | 62·32 | 14 400 | 89 741 |
| 0·33 | 2·08 | 20· | 124·64 | 28 800 | 179 842 |
| 0·5 | 3·12 | 30· | 186·96 | 43 200 | 269 223 |
| 0·66 | 4·16 | 40· | 249·28 | 47 600 | 358 964 |
| 0·83 | 5·20 | 50· | 311·60 | 72 000 | 448 704 |
| 1· | 6·23 | 60· | 373·92 | 86 400 | 538 446 |
| 1·16 | 7·27 | 70· | 436·24 | 100 800 | 628 187 |
| 1·33 | 8·31 | 80· | 498·56 | 115 200 | 717 928 |
| 1·5 | 9·35 | 90· | 560·88 | 129 600 | 807 669 |
| 1·66 | 10·39 | 100 | 623·20 | 144 000 | 897 408 |
| 1·15 | 7·21 | 69·4 | 432·7 | 100 000 | 623 200 |
| 1·93 | 14·42 | 115·7 | 865·4 | 200 000 | 1 246 400 |
| 3·47 | 21·63 | 208·3 | 1 298·1 | 300 000 | 1 869 600 |
| 4·63 | 28·84 | 277·7 | 1 730·8 | 400 000 | 2 492 800 |
| 5·78 | 36·05 | 346·8 | 2 163·5 | 500 000 | 3 116 000 |
| 6·94 | 43·26 | 416·6 | 2 596·2 | 600 000 | 3 739 200 |
| 8·10 | 50·47 | 486· | 3 028·9 | 700 000 | 4 362 400 |
| 9·26 | 57·68 | 555·5 | 3 461·6 | 800 000 | 4 985 600 |
| 10·41 | 64·89 | 624·9 | 3 894·3 | 900 000 | 5 608 800 |
| 11·57 | 72·10 | 694·4 | 4 327·5 | 1 million | 6 232 000 |

PART 4 (continued).—Equivalent supply.

Gallons per second, per minute, and per day, into Cubic Feet per second, per minute, and per day.

| Per second | | Per minute | | Per day of 24 hours | |
|------------|------------|------------|------------|---------------------|------------|
| Gallons | Cubic feet | Gallons | Cubic feet | Gallons | Cubic feet |
| 0.1 | 0.016 | 6 | 0.96 | 8640 | 1385 |
| 0.2 | 0.032 | 12 | 1.92 | 17280 | 2772 |
| 0.3 | 0.048 | 18 | 2.88 | 25920 | 4158 |
| 0.4 | 0.064 | 24 | 3.84 | 34560 | 5543 |
| 0.5 | 0.080 | 30 | 4.80 | 43200 | 6929 |
| 0.6 | 0.096 | 36 | 5.76 | 51840 | 8315 |
| 0.7 | 0.112 | 42 | 6.72 | 60480 | 9701 |
| 0.8 | 0.128 | 48 | 7.68 | 69120 | 11087 |
| 0.9 | 0.144 | 54 | 8.64 | 77760 | 12473 |
| 1. | 0.160 | 60 | 9.62 | 86400 | 13858 |
| 0.166 | 0.027 | 10 | 1.60 | 14400 | 2310 |
| 0.333 | 0.053 | 20 | 3.21 | 28800 | 4619 |
| 0.5 | 0.080 | 30 | 4.81 | 43200 | 6929 |
| 0.666 | 0.107 | 40 | 6.42 | 57600 | 9239 |
| 0.833 | 0.134 | 50 | 8.02 | 72000 | 11549 |
| 1. | 0.160 | 60 | 9.62 | 86400 | 13858 |
| 1.166 | 0.187 | 70 | 11.23 | 100800 | 16168 |
| 1.333 | 0.214 | 80 | 12.83 | 115200 | 18478 |
| 1.5 | 0.241 | 90 | 14.44 | 129600 | 20788 |
| 1.666 | 0.267 | 100 | 16.04 | 144000 | 23097 |
| 1.15 | 0.186 | 69.4 | 111.4 | 100000 | 16040 |
| 1.93 | 0.371 | 115.7 | 222.8 | 200000 | 32079 |
| 3.47 | 0.557 | 208.3 | 334.2 | 300000 | 48119 |
| 4.63 | 0.742 | 277.7 | 445.6 | 400000 | 64159 |
| 5.78 | 0.928 | 346.8 | 556.9 | 500000 | 80199 |
| 6.94 | 1.114 | 416.6 | 667.3 | 600000 | 96239 |
| 8.10 | 1.299 | 486. | 779.7 | 700000 | 112278 |
| 9.26 | 1.485 | 555.5 | 891.1 | 800000 | 128318 |
| 10.41 | 1.670 | 624.9 | 1002.5 | 900000 | 144358 |
| 11.57 | 1.856 | 694.4 | 1113.9 | 1 million | 160398 |

TABLE III.—STORAGE AND SUPPLY.

- Part 1.** Capacity of reservoirs and supply from catchment.
- Part 2.** Utilisation of a continuous supply of water.
- Part 3.** Equivalent of continuous supply.

EXAMPLES.

PART I.—Capacity of reservoirs and supply from catchment.

FOR A NINE MONTHS' SUPPLY.

| Supply afforded during 270 days or nine months | Contents of reservoir to hold that supply | Surface of that reservoir if 3 feet deep on the average | Catchment necessary to fill that reservoir in three months; with 1 foot available rainfall in that time |
|--|---|---|---|
| Cubic feet per second | Cubic feet | Square feet | Square miles |
| 1 | 23 328 000 | 7 776 000 | 0·83678 |
| 2 | 46 656 000 | 15 552 000 | 1·67355 |
| 3 | 69 984 000 | 23 328 000 | 2·51033 |
| 4 | 93 312 000 | 31 104 000 | 3·34711 |
| 5 | 116 640 000 | 38 880 000 | 4·18388 |
| 6 | 139 968 000 | 46 656 000 | 5·02066 |
| 7 | 163 296 000 | 54 432 000 | 5·85743 |
| 8 | 186 624 000 | 62 208 000 | 6·69422 |
| 9 | 209 952 000 | 69 984 000 | 7·53099 |
| 10 | 233 280 000 | 77 760 000 | 8·36775 |
| 1·1951 | 27 878 400 | 9 292 800 | 1 |
| 2·3901 | 55 756 800 | 18 585 600 | 2 |
| 3·5852 | 83 635 200 | 27 878 400 | 3 |
| 4·7802 | 111 513 600 | 37 171 200 | 4 |
| 5·9753 | 139 392 000 | 46 464 000 | 5 |
| 7·1704 | 167 270 400 | 55 756 800 | 6 |
| 8·3654 | 195 148 800 | 65 049 600 | 7 |
| 9·5604 | 223 027 200 | 74 342 000 | 8 |
| 10·7555 | 250 905 600 | 83 635 200 | 9 |
| 11·9506 | 278 784 000 | 92 928 000 | 10 |

NOTE.—The reduction of similar quantities in decimal scientific units is so simple as not to require the aid of tables.

PART I (continued).—Capacity of reservoirs and supply
from catchment.

FOR AN EIGHT MONTHS' SUPPLY.

| Supply afforded during 240 days or eight months | Contents of reservoir to hold that supply | Surface of that reservoir if 3 feet deep on the average | Catchment area necessary to fill that reservoir in four months, having one foot available rainfall in that time |
|---|---|---|---|
| Cubic feet per second | Cubic feet | Square feet | Square miles |
| 1 | 20 736 000 | 6 912 000 | ·7438 |
| 2 | 41 472 000 | 13 824 000 | 1·4876 |
| 3 | 62 208 000 | 20 736 000 | 2·2314 |
| 4 | 82 944 000 | 27 648 000 | 2·9752 |
| 5 | 103 680 000 | 34 560 000 | 3·7190 |
| 6 | 124 416 000 | 41 472 000 | 4·4628 |
| 7 | 145 152 000 | 48 384 000 | 5·2066 |
| 8 | 165 888 000 | 55 296 000 | 5·9504 |
| 9 | 186 624 000 | 62 208 000 | 6·6942 |
| 10 | 207 360 000 | 69 120 000 | 7·4380 |
| 1·3444 | 27 878 400 | 9 292 800 | 1 |
| 2·6888 | 55 756 800 | 18 585 600 | 2 |
| 4·0333 | 83 635 200 | 27 878 400 | 3 |
| 5·3777 | 111 513 600 | 37 171 200 | 4 |
| 6·7222 | 139 392 000 | 46 464 000 | 5 |
| 8·0666 | 167 270 400 | 55 756 800 | 6 |
| 9·4100 | 195 148 800 | 65 049 600 | 7 |
| 10·7555 | 223 027 200 | 74 342 000 | 8 |
| 12·0999 | 250 905 600 | 83 635 200 | 9 |
| 13·4444 | 278 784 000 | 92 928 000 | 10 |

NOTE.—See explanatory examples following Table III.

PART I (continued).—Capacity of reservoirs and supply from catchment.

FOR A SIX MONTHS' SUPPLY.

| Supply afforded during 180 days or six months | Contents of reservoir to hold that supply | Surface of that reservoir if 34 feet deep on an average | Catchment necessary; with 1 foot available rainfall in 180 days |
|---|---|---|---|
| Cubic feet per second | Cubic feet | Square feet | Square miles |
| 1 | 15 552 000 | 5 184 000 | 0·55785 |
| 2 | 31 104 000 | 10 368 000 | 1·11570 |
| 3 | 46 656 000 | 15 552 000 | 1·67355 |
| 4 | 62 208 000 | 20 736 000 | 2·23140 |
| 5 | 77 760 000 | 25 920 000 | 2·78926 |
| 6 | 93 312 000 | 31 104 000 | 3·34711 |
| 7 | 108 864 000 | 36 288 000 | 3·90496 |
| 8 | 124 416 000 | 41 472 000 | 4·46281 |
| 9 | 139 968 000 | 46 656 000 | 5·02066 |
| 10 | 155 520 000 | 51 840 400 | 5·57851 |
| 1·7926 | 27 878 400 | 9 292 800 | 1 |
| 3·5852 | 55 756 800 | 18 585 600 | 2 |
| 5·3778 | 83 635 200 | 27 878 400 | 3 |
| 7·1704 | 111 513 600 | 37 171 200 | 4 |
| 8·9630 | 139 392 000 | 46 464 000 | 5 |
| 10·7556 | 167 270 400 | 55 756 800 | 6 |
| 12·5482 | 195 148 800 | 65 049 600 | 7 |
| 14·3407 | 223 027 200 | 74 342 000 | 8 |
| 16·1333 | 250 905 600 | 83 635 200 | 9 |
| 17·9259 | 278 784 000 | 92 928 000 | 10 |

PART I (continued).—Capacity of reservoirs and supply from catchment.

FOR A FOUR MONTHS' SUPPLY.

| Supply afforded during 120 days or four months | Contents of reservoir to hold that supply | Surface of that reservoir, if 3 feet deep on the average | Catchment necessary; with 1 foot available rainfall in 240 days |
|--|---|--|---|
| Cubic feet per second | Cubic feet | Square feet | Square miles |
| 1 | 10 368 000 | 3 456 000 | 0·3719 |
| 2 | 20 736 000 | 6 912 000 | 0·7438 |
| 3 | 31 104 000 | 10 368 000 | 1·1157 |
| 4 | 41 472 000 | 13 824 000 | 1·4876 |
| 5 | 51 840 000 | 17 280 000 | 1·8595 |
| 6 | 62 208 000 | 20 736 000 | 2·2314 |
| 7 | 72 576 000 | 24 192 000 | 2·6033 |
| 8 | 82 944 000 | 27 648 000 | 2·9752 |
| 9 | 93 312 000 | 31 104 000 | 3·3471 |
| 10 | 103 680 000 | 34 560 000 | 3·7190 |
| 2·6889 | 27 878 400 | 9 292 800 | 1 |
| 5·3777 | 55 756 800 | 18 585 600 | 2 |
| 8·0666 | 83 635 200 | 27 878 400 | 3 |
| 10·7555 | 111 513 600 | 37 171 200 | 4 |
| 13·4444 | 139 392 000 | 46 464 000 | 5 |
| 16·2000 | 167 270 400 | 55 756 800 | 6 |
| 18·8200 | 195 148 800 | 65 049 600 | 7 |
| 21·5111 | 223 027 200 | 74 342 000 | 8 |
| 24·1999 | 250 905 600 | 83 635 200 | 9 |
| 26·8889 | 278 784 000 | 92 928 000 | 10 |

PART 2.—Utilisation of a continuous supply of water.

| Cub. feet per second | At 5 gallons per head daily | At 7½ gallons per head daily | At 10 gallons per head daily | At 15 gallons per head daily | At 20 gallons per head daily | At 25 gallons per head daily | At 30 gallons per head daily |
|----------------------|-----------------------------|------------------------------|------------------------------|------------------------------|------------------------------|------------------------------|------------------------------|
| | Population supplied | | | | | | |
| 1 | 107732 | 71820 | 53866 | 35910 | 26933 | 21546 | 17955 |
| 2 | 215464 | 143640 | 107732 | 71820 | 53866 | 43093 | 35910 |
| 3 | 323196 | 215410 | 161598 | 107730 | 80799 | 64639 | 53865 |
| 4 | 430928 | 287280 | 215464 | 143640 | 107732 | 86186 | 71820 |
| 5 | 538660 | 359100 | 269330 | 179550 | 134665 | 107932 | 89775 |
| 6 | 646392 | 430920 | 323196 | 215460 | 161598 | 129278 | 107730 |
| 7 | 754124 | 474740 | 377062 | 237370 | 188531 | 150825 | 118685 |
| 8 | 861856 | 574560 | 430928 | 287280 | 215464 | 172371 | 143640 |
| 9 | 969588 | 646380 | 484794 | 323190 | 242397 | 193917 | 161595 |
| 10 | 1077320 | 718200 | 538660 | 359100 | 269330 | 215464 | 179550 |

| Cub. feet per second | At 1 cub. foot per head daily | At 1½ cub. feet per head daily | At 2 cub. feet per head daily | At 2½ cub. feet per head daily | At 3 cub. feet per head daily | At 4 cub. feet per head daily | At 5 cub. feet per head daily |
|----------------------|-------------------------------|--------------------------------|-------------------------------|--------------------------------|-------------------------------|-------------------------------|-------------------------------|
| | Population supplied | | | | | | |
| 1 | 86400 | 57600 | 43200 | 34560 | 28800 | 21600 | 17280 |
| 2 | 172800 | 115200 | 86400 | 69120 | 57600 | 43200 | 34560 |
| 3 | 259200 | 172800 | 129600 | 103680 | 86400 | 64800 | 51840 |
| 4 | 345600 | 230400 | 172800 | 138240 | 115200 | 86400 | 69120 |
| 5 | 432000 | 288000 | 216000 | 172800 | 144000 | 108000 | 86400 |
| 6 | 518400 | 345600 | 259200 | 207360 | 172800 | 125600 | 103680 |
| 7 | 604800 | 403200 | 302400 | 241920 | 201600 | 151200 | 120960 |
| 8 | 691200 | 460800 | 345600 | 276480 | 230400 | 172800 | 138240 |
| 9 | 777600 | 518400 | 388800 | 311040 | 259200 | 194400 | 155520 |
| 10 | 864000 | 576000 | 432000 | 345600 | 288000 | 216000 | 172800 |

NOTE.—See explanatory examples following Table III.

PART 2.—(continued).

| Cub. feet per second | At 50 acres per cub. foot per sec. | At 75 acres per cub. foot per sec. | At 100 acres per cub. foot per sec. | At 150 acres per cub. foot per sec. | At 200 acres per cub. foot per sec. | At 250 acres per cub. foot per sec. | At 300 acres per cub. foot per sec. |
|---------------------------|------------------------------------|------------------------------------|-------------------------------------|-------------------------------------|-------------------------------------|-------------------------------------|-------------------------------------|
| Number of acres irrigated | | | | | | | |
| 1 | 50 | 75 | 100 | 150 | 200 | 250 | 300 |
| 2 | 100 | 150 | 200 | 300 | 400 | 500 | 600 |
| 3 | 150 | 225 | 300 | 450 | 600 | 750 | 900 |
| 4 | 200 | 300 | 400 | 600 | 800 | 1000 | 1200 |
| 5 | 250 | 375 | 500 | 750 | 1000 | 1250 | 1500 |
| 6 | 300 | 450 | 600 | 900 | 1200 | 1500 | 1800 |
| 7 | 350 | 525 | 700 | 1050 | 1400 | 1750 | 2100 |
| 8 | 400 | 600 | 800 | 1200 | 1600 | 2000 | 2400 |
| 9 | 450 | 675 | 900 | 1350 | 1800 | 2250 | 2700 |
| 10 | 500 | 750 | 1000 | 1500 | 2000 | 2500 | 3000 |

| Cub. feet per second | At 200 sq. chains per cub. foot per second | At 300 sq. chains per cub. foot per second | At 400 sq. chains per cub. foot per second | At 600 sq. chains per cub. foot per second | At 800 sq. chains per cub. foot per second | At 1000 sq. chains per cub. foot per second | At 1200 sq. chains per cub. foot per second |
|---|--|--|--|--|--|---|---|
| Number of square chains (Ramsden) irrigated | | | | | | | |
| 1 | 200 | 300 | 400 | 600 | 800 | 1000 | 1200 |
| 2 | 400 | 600 | 800 | 1200 | 1600 | 2000 | 2400 |
| 3 | 600 | 900 | 1200 | 1800 | 2400 | 3000 | 3600 |
| 4 | 800 | 1200 | 1600 | 2400 | 3200 | 4000 | 4800 |
| 5 | 1000 | 1500 | 2000 | 3000 | 4000 | 5000 | 6000 |
| 6 | 1200 | 1800 | 2400 | 3600 | 4800 | 6000 | 7200 |
| 7 | 1400 | 2100 | 2800 | 4200 | 5600 | 7000 | 8400 |
| 8 | 1600 | 2400 | 3200 | 4800 | 6400 | 8000 | 9600 |
| 9 | 1800 | 2700 | 3600 | 5400 | 7200 | 9000 | 10800 |
| 10 | 2000 | 3000 | 4000 | 6000 | 8000 | 10000 | 12000 |

PART 3.—*Equivalent of continuous supply.*

Continuous supply in cubic feet per second into total quantities and vice versâ.

| Total quantity in cubic feet | Continuous supply in cubic feet per second. | | | | | |
|---------------------------------|---|-----------------|-----------------|-----------------|-----------------|------------------|
| | For 2 months | For 3 months | For 6 months | For 8 months | For 9 months | For 12 months |
| 315 360 | ·06 | ·04 | ·02 | ·015 | ·013 | ·01 |
| 630 720 | ·12 | ·08 | ·04 | ·030 | ·027 | ·02 |
| 946 080 | ·18 | ·12 | ·06 | ·045 | ·040 | ·03 |
| 1 261 440 | ·24 | ·16 | ·08 | ·060 | ·053 | ·04 |
| 1 576 800 | ·30 | ·20 | ·10 | ·075 | ·067 | ·05 |
| 1 892 160 | ·36 | ·24 | ·12 | ·090 | ·080 | ·06 |
| 2 207 520 | ·42 | ·28 | ·14 | ·105 | ·093 | ·07 |
| 2 522 880 | ·48 | ·32 | ·16 | ·120 | ·107 | ·08 |
| 2 838 240 | ·54 | ·36 | ·18 | ·135 | ·120 | ·09 |
| 1 million | ·1903 | ·1268 | ·0634 | ·0476 | ·0423 | ·031710 |
| 2 millions | ·3805 | ·2537 | ·1268 | ·0851 | ·0846 | ·063420 |
| 3 „ | ·5708 | ·3805 | ·1903 | ·1427 | ·1268 | ·095129 |
| 4 „ | ·7610 | ·5074 | ·2537 | ·1902 | ·1691 | ·126839 |
| 5 „ | ·9513 | ·6342 | ·3171 | ·2378 | ·2114 | ·158549 |
| 6 „ | 1·1416 | ·7610 | ·3805 | ·2854 | ·2537 | ·190259 |
| 7 „ | 1·3318 | ·8879 | ·4439 | ·3119 | ·2960 | ·221969 |
| 8 „ | 1·5221 | 1·0147 | ·5074 | ·3405 | ·3382 | ·253678 |
| 9 „ | 1·7123 | 1·1416 | ·5708 | ·4280 | ·3805 | ·285388 |
| 10 „ | 1·9026 | 1·2684 | ·6342 | ·4756 | ·4228 | ·317098 |

PART 3 (continued).—Equivalent of continuous supply.

Continuous supply in cubic feet per second throughout a month of 30 days that is equivalent to a certain number of waterings in a month.

| Amounts given at each watering to one acre | At 30 waterings per month | At 15 waterings per month | At 10 waterings per month | At 4 waterings per month | At 2 waterings per month | At 1 watering per month |
|--|---|---------------------------|---------------------------|--------------------------|--------------------------|-------------------------|
| Cubic feet | Monthly supply in cubic feet per second | | | | | |
| 10000 | ·1157 | ·0579 | ·0386 | ·0154 | ·0077 | ·0039 |
| 9000 | ·1041 | ·0520 | ·0347 | ·0139 | ·0069 | ·0035 |
| 8000 | ·0926 | ·0463 | ·0309 | ·0123 | ·0062 | ·0031 |
| 7000 | ·0810 | ·0405 | ·0271 | ·0108 | ·0054 | ·0027 |
| 6000 | ·0694 | ·0347 | ·0231 | ·0092 | ·0046 | ·0023 |
| 5000 | ·0579 | ·0289 | ·0193 | ·0077 | ·0039 | ·0019 |
| 4000 | ·0463 | ·0231 | ·0154 | ·0062 | ·0031 | ·0015 |
| 3000 | ·0347 | ·0173 | ·0116 | ·0046 | ·0023 | ·0011 |
| 2000 | ·0231 | ·0116 | ·0077 | ·0031 | ·0015 | ·0008 |
| 1000 | ·0116 | ·0058 | ·0039 | ·0015 | ·0008 | ·0004 |
| 8640 | ·1 | ·050 | ·0333 | ·0133 | ·0066 | ·0033 |
| 7776 | ·09 | ·045 | ·0300 | ·0120 | ·0060 | ·0030 |
| 6912 | ·08 | ·040 | ·0267 | ·0107 | ·0054 | ·0027 |
| 6048 | ·07 | ·035 | ·0233 | ·0093 | ·0046 | ·0023 |
| 5184 | ·06 | ·030 | ·0200 | ·0080 | ·0040 | ·0020 |
| 4320 | ·05 | ·025 | ·0167 | ·0067 | ·0032 | ·0016 |
| 3456 | ·04 | ·020 | ·0133 | ·0053 | ·0026 | ·0013 |
| 2592 | ·03 | ·015 | ·0100 | ·0040 | ·0020 | ·0010 |
| 1728 | ·02 | ·010 | ·0067 | ·0027 | ·0014 | ·0007 |
| 864 | ·01 | ·005 | ·0033 | ·0013 | ·0007 | ·0003 |

EXAMPLE I.

A supply of 18·234 cubic feet per second is wanted during eight months of the year from a reservoir which is to be supplied by a catchment area yielding an available rainfall of 1·32 feet during the remaining four months; required the contents of the reservoir, and the size of the catchment area.

Obtain from the Table the quantities due to 1 foot of rainfall,

| Supply, cubic feet per second. | Contents of reservoir cubic feet. | Catchment area, square miles. |
|--------------------------------|-----------------------------------|-------------------------------|
| 10 | 207 360 000 | 7·4380 |
| 8 | 165 888 000 | 5·9504 |
| ·2 | 4 147 200 | ·1488 |
| ·03 | 622 080 | ·0223 |
| ·004 | 82 944 | ·0030 |
| <hr/> 18·234 | <hr/> 378 100 224 | <hr/> 13·5625 |

Catchment area for 1·32 feet of fall = $\frac{13\cdot5625}{1\cdot32} = 10\cdot274$ sq. miles.

EXAMPLE II.

A catchment area of 21·963 square miles, having an available rainfall of 1·32 feet in four months of rainy season, supplies a reservoir which is to hold water for eight months' supply; what should be the full contents of the reservoir, and the supply in cubic feet per second during the eight months?

The proportionate catchment area for an available rainfall of one foot will = $21\cdot963 \times 1\cdot32 = 29\cdot001$ square miles.

| Catchment area | Contents of reservoir cubic feet | Supply, cub. ft. per second |
|----------------|----------------------------------|-----------------------------|
| 20 | 557 568 000 | 26·888 |
| 9 | 250 905 600 | 12·0999 |
| ·001 | 27 878 | ·0013 |
| <hr/> 29·001 | <hr/> 808 501 478 | <hr/> 38·9892 |

EXAMPLE III.

A combined irrigation and water-work scheme yields 18·234 cubic feet per second; what amount of land and of population could it supply, at the rates of 150 acres per cubic foot per second, and of $7\frac{1}{2}$ gallons per head per diem, if one-fourth is to be used for the water-works?

The supply available for irrigation will be $= 18\cdot234 - 4\cdot558 = 13\cdot676$ cubic feet per second; and from Table III., Part 2, we obtain the required results, thus—

| | | | |
|------------------------|----------------|------------------------|--------------|
| Cubic feet per second. | Population. | Cubic feet per second. | Acres. |
| 4 | 287 280 | 10 | 1500 |
| ·5 | 35 910 | 3 | 450 |
| ·05 | 0 591 | ·6 | 90 |
| ·008 | 574 | ·07 | 10·5 |
| <u>4·558</u> | <u>327 355</u> | ·006 | ·9 |
| | | <u>13·676</u> | <u>2051·</u> |

EXAMPLE IV.

A town has a population of 40 000, requiring water supply at 3 cubic feet per head daily, and has suburbs to the extent of 1 400 acres requiring irrigation at 150 acres per cubic foot per second of supply:—what catchment area will be necessary to provide this, if the annual rainfall is 60 inches, out of which a half can be utilised?

According to Table III., Part 2, the supply necessary will be

| | | | | |
|-----------------|-------------------|-----------------|-------------------|---------------------------------|
| For population. | | For irrigation. | | Total cubic feet per second. |
| Persons. | Cub. ft. per sec. | Acres. | Cub. ft. per sec. | |
| 28 800 | 1 | 1 350 | 9 | 10·721 |
| 8 640 | 0·3 | 50 | 0·333 | |
| 2 304 | 0·08 | | | |
| 230 | 0·008 | | | |
| 28 | 0·0001 | | | |
| <u>40 000</u> | <u>1·3881</u> | <u>1 400</u> | <u>9·333</u> | |

According to Part 2, Table II., 30 inches of effective annual rainfall is equivalent to a supply of 2·2083 cubic feet per second from one square mile, hence the minimum catchment area necessary will $= \frac{10\cdot721}{2\cdot208} = 4\cdot86$ square miles.

TABLE IV.—FLOOD DISCHARGE.

Part 1. Table of flood discharges in cubic feet per second, due to catchment areas in square miles, and corresponding to a coefficient $k=1$ in the formula—

$$Q = k \times 100 (K)^{\frac{3}{4}}.$$

Part 2. Flood discharges in cubic feet per second due to catchment areas, with values of k from 1 to 20

Part 3. Flood waterway for bridge-openings under coefficients $k=8.25$; and $k=12$.

PART I.—Table of flood discharges in cubic feet per second, due coefficient $k=1$ in the formula $Q=k \times 100 (K)^{\frac{3}{4}}$.

| Catchment area | Flood discharge | Catchment area | Flood discharge | Catchment area | Flood discharge | Catchment area | Flood discharge |
|----------------|-----------------|----------------|-----------------|----------------|-----------------|----------------|-----------------|
| .01 | 3 | 11 | 604 | 41 | 1620 | 71 | 2446 |
| .02 | 5 | 12 | 645 | 42 | 1650 | 72 | 2472 |
| .03 | 7 | 13 | 685 | 43 | 1679 | 73 | 2498 |
| .04 | 9 | 14 | 724 | 44 | 1708 | 74 | 2523 |
| .05 | 11 | 15 | 762 | 45 | 1737 | 75 | 2549 |
| .06 | 12 | 16 | 800 | 46 | 1766 | 76 | 2574 |
| .07 | 14 | 17 | 837 | 47 | 1795 | 77 | 2599 |
| .08 | 15 | 18 | 874 | 48 | 1824 | 78 | 2625 |
| .09 | 16 | 19 | 910 | 49 | 1852 | 79 | 2650 |
| | | 20 | 946 | 50 | 1880 | 80 | 2675 |
| .1 | 18 | 21 | 981 | 51 | 1908 | 81 | 2700 |
| .2 | 30 | 22 | 1016 | 52 | 1936 | 82 | 2725 |
| .3 | 41 | 23 | 1050 | 53 | 1964 | 83 | 2750 |
| .4 | 50 | 24 | 1084 | 54 | 1992 | 84 | 2775 |
| .5 | 59 | 25 | 1118 | 55 | 2020 | 85 | 2799 |
| .6 | 68 | 26 | 1151 | 56 | 2047 | 86 | 2824 |
| .7 | 76 | 27 | 1184 | 57 | 2074 | 87 | 2849 |
| .8 | 85 | 28 | 1217 | 58 | 2102 | 88 | 2873 |
| .9 | 92 | 29 | 1250 | 59 | 2129 | 89 | 2898 |
| | | 30 | 1282 | 60 | 2155 | 90 | 2922 |
| 1 | 100 | 31 | 1314 | 61 | 2183 | 91 | 2946 |
| 2 | 168 | 32 | 1345 | 62 | 2210 | 92 | 2971 |
| 3 | 238 | 33 | 1377 | 63 | 2236 | 93 | 2995 |
| 4 | 283 | 34 | 1408 | 64 | 2263 | 94 | 3019 |
| 5 | 334 | 35 | 1439 | 65 | 2289 | 95 | 3043 |
| 6 | 383 | 36 | 1470 | 66 | 2316 | 96 | 3067 |
| 7 | 430 | 37 | 1500 | 67 | 2342 | 97 | 3091 |
| 8 | 476 | 38 | 1531 | 68 | 2368 | 98 | 3115 |
| 9 | 520 | 39 | 1561 | 69 | 2394 | 99 | 3139 |
| 10 | 562 | 40 | 1590 | 70 | 2420 | 100 | 3162 |

to catchment areas in square miles, and corresponding to a
For local values of coefficients, see Part 2, Table XII.

| Catchment area | Flood discharge | Catchment area | Flood discharge | Catchment area | Flood discharge | Catchment area | Flood discharge |
|----------------|-----------------|----------------|-----------------|----------------|-----------------|----------------|-----------------|
| 110 | 3397 | 410 | 9112 | 710 | 13 754 | 1250 | 21 022 |
| 120 | 3625 | 420 | 9278 | 720 | 13 900 | 1500 | 24 103 |
| 130 | 3850 | 430 | 9443 | 730 | 14 044 | 1750 | 27 057 |
| 140 | 4070 | 440 | 9607 | 740 | 14 188 | 2000 | 29 907 |
| 150 | 4286 | 450 | 9770 | 750 | 14 332 | 2500 | 35 355 |
| 160 | 4499 | 460 | 9933 | 760 | 14 475 | 3000 | 40 536 |
| 170 | 4708 | 470 | 10 094 | 770 | 14 617 | 3500 | 45 504 |
| 180 | 4914 | 480 | 10 255 | 780 | 14 760 | 4000 | 50 297 |
| 190 | 5117 | 490 | 10 415 | 790 | 14 901 | 4500 | 54 943 |
| 200 | 5318 | 500 | 10 574 | 800 | 15 042 | 5000 | 59 460 |
| 210 | 5517 | 510 | 10 732 | 810 | 15 183 | 5500 | 63 867 |
| 220 | 5712 | 520 | 10 890 | 820 | 15 324 | 6000 | 68 173 |
| 230 | 5906 | 530 | 11 046 | 830 | 15 463 | 6500 | 72 391 |
| 240 | 6098 | 540 | 11 202 | 840 | 15 603 | 7000 | 76 529 |
| 250 | 6287 | 550 | 11 357 | 850 | 15 742 | 7500 | 80 593 |
| 260 | 6475 | 560 | 11 512 | 860 | 15 881 | 8000 | 84 590 |
| 270 | 6661 | 570 | 11 666 | 870 | 16 019 | 8500 | 88 525 |
| 280 | 6845 | 580 | 11 819 | 880 | 16 157 | 9000 | 92 402 |
| 290 | 7027 | 590 | 11 971 | 890 | 16 295 | 9500 | 96 448 |
| 300 | 7208 | 600 | 12 123 | 900 | 16 432 | 10 000 | 100 000 |
| 310 | 7388 | 610 | 12 204 | 910 | 16 568 | | |
| 320 | 7566 | 620 | 12 425 | 920 | 16 705 | 20 000 | 168 179 |
| 330 | 7743 | 630 | 12 575 | 930 | 16 841 | 30 000 | 238 285 |
| 340 | 7918 | 640 | 12 724 | 940 | 16 976 | 40 000 | 282 355 |
| 350 | 8092 | 650 | 12 873 | 950 | 17 112 | 50 000 | 334 370 |
| 360 | 8265 | 660 | 13 021 | 960 | 17 246 | 60 000 | 383 366 |
| 370 | 8436 | 670 | 13 169 | 970 | 17 381 | 70 000 | 430 352 |
| 380 | 8607 | 680 | 13 316 | 980 | 17 511 | 80 000 | 475 683 |
| 390 | 8776 | 690 | 13 463 | 990 | 17 649 | 90 000 | 519 615 |
| 400 | 8944 | 700 | 13 609 | 1000 | 17 783 | 100 000 | 562 341 |

PART 2.—*Flood discharges in cubic feet per second due to*

| Catchment in square miles | $k=2$ | $k=3$ | $k=4$ | $k=5$ | $k=6$ |
|---------------------------|-------|-------|--------|--------|--------|
| 0.05 | 22 | 33 | 44 | 55 | 66 |
| 0.1 | 36 | 54 | 72 | 90 | 108 |
| 0.2 | 60 | 90 | 120 | 150 | 180 |
| 0.3 | 82 | 123 | 164 | 205 | 246 |
| 0.4 | 100 | 150 | 200 | 250 | 300 |
| 0.5 | 118 | 177 | 236 | 295 | 354 |
| 0.6 | 136 | 204 | 272 | 340 | 408 |
| 0.7 | 152 | 228 | 304 | 380 | 456 |
| 0.8 | 170 | 255 | 340 | 425 | 510 |
| 0.9 | 184 | 276 | 368 | 460 | 552 |
| 1 | 200 | 300 | 400 | 500 | 600 |
| 2 | 336 | 504 | 672 | 840 | 1 008 |
| 3 | 476 | 714 | 952 | 1 190 | 1 428 |
| 4 | 566 | 849 | 1 132 | 1 415 | 1 698 |
| 5 | 668 | 1002 | 1 336 | 1 670 | 2 004 |
| 6 | 766 | 1149 | 1 532 | 1 915 | 2 298 |
| 7 | 860 | 1290 | 1 720 | 2 150 | 2 580 |
| 8 | 952 | 1428 | 1 904 | 2 380 | 2 856 |
| 9 | 1040 | 1560 | 2 080 | 2 600 | 3 120 |
| 10 | 1124 | 1686 | 2 248 | 2 810 | 3 372 |
| 20 | 1802 | 2838 | 3 784 | 4 730 | 5 676 |
| 30 | 2564 | 3846 | 5 128 | 6 410 | 7 692 |
| 40 | 3180 | 4770 | 6 360 | 7 950 | 9 540 |
| 50 | 3760 | 5640 | 7 520 | 9 400 | 11 280 |
| 60 | 4310 | 6465 | 8 620 | 10 775 | 12 930 |
| 70 | 4840 | 7260 | 9 680 | 12 100 | 14 520 |
| 80 | 5350 | 8025 | 10 700 | 13 375 | 16 050 |
| 90 | 5844 | 8766 | 11 688 | 14 610 | 17 532 |
| 100 | 6324 | 9486 | 12 648 | 15 810 | 18 972 |

catchment areas, with other values of the coefficient k.

| Catchment in square miles | $k=2$ | $k=3$ | $k=4$ | $k=5$ |
|---------------------------------|-----------|-----------|-----------|-----------|
| 100 | 6 324 | 9 486 | 12 648 | 15 810 |
| 200 | 10 636 | 15 954 | 21 272 | 26 590 |
| 300 | 14 416 | 21 624 | 28 832 | 36 040 |
| 400 | 17 888 | 26 832 | 35 776 | 44 720 |
| 500 | 21 148 | 31 722 | 42 296 | 52 870 |
| 600 | 24 246 | 36 369 | 48 492 | 60 615 |
| 700 | 27 218 | 40 827 | 54 436 | 68 045 |
| 800 | 30 084 | 45 126 | 60 168 | 75 210 |
| 900 | 32 864 | 49 296 | 65 728 | 82 160 |
| 1 000 | 35 566 | 53 349 | 71 132 | 88 915 |
| 2 000 | 59 814 | 89 721 | 119 628 | 149 535 |
| 3 000 | 81 072 | 121 608 | 162 144 | 202 680 |
| 4 000 | 100 594 | 150 891 | 201 188 | 251 485 |
| 5 000 | 118 920 | 178 380 | 237 840 | 297 300 |
| 6 000 | 136 346 | 204 519 | 272 692 | 340 865 |
| 7 000 | 153 058 | 229 587 | 316 116 | 382 645 |
| 8 000 | 169 180 | 253 770 | 338 360 | 422 950 |
| 9 000 | 184 804 | 277 206 | 369 608 | 462 010 |
| 10 000 | 200 000 | 300 000 | 400 000 | 500 000 |
| 20 000 | 336 358 | 504 537 | 672 716 | 840 895 |
| 30 000 | 476 570 | 714 855 | 953 140 | 1 191 425 |
| 40 000 | 564 710 | 847 065 | 1 129 420 | 1 411 775 |
| 50 000 | 668 740 | 1 003 110 | 1 337 480 | 1 671 850 |
| 60 000 | 766 732 | 1 150 098 | 1 533 464 | 1 916 830 |
| 70 000 | 860 704 | 1 291 056 | 1 721 408 | 2 151 760 |
| 80 000 | 951 366 | 1 427 049 | 1 902 732 | 2 378 415 |
| 90 000 | 1 039 230 | 1 558 845 | 2 078 460 | 2 598 075 |
| 100 000 | 1 124 682 | 1 687 023 | 2 249 364 | 2 811 705 |

PART 2 (cont.).—Flood discharges in cubic feet per second

| Catchment in square miles | $k=8$ | $k=10$ | $k=12$ | $k=16$ | $k=20$ |
|---------------------------|--------|--------|--------|--------|--------|
| 0.05 | 88 | 110 | 132 | 176 | 220 |
| 0.1 | 144 | 180 | 216 | 288 | 360 |
| 0.2 | 240 | 300 | 360 | 480 | 600 |
| 0.3 | 328 | 410 | 492 | 656 | 820 |
| 0.4 | 400 | 500 | 600 | 800 | 1 000 |
| 0.5 | 472 | 590 | 708 | 944 | 1 180 |
| 0.6 | 544 | 680 | 816 | 1 088 | 1 360 |
| 0.7 | 608 | 760 | 912 | 1 216 | 1 520 |
| 0.8 | 680 | 850 | 1 020 | 1 360 | 1 700 |
| 0.9 | 736 | 920 | 1 104 | 1 472 | 1 840 |
| 1 | 800 | 1 000 | 1 200 | 1 600 | 2 000 |
| 2 | 1 344 | 1 680 | 2 016 | 2 688 | 3 360 |
| 3 | 1 904 | 2 380 | 2 856 | 3 808 | 4 760 |
| 4 | 2 264 | 2 830 | 3 396 | 4 528 | 5 660 |
| 5 | 2 672 | 3 340 | 4 008 | 5 344 | 6 680 |
| 6 | 2 904 | 3 830 | 4 596 | 5 808 | 7 660 |
| 7 | 3 440 | 4 300 | 5 160 | 6 880 | 8 600 |
| 8 | 3 808 | 4 760 | 5 712 | 7 616 | 9 520 |
| 9 | 4 160 | 5 200 | 6 240 | 8 320 | 10 400 |
| 10 | 4 496 | 5 620 | 6 744 | 8 992 | 11 240 |
| 20 | 7 568 | 9 460 | 11 352 | 15 136 | 18 920 |
| 30 | 10 256 | 12 820 | 15 384 | 20 512 | 25 640 |
| 40 | 12 720 | 15 900 | 19 080 | 25 440 | 31 800 |
| 50 | 15 040 | 18 800 | 22 560 | 30 080 | 37 600 |
| 60 | 17 240 | 21 550 | 25 860 | 34 480 | 43 100 |
| 70 | 19 360 | 24 200 | 29 040 | 38 720 | 48 400 |
| 80 | 21 400 | 26 750 | 32 100 | 42 800 | 53 500 |
| 90 | 23 376 | 29 220 | 35 064 | 46 752 | 58 440 |
| 100 | 25 296 | 31 620 | 37 944 | 50 592 | 63 240 |

due to catchment areas with other values of the coefficient k .

| Catchment in square miles | $k=8$ | $k=12$ | $k=16$ | $k=20$ |
|---------------------------|-----------|-----------|-----------|------------|
| 100 | 25 296 | 37 944 | 50 592 | 63 240 |
| 200 | 42 544 | 63 816 | 85 088 | 106 360 |
| 300 | 57 664 | 86 496 | 115 328 | 144 160 |
| 400 | 71 552 | 107 328 | 143 104 | 178 880 |
| 500 | 84 592 | 126 888 | 169 184 | 211 480 |
| 600 | 96 984 | 145 476 | 193 968 | 242 460 |
| 700 | 108 872 | 163 308 | 217 754 | 272 180 |
| 800 | 120 336 | 180 504 | 240 672 | 300 840 |
| 900 | 131 456 | 197 184 | 262 212 | 328 640 |
| 1 000 | 142 264 | 213 396 | 284 528 | 355 660 |
| 2 000 | 239 256 | 358 884 | 478 512 | 598 140 |
| 3 000 | 324 288 | 486 432 | 648 576 | 810 720 |
| 4 000 | 402 376 | 603 564 | 804 752 | 1 005 940 |
| 5 000 | 475 680 | 713 520 | 951 360 | 1 189 200 |
| 6 000 | 545 384 | 818 076 | 1 090 768 | 1 363 460 |
| 7 000 | 632 232 | 918 348 | 1 264 464 | 1 530 580 |
| 8 000 | 676 720 | 1 015 080 | 1 353 480 | 1 691 800 |
| 9 000 | 739 216 | 1 108 824 | 1 478 432 | 1 848 040 |
| 10 000 | 800 000 | 1 200 000 | 1 600 000 | 2 000 000 |
| 20 000 | 1 345 432 | 2 018 148 | 2 690 864 | 3 363 580 |
| 30 000 | 1 906 280 | 2 859 420 | 3 812 560 | 4 765 700 |
| 40 000 | 2 258 840 | 3 388 260 | 4 517 680 | 5 647 100 |
| 50 000 | 2 674 960 | 4 012 440 | 5 349 920 | 6 687 400 |
| 60 000 | 3 066 928 | 4 600 392 | 6 133 856 | 7 667 320 |
| 70 000 | 3 442 816 | 5 164 224 | 6 885 632 | 8 607 040 |
| 80 000 | 3 805 464 | 5 708 196 | 7 610 928 | 9 513 660 |
| 90 000 | 4 156 920 | 6 235 380 | 8 313 840 | 10 392 300 |
| 100 000 | 4 498 728 | 6 748 092 | 8 977 456 | 11 246 820 |

PART 3.—*Flood waterway for bridge openings under a coefficient $k=8.25$.*

(By Colonel Dickens.)

| Catchment area | Flood discharge | Assumed velocity | Flood waterway | Number of square openings | Span | Height of pier |
|----------------|-----------------------|------------------|----------------|---------------------------|------|----------------|
| Square miles | Cubic feet per second | Feet per sec. | Square feet | Number | Feet | Feet |
| .0016 | 6.5 | 5 | 1.5 | 1 | 1½ | 1 |
| .0031 | 11 | 5 | 2.25 | 1 | 2 | 1½ |
| .0047 | 15 | 5 | 3 | 1 | 2 | 1½ |
| .0078 | 22 | 5 | 4.5 | 1 | 3 | 1½ |
| .0125 | 31 | 5 | 6 | 1 | 3 | 2 |
| .0250 | 52 | 5 | 10.5 | 1 | 4 | 2½ |
| .0625 | 103 | 6 | 18 | 1 | 6 | 3 |
| .1250 | 173 | 6 | 29 | 1 | 7 | 4 |
| .2500 | 292 | 6 | 49 | 1 | 10 | 5 |
| .5000 | 490 | 6 | 81 | 1 | 12 | 7 |
| 1 | 5 | 7 | 137 | 2 | 12 | 6 |
| 2 | 1 388 | 7 | 200 | 3 | 12 | 6 |
| 3 | 1 881 | 7 | 270 | 3 | 14 | 7 |
| 5 | 2 760 | 7 | 400 | 3 | 16 | 8 |
| 7 | 3 550 | 7 | 507 | 3 | 18 | 9 |
| 10 | 4 640 | 7 | 663 | 3 | 20 | 11 |
| 20 | 7 804 | 8 | 975 | 5 | 20 | 10 |
| 30 | 10 577 | 8 | 1 322 | 5 | 24 | 11 |
| 50 | 15 605 | 9 | 1 734 | 5 | 30 | 11½ |
| 100 | 26 094 | 9 | 2 899 | 5 | 40 | 14½ |
| 200 | 43 884 | 10 | 4 388 | 7 | 40 | 15½ |
| 300 | 59 481 | 10 | 5 948 | 9 | 40 | 16½ |
| 500 | 87 255 | 10 | 8 725 | 9 | 50 | 19 |
| 1 000 | 146 737 | 10 | 14 673 | 15 | 50 | 19 |
| 2 000 | 246 780 | 11 | 22 434 | 15 | 60 | 24 |
| 3 000 | 334 487 | 11 | 30 408 | 20 | 60 | 25 |
| 5 000 | 490 636 | 12 | 40 886 | 20 | 75 | 27 |
| 10 000 | 825 000 | 12 | 68 750 | 30 | 75 | 30 |
| 20 000 | 1 385 746 | 13 | 106 749 | 40 | 75 | 35 |
| 30 000 | 1 870 962 | 13 | 143 920 | 45 | 80 | 40 |
| 50 000 | 2 695 690 | 14 | 190 256 | 50 | 90 | 42 |
| 100 000 | 4 639 274 | 15 | 306 285 | 60 | 100 | 50 |

PART 3 (cont.).—Flood waterway for bridge-openings under a coefficient $k=12$.

(By the Author.)

| Catchment area | Flood discharge | Assumed velocity | Flood waterway | No. of sq. openings | Span | Height |
|----------------|--------------------|------------------|----------------|---------------------|------|--------|
| Square miles | Cub. feet per sec. | Feet per sec. | Square feet | No. | Feet | Feet |
| .0016 | 9.6 | 5 | 2 | 1 | 2 | 1 |
| .0031 | 15.8 | 5 | 3 | 1 | 3 | 1 |
| .0047 | 21.5 | 5 | 4 | 1 | 3 | 1½ |
| .0078 | 31.5 | 5 | 6 | 1 | 3 | 2 |
| .0125 | 44.9 | 5 | 9 | 1 | 3 | 3 |
| .0250 | 75.4 | 5 | 15 | 1 | 4 | 4 |
| .0625 | 150 | 6 | 25 | 1 | 5 | 5 |
| .1250 | 252 | 6 | 42 | 1 | 9 | 5 |
| .2500 | 424 | 6 | 71 | 1 | 12 | 6 |
| .5000 | 708 | 6 | 118 | 2 | 10 | 6 |
| 1 | 1 200 | 7 | 172 | 3 | 10 | 6 |
| 2 | 2 016 | 7 | 288 | 3 | 12 | 8 |
| 3 | 2 856 | 7 | 408 | 3 | 16 | 9 |
| 5 | 4 008 | 7 | 573 | 5 | 14 | 9 |
| 7 | 5 160 | 7 | 738 | 5 | 15 | 10 |
| 10 | 6 744 | 7 | 964 | 5 | 19 | 10 |
| 20 | 11 352 | 8 | 1 694 | 5 | 28 | 12 |
| 30 | 15 384 | 8 | 1 924 | 5 | 30 | 13 |
| 50 | 22 560 | 9 | 2 508 | 5 | 40 | 13 |
| 100 | 37 944 | 9 | 4 216 | 7 | 40 | 15 |
| 200 | 63 816 | 10 | 6 382 | 9 | 40 | 18 |
| 300 | 86 496 | 10 | 8 650 | 9 | 50 | 20 |
| 500 | 126 588 | 10 | 12 660 | 11 | 60 | 20 |
| 1 000 | 213 396 | 10 | 21 340 | 15 | 60 | 25 |
| 2 000 | 358 884 | 11 | 32 626 | 17 | 80 | 25 |
| 3 000 | 486 432 | 11 | 44 222 | 23 | 80 | 25 |
| 5 000 | 713 520 | 12 | 59 460 | 20 | 100 | 30 |
| 10 000 | 1 200 000 | 12 | 100 000 | 25 | 100 | 40 |
| 20 000 | 2 018 148 | 13 | 155 244 | 26 | 150 | 40 |
| 30 000 | 2 859 420 | 13 | 219 956 | 28 | 200 | 40 |
| 50 000 | 4 012 440 | 14 | 286 604 | 29 | 250 | 40 |
| 100 000 | 6 748 092 | 15 | 449 874 | 45 | 250 | 40 |

TABLE V.—SECTIONAL DATA.

SECTIONAL AREAS (A) AND HYDRAULIC RADII (R),

- Part 1. For Rectangular Canal Sections
- Part 2. For Trapezoidal Canal Sections having*side-slopes of one to one.
- Part 3. Dimensions of Channel Sections of equal discharge.
- Part 4. Values of A and R for Cylindrical and Ovoidal Pipes and Culverts.

FOR USE IN THE GENERAL FORMULÆ,

$$Q = A \cdot c \cdot 100 \sqrt{RS},$$
$$V = c \cdot 100 \sqrt{RS}.$$

This Table may be used with any unit of measurement.

PART I.—Sectional Areas (A) and Hydraulic Radii (R),

Corresponding to various Bed-

| a | b=2 | | b=3 | | b=4 | | b=5 | |
|------|-----|-------|------|-------|-----|-------|-------|-------|
| | A | R | A | R | A | R | A | R |
| 0.5 | 1. | 0.333 | 1.5 | 0.375 | 2 | 0.4 | 2.5 | 0.416 |
| 0.75 | 1.5 | 0.429 | 2.25 | 0.5 | 3 | 0.545 | 3.75 | 0.577 |
| 1. | 2. | 0.5 | 3. | 0.6 | 4 | 0.666 | 5. | 0.714 |
| 1.25 | 2.5 | 0.555 | 3.75 | 0.682 | 5 | 0.769 | 6.25 | 0.833 |
| 1.5 | 3. | 0.600 | 4.5 | 0.750 | 6 | 0.857 | 7.5 | 0.937 |
| 1.75 | 3.5 | 0.636 | 5.25 | 0.808 | 7 | 0.933 | 8.75 | 1.029 |
| 2. | 4. | 0.666 | 6. | 0.857 | 8 | 1. | 10. | 1.111 |
| 2.25 | 4.5 | 0.692 | 6.75 | 0.9 | 9 | 1.058 | 11.25 | 1.184 |
| 2.5 | 5. | 0.714 | 7.5 | 0.937 | 10 | 1.111 | 12.5 | 1.250 |
| 2.75 | 5.5 | 0.733 | 8.25 | 0.971 | 11 | 1.158 | 13.75 | 1.309 |
| 3. | 6. | 0.750 | 9. | 1. | 12 | 1.200 | 15. | 1.364 |
| 3.5 | 7. | 0.777 | 10.5 | 1.050 | 14 | 1.273 | 17.50 | 1.439 |
| 4. | 8. | 0.800 | 12. | 1.091 | 16 | 1.333 | 20. | 1.538 |
| 5. | 10. | 0.833 | 15. | 1.154 | 20 | 1.428 | 25. | 1.666 |

| a | b=14 | | b=16 | | b=18 | | b=20 | |
|------|------|-------|------|-------|------|-------|------|-------|
| | A | R | A | R | A | R | A | R |
| 1. | 14. | 0.875 | 16 | 0.888 | 18 | 0.900 | 20 | 0.909 |
| 1.25 | 17.5 | 1.061 | 20 | 1.080 | 22.5 | 1.098 | 25 | 1.111 |
| 1.5 | 21. | 1.244 | 24 | 1.262 | 27 | 1.286 | 30 | 1.305 |
| 1.75 | 24.5 | 1.397 | 28 | 1.434 | 31.5 | 1.468 | 35 | 1.491 |
| 2. | 28. | 1.555 | 32 | 1.600 | 36 | 1.636 | 40 | 1.666 |
| 2.25 | 31.5 | 1.701 | 36 | 1.757 | 40.5 | 1.800 | 45 | 1.836 |
| 2.5 | 35. | 1.841 | 40 | 1.904 | 45 | 1.953 | 50 | 2. |
| 2.75 | 38.5 | 1.971 | 44 | 2.050 | 49.5 | 2.109 | 55 | 2.156 |
| 3. | 42. | 2.100 | 48 | 2.182 | 54 | 2.250 | 60 | 2.307 |
| 3.25 | 45.5 | 2.230 | 52 | 2.311 | 58.5 | 2.387 | 65 | 2.457 |
| 3.5 | 49. | 2.333 | 56 | 2.346 | 63 | 2.520 | 70 | 2.590 |
| 3.75 | 52.5 | 2.447 | 60 | 2.556 | 67.5 | 2.646 | 75 | 2.727 |
| 4. | 56. | 2.545 | 64 | 2.666 | 72 | 2.768 | 80 | 2.857 |
| 4.25 | 59.5 | 2.644 | 68 | 2.774 | 76.5 | 2.892 | 85 | 2.975 |
| 4.5 | 63. | 2.741 | 72 | 2.880 | 81 | 3. | 90 | 3.105 |
| 4.75 | 66.5 | 2.833 | 76 | 2.979 | 85.5 | 3.109 | 95 | 3.211 |
| 5. | 70. | 2.917 | 80 | 3.080 | 90 | 3.214 | 100 | 3.333 |
| 5.5 | 77. | 3.080 | 88 | 3.256 | 99 | 3.416 | 110 | 3.553 |
| 6. | 84. | 3.230 | 96 | 3.429 | 108 | 3.600 | 120 | 3.750 |
| 7. | 98. | 3.500 | 112 | 3.733 | 126 | 3.938 | 140 | 4.116 |

for Rectangular sections of Channels, Canals, and Aqueducts.

widths (*b*) and Depths of Water (*d*).

| <i>d</i> | <i>b</i> =6 | | <i>b</i> =8 | | <i>b</i> =10 | | <i>b</i> =12 | |
|----------|-------------|----------|-------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1.0 | 6. | 0.750 | 8 | 0.800 | 10. | 0.833 | 12 | 0.857 |
| 1.25 | 7.5 | 0.882 | 9 | 0.857 | 12.5 | 1. | 15 | 1.035 |
| 1.5 | 9. | 1. | 12 | 1.091 | 15. | 1.154 | 18 | 1.200 |
| 1.75 | 10.5 | 1.106 | 14 | 1.218 | 17.5 | 1.295 | 21 | 1.357 |
| 2. | 12. | 1.200 | 16 | 1.333 | 20. | 1.429 | 24 | 1.5 |
| 2.25 | 13.5 | 1.286 | 18 | 1.440 | 22.5 | 1.553 | 27 | 1.636 |
| 2.5 | 15. | 1.364 | 20 | 1.538 | 25. | 1.666 | 30 | 1.764 |
| 2.75 | 16.5 | 1.436 | 22 | 1.628 | 27.5 | 1.777 | 33 | 1.887 |
| 3. | 18. | 1.5 | 24 | 1.714 | 30. | 1.875 | 36 | 2. |
| 3.25 | 19.5 | 1.560 | 26 | 1.794 | 32.5 | 1.970 | 39 | 2.106 |
| 3.5 | 21. | 1.615 | 28 | 1.866 | 35. | 2.058 | 42 | 2.209 |
| 3.75 | 22.5 | 1.666 | 30 | 1.938 | 37.5 | 2.143 | 45 | 2.304 |
| 4. | 24. | 1.714 | 32 | 2. | 40. | 2.222 | 48 | 2.4 |
| 5. | 30. | 1.875 | 40 | 2.222 | 50. | 2.500 | 60 | 2.727 |

| <i>d</i> | <i>b</i> =25 | | <i>b</i> =30 | | <i>b</i> =35 | | <i>b</i> =40 | |
|----------|--------------|----------|--------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1. | 25. | 0.925 | 30 | 0.938 | 35. | 0.945 | 40 | 0.952 |
| 1.5 | 37.5 | 1.338 | 45 | 1.364 | 52.5 | 1.382 | 60 | 1.398 |
| 2. | 50. | 1.725 | 60 | 1.764 | 70. | 1.792 | 80 | 1.818 |
| 2.25 | 56.25 | 1.901 | 67.5 | 1.957 | 78.75 | 1.994 | 90 | 2.023 |
| 2.5 | 62.5 | 2.083 | 75. | 2.143 | 87.5 | 2.187 | 100 | 2.222 |
| 2.75 | 68.75 | 2.255 | 82.5 | 2.326 | 96.25 | 2.377 | 110 | 2.418 |
| 3. | 75. | 2.422 | 90 | 2.500 | 105. | 2.562 | 120 | 2.610 |
| 3.25 | 81.25 | 2.579 | 97.5 | 2.672 | 113.75 | 2.741 | 130 | 2.795 |
| 3.5 | 87.5 | 2.734 | 105 | 2.835 | 122.5 | 2.919 | 140 | 2.982 |
| 3.75 | 93.75 | 2.884 | 112.5 | 3. | 131.25 | 3.071 | 150 | 3.099 |
| 4. | 100. | 3.030 | 120 | 3.156 | 140. | 3.162 | 160 | 3.333 |
| 4.25 | 106.25 | 3.166 | 127.5 | 3.312 | 148.75 | 3.421 | 170 | 3.505 |
| 4.5 | 112.5 | 3.308 | 135 | 3.456 | 157.5 | 3.579 | 180 | 3.672 |
| 4.75 | 118.75 | 3.327 | 142.5 | 3.608 | 166.25 | 3.737 | 190 | 3.838 |
| 5. | 125. | 3.571 | 150 | 3.750 | 175. | 3.944 | 200 | 4. |
| 5.5 | 137.5 | 3.820 | 165 | 4.026 | 192.5 | 4.177 | 220 | 4.314 |
| 6. | 150. | 4.050 | 180 | 4.286 | 210. | 4.473 | 240 | 4.614 |
| 6.5 | 162.5 | 4.274 | 195 | 4.544 | 227.5 | 4.739 | 260 | 4.906 |
| 7. | 175. | 4.480 | 210 | 4.773 | 245. | 5. | 280 | 5.180 |
| 8. | 200. | 4.880 | 240 | 5.220 | 280. | 5.491 | 320 | 5.714 |

PART I (cont.).—Sectional Areas (A) and Hydraulic Radii (R),
Corresponding to Different Bed-

| d | b=50 | | b=60 | | b=70 | | b=80 | |
|------|-------|-------|------|-------|-------|-------|------|-------|
| | A | R | A | R | A | R | A | R |
| 1.0 | 50. | 0.962 | 60 | 0.968 | 70. | 0.972 | 80 | 0.976 |
| 2.0 | 100. | 1.852 | 120 | 1.875 | 140. | 1.892 | 160 | 1.905 |
| 2.25 | 112.5 | 2.063 | 135 | 2.093 | 157.5 | 2.114 | 180 | 2.130 |
| 2.5 | 125. | 2.273 | 150 | 2.308 | 175. | 2.333 | 200 | 2.353 |
| 2.75 | 137.5 | 2.477 | 165 | 2.519 | 192.5 | 2.549 | 220 | 2.573 |
| 3. | 150. | 2.679 | 180 | 2.727 | 210. | 2.764 | 240 | 2.791 |
| 3.25 | 162.5 | 2.876 | 195 | 2.932 | 227.5 | 2.975 | 260 | 3.006 |
| 3.5 | 175. | 3.069 | 210 | 3.134 | 245. | 3.182 | 280 | 3.217 |
| 3.75 | 187.5 | 3.261 | 225 | 3.333 | 262.5 | 3.387 | 300 | 3.427 |
| 4. | 200. | 3.448 | 240 | 3.529 | 280. | 3.590 | 320 | 3.636 |
| 4.25 | 212.5 | 3.632 | 255 | 3.722 | 297.5 | 3.790 | 340 | 3.842 |
| 4.5 | 225. | 3.814 | 270 | 3.913 | 315. | 3.988 | 360 | 4.046 |
| 4.75 | 237.5 | 3.991 | 285 | 4.101 | 332.5 | 4.182 | 380 | 4.245 |
| 5. | 250. | 4.167 | 300 | 4.286 | 350. | 4.375 | 400 | 4.444 |
| 5.25 | 262.5 | 4.339 | 315 | 4.468 | 367.5 | 4.565 | 420 | 4.641 |
| 5.5 | 275. | 4.507 | 330 | 4.646 | 385. | 4.753 | 440 | 4.836 |
| 5.75 | 287.5 | 4.675 | 345 | 4.825 | 402.5 | 4.939 | 460 | 5.027 |
| 6. | 300. | 4.839 | 360 | 5. | 420. | 5.122 | 480 | 5.217 |
| 6.25 | 312.5 | 5. | 375 | 5.172 | 437.5 | 5.302 | 500 | 5.405 |
| 6.5 | 325. | 5.158 | 390 | 5.343 | 455. | 5.483 | 520 | 5.590 |
| 6.75 | 337.5 | 5.315 | 405 | 5.510 | 472.5 | 5.659 | 540 | 5.775 |
| 7. | 350. | 5.470 | 420 | 5.676 | 490. | 5.833 | 560 | 5.958 |
| 7.25 | 362.5 | 5.620 | 435 | 5.839 | 507.5 | 6.006 | 580 | 6.138 |
| 7.5 | 375. | 5.767 | 450 | 6. | 525. | 6.176 | 600 | 6.315 |
| 7.75 | 387.5 | 5.916 | 465 | 6.158 | 542.5 | 6.345 | 620 | 6.493 |
| 8. | 400. | 6.060 | 480 | 6.316 | 560. | 6.513 | 640 | 6.666 |
| 8.25 | 412.5 | 6.203 | 495 | 6.471 | 577.5 | 6.676 | 660 | 6.839 |
| 8.5 | 425. | 6.345 | 510 | 6.624 | 595. | 6.842 | 680 | 7.004 |
| 8.75 | 437.5 | 6.481 | 525 | 6.775 | 612.5 | 7. | 700 | 7.179 |
| 9. | 450. | 6.619 | 540 | 6.923 | 630. | 7.160 | 720 | 7.344 |
| 9.25 | 462.5 | 6.752 | 555 | 7.010 | 647.5 | 7.317 | 740 | 7.512 |
| 9.5 | 475. | 6.883 | 570 | 7.216 | 665. | 7.475 | 760 | 7.676 |
| 9.75 | 487.5 | 7.014 | 585 | 7.358 | 682.5 | 7.626 | 780 | 7.839 |
| 10. | 500. | 7.145 | 600 | 7.500 | 700. | 7.777 | 800 | 8.000 |
| 11. | 550. | 9.014 | 660 | 8.048 | 770. | 8.370 | 880 | 8.624 |
| 12. | 600. | 9.678 | 720 | 8.571 | 840. | 8.938 | 960 | 9.230 |

for Rectangular Sections of Channels, Canals, and Aqueducts.

widths (b) and Depths of Water (d).

| d | $b=90$ | | $b=100$ | | $b=120$ | | $b=140$ | |
|------|--------|-------|---------|-------|---------|-------|---------|--------|
| | A | R | A | R | A | R | A | R |
| 1.0 | 90 | 0.978 | 100 | 0.980 | 120 | 0.984 | 140 | 0.986 |
| 2.0 | 180 | 1.915 | 200 | 1.923 | 240 | 1.936 | 280 | 1.944 |
| 2.25 | 202.5 | 2.143 | 225 | 2.153 | 270 | 2.169 | 315 | 2.180 |
| 2.5 | 225 | 2.369 | 250 | 2.381 | 300 | 2.400 | 350 | 2.414 |
| 2.75 | 247.5 | 2.592 | 275 | 2.606 | 330 | 2.629 | 385 | 2.646 |
| 3 | 270 | 2.813 | 300 | 2.830 | 360 | 2.857 | 420 | 2.877 |
| 3.25 | 292.5 | 3.031 | 325 | 3.052 | 390 | 3.083 | 455 | 3.106 |
| 3.5 | 315 | 3.245 | 350 | 3.271 | 420 | 3.307 | 490 | 3.333 |
| 3.75 | 337.5 | 3.461 | 375 | 3.488 | 450 | 3.529 | 525 | 3.560 |
| 4 | 360 | 3.672 | 400 | 3.704 | 480 | 3.750 | 560 | 3.784 |
| 4.25 | 382.5 | 3.883 | 425 | 3.917 | 510 | 3.969 | 595 | 4.007 |
| 4.5 | 405 | 4.091 | 450 | 4.128 | 540 | 4.186 | 630 | 4.228 |
| 4.75 | 427.5 | 4.296 | 475 | 4.338 | 570 | 4.402 | 665 | 4.448 |
| 5 | 450 | 4.500 | 500 | 4.545 | 600 | 4.615 | 700 | 4.667 |
| 5.25 | 472.5 | 4.701 | 525 | 4.751 | 630 | 4.828 | 735 | 4.883 |
| 5.5 | 495 | 4.900 | 550 | 4.955 | 660 | 5.038 | 770 | 5.100 |
| 5.75 | 517.5 | 5.098 | 575 | 5.157 | 700 | 5.247 | 805 | 5.313 |
| 6 | 540 | 5.292 | 600 | 5.357 | 720 | 5.455 | 840 | 5.527 |
| 6.25 | 562.5 | 5.488 | 625 | 5.555 | 750 | 5.659 | 875 | 5.738 |
| 6.5 | 585 | 5.679 | 650 | 5.752 | 780 | 5.865 | 910 | 5.948 |
| 6.75 | 607.5 | 5.870 | 675 | 5.947 | 810 | 6.068 | 945 | 6.156 |
| 7 | 630 | 6.057 | 700 | 6.140 | 840 | 6.269 | 980 | 6.364 |
| 7.25 | 652.5 | 6.244 | 725 | 6.332 | 870 | 6.468 | 1015 | 6.569 |
| 7.5 | 675 | 6.429 | 750 | 6.522 | 900 | 6.667 | 1050 | 6.775 |
| 7.75 | 697.5 | 6.611 | 775 | 6.720 | 930 | 6.863 | 1085 | 6.977 |
| 8 | 720 | 6.792 | 800 | 6.897 | 960 | 7.059 | 1120 | 7.179 |
| 8.25 | 742.5 | 6.972 | 825 | 7.082 | 990 | 7.253 | 1155 | 7.380 |
| 8.5 | 765 | 7.150 | 850 | 7.265 | 1020 | 7.445 | 1190 | 7.579 |
| 8.75 | 787.5 | 7.325 | 875 | 7.445 | 1050 | 7.637 | 1225 | 7.778 |
| 9 | 810 | 7.505 | 900 | 7.627 | 1080 | 7.826 | 1260 | 7.976 |
| 9.25 | 832.5 | 7.672 | 925 | 7.805 | 1110 | 8.015 | 1295 | 8.171 |
| 9.5 | 855 | 7.844 | 950 | 7.983 | 1140 | 8.201 | 1330 | 8.364 |
| 9.75 | 877.5 | 8.013 | 975 | 8.159 | 1170 | 8.387 | 1365 | 8.559 |
| 10 | 900 | 8.182 | 1000 | 8.333 | 1200 | 8.571 | 1400 | 8.750 |
| 11 | 990 | 8.839 | 1100 | 9.017 | 1320 | 9.295 | 1540 | 9.510 |
| 12 | 1080 | 9.472 | 1200 | 9.677 | 1440 | 10 | 1680 | 10.244 |

PART I (cont.).—Sectional Areas (A) and Hydraulic Radii (R),
Corresponding to Various Bed-

| d | $b=160$ | | $b=180$ | | $b=200$ | | $b=220$ | |
|------|---------|--------|---------|--------|---------|--------|---------|--------|
| | A | R | A | R | A | R | A | R |
| 1.0 | 160 | 0.987 | 180 | 0.989 | 200 | 0.990 | 220 | 0.991 |
| 2.0 | 320 | 1.951 | 360 | 1.956 | 400 | 1.961 | 440 | 1.965 |
| 2.25 | 360 | 2.188 | 405 | 2.195 | 450 | 2.201 | 495 | 2.205 |
| 2.5 | 400 | 2.424 | 450 | 2.432 | 500 | 2.439 | 550 | 2.444 |
| 2.75 | 440 | 2.658 | 495 | 2.668 | 550 | 2.676 | 605 | 2.683 |
| 3. | 480 | 2.891 | 540 | 2.903 | 600 | 2.913 | 660 | 2.921 |
| 3.25 | 520 | 3.123 | 585 | 3.137 | 650 | 3.148 | 715 | 3.157 |
| 3.5 | 560 | 3.353 | 630 | 3.369 | 700 | 3.382 | 770 | 3.392 |
| 3.75 | 600 | 3.582 | 675 | 3.599 | 750 | 3.614 | 825 | 3.627 |
| 4. | 640 | 3.809 | 720 | 3.830 | 800 | 3.846 | 880 | 3.860 |
| 4.25 | 680 | 4.036 | 765 | 4.059 | 850 | 4.077 | 935 | 4.092 |
| 4.5 | 720 | 4.260 | 810 | 4.285 | 900 | 4.306 | 990 | 4.323 |
| 4.75 | 760 | 4.484 | 855 | 4.513 | 950 | 4.534 | 1045 | 4.554 |
| 5. | 800 | 4.706 | 900 | 4.737 | 1000 | 4.762 | 1100 | 4.783 |
| 5.25 | 840 | 4.927 | 945 | 4.961 | 1050 | 4.989 | 1155 | 5.010 |
| 5.5 | 880 | 5.146 | 990 | 5.184 | 1100 | 5.213 | 1210 | 5.238 |
| 5.75 | 920 | 5.365 | 1035 | 5.405 | 1150 | 5.437 | 1265 | 5.465 |
| 6. | 960 | 5.581 | 1080 | 5.625 | 1200 | 5.660 | 1320 | 5.689 |
| 6.25 | 1000 | 5.797 | 1125 | 5.843 | 1250 | 5.883 | 1375 | 5.913 |
| 6.5 | 1040 | 6.011 | 1170 | 6.062 | 1300 | 6.104 | 1430 | 6.138 |
| 6.75 | 1080 | 6.225 | 1215 | 6.279 | 1350 | 6.323 | 1485 | 6.359 |
| 7. | 1120 | 6.437 | 1260 | 6.495 | 1400 | 6.542 | 1540 | 6.582 |
| 7.25 | 1160 | 6.648 | 1305 | 6.710 | 1450 | 6.760 | 1595 | 6.801 |
| 7.5 | 1200 | 6.857 | 1350 | 6.923 | 1500 | 6.977 | 1650 | 7.021 |
| 7.75 | 1240 | 7.057 | 1395 | 7.134 | 1550 | 7.192 | 1705 | 7.239 |
| 8. | 1280 | 7.273 | 1440 | 7.348 | 1600 | 7.408 | 1760 | 7.457 |
| 8.5 | 1360 | 7.684 | 1530 | 7.766 | 1700 | 7.834 | 1870 | 7.890 |
| 9. | 1440 | 8.090 | 1620 | 8.182 | 1800 | 8.257 | 1980 | 8.319 |
| 9.5 | 1520 | 8.492 | 1710 | 8.593 | 1900 | 8.675 | 2090 | 8.745 |
| 10. | 1600 | 8.888 | 1800 | 9. | 2000 | 9.091 | 2200 | 9.167 |
| 11. | 1760 | 9.167 | 1980 | 9.801 | 2200 | 9.910 | 2420 | 10. |
| 12. | 1920 | 10.435 | 2160 | 10.588 | 2400 | 10.715 | 2640 | 10.820 |
| 13. | 2080 | 11.183 | 2340 | 11.359 | 2600 | 11.505 | 2860 | 11.626 |
| 14. | 2240 | 11.915 | 2520 | 12.115 | 2800 | 12.280 | 3080 | 12.419 |
| 15. | 2400 | 12.632 | 2700 | 12.857 | 3000 | 13.043 | 3300 | 13.200 |
| 16. | 2560 | 13.333 | 2880 | 13.585 | 3200 | 13.793 | 3520 | 13.968 |

for Rectangular Sections of Channels, Canals, and Aqueducts.

widths (*b*) and Depths of Water (*d*).

| <i>d</i> | <i>b</i> =240 | | <i>b</i> =260 | | <i>b</i> =280 | | <i>b</i> =300 | |
|----------|---------------|----------|---------------|----------|---------------|----------|---------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 2.0 | 480 | 1.967 | 520 | 1.969 | 560 | 1.971 | 600 | 1.974 |
| 2.5 | 600 | 2.449 | 650 | 2.453 | 700 | 2.456 | 750 | 2.459 |
| 3. | 720 | 2.927 | 780 | 2.932 | 840 | 2.937 | 900 | 2.941 |
| 3.25 | 780 | 3.164 | 845 | 3.170 | 910 | 3.176 | 975 | 3.181 |
| 3.5 | 840 | 3.401 | 910 | 3.408 | 980 | 3.414 | 1050 | 3.420 |
| 3.75 | 900 | 3.636 | 975 | 3.645 | 1050 | 3.652 | 1125 | 3.659 |
| 4. | 960 | 3.871 | 1040 | 3.880 | 1120 | 3.889 | 1200 | 3.896 |
| 4.25 | 1020 | 4.104 | 1105 | 4.115 | 1190 | 4.125 | 1275 | 4.132 |
| 4.5 | 1080 | 4.337 | 1170 | 4.349 | 1260 | 4.360 | 1350 | 4.369 |
| 4.75 | 1140 | 4.569 | 1235 | 4.582 | 1330 | 4.594 | 1425 | 4.604 |
| 5. | 1200 | 4.800 | 1300 | 4.815 | 1400 | 4.827 | 1500 | 4.839 |
| 5.25 | 1260 | 5.030 | 1365 | 5.045 | 1470 | 5.060 | 1575 | 5.073 |
| 5.5 | 1320 | 5.259 | 1430 | 5.277 | 1540 | 5.291 | 1650 | 5.305 |
| 5.75 | 1380 | 5.487 | 1495 | 5.508 | 1610 | 5.522 | 1725 | 5.537 |
| 6. | 1440 | 5.714 | 1560 | 5.735 | 1680 | 5.754 | 1800 | 5.769 |
| 6.25 | 1500 | 5.940 | 1625 | 5.963 | 1750 | 5.983 | 1875 | 6. |
| 6.5 | 1560 | 6.167 | 1690 | 6.192 | 1820 | 6.212 | 1950 | 6.230 |
| 6.75 | 1620 | 6.391 | 1755 | 6.416 | 1890 | 6.439 | 2025 | 6.460 |
| 7. | 1680 | 6.614 | 1820 | 6.643 | 1960 | 6.666 | 2100 | 6.689 |
| 7.25 | 1740 | 6.836 | 1885 | 6.869 | 2030 | 6.894 | 2175 | 6.916 |
| 7.5 | 1800 | 7.060 | 1950 | 7.090 | 2100 | 7.119 | 2250 | 7.144 |
| 7.75 | 1860 | 7.274 | 2015 | 7.314 | 2170 | 7.343 | 2325 | 7.370 |
| 8. | 1920 | 7.500 | 2080 | 7.536 | 2240 | 7.567 | 2400 | 7.596 |
| 8.5 | 2040 | 7.938 | 2210 | 7.978 | 2380 | 8.013 | 2550 | 8.055 |
| 9. | 2160 | 8.372 | 2340 | 8.417 | 2520 | 8.457 | 2700 | 8.492 |
| 9.5 | 2280 | 8.803 | 2470 | 8.852 | 2660 | 8.895 | 2850 | 8.935 |
| 10. | 2400 | 9.230 | 2600 | 9.286 | 2800 | 9.333 | 3000 | 9.375 |
| 10.5 | 2520 | 9.654 | 2730 | 9.716 | 2940 | 9.767 | 3150 | 9.807 |
| 11. | 2640 | 10.076 | 2860 | 10.142 | 3080 | 10.198 | 3300 | 10.250 |
| 11.5 | 2760 | 10.494 | 2990 | 10.565 | 3220 | 10.627 | 3450 | 10.681 |
| 12. | 2880 | 10.909 | 3120 | 10.986 | 3360 | 11.052 | 3600 | 11.111 |
| 12.5 | 3000 | 11.321 | 3250 | 11.404 | 3500 | 11.475 | 3750 | 11.538 |
| 13. | 3120 | 11.728 | 3380 | 11.818 | 3640 | 11.895 | 3900 | 11.961 |
| 14. | 3360 | 12.546 | 3640 | 12.639 | 3920 | 12.727 | 4200 | 12.793 |
| 15. | 3600 | 13.333 | 3900 | 13.448 | 4200 | 13.549 | 4500 | 13.635 |
| 16. | 3840 | 14.116 | 4160 | 14.247 | 4480 | 14.359 | 4800 | 14.458 |
| 20. | 4800 | 17.143 | 5200 | 17.333 | 5600 | 17.500 | 6000 | 17.646 |

PART 2.—Sectional Areas (A) and Hydraulic Radii (R), for

Corresponding to Different Bed-

| <i>a</i> | <i>b</i> =2 | | <i>b</i> =3 | | <i>b</i> =4 | | <i>b</i> =5 | |
|----------|-------------|----------|-------------|----------|-------------|----------|-------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 0.5 | 1.25 | 0.366 | 1.75 | 0.396 | 2.25 | 0.416 | 2.75 | 0.429 |
| 0.75 | 2.06 | 0.500 | 2.81 | 0.549 | 3.56 | 0.582 | 4.31 | 0.607 |
| 1. | 3. | 0.621 | 4. | 0.686 | 5. | 0.732 | 6. | 0.766 |
| 1.25 | 4.06 | 0.734 | 5.31 | 0.812 | 6.56 | 0.871 | 7.81 | 0.915 |
| 1.5 | 5.25 | 0.841 | 6.75 | 0.932 | 8.25 | 1.000 | 9.75 | 1.054 |
| 1.75 | 6.56 | 0.942 | 8.31 | 1.045 | 10.06 | 1.124 | 11.81 | 1.186 |
| 2. | 8. | 1.045 | 10. | 1.155 | 12. | 1.243 | 14. | 1.314 |
| 2.25 | 9.56 | 1.143 | 11.81 | 1.261 | 14.06 | 1.357 | 16.31 | 1.436 |
| 2.5 | 11.25 | 1.240 | 13.75 | 1.365 | 16.25 | 1.468 | 18.75 | 1.553 |
| 2.75 | 13.06 | 1.336 | 15.81 | 1.466 | 18.56 | 1.576 | 21.31 | 1.668 |
| 3. | 15. | 1.431 | 18. | 1.567 | 21. | 1.682 | 24. | 1.780 |
| 3.5 | 19.25 | 1.618 | 22.75 | 1.764 | 26.25 | 1.889 | 29.75 | 1.997 |
| 4. | 24. | 1.803 | 28. | 1.956 | 32. | 2.090 | 36. | 2.207 |
| 5. | | | 40. | 2.333 | 45. | 2.480 | 50. | 2.612 |

| <i>a</i> | <i>b</i> =14 | | <i>b</i> =16 | | <i>b</i> =18 | | <i>b</i> =20 | |
|----------|--------------|----------|--------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1. | 15. | 0.891 | 17. | 0.903 | 19. | 0.912 | 21. | 0.920 |
| 1.25 | 19.06 | 1.087 | 21.56 | 1.104 | 24.06 | 1.117 | 26.56 | 1.129 |
| 1.5 | 23.25 | 1.275 | 26.25 | 1.297 | 29.25 | 1.315 | 32.25 | 1.330 |
| 1.75 | 27.56 | 1.454 | 31.06 | 1.482 | 34.56 | 1.506 | 38.06 | 1.525 |
| 2. | 32. | 1.628 | 36. | 1.662 | 40. | 1.691 | 44. | 1.715 |
| 2.25 | 36.56 | 1.795 | 41.06 | 1.835 | 45.56 | 1.870 | 50.06 | 1.898 |
| 2.5 | 41.25 | 1.958 | 46.25 | 2.005 | 51.25 | 2.044 | 56.25 | 2.078 |
| 2.75 | 46.06 | 2.115 | 51.56 | 2.168 | 57.06 | 2.213 | 62.56 | 2.252 |
| 3. | 51. | 2.268 | 57. | 2.328 | 63. | 2.379 | 69. | 2.422 |
| 3.25 | 56.06 | 2.417 | 62.56 | 2.484 | 69.06 | 2.541 | 75.56 | 2.589 |
| 3.5 | 61.25 | 2.563 | 68.25 | 2.635 | 75.25 | 2.697 | 82.25 | 2.751 |
| 3.75 | 66.56 | 2.709 | 74.06 | 2.783 | 81.56 | 2.851 | 89.06 | 2.998 |
| 4. | 72. | 2.845 | 80. | 2.929 | 88. | 3.002 | 96. | 3.066 |
| 4.25 | 77.56 | 2.981 | 86.06 | 3.071 | 94.56 | 3.150 | 103.06 | 3.219 |
| 4.5 | 83.25 | 3.115 | 92.25 | 3.211 | 101.25 | 3.296 | 110.25 | 3.369 |
| 4.75 | 89.06 | 3.246 | 98.56 | 3.348 | 108.06 | 3.437 | 117.56 | 3.516 |
| 5. | 95. | 3.376 | 105. | 3.484 | 115. | 3.578 | 125. | 3.661 |
| 5.5 | 107.25 | 3.630 | 118.25 | 3.748 | 129.25 | 3.852 | 140.25 | 3.944 |
| 6. | 120. | 3.875 | 132. | 4.004 | 144. | 4.117 | 156. | 4.220 |
| 7. | 147. | 4.350 | 161. | 4.497 | 175. | 4.630 | 189. | 4.748 |

Trapezoidal Sections of Canals with Side Slopes of One to One.
widths (*b*) and Depths of Water (*d*).

| <i>d</i> | <i>b</i> =6 | | <i>b</i> =8 | | <i>b</i> =10 | | <i>b</i> =12 | |
|----------|-------------|----------|-------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1.0 | 7. | 0.793 | 9. | 0.831 | 11. | 0.858 | 13. | 0.877 |
| 1.25 | 9.06 | 0.950 | 11.56 | 1.002 | 14.06 | 1.039 | 16.56 | 1.066 |
| 1.5 | 11.25 | 1.098 | 14.25 | 1.164 | 17.25 | 1.211 | 20.25 | 1.246 |
| 1.75 | 13.56 | 1.238 | 17.06 | 1.318 | 20.56 | 1.375 | 24.06 | 1.420 |
| 2. | 16. | 1.373 | 20. | 1.464 | 24. | 1.533 | 28. | 1.586 |
| 2.25 | 18.56 | 1.502 | 23.06 | 1.606 | 27.56 | 1.684 | 32.06 | 1.746 |
| 2.5 | 21.25 | 1.626 | 26.25 | 1.742 | 31.25 | 1.831 | 36.25 | 1.901 |
| 2.75 | 24.06 | 1.747 | 29.56 | 1.873 | 35.06 | 1.972 | 40.56 | 2.051 |
| 3. | 27. | 1.864 | 33. | 2.002 | 39. | 2.110 | 45. | 2.197 |
| 3.25 | 30.06 | 1.979 | 35.56 | 2.069 | 43.06 | 2.244 | 49.56 | 2.339 |
| 3.5 | 33.25 | 2.091 | 40.25 | 2.249 | 47.25 | 2.375 | 54.25 | 2.477 |
| 3.75 | 36.56 | 2.201 | 44.06 | 2.368 | 51.56 | 2.502 | 59.06 | 2.612 |
| 4. | 40. | 2.311 | 48. | 2.486 | 56. | 2.628 | 64. | 2.745 |
| 5. | 55. | 2.731 | 65. | 2.936 | 75. | 3.107 | 85. | 3.252 |

| <i>d</i> | <i>b</i> =25 | | <i>b</i> =30 | | <i>b</i> =35 | | <i>b</i> =40 | |
|----------|--------------|----------|--------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1.0 | 26. | 0.934 | 31. | 0.944 | 36. | 0.952 | 41. | 0.957 |
| 1.5 | 39.75 | 1.359 | 47.25 | 1.380 | 54.75 | 1.395 | 62.25 | 1.407 |
| 2. | 54. | 1.761 | 64. | 1.795 | 74. | 1.820 | 84. | 1.840 |
| 2.25 | 61.31 | 1.954 | 72.56 | 1.995 | 83.81 | 2.026 | 95.06 | 2.050 |
| 2.5 | 68.75 | 2.144 | 81.25 | 2.172 | 93.75 | 2.228 | 106.25 | 2.257 |
| 2.75 | 76.31 | 2.328 | 90.06 | 2.384 | 103.81 | 2.426 | 117.56 | 2.460 |
| 3. | 84. | 2.509 | 99. | 2.573 | 114. | 2.622 | 129. | 2.661 |
| 3.25 | 91.81 | 2.684 | 108.06 | 2.758 | 124.31 | 2.815 | 140.56 | 2.838 |
| 3.5 | 99.75 | 2.858 | 117.25 | 2.939 | 134.75 | 3.001 | 152.25 | 3.051 |
| 3.75 | 107.81 | 3.028 | 126.56 | 3.141 | 145.31 | 3.197 | 164.06 | 3.242 |
| 4. | 116. | 3.193 | 136. | 3.291 | 156. | 3.368 | 176. | 3.431 |
| 4.25 | 124.31 | 3.358 | 145.56 | 3.464 | 166.81 | 3.547 | 188.06 | 3.615 |
| 4.5 | 132.75 | 3.519 | 155.25 | 3.633 | 177.75 | 3.724 | 200.25 | 3.798 |
| 4.75 | 141.31 | 3.677 | 165.06 | 3.800 | 188.81 | 3.898 | 212.56 | 3.977 |
| 5. | 150. | 3.831 | 175. | 3.965 | 200. | 4.070 | 225. | 4.155 |
| 5.5 | 167.75 | 4.136 | 195.25 | 4.286 | 222.75 | 4.406 | 250.25 | 4.504 |
| 6. | 186. | 4.432 | 216. | 4.599 | 246. | 4.733 | 276. | 4.844 |
| 6.5 | 204.75 | 4.720 | 237.25 | 4.903 | 269.75 | 5.053 | 302.25 | 5.177 |
| 7. | 224. | 5.000 | 259. | 5.201 | 294. | 5.365 | 329. | 5.501 |
| 8. | 264. | 5.541 | 304. | 5.776 | 344. | 5.968 | 384. | 6.132 |

PART 2 (cont.).—Sectional Areas (A) and Hydraulic Radii (R), for
Corresponding to Various Bed-

| <i>d</i> | <i>b</i> =50 | | <i>b</i> =60 | | <i>b</i> =70 | | <i>b</i> =80 | |
|----------|--------------|----------|--------------|----------|--------------|----------|--------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1.0 | 51. | 0.964 | 61. | 0.971 | 71. | 0.975 | 81. | 0.978 |
| 2.0 | 104. | 1.868 | 124. | 1.889 | 144. | 1.903 | 164. | 1.915 |
| 2.25 | 117.56 | 2.086 | 140.06 | 2.110 | 162.56 | 2.129 | 185.06 | 2.143 |
| 2.5 | 131.25 | 2.300 | 156.25 | 2.330 | 181.25 | 2.352 | 206.25 | 2.369 |
| 2.75 | 145.06 | 2.511 | 172.56 | 2.546 | 200.06 | 2.572 | 227.56 | 2.592 |
| 3. | 159. | 2.719 | 189. | 2.760 | 219. | 2.790 | 249. | 2.814 |
| 3.25 | 173.06 | 2.927 | 205.56 | 2.971 | 238.06 | 3.006 | 270.56 | 3.034 |
| 3.5 | 187.25 | 3.126 | 222.25 | 3.180 | 257.25 | 3.220 | 292.25 | 3.251 |
| 3.75 | 201.56 | 3.326 | 239.06 | 3.386 | 276.56 | 3.431 | 314.06 | 3.466 |
| 4. | 216. | 3.523 | 256. | 3.590 | 296. | 3.640 | 336. | 3.680 |
| 4.25 | 230.56 | 3.717 | 273.06 | 3.791 | 315.56 | 3.847 | 358.06 | 3.891 |
| 4.5 | 245.25 | 3.910 | 290.25 | 3.991 | 335.25 | 4.052 | 380.25 | 4.101 |
| 4.75 | 260.06 | 4.100 | 307.56 | 4.188 | 355.06 | 4.256 | 402.56 | 4.308 |
| 5. | 275. | 4.287 | 325. | 4.384 | 375. | 4.457 | 425. | 4.514 |
| 5.25 | 290.06 | 4.473 | 342.56 | 4.577 | 395.06 | 4.656 | 447.56 | 4.719 |
| 5.5 | 305.25 | 4.656 | 360.25 | 4.768 | 415.25 | 4.853 | 470.25 | 4.921 |
| 5.75 | 320.56 | 4.838 | 378.06 | 4.957 | 435.56 | 5.049 | 493.06 | 5.122 |
| 6. | 336. | 5.017 | 396. | 5.145 | 456. | 5.243 | 516. | 5.321 |
| 6.25 | 351.56 | 5.195 | 414.06 | 5.330 | 476.56 | 5.435 | 539.06 | 5.519 |
| 6.5 | 367.25 | 5.371 | 432.25 | 5.515 | 497.25 | 5.626 | 562.25 | 5.715 |
| 6.75 | 383.06 | 5.544 | 450.56 | 5.697 | 518.06 | 5.815 | 585.56 | 5.909 |
| 7. | 399. | 5.716 | 469. | 5.877 | 539. | 6.002 | 609. | 6.102 |
| 7.25 | 415.06 | 5.887 | 487.59 | 6.056 | 560.06 | 6.188 | 632.56 | 6.293 |
| 7.5 | 431.25 | 6.056 | 506.25 | 6.234 | 581.25 | 6.373 | 656.25 | 6.484 |
| 7.75 | 447.56 | 6.223 | 525.06 | 6.409 | 602.56 | 6.555 | 680.06 | 6.672 |
| 8. | 464. | 6.389 | 544. | 6.584 | 624. | 6.736 | 704. | 6.860 |
| 8.25 | 480.56 | 6.553 | 563.06 | 6.757 | 645.56 | 6.917 | 728.06 | 7.046 |
| 8.5 | 497.25 | 6.716 | 582.25 | 6.928 | 667.25 | 7.095 | 752.25 | 7.230 |
| 8.75 | 514.06 | 6.877 | 601.56 | 7.098 | 689.06 | 7.272 | 776.56 | 7.414 |
| 9. | 531. | 7.037 | 621. | 7.267 | 711. | 7.448 | 801. | 7.595 |
| 9.25 | 548.06 | 7.196 | 640.56 | 7.434 | 733.06 | 7.623 | 825.56 | 7.777 |
| 9.5 | 565.25 | 7.353 | 660.25 | 7.600 | 755.25 | 7.797 | 850.25 | 7.956 |
| 9.75 | 582.56 | 7.509 | 680.06 | 7.765 | 777.56 | 7.968 | 875.06 | 8.134 |
| 10. | 600. | 7.665 | 700. | 7.929 | 800. | 8.140 | 900. | 8.312 |
| 11. | 671. | 8.273 | 781. | 8.572 | 891. | 8.812 | 1001. | 9.009 |
| 12. | 744. | 8.863 | 864. | 9.197 | 984. | 9.467 | 1104. | 9.689 |

Trapezoidal Sections of Canals with Side Slopes of One to One.
widths (*b*) and Depths of Water (*d*).

| <i>d</i> | <i>b</i> =90 | | <i>b</i> =100 | | <i>b</i> =120 | | <i>b</i> =140 | |
|----------|--------------|----------|---------------|----------|---------------|----------|---------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1.0 | 91. | 0.980 | 101. | 0.982 | 121. | 0.985 | 141. | 0.987 |
| 2.0 | 184. | 1.923 | 204. | 1.931 | 244. | 1.942 | 284. | 1.950 |
| 2.25 | 207.56 | 2.154 | 230.06 | 2.163 | 275.06 | 2.177 | 320.06 | 2.187 |
| 2.5 | 231.25 | 2.382 | 256.25 | 2.393 | 306.25 | 2.410 | 356.25 | 2.422 |
| 2.75 | 255.06 | 2.609 | 282.56 | 2.622 | 337.56 | 2.642 | 392.56 | 2.656 |
| 3. | 279. | 2.833 | 309. | 2.848 | 369. | 2.872 | 429. | 2.889 |
| 3.25 | 303.06 | 3.055 | 335.56 | 3.073 | 400.56 | 3.101 | 465.56 | 3.121 |
| 3.5 | 327.25 | 3.276 | 362.25 | 3.296 | 432.25 | 3.328 | 502.25 | 3.351 |
| 3.75 | 351.56 | 3.494 | 389.06 | 3.517 | 464.06 | 3.553 | 539.06 | 3.579 |
| 4. | 376. | 3.711 | 416. | 3.737 | 496. | 3.777 | 576. | 3.807 |
| 4.25 | 400.56 | 3.926 | 443.06 | 3.955 | 528.06 | 4. | 613.06 | 4.033 |
| 4.5 | 425.25 | 4.139 | 470.25 | 4.171 | 560.25 | 4.221 | 650.25 | 4.258 |
| 4.75 | 450.06 | 4.351 | 497.56 | 4.386 | 592.56 | 4.441 | 687.56 | 4.481 |
| 5. | 475. | 4.562 | 525. | 4.600 | 625. | 4.659 | 725. | 4.703 |
| 5.25 | 500.06 | 4.769 | 552.56 | 4.811 | 657.56 | 4.876 | 762.56 | 4.924 |
| 5.5 | 525.25 | 4.976 | 580.25 | 5.021 | 690.25 | 5.092 | 800.25 | 5.144 |
| 5.75 | 550.56 | 5.181 | 608.06 | 5.230 | 723.06 | 5.306 | 838.06 | 5.303 |
| 6. | 576. | 5.397 | 636. | 5.437 | 756. | 5.519 | 876. | 5.581 |
| 6.25 | 601.56 | 5.587 | 664.06 | 5.643 | 789.06 | 5.731 | 914.06 | 5.797 |
| 6.5 | 627.25 | 5.788 | 692.25 | 5.848 | 822.25 | 5.942 | 952.25 | 6.013 |
| 6.75 | 653.06 | 5.986 | 720.56 | 6.050 | 855.56 | 6.151 | 990.56 | 6.226 |
| 7. | 679. | 6.184 | 749. | 6.252 | 889. | 6.359 | 1029. | 6.439 |
| 7.25 | 705.06 | 6.380 | 777.56 | 6.452 | 922.56 | 6.566 | 1067.56 | 6.651 |
| 7.5 | 731.25 | 6.575 | 806.25 | 6.652 | 956.25 | 6.772 | 1106.25 | 6.862 |
| 7.75 | 757.56 | 6.769 | 835.06 | 6.849 | 990.06 | 6.976 | 1145.06 | 7.072 |
| 8. | 784. | 6.961 | 864. | 7.046 | 1024. | 7.179 | 1184. | 7.280 |
| 8.25 | 810.56 | 7.152 | 892.06 | 7.241 | 1058.06 | 7.382 | 1223.06 | 7.488 |
| 8.5 | 837.25 | 7.342 | 922.25 | 7.435 | 1092.25 | 7.583 | 1262.25 | 7.695 |
| 8.75 | 864.06 | 7.530 | 951.56 | 7.628 | 1126.56 | 7.783 | 1301.56 | 7.900 |
| 9. | 891. | 7.717 | 981. | 7.819 | 1161. | 7.982 | 1341. | 8.105 |
| 9.25 | 918.06 | 7.903 | 1010.56 | 8.010 | 1195.56 | 8.180 | 1380.56 | 8.289 |
| 9.5 | 945.25 | 8.088 | 1040.25 | 8.199 | 1230.25 | 8.376 | 1420.25 | 8.511 |
| 9.75 | 972.56 | 8.271 | 1070.06 | 8.387 | 1265.06 | 8.572 | 1460.06 | 8.713 |
| 10. | 1000. | 8.454 | 1100. | 8.575 | 1300. | 8.767 | 1500. | 8.912 |
| 11. | 1111. | 9.173 | 1221. | 9.313 | 1441. | 9.536 | 1661. | 9.707 |
| 12. | 1224. | 9.876 | 1344. | 10.03 | 1584. | 10.29 | 1824. | 10.49 |

PART 2 (cont.).—Sectional Areas (A) and Hydraulic Radii (R), for
Corresponding to Various Bed-

| d | b=180 | | b=180 | | b=200 | | b=220 | |
|------|---------|-------|---------|-------|---------|-------|---------|-------|
| | A | R | A | R | A | R | A | R |
| 1.0 | 161. | 0.989 | 181. | 0.990 | 201. | 0.991 | 221. | 0.992 |
| 2.0 | 324. | 1.956 | 364. | 1.961 | 404. | 1.964 | 444. | 1.968 |
| 2.25 | 365.06 | 2.194 | 410.06 | 2.200 | 455.06 | 2.205 | 500.06 | 2.209 |
| 2.5 | 406.25 | 2.432 | 456.25 | 2.439 | 506.25 | 2.445 | 556.25 | 2.450 |
| 2.75 | 447.56 | 2.668 | 502.56 | 2.676 | 557.56 | 2.683 | 612.56 | 2.689 |
| 3. | 489. | 2.902 | 549. | 2.913 | 609. | 2.921 | 669. | 2.928 |
| 3.25 | 530.56 | 3.136 | 595.56 | 3.148 | 660.56 | 3.158 | 725.56 | 3.166 |
| 3.5 | 572.25 | 3.368 | 642.25 | 3.382 | 712.25 | 3.393 | 782.25 | 3.403 |
| 3.75 | 614.06 | 3.599 | 689.06 | 3.615 | 764.06 | 3.628 | 839.06 | 3.638 |
| 4. | 656. | 3.829 | 736. | 3.847 | 816. | 3.862 | 896. | 3.874 |
| 4.25 | 698.06 | 4.058 | 783.06 | 4.078 | 868.06 | 4.094 | 953.06 | 4.108 |
| 4.5 | 740.25 | 4.286 | 830.25 | 4.308 | 920.25 | 4.326 | 1010.25 | 4.341 |
| 4.75 | 782.56 | 4.512 | 877.56 | 4.537 | 972.56 | 4.557 | 1067.56 | 4.573 |
| 5. | 825. | 4.738 | 925. | 4.765 | 1025. | 4.787 | 1125. | 4.805 |
| 5.25 | 867.56 | 4.962 | 972.56 | 4.991 | 1077.56 | 5.015 | 1182.56 | 5.035 |
| 5.5 | 910.25 | 5.185 | 1020.25 | 5.217 | 1130.25 | 5.243 | 1240.25 | 5.265 |
| 5.75 | 953.06 | 5.407 | 1068.06 | 5.442 | 1183.06 | 5.470 | 1298.06 | 5.494 |
| 6. | 996. | 5.628 | 1116. | 5.666 | 1236. | 5.697 | 1356. | 5.722 |
| 6.25 | 1039.06 | 5.848 | 1164.06 | 5.889 | 1289.06 | 5.921 | 1414.06 | 5.949 |
| 6.5 | 1082.25 | 6.067 | 1212.25 | 6.111 | 1342.25 | 6.146 | 1472.25 | 6.176 |
| 6.75 | 1125.56 | 6.285 | 1260.56 | 6.332 | 1395.56 | 6.370 | 1530.56 | 6.402 |
| 7. | 1169. | 6.498 | 1309. | 6.552 | 1449. | 6.592 | 1589. | 6.626 |
| 7.25 | 1212.56 | 6.717 | 1357.56 | 6.770 | 1502.56 | 6.814 | 1647.56 | 6.850 |
| 7.5 | 1256.25 | 6.927 | 1406.25 | 6.973 | 1556.25 | 7.035 | 1706.25 | 7.074 |
| 7.75 | 1300.06 | 7.146 | 1455.06 | 7.206 | 1610.06 | 7.255 | 1765.06 | 7.296 |
| 8. | 1344. | 7.359 | 1504. | 7.422 | 1664. | 7.474 | 1824. | 7.518 |
| 8.5 | 1432.25 | 7.782 | 1602.25 | 7.853 | 1772.25 | 7.910 | 1942.25 | 7.959 |
| 9. | 1521. | 8.201 | 1701. | 8.279 | 1881. | 8.343 | 2061. | 8.397 |
| 9.5 | 1610.25 | 8.617 | 1800.25 | 8.702 | 1990.25 | 8.773 | 2180.25 | 8.832 |
| 10. | 1700. | 9.029 | 1900. | 9.122 | 2100. | 9.199 | 2300. | 9.264 |
| 11. | 1881. | 9.843 | 2101. | 9.952 | 2321. | 10.04 | 2541. | 10.12 |
| 12. | 2064. | 10.64 | 2304. | 10.77 | 2544. | 10.87 | 2784. | 10.96 |
| 13. | 2249. | 11.43 | 2509. | 11.59 | 2769. | 11.69 | 3029. | 11.80 |
| 14. | 2436. | 12.20 | 2716. | 12.37 | 2996. | 12.50 | 3276. | 12.62 |
| 15. | 2625. | 12.97 | 2925. | 13.15 | 3225. | 13.30 | 3525. | 13.46 |
| 16. | 2816. | 13.72 | 3136. | 13.92 | 3456. | 14.09 | 3776. | 14.24 |

*Trapezoidal Sections of Canals with Side Slopes of One to One.**widths (b) and depths of water (d).*

| d | b=240 | | b=260 | | b=280 | | b=300 | |
|------|---------|--------------|---------|--------------|---------|--------------|---------|--------------|
| | A | R | A | R | A | R | A | R |
| 2.0 | 484. | 1.970 | 524. | 1.972 | 564. | 1.974 | 604. | 1.976 |
| 2.5 | 606.25 | 2.454 | 656.25 | 2.457 | 706.25 | 2.460 | 756.25 | 2.463 |
| 3. | 729. | 2.934 | 789. | 2.939 | 849. | 2.943 | 909. | 2.947 |
| 3.25 | 790.56 | 3.173 | 855.56 | 3.178 | 920.56 | 3.183 | 985.56 | 3.188 |
| 3.5 | 852.25 | 3.411 | 922.25 | 3.417 | 992.25 | 3.423 | 1062.25 | 3.428 |
| 3.75 | 914.06 | 3.647 | 989.06 | 3.655 | 1064.06 | 3.662 | 1139.06 | 3.667 |
| 4. | 976. | 3.884 | 1056. | 3.892 | 1136. | 3.900 | 1216. | 3.906 |
| 4.25 | 1038.06 | 4.119 | 1123.06 | 4.129 | 1208.06 | 4.136 | 1293.06 | 4.144 |
| 4.5 | 1100.25 | 4.353 | 1190.25 | 4.364 | 1280.25 | 4.373 | 1370.25 | 4.382 |
| 4.75 | 1162.56 | 4.587 | 1257.56 | 4.599 | 1352.56 | 4.610 | 1447.56 | 4.619 |
| 5. | 1225. | 4.820 | 1325. | 4.833 | 1425. | 4.845 | 1525. | 4.855 |
| 5.25 | 1287.56 | 5.053 | 1392.56 | 5.067 | 1497.56 | 5.079 | 1602.56 | 5.090 |
| 5.5 | 1350.25 | 5.283 | 1460.25 | 5.299 | 1570.25 | 5.313 | 1680.25 | 5.325 |
| 5.75 | 1413.06 | 5.514 | 1528.06 | 5.531 | 1643.06 | 5.546 | 1758.06 | 5.559 |
| 6. | 1476. | 5.744 | 1596. | 5.762 | 1716. | 5.778 | 1836. | 5.792 |
| 6.25 | 1539.06 | 5.973 | 1664.06 | 5.993 | 1789.06 | 6.010 | 1914.06 | 6.025 |
| 6.5 | 1602.25 | 6.201 | 1732.25 | 6.223 | 1862.25 | 6.241 | 1992.25 | 6.257 |
| 6.75 | 1665.56 | 6.429 | 1800.56 | 6.452 | 1935.56 | 6.470 | 2070.56 | 6.489 |
| 7. | 1729. | 6.655 | 1869. | 6.680 | 2009. | 6.701 | 2149. | 6.720 |
| 7.25 | 1792.56 | 6.881 | 1937.56 | 6.908 | 2082.56 | 6.930 | 2227.56 | 6.950 |
| 7.5 | 1856.25 | 7.106 | 2006.25 | 7.134 | 2156.25 | 7.159 | 2306.25 | 7.180 |
| 7.75 | 1920.06 | 7.331 | 2075.06 | 7.361 | 2230.06 | 7.386 | 2385.06 | 7.392 |
| 8. | 1984. | 7.554 | 2144. | 7.586 | 2304. | 7.613 | 2464. | 7.637 |
| 8.5 | 2112.25 | 8.000 | 2282.25 | 8.035 | 2452.25 | 8.066 | 2622.25 | 8.092 |
| 9. | 2241. | 8.442 | 2421. | 8.481 | 2601. | 8.515 | 2781. | 8.545 |
| 9.5 | 2370.25 | 8.882 | 2560.25 | 8.925 | 2750.25 | 8.962 | 2940.25 | 8.995 |
| 10. | 2500. | 9.319 | 2700. | 9.366 | 2900. | 9.407 | 3100. | 9.443 |
| 10.5 | 2630.25 | <u>9.753</u> | 2840.25 | <u>9.804</u> | 3050.25 | <u>9.849</u> | 3260.25 | <u>9.889</u> |
| 11. | 2761. | 10.18 | 2981. | 10.24 | 3201. | 10.29 | 3421. | 10.33 |
| 11.5 | 2892.25 | 10.61 | 3122.25 | 10.67 | 3352.25 | 10.73 | 3582.25 | 10.77 |
| 12. | 3024. | 11.04 | 3264. | 11.10 | 3504. | 11.16 | 3744. | 11.21 |
| 12.5 | 3156.25 | 11.46 | 3406.25 | 11.53 | 3656.25 | 11.59 | 3906.25 | 11.65 |
| 13. | 3289. | 11.88 | 3549. | 11.96 | 3809. | 12.02 | 4069. | 12.08 |
| 14. | 3556. | 12.72 | 3836. | 12.80 | 4116. | 12.88 | 4276. | 12.94 |
| 15. | 3825. | 13.54 | 4125. | 13.64 | 4425. | 13.72 | 4725. | 13.80 |
| 16. | 4096. | 14.36 | 4416. | 14.47 | 4736. | 14.56 | 5056. | 14.64 |
| 20. | 5200. | 17.53 | 5600. | 17.69 | 6000. | 17.83 | 6400. | 17.95 |

REDUCTION MULTIPLIERS FOR R .

For obtaining Values of R' , the Hydraulic Radius, for any Trapezoidal Section, from those of R given for Rectangular Sections in Part I.

$\frac{b}{d}$ is the ratio of the bed-width to the depth of water.

| $\frac{b}{d}$ | Ratios of Side Slopes. | | | | | | | | |
|---------------|------------------------|---------------------|---------------------|---------------------|---------------------|---------|----------------------|----------------------|---------|
| | Rectr. | $\frac{1}{2}$ to 1. | $\frac{3}{4}$ to 1. | $\frac{1}{2}$ to 1. | $\frac{3}{4}$ to 1. | 1 to 1. | $1\frac{1}{2}$ to 1. | $1\frac{3}{4}$ to 1. | 2 to 1. |
| 0.5 | 1.0 | 1.179 | 1.242 | 1.828 | 2.083 | 2.254 | 2.332 | 2.435 | 2.514 |
| 0.75 | | 1.105 | 1.160 | 1.536 | 1.692 | 1.793 | 1.855 | 1.894 | 1.931 |
| 1. | | 1.081 | 1.119 | 1.391 | 1.500 | 1.567 | 1.606 | 1.628 | 1.645 |
| 1.25 | | 1.064 | 1.095 | 1.305 | 1.386 | 1.434 | 1.460 | 1.473 | 1.477 |
| 1.5 | | 1.054 | 1.078 | 1.249 | 1.313 | 1.348 | 1.364 | 1.371 | 1.368 |
| 2. | | 1.040 | 1.058 | 1.180 | 1.222 | 1.243 | 1.249 | 1.249 | 1.236 |
| 2.5 | | 1.032 | 1.046 | 1.140 | 1.170 | 1.183 | 1.184 | 1.179 | 1.162 |
| 3.0 | | 1.026 | 1.038 | 1.114 | 1.136 | 1.144 | 1.142 | 1.135 | 1.115 |
| 3.5 | | 1.023 | 1.033 | 1.096 | 1.113 | 1.117 | 1.114 | 1.106 | 1.084 |
| 4.0 | | 1.020 | 1.029 | 1.082 | 1.096 | 1.099 | 1.093 | 1.085 | 1.062 |
| 4.5 | | 1.016 | 1.025 | 1.072 | 1.085 | 1.084 | 1.078 | 1.069 | 1.046 |
| 5. | | 1.016 | 1.023 | 1.064 | 1.073 | 1.073 | 1.067 | 1.057 | 1.038 |
| 6. | | 1.013 | 1.018 | 1.052 | 1.059 | 1.057 | 1.051 | 1.041 | 1.019 |
| 7. | | 1.011 | 1.016 | 1.044 | 1.049 | 1.047 | 1.039 | 1.031 | 1.009 |
| 8. | | 1.010 | 1.014 | 1.038 | 1.042 | 1.039 | 1.032 | 1.023 | 1.002 |
| 9. | | 1.009 | 1.012 | 1.033 | 1.036 | 1.033 | 1.027 | 1.018 | 0.998 |
| 10. | | 1.008 | 1.011 | 1.030 | 1.032 | 1.029 | 1.023 | 1.014 | 0.995 |
| 12. | | 1.006 | 1.009 | 1.024 | 1.026 | 1.023 | 1.017 | 1.009 | 0.992 |
| 14. | | 1.005 | 1.008 | 1.021 | 1.022 | 1.019 | 1.013 | 1.006 | 0.990 |
| 16. | | 1.004 | 1.007 | 1.018 | 1.018 | 1.016 | 1.011 | 1.004 | 0.989 |
| 18. | | 1.004 | 1.006 | 1.016 | 1.016 | 1.014 | 1.009 | 1.003 | 0.989 |
| 20. | | 1.004 | 1.005 | 1.014 | 1.014 | 1.012 | 1.007 | 1.002 | 0.989 |
| 30. | | 1.003 | 1.003 | 1.009 | 1.009 | 1.007 | 1.004 | 1.000 | 0.990 |
| 40. | | 1.002 | 1.003 | 1.007 | 1.007 | 1.005 | 1.002 | 1.000 | 0.992 |
| 50. | | 1.001 | 1.002 | 1.005 | 1.005 | 1.004 | 1.002 | 0.999 | 0.992 |
| 60. | | 1.001 | 1.002 | 1.005 | 1.005 | 1.004 | 1.001 | 0.999 | 0.994 |
| 70. | | 1.001 | 1.001 | 1.004 | 1.004 | 1.003 | 1.001 | 0.999 | 0.995 |
| 80. | | 1.001 | 1.001 | 1.003 | 1.003 | 1.002 | 1.001 | 0.999 | 0.995 |
| 90. | | 1.001 | 1.001 | 1.003 | 1.003 | 1.002 | 1.001 | 0.999 | 0.995 |
| 100. | 1.0 | 1.001 | 1.001 | 1.003 | 1.003 | 1.002 | 1.001 | 0.999 | 0.996 |

To obtain values of A' the sectional area for any trapezoidal section having t to 1 as the ratio of the side slopes, add td^2 to the values of A given for rectangular sections in Part I.

REDUCTION MULTIPLIERS FOR R .

For obtaining Values of R' , the Hydraulic Radius, for any Trapezoidal Section, from those of R given for Trapezoidal Sections having Side Slopes of One to One in Part 2.

$\frac{b}{d}$ is the ratio of the bed-width to the depth of water.

| $\frac{b}{d}$ | Ratios of Side Slopes. | | | | | | | | |
|---------------|------------------------|---------------------|---------------------|---------------------|---------------------|---------|----------------------|----------------------|---------|
| | 0 to 1. | $\frac{1}{2}$ to 1. | $\frac{1}{3}$ to 1. | $\frac{1}{4}$ to 1. | $\frac{1}{5}$ to 1. | 1 to 1. | $1\frac{1}{2}$ to 1. | $1\frac{2}{3}$ to 1. | 2 to 1. |
| 0.5 | .4437 | 0.523 | 0.551 | 0.811 | 0.924 | 1.0 | 1.035 | 1.080 | 1.116 |
| 0.75 | .5577 | 0.616 | 0.647 | 0.857 | 0.944 | | 1.035 | 1.056 | 1.077 |
| 1. | .6382 | 0.690 | 0.714 | 0.888 | 0.957 | | 1.025 | 1.039 | 1.050 |
| 1.25 | .6974 | 0.742 | 0.764 | 0.910 | 0.967 | | 1.018 | 1.027 | 1.030 |
| 1.5 | .7418 | 0.782 | 0.800 | 0.927 | 0.974 | | 1.012 | 1.017 | 1.015 |
| 2. | .8045 | 0.837 | 0.851 | 0.949 | 0.983 | | 1.005 | 1.005 | 0.994 |
| 2.5 | .8453 | 0.872 | 0.884 | 0.964 | 0.989 | | 1.001 | 0.997 | 0.982 |
| 3. | .8741 | 0.897 | 0.907 | 0.974 | 0.993 | | 0.998 | 0.992 | 0.975 |
| 3.5 | .8953 | 0.916 | 0.919 | 0.979 | 0.996 | | 0.997 | 0.989 | 0.971 |
| 4. | .9099 | 0.928 | 0.933 | 0.983 | 0.997 | | 0.994 | 0.986 | 0.966 |
| 4.5 | .9225 | 0.937 | 0.944 | 0.988 | 1.000 | | 0.994 | 0.985 | 0.965 |
| 5. | .9320 | 0.947 | 0.953 | 0.991 | 1.000 | | 0.994 | 0.984 | 0.964 |
| 6 | .9461 | 0.958 | 0.963 | 0.995 | 1.002 | | 0.994 | 0.984 | 0.963 |
| 7 | .9551 | 0.966 | 0.970 | 0.997 | 1.002 | | 0.992 | 0.984 | 0.963 |
| 8 | .9625 | 0.972 | 0.976 | 0.999 | 1.003 | | 0.993 | 0.984 | 0.964 |
| 9 | .9681 | 0.977 | 0.980 | 1.000 | 1.003 | | 0.994 | 0.985 | 0.966 |
| 10 | .9718 | 0.980 | 0.983 | 1.001 | 1.003 | | 0.994 | 0.985 | 0.967 |
| 12 | .9775 | 0.983 | 0.986 | 1.001 | 1.003 | | 0.994 | 0.986 | 0.970 |
| 14 | .9814 | 0.986 | 0.989 | 1.002 | 1.003 | | 0.994 | 0.987 | 0.972 |
| 16 | .9843 | 0.988 | 0.991 | 1.002 | 1.002 | | 0.995 | 0.988 | 0.974 |
| 18 | .9862 | 0.990 | 0.992 | 1.002 | 1.002 | | 0.995 | 0.989 | 0.976 |
| 20 | .9881 | 0.992 | 0.993 | 1.002 | 1.002 | | 0.995 | 0.990 | 0.978 |
| 30 | .9930 | 0.996 | 0.996 | 1.002 | 1.002 | | 0.997 | 0.993 | 0.983 |
| 40 | .9950 | 0.997 | 0.998 | 1.002 | 1.002 | | 0.997 | 0.995 | 0.987 |
| 50 | .9960 | 0.997 | 0.998 | 1.001 | 1.001 | | 0.998 | 0.995 | 0.988 |
| 60 | .9960 | 0.997 | 0.998 | 1.001 | 1.001 | | 0.997 | 0.995 | 0.990 |
| 70 | .9970 | 0.998 | 0.998 | 1.001 | 1.001 | | 0.998 | 0.996 | 0.992 |
| 80 | .9980 | 0.999 | 0.999 | 1.001 | 1.001 | | 0.999 | 0.997 | 0.993 |
| 90 | .9980 | 0.999 | 0.999 | 1.001 | 1.001 | | 0.999 | 0.997 | 0.993 |
| 100 | .9980 | 0.999 | 0.999 | 1.001 | 1.001 | 1.0 | 0.999 | 0.997 | 0.994 |

To obtain values of A' the sectional area for any trapezoidal section, having t to 1 as the ratio of the side slopes, add $d^2(t-1)$ to the values of A given for trapezoids of one to one in Part 2.

PART 3.—*Dimensions of equal-discharging*

| MEAN WIDTHS | | | | | MEAN WIDTHS | | | | |
|----------------------|-------|-------|-------|-------|----------------------|-------|-------|-------|-------|
| 100 | 90 | 80 | 70 | 60 | 60 | 50 | 40 | 30 | 20 |
| Corresponding depths | | | | | Corresponding depths | | | | |
| 1 | 1·074 | 1·164 | 1·276 | 1·408 | 1 | 1·135 | 1·324 | 1·625 | 2·198 |
| 1·5 | 1·612 | 1·748 | 1·919 | 2·135 | 1·5 | 1·704 | 1·998 | 2·466 | 3·385 |
| 2 | 2·151 | 2·333 | 2·564 | 2·862 | 2 | 2·275 | 2·674 | 3·320 | 4·621 |
| 2·5 | 2·689 | 2·921 | 3·211 | 3·591 | 2·5 | 2·850 | 3·359 | 4·196 | 5·910 |
| 3 | 3·230 | 3·511 | 3·864 | 4·327 | 3 | 3·425 | 4·050 | 5·088 | 7·250 |
| 3·5 | 3·771 | 4·102 | 4·521 | 5·066 | 3·5 | 4·003 | 4·744 | 5·993 | 8·638 |
| 4 | 4·312 | 4·695 | 5·179 | 5·814 | 4 | 4·581 | 5·445 | 6·912 | 10·07 |
| 4·5 | 4·854 | 5·289 | 5·838 | 6·567 | 4·5 | 5·162 | 6·154 | 7·847 | 11·54 |
| 5 | 5·391 | 5·884 | 6·503 | 7·322 | 5 | 5·746 | 6·868 | 8·795 | 13·05 |
| 5·5 | 5·935 | 6·481 | 7·169 | 8·087 | 5·5 | 6·331 | 7·585 | 9·753 | 14·60 |
| 6 | 6·483 | 7·079 | 7·840 | 8·854 | 6 | 6·917 | 8·306 | 10·73 | 16·19 |
| 6·5 | 7·026 | 7·678 | 8·512 | 9·624 | 6·5 | 7·504 | 9·034 | 11·72 | 17·81 |
| 7 | 7·570 | 8·278 | 9·184 | 10·40 | 7 | 8·092 | 9·766 | 12·72 | 19·46 |
| 7·5 | 8·115 | 8·880 | 9·861 | 11·18 | 7·5 | 8·682 | 10·50 | 13·73 | 21·15 |
| 8 | 8·661 | 9·486 | 10·54 | 11·97 | 8 | 9·274 | 11·24 | 14·75 | 22·87 |
| 9 | 9·754 | 10·69 | 11·91 | 13·56 | 8·5 | 9·866 | 11·98 | 15·78 | 24·61 |
| 10 | 10·85 | 11·91 | 13·29 | 15·16 | 9 | 10·46 | 12·73 | 16·82 | 26·38 |
| 11 | 11·94 | 13·13 | 14·67 | 16·78 | 9·5 | 11·06 | 13·49 | 17·87 | 28·18 |
| 12 | 13·04 | 14·35 | 16·07 | 18·41 | 10 | 11·66 | 14·24 | 18·93 | 30·00 |

Sections of Flow in Canals and Channels.

| MEAN WIDTHS | | | | | MEAN WIDTHS | | | | |
|----------------------|-------|-------|-------|-------|----------------------|-------|-------|-------|-------|
| 20 | 18 | 16 | 14 | 12 | 12 | 10 | 8 | 6 | 4 |
| Corresponding depths | | | | | Corresponding depths | | | | |
| 1 | 1·079 | 1·177 | 1·301 | 1·465 | 1 | 1·149 | 1·374 | 1·759 | 2·610 |
| 1·5 | 1·623 | 1·776 | 1·972 | 2·237 | 1·25 | 1·442 | 1·734 | 2·244 | 3·399 |
| 2 | 2·170 | 2·382 | 2·657 | 3·031 | 1·5 | 1·737 | 2·100 | 2·751 | 4·230 |
| 2·5 | 2·718 | 2·993 | 3·354 | 3·847 | 1·75 | 2·033 | 2·473 | 3·266 | 5·106 |
| 3 | 3·270 | 3·611 | 4·061 | 4·683 | 2 | 2·331 | 2·849 | 3·787 | 6·000 |
| 3·5 | 3·822 | 4·232 | 4·777 | 5·536 | 2·25 | 2·630 | 3·230 | 4·325 | 6·931 |
| 4 | 4·377 | 4·860 | 5·502 | 6·404 | 2·5 | 2·931 | 3·615 | 4·875 | 7·888 |
| 4·5 | 4·933 | 5·491 | 6·237 | 7·286 | 2·75 | 3·233 | 4·004 | 5·431 | 8·857 |
| 5 | 5·492 | 6·126 | 6·979 | 8·179 | 3 | 3·537 | 4·397 | 6·000 | 9·869 |
| 5·5 | 6·051 | 6·763 | 7·724 | 9·084 | 3·5 | 4·147 | 5·192 | 7·158 | 11·93 |
| 6 | 6·612 | 7·404 | 8·475 | 10· | 4 | 4·761 | 6·000 | 8·345 | 14·05 |
| 6·5 | 7·173 | 8·047 | 9·234 | 10·93 | 4·5 | 5·379 | 6·817 | 9·550 | 16·22 |
| 7 | 7·737 | 8·695 | 9·998 | 11·86 | 5 | 6·000 | 7·644 | 10·78 | 18·44 |
| 7·5 | 8·301 | 9·345 | 10·77 | 12·80 | 5·5 | 6·624 | 8·478 | 12·03 | 20·69 |
| 8 | 8·867 | 9·999 | 11·54 | 13·75 | 6 | 7·250 | 9·318 | 13·29 | 22·98 |
| 8·5 | 9·433 | 10·65 | 12·32 | 14·70 | 6·5 | 7·878 | 10·17 | 14·56 | 25·29 |
| 9 | 9·999 | 11·31 | 13·10 | 15·67 | 7 | 8·508 | 11·02 | 15·85 | 27·64 |
| 9·5 | 10·57 | 11·97 | 13·88 | 16·64 | 7·5 | 9·139 | 11·87 | 17·14 | 29·95 |
| 10 | 11·13 | 12·63 | 14·70 | 17·62 | 8 | 9·773 | 12·74 | 18·44 | 32·37 |

PART 3 (cont.).—Dimensions of equal-discharging

| DEPTHS OF WATER | | | | | DEPTHS OF WATER | | | | |
|---------------------------|-------|-------|-------|-------|---------------------------|-------|-------|-------|-------|
| 1 | 1.5 | 2 | 2.5 | 3 | 3 | 3.5 | 4 | 4.5 | 5 |
| Corresponding mean-widths | | | | | Corresponding mean-widths | | | | |
| 100 | 55.32 | 36.85 | 27.13 | 21.55 | 100 | 80.37 | 66.77 | 56.90 | 49.48 |
| 90 | 49.88 | 33.30 | 24.59 | 19.58 | 90 | 72.44 | 60.25 | 51.42 | 44.78 |
| 80 | 44.44 | 29.75 | 22.04 | 17.61 | 80 | 64.48 | 53.72 | 45.93 | 40.08 |
| 70 | 38.99 | 26.20 | 19.48 | 15.63 | 70 | 56.52 | 47.19 | 40.44 | 35.36 |
| 60 | 33.54 | 22.65 | 16.92 | 13.63 | 60 | 48.56 | 40.65 | 34.92 | 30.62 |
| 50 | 28.08 | 19.80 | 14.34 | 11.62 | 50 | 40.60 | 34.10 | 29.39 | 25.86 |
| 40 | 22.62 | 15.47 | 11.73 | 9.58 | 40 | 32.62 | 27.52 | 23.86 | 21.06 |
| 30 | 17.14 | 11.87 | 9.10 | 7.50 | 30 | 24.63 | 20.91 | 18.30 | 16.18 |
| 20 | 11.64 | 8.22 | 6.41 | 5.35 | 20 | 16.62 | 14.24 | 12.52 | 11.21 |

| DEPTHS OF WATER | | | | | DEPTHS OF WATER | | | | |
|---------------------------|-------|-------|------|------|---------------------------|-------|-------|-------|-------|
| 1 | 1.25 | 1.5 | 1.75 | 2 | 2 | 2.25 | 2.5 | 2.75 | 3 |
| Corresponding mean-widths | | | | | Corresponding mean-widths | | | | |
| 20 | 14.75 | 11.64 | 9.62 | 8.22 | 20 | 17.19 | 15.01 | 13.44 | 12.15 |
| 18 | 13.31 | 10.53 | 8.73 | 7.48 | 18 | 15.50 | 13.56 | 12.17 | 11.02 |
| 16 | 11.87 | 9.42 | 7.83 | 6.73 | 16 | 13.81 | 12.11 | 10.89 | 9.86 |
| 14 | 10.43 | 8.31 | 6.93 | 5.98 | 14 | 12.12 | 10.66 | 9.60 | 8.73 |
| 12 | 8.98 | 7.19 | 6.02 | 5.22 | 12 | 10.42 | 9.20 | 8.31 | 7.58 |
| 10 | 7.53 | 6.07 | 5.12 | 4.45 | 10 | 8.72 | 7.73 | 7.00 | 6.40 |
| 8 | 6.08 | 4.93 | 4.18 | 3.66 | 8 | 7.01 | 6.24 | 5.68 | 5.21 |
| 6 | 4.62 | 3.79 | 3.24 | 2.85 | 6 | 5.29 | 4.74 | 4.34 | 4.00 |
| 4 | 3.13 | 2.61 | 2.26 | 2. | 4 | 3.57 | 3.22 | 2.96 | 2.75 |

Sections of Flow in Canals and Channels.

| DEPTHS OF WATER | | | | | DEPTHS OF WATER | | | | |
|---------------------------|-------|-------|-------|-------|---------------------------|-------|-------|-------|-------|
| 5 | 6 | 7 | 7.5 | 8 | 8 | 9 | 10 | 11 | 12 |
| Corresponding mean-widths | | | | | Corresponding mean-widths | | | | |
| 100 | 77.90 | 63.57 | 58.20 | 53.67 | 100 | 85.59 | 74.78 | 66.42 | 59.82 |
| 90 | 70.27 | 57.47 | 52.68 | 48.63 | 90 | 77.18 | 67.56 | 60.11 | 54.21 |
| 80 | 62.63 | 51.36 | 41.15 | 43.58 | 80 | 68.75 | 60.28 | 53.74 | 48.56 |
| 70 | 54.98 | 45.23 | 41.58 | 38.50 | 70 | 60.31 | 53.00 | 47.36 | 42.90 |
| 60 | 47.32 | 39.07 | 35.99 | 33.39 | 60 | 51.87 | 45.71 | 40.96 | 37.20 |
| 50 | 39.63 | 32.90 | 30.37 | 28.23 | 50 | 43.38 | 38.40 | 34.53 | 31.45 |
| 40 | 31.91 | 26.66 | 24.67 | 23.00 | 40 | 34.87 | 31.01 | 28.04 | 25.59 |
| 30 | 24.17 | 20.37 | 18.94 | 17.72 | 30 | 26.34 | 23.57 | 21.49 | 19.66 |
| 20 | 16.36 | 13.95 | 13.03 | 12.25 | 20 | 17.73 | 16.00 | 14.63 | 13.51 |

| DEPTHS OF WATER | | | | | DEPTHS OF WATER | | | | |
|---------------------------|-------|-------|-------|-------|---------------------------|-------|-------|-------|-------|
| 3 | 3.5 | 4 | 4.5 | 5 | 5 | 6 | 7 | 7.5 | 8 |
| Corresponding mean-widths | | | | | Corresponding mean-widths | | | | |
| 20 | 16.62 | 14.24 | 12.52 | 11.21 | 20 | 16.36 | 13.95 | 13.03 | 12.25 |
| 18 | 15.00 | 12.89 | 11.36 | 10.19 | 18 | 14.78 | 12.65 | 11.83 | 11.13 |
| 16 | 13.37 | 11.53 | 10.19 | 9.16 | 16 | 13.19 | 11.33 | 10.61 | 10.00 |
| 14 | 11.75 | 10.17 | 9.01 | 8.13 | 14 | 11.59 | 10.00 | 9.38 | 8.86 |
| 12 | 10.12 | 8.79 | 7.82 | 7.07 | 12 | 9.99 | 8.65 | 8.13 | 7.68 |
| 10 | 8.48 | 7.41 | 6.61 | 6.00 | 10 | 8.39 | 7.29 | 6.87 | 6.50 |
| 8 | 6.83 | 5.99 | 5.37 | 4.89 | 8 | 6.78 | 5.90 | 5.57 | 5.28 |
| 6 | 5.17 | 4.57 | 4.12 | 3.77 | 6 | 5.11 | 4.50 | 4.26 | 4.04 |
| 4 | 3.48 | 3.11 | 2.82 | 2.59 | 4 | 3.44 | 3.05 | 2.89 | 2.76 |

PART 4.—Sectional Areas (A) in square feet and

CYLINDRICAL CULVERTS AND PIPES.

| Diameter | Full. | | Two-thirds full. | | One-third full. | |
|----------|--------|--------|------------------|-------|-----------------|-------|
| | A | R | A | R | A | R |
| 3 inches | 0·0491 | 0·0625 | 0·0347 | 0·073 | 0·0143 | 0·046 |
| 4 " | 0·0872 | 0·0833 | 0·0618 | 0·097 | 0·0254 | 0·062 |
| 6 " | 0·1963 | 0·125 | 0·1390 | 0·145 | 0·0573 | 0·093 |
| 8 " | 0·3490 | 0·1666 | 0·2472 | 0·194 | 0·1018 | 0·124 |
| 9 " | 0·4418 | 0·1875 | 0·3128 | 0·218 | 0·1289 | 0·140 |
| 10 " | 0·5454 | 0·2083 | 0·3807 | 0·243 | 0·1592 | 0·155 |
| Feet | | | | | | |
| 1 | 0·7854 | 0·25 | 0·5562 | 0·291 | 0·2292 | 0·186 |
| 1·25 | 1·2272 | 0·3125 | 0·8565 | 0·364 | 0·3581 | 0·233 |
| 1·5 | 1·7671 | 0·375 | 1·2514 | 0·436 | 0·5157 | 0·280 |
| 1·75 | 2·4053 | 0·4375 | 1·6409 | 0·509 | 0·7019 | 0·326 |
| 2 | 3·1416 | 0·5 | 2·2248 | 0·582 | 0·9168 | 0·372 |
| 2·25 | 3·9760 | 0·5625 | 2·8157 | 0·655 | 1·1609 | 0·419 |
| 2·5 | 4·9087 | 0·625 | 3·4262 | 0·728 | 1·4325 | 0·465 |
| 2·75 | 5·9395 | 0·6875 | 4·2062 | 0·800 | 1·7333 | 0·512 |
| 3 | 7·0686 | 0·75 | 5·0058 | 0·873 | 2·0628 | 0·559 |
| 3·25 | 8·2957 | 0·8125 | 5·8747 | 0·996 | 2·4209 | 0·605 |
| 3·5 | 9·6211 | 0·875 | 6·5635 | 1·019 | 2·8077 | 0·652 |
| 3·75 | 11·045 | 0·9375 | 7·8215 | 1·092 | 3·2230 | 0·698 |
| 4 | 12·566 | 1 | 8·8992 | 1·164 | 3·6672 | 0·744 |
| 4·5 | 15·904 | 1·125 | 11·263 | 1·310 | 4·6437 | 0·838 |
| 5 | 19·635 | 1·25 | 13·905 | 1·455 | 5·7300 | 0·931 |
| 5·5 | 23·758 | 1·375 | 16·825 | 1·601 | 6·9333 | 1·024 |
| 6 | 28·274 | 1·5 | 20·023 | 1·747 | 8·2512 | 1·117 |
| 6·5 | 33·183 | 1·625 | 23·499 | 1·992 | 9·6837 | 1·210 |
| 7 | 38·485 | 1·75 | 27·254 | 2·038 | 11·231 | 1·303 |
| 7·5 | 44·179 | 1·875 | 31·286 | 2·183 | 12·892 | 1·396 |
| 8 | 50·265 | 2 | 35·597 | 2·329 | 14·669 | 1·490 |
| 8·5 | 56·745 | 2·125 | 40·185 | 2·475 | 16·560 | 1·583 |
| 9 | 63·617 | 2·25 | 45·052 | 2·620 | 18·565 | 1·676 |
| 9·5 | 70·882 | 2·375 | 50·197 | 2·765 | 20·685 | 1·769 |
| 10 | 78·540 | 2·5 | 55·620 | 2·911 | 22·920 | 1·862 |

The values of R for cylindrical culverts half full are the same as those for full cylindrical culverts of the same diameter.

Hydraulic Radii (R) in Feet, for Culverts and Pipes.

HAWKSLEY'S OVOID CULVERT.

| Transverse Diameter | Full. | | Two-thirds full. | | One-third full. | |
|------------------------|----------|----------|------------------|----------|-----------------|----------|
| | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> | <i>A</i> | <i>R</i> |
| 1' 0" | 0·9955 | 0·2766 | 0·6714 | 0·310 | 0·2569 | 0·198 |
| 1' 2" | 1·3550 | 0·3227 | 0·9138 | 0·362 | 0·3496 | 0·231 |
| 1' 4" | 1·7697 | 0·3688 | 1·1936 | 0·413 | 0·4566 | 0·264 |
| 1' 6" | 2·2424 | 0·4149 | 1·5106 | 0·465 | 0·5780 | 0·297 |
| 1' 8" | 2·7653 | 0·4610 | 1·8650 | 0·517 | 0·7136 | 0·330 |
| 1' 10" | 3·3457 | 0·5071 | 2·2506 | 0·568 | 0·8627 | 0·363 |
| 2' 0" | 3·9820 | 0·5532 | 2·6856 | 0·620 | 1·0276 | 0·396 |
| 2' 2" | 4·6728 | 0·5993 | 3·1434 | 0·672 | 1·2050 | 0·429 |
| 2' 4" | 5·4199 | 0·6454 | 3·6554 | 0·723 | 1·3985 | 0·462 |
| 2' 6" | 6·2219 | 0·6915 | 4·1962 | 0·775 | 1·6054 | 0·495 |
| 2' 8" | 7·0790 | 0·7376 | 4·7744 | 0·826 | 1·8265 | 0·528 |
| 2' 10" | 7·8908 | 0·7837 | 5·3754 | 0·878 | 2·0606 | 0·561 |
| 3' 0" | 8·9695 | 0·8298 | 6·0426 | 0·930 | 2·3121 | 0·594 |
| 3' 2" | 9·9822 | 0·8759 | 6·7324 | 0·981 | 2·5760 | 0·627 |
| 3' 4" | 11·061 | 0·9220 | 7·4600 | 1·033 | 2·8544 | 0·660 |
| 3' 6" | 12·195 | 0·9681 | 8·2242 | 1·085 | 3·1464 | 0·693 |
| 3' 8" | 13·383 | 1·0142 | 9·0024 | 1·136 | 3·4508 | 0·726 |
| 3' 10" | 14·628 | 1·0603 | 9·8657 | 1·188 | 3·7749 | 0·759 |
| 4' 0" | 15·928 | 1·1064 | 10·742 | 1·240 | 4·1104 | 0·792 |
| 4' 2" | 17·282 | 1·1525 | 11·656 | 1·291 | 4·4600 | 0·825 |
| 4' 4" | 18·691 | 1·1986 | 12·574 | 1·343 | 4·8200 | 0·858 |
| 4' 6" | 20·182 | 1·2447 | 13·595 | 1·395 | 5·2020 | 0·891 |
| 4' 8" | 21·680 | 1·2908 | 14·622 | 1·446 | 5·5942 | 0·924 |
| 4' 10" | 23·253 | 1·3369 | 15·683 | 1·498 | 6·0006 | 0·957 |
| 5' 0" | 24·887 | 1·3830 | 16·785 | 1·550 | 6·4225 | 0·990 |
| 5' 2" | 26·567 | 1·4291 | 17·918 | 1·601 | 6·8560 | 1·023 |
| 5' 4" | 28·316 | 1·4752 | 19·098 | 1·653 | 7·3062 | 1·056 |
| 5' 6" | 30·111 | 1·5213 | 20·255 | 1·705 | 7·7643 | 1·089 |
| 5' 8" | 31·563 | 1·5674 | 21·502 | 1·756 | 8·2424 | 1·122 |
| 5' 10" | 33·871 | 1·6135 | 22·844 | 1·808 | 8·7407 | 1·155 |
| 6' 0" | 35·838 | 1·6596 | 24·170 | 1·859 | 9·2484 | 1·188 |

The long diameter = $1·2929 \times$ transverse diameter in Hawksley's Ovoid.

PART 4 (cont.).—Sectional Areas (A) in square feet

PHILLIPS' METROPOLITAN OVOID.

| Dimensions | Full. | | Two-thirds full. | | One-third full. | |
|----------------|--------|-------|------------------|-------|-----------------|-------|
| | A | R | A | R | A | R |
| 1' 0" x 1' 6" | 1·1485 | 0·290 | 0·7558 | 0·316 | 0·2840 | 0·207 |
| 1' 2" x 1' 9" | 1·5632 | 0·338 | 1·0287 | 0·368 | 0·3865 | 0·241 |
| 1' 4" x 2' 0" | 2·0418 | 0·386 | 1·3436 | 0·421 | 0·5049 | 0·276 |
| 1' 6" x 2' 3" | 2·5841 | 0·434 | 1·7005 | 0·474 | 0·6390 | 0·310 |
| 1' 8" x 2' 6" | 3·1903 | 0·483 | 2·0994 | 0·526 | 0·7889 | 0·344 |
| 1' 10" x 2' 9" | 3·8602 | 0·531 | 2·5402 | 0·579 | 0·9545 | 0·379 |
| 2' 0" x 3' 0" | 4·5940 | 0·579 | 3·0232 | 0·631 | 1·1360 | 0·413 |
| 2' 2" x 3' 3" | 5·3916 | 0·628 | 3·5480 | 0·684 | 1·3332 | 0·448 |
| 2' 4" x 3' 6" | 6·2529 | 0·676 | 4·1149 | 0·737 | 1·5462 | 0·482 |
| 2' 6" x 3' 9" | 7·1781 | 0·724 | 4·7237 | 0·789 | 1·7750 | 0·517 |
| 2' 8" x 4' 0" | 8·1671 | 0·773 | 5·3746 | 0·842 | 2·0195 | 0·541 |
| 2' 10" x 4' 3" | 9·2199 | 0·821 | 6·0674 | 0·894 | 2·2799 | 0·585 |
| 3' 0" x 4' 6" | 10·336 | 0·869 | 6·8022 | 0·947 | 2·5560 | 0·620 |
| 3' 2" x 4' 9" | 11·517 | 0·917 | 7·5790 | 1· | 2·8479 | 0·654 |
| 3' 4" x 5' 0" | 12·761 | 0·966 | 8·3978 | 1·052 | 3·1556 | 0·689 |
| 3' 6" x 5' 3" | 14·069 | 1·014 | 9·2585 | 1·105 | 3·4790 | 0·723 |
| 3' 8" x 5' 6" | 15·410 | 1·062 | 10·161 | 1·158 | 3·8182 | 0·758 |
| 3' 10" x 5' 9" | 16·877 | 1·111 | 11·106 | 1·210 | 4·1732 | 0·792 |
| 4' 0" x 6' 0" | 18·376 | 1·159 | 12·093 | 1·263 | 4·5440 | 0·826 |
| 4' 2" x 6' 3" | 19·939 | 1·207 | 13·122 | 1·315 | 4·9306 | 0·861 |
| 4' 4" x 6' 6" | 21·566 | 1·255 | 14·192 | 1·368 | 5·3329 | 0·895 |
| 4' 6" x 6' 9" | 23·257 | 1·304 | 15·305 | 1·421 | 5·7510 | 0·930 |
| 4' 8" x 7' 0" | 25·012 | 1·352 | 16·460 | 1·473 | 6·1849 | 0·964 |
| 4' 10" x 7' 3" | 26·830 | 1·400 | 17·656 | 1·526 | 6·6346 | 0·999 |
| 5' 0" x 7' 6" | 28·713 | 1·449 | 18·895 | 1·579 | 7·1000 | 1·033 |
| 5' 2" x 7' 9" | 30·665 | 1·497 | 20·176 | 1·631 | 7·5812 | 1·068 |
| 5' 4" x 8' 0" | 32·668 | 1·545 | 21·498 | 1·684 | 8·0782 | 1·102 |
| 5' 6" x 8' 3" | 34·742 | 1·593 | 22·863 | 1·736 | 8·5910 | 1·136 |
| 5' 8" x 8' 6" | 36·880 | 1·642 | 24·270 | 1·789 | 9·1196 | 1·171 |
| 5' 10" x 8' 9" | 39·081 | 1·690 | 25·718 | 1·842 | 9·6639 | 1·205 |
| 6' 0" x 9' 0" | 41·346 | 1·738 | 27·209 | 1·894 | 10·224 | 1·240 |

and Hydraulic Radii (R) in feet, for Culverts.

JACKSON'S PEGTOP SECTION.

| Dimensions | Full | | Two-thirds full | | One-third full | |
|----------------|--------|-------|-----------------|-------|----------------|-------|
| | A | R | A | R | A | R |
| 1' 0" x 1' 6" | 1.0385 | 0.268 | 0.6458 | 0.280 | 0.2422 | 0.190 |
| 1' 2" x 1' 9" | 1.4136 | 0.312 | 0.8790 | 0.326 | 0.3296 | 0.222 |
| 1' 4" x 2' 0" | 1.8463 | 0.357 | 1.1482 | 0.373 | 0.4305 | 0.254 |
| 1' 6" x 2' 3" | 2.3367 | 0.402 | 1.4531 | 0.420 | 0.5448 | 0.286 |
| 1' 8" x 2' 6" | 2.8848 | 0.447 | 1.7929 | 0.466 | 0.6504 | 0.317 |
| 1' 10" x 2' 9" | 3.4906 | 0.492 | 2.1152 | 0.513 | 0.8134 | 0.349 |
| 2' 0" x 3' 0" | 4.1542 | 0.536 | 2.5834 | 0.560 | 0.9686 | 0.381 |
| 2' 2" x 3' 3" | 4.8735 | 0.580 | 3.0317 | 0.606 | 1.1355 | 0.412 |
| 2' 4" x 3' 6" | 5.6542 | 0.624 | 3.5162 | 0.653 | 1.3186 | 0.444 |
| 2' 6" x 3' 9" | 6.4909 | 0.669 | 4.0340 | 0.699 | 1.5134 | 0.476 |
| 2' 8" x 4' 0" | 7.3851 | 0.714 | 4.5928 | 0.746 | 1.7220 | 0.508 |
| 2' 10" x 4' 3" | 8.3371 | 0.759 | 5.1843 | 0.793 | 1.9425 | 0.539 |
| 3' 0" x 4' 6" | 9.3469 | 0.803 | 5.8126 | 0.839 | 2.1794 | 0.571 |
| 3' 2" x 4' 9" | 10.414 | 0.848 | 6.4776 | 0.886 | 2.4265 | 0.603 |
| 3' 4" x 5' 0" | 11.539 | 0.893 | 7.1716 | 0.933 | 2.6016 | 0.634 |
| 3' 6" x 5' 3" | 12.722 | 0.937 | 7.9115 | 0.979 | 2.9668 | 0.666 |
| 3' 8" x 5' 6" | 13.963 | 0.982 | 8.4608 | 1.026 | 3.2536 | 0.698 |
| 3' 10" x 5' 9" | 15.261 | 1.027 | 9.4922 | 1.072 | 3.5558 | 0.730 |
| 4' 0" x 6' 0" | 16.617 | 1.071 | 10.334 | 1.119 | 3.8744 | 0.761 |
| 4' 2" x 6' 3" | 18.030 | 1.115 | 11.215 | 1.165 | 4.2011 | 0.793 |
| 4' 4" x 6' 6" | 19.501 | 1.160 | 12.127 | 1.212 | 4.5420 | 0.825 |
| 4' 6" x 6' 9" | 21.030 | 1.205 | 13.078 | 1.259 | 4.9032 | 0.856 |
| 4' 8" x 7' 0" | 22.617 | 1.249 | 14.065 | 1.305 | 5.2744 | 0.888 |
| 4' 10" x 7' 3" | 24.261 | 1.294 | 15.091 | 1.352 | 5.6529 | 0.920 |
| 5' 0" x 7' 6" | 25.964 | 1.339 | 16.136 | 1.399 | 6.0538 | 0.952 |
| 5' 2" x 7' 9" | 27.723 | 1.384 | 17.244 | 1.445 | 6.4595 | 0.983 |
| 5' 4" x 8' 0" | 29.540 | 1.428 | 18.371 | 1.492 | 6.8440 | 1.015 |
| 5' 6" x 8' 3" | 31.416 | 1.472 | 19.537 | 1.539 | 7.3206 | 1.047 |
| 5' 8" x 8' 6" | 33.348 | 1.517 | 20.737 | 1.585 | 7.7700 | 1.078 |
| 5' 10" x 8' 9" | 35.339 | 1.562 | 21.981 | 1.632 | 8.2340 | 1.110 |
| 6' 0" x 9' 0" | 37.388 | 1.607 | 23.250 | 1.679 | 8.7175 | 1.142 |

TABLE VI.—HYDRAULIC SLOPES AND GRADIENTS.

- Part 1. Reduction of hydraulic slopes and inclinations.
- Part 2. Reduction of angular declivities and gradients.
- Part 3. Limiting Inclinations, Maximum Gradients, Angles of Repose.

PART I.—*Reduction of hydraulic slopes.*

| S per thousand | One in | Feet per mile | S per thousand | One in | Feet per mile |
|----------------|---------|---------------|----------------|--------|---------------|
| 0.01 | 100 000 | 0.0528 | 1 | 1000 | 5.28 |
| 0.02 | 50 000 | 0.1056 | 1.25 | 800 | 6.60 |
| 0.03 | 33 333 | 0.1584 | 1.5 | 666 | 7.92 |
| 0.04 | 25 000 | 0.2112 | 1.75 | 571 | 9.24 |
| 0.05 | 20 000 | 0.2640 | 2 | 500 | 10.56 |
| 0.06 | 16 666 | 0.3168 | 2.25 | 444 | 11.88 |
| 0.07 | 14 286 | 0.3696 | 2.5 | 400 | 13.20 |
| 0.08 | 12 500 | 0.4224 | 2.75 | 364 | 14.52 |
| 0.09 | 11 111 | 0.4752 | 3 | 333 | 15.84 |
| | | | 3.25 | 308 | 16.66 |
| 0.1 | 10 000 | 0.528 | 3.5 | 286 | 18.48 |
| 0.15 | 6 666 | 0.792 | 3.75 | 266 | 19.80 |
| 0.2 | 5 000 | 1.056 | 4 | 250 | 21.12 |
| 0.25 | 4 000 | 1.320 | 4.25 | 235 | 22.44 |
| 0.3 | 3 333 | 1.584 | 4.5 | 222 | 23.76 |
| 0.35 | 2 857 | 1.848 | 4.75 | 210 | 25.08 |
| 0.4 | 2 500 | 2.112 | 5 | 200 | 26.40 |
| 0.45 | 2 222 | 2.376 | | | |
| 0.5 | 2 000 | 2.640 | 6 | 167 | 31.68 |
| | | | 7 | 143 | 36.96 |
| 0.55 | 1 818 | 2.904 | 8 | 125 | 42.25 |
| 0.6 | 1 666 | 3.168 | 9 | 111 | 47.52 |
| 0.65 | 1 538 | 3.332 | | | |
| 0.7 | 1 429 | 3.696 | 10 | 100 | 52.80 |
| 0.75 | 1 333 | 3.960 | 20 | 50 | 105.6 |
| 0.8 | 1 250 | 4.224 | 30 | 33 | 158.4 |
| 0.85 | 1 176 | 4.488 | 40 | 25 | 211.2 |
| 0.9 | 1 111 | 4.752 | 50 | 20 | 264.0 |
| 0.95 | 1 053 | 5.016 | | | |

PART I (continued).

| One in | S per thousand | Feet per mile | One in | S per thousand | Feet per mile |
|---------|----------------|---------------|--------|----------------|---------------|
| 100 000 | 0·0100 | 0·0528 | 1 000 | 1· | 5·280 |
| 90 000 | 0·0111 | 0·0587 | 900 | 1·111 | 5·866 |
| 80 000 | 0·0125 | 0·0660 | 800 | 1·250 | 6·6 |
| 70 000 | 0·0143 | 0·0754 | 700 | 1·428 | 7·54 |
| 60 000 | 0·0167 | 0·0880 | 600 | 1·666 | 8·8 |
| 50 000 | 0·0200 | 0·1056 | 500 | 2· | 10·56 |
| 40 000 | 0·0250 | 0·1320 | 400 | 2·5 | 13·20 |
| 30 000 | 0·0333 | 0·1760 | 300 | 3·333 | 17·60 |
| 20 000 | 0·0500 | 0·2640 | 200 | 5· | 26·40 |
| 10 000 | 0·1000 | 0·5280 | 190 | 5·263 | 27·78 |
| 9 500 | 0·1053 | 0·5557 | 180 | 5·555 | 29·33 |
| 9 000 | 0·1111 | 0·5866 | 170 | 5·882 | 31·05 |
| 8 500 | 0·1177 | 0·6211 | 160 | 6·250 | 33· |
| 8 000 | 0·1250 | 0·6600 | 150 | 6·667 | 35·20 |
| 7 500 | 0·1333 | 0·7040 | 140 | 7·143 | 37·71 |
| 7 000 | 0·1428 | 0·7543 | 130 | 7·692 | 40·60 |
| 6 500 | 0·1539 | 0·8123 | 120 | 8·333 | 44· |
| 6 000 | 0·1666 | 0·8800 | 110 | 9·091 | 48· |
| 5 500 | 0·1818 | 0·9600 | 100 | 10· | 52·80 |
| 5 000 | 0·2 | 1·0560 | 90 | 11·111 | 58·66 |
| 4 500 | 0·2222 | 1·1733 | 80 | 12·5 | 66· |
| 4 000 | 0·25 | 1·3200 | 70 | 14·286 | 75·42 |
| 3 500 | 0·2856 | 1·5086 | 60 | 16·667 | 88· |
| 3 000 | 0·3333 | 1·7600 | 50 | 20· | 105·6 |
| 2 500 | 0·4 | 2·1120 | 40 | 25· | 132· |
| 2 000 | 0·5 | 2·6400 | 30 | 33·333 | 176· |
| 1 500 | 0·6666 | 3·5200 | | | |

PART 2.—*Reduction of gradients.*

| Angle in degrees | Ratio to one vertical | Reduction of 100 feet horizontal | Angle in degrees | Ratio to one vertical | Reduction of 100 feet horizontal |
|------------------|-----------------------|----------------------------------|------------------|-----------------------|----------------------------------|
| 1° | 57·29 | 100·02 | 21° | 2·61 | 107·11 |
| 1½ | 38·19 | 100·03 | 22 | 2·48 | 107·85 |
| 2 | 28·64 | 100·06 | 23 | 2·36 | 108·64 |
| 2½ | 22·90 | 100·10 | 24 | 2·25 | 109·46 |
| 3 | 19·08 | 100·14 | 25 | 2·15 | 110·34 |
| 3½ | 16·35 | 100·19 | 26 | 2·05 | 111·26 |
| 4 | 14·30 | 100·24 | 27 | 1·96 | 112·23 |
| 4½ | 12·71 | 100·31 | 28 | 1·88 | 113·26 |
| 5 | 11·43 | 100·38 | 29 | 1·80 | 114·34 |
| 5½ | 10·39 | 100·46 | 30 | 1·73 | 115·47 |
| 6 | 9·51 | 100·55 | 31 | 1·66 | 116·66 |
| 6½ | 8·78 | 100·65 | 32 | 1·60 | 117·92 |
| 7 | 8·14 | 100·75 | 33 | 1·54 | 119·24 |
| 7½ | 7·60 | 100·86 | 34 | 1·48 | 120·62 |
| 8 | 7·12 | 100·98 | 35 | 1·43 | 122·08 |
| 8½ | 6·69 | 101·11 | 36 | 1·38 | 123·61 |
| 9 | 6·31 | 101·25 | 37 | 1·33 | 125·21 |
| 9½ | 5·98 | 101·39 | 38 | 1·28 | 126·90 |
| 10 | 5·67 | 101·54 | 39 | 1·24 | 128·68 |
| 11 | 5·15 | 101·87 | 40 | 1·19 | 130·54 |
| 12 | 4·71 | 102·23 | 41 | 1·15 | 132·51 |
| 13 | 4·33 | 102·63 | 42 | 1·11 | 134·56 |
| 14 | 4·01 | 103·06 | 43 | 1·07 | 136·73 |
| 15 | 3·73 | 103·53 | 44 | 1·04 | 139·02 |
| 16 | 3·49 | 104·03 | 45 | 1· | 141·4 |
| 17 | 3·27 | 104·57 | | | |
| 18 | 3·08 | 105·15 | 50 | 0·84 | 155·6 |
| 19 | 2·90 | 105·76 | 55 | 0·70 | 174·3 |
| 20 | 2·75 | 106·42 | 60 | 0·58 | 200· |

PART 2 (*continued*).

| Ratio to one vertical | Angle | Reduction of 100 feet horizontal | Ratio to one vertical | Angle | Reduction of 100 feet horizontal |
|-----------------------|--------|----------------------------------|-----------------------|-------|----------------------------------|
| 100 | 0° 34' | 100·01 | 9·5 | 6° 1' | 100·55 |
| 60 | 0 57 | 100·01 | 9· | 6 20 | 100·61 |
| 55 | 1 2 | 100·02 | 8·5 | 6 43 | 100·69 |
| 50 | 1 9 | 100·02 | 8· | 7 8 | 100·78 |
| 45 | 1 16 | 100·02 | 7·5 | 7 36 | 100·88 |
| 40 | 1 26 | 100·03 | 7· | 8 8 | 101·01 |
| 35 | 1 36 | 100·04 | 6·75 | 8 26 | 101·09 |
| 30 | 1 55 | 100·06 | 6·5 | 8 45 | 101·17 |
| 29 | 1 58 | 100·06 | 6·25 | 9 5 | 101·27 |
| 28 | 2 3 | 100·06 | 6· | 9 28 | 101·38 |
| 27 | 2 7 | 100·07 | 5·75 | 9 52 | 101·50 |
| 26 | 2 12 | 100·07 | 5·5 | 10 18 | 101·64 |
| 25 | 2 17 | 100·08 | 5·25 | 10 45 | 101·78 |
| 24 | 2 23 | 100·09 | 5· | 11 19 | 101·99 |
| 23 | 2 29 | 100·09 | 4·75 | 11 53 | 102·19 |
| 22 | 2 36 | 100·10 | 4·5 | 12 32 | 102·44 |
| 21 | 2 44 | 100·11 | 4·25 | 13 14 | 102·73 |
| 20 | 2 52 | 100·12 | 4· | 14 2 | 103·08 |
| 19 | 3 1 | 100·14 | 3·75 | 14 56 | 103·50 |
| 18 | 3 11 | 100·15 | 3·5 | 15 57 | 104·00 |
| 17 | 3 22 | 100·17 | 3·25 | 17 6 | 104·62 |
| 16 | 3 35 | 100·20 | 3· | 18 26 | 105·41 |
| 15 | 3 49 | 100·22 | 2·75 | 19 59 | 106·41 |
| 14 | 4 5 | 100·25 | 2·5 | 21 48 | 107·70 |
| 13 | 4 24 | 100·30 | 2·25 | 23 58 | 109·43 |
| 12 | 4 46 | 100·34 | 2· | 26 34 | 111·80 |
| 11 | 5 12 | 100·41 | 1·75 | 29 45 | 115·18 |
| 10 | 5 43 | 100·50 | 1·5 | 33 41 | 120·17 |
| | | | 1·25 | 38 40 | 128·08 |
| | | | 1· | 45 0 | 141·4 |

PART 3.—*Various Slopes and Gradients.*

ORDINARY LIMITS OF INCLINATION IN CHANNELS.

| | | |
|---------------------|---|--|
| Reciprocal of slope | | |
| 1 in 500 000 | | Least canal slope to produce motion. |
| 1 in 16 000 | } | Limits of tidal navigation for large canals. |
| 1 in 6 000 | | |
| 1 in 15 000 | } | Fall of most deltaic or inundation canals. |
| 1 in 5 000 | | |
| 1 in 6 000 | } | Fall of most canals. |
| 1 in 2 000 | | |
| 1 in 3 000 | } | Fall of smaller canals, channels. |
| 1 in 1 000 | | |
| 1 in 5 000 | } | Fall of most rivers. |
| 1 in 500 | | |
| 1 in 300 | } | Fall of torrents. |
| 1 in 80 | | |

VARIOUS GRADIENTS.

For sewerage unaided by flushing.

| | | |
|----------|------------------|------------------------------|
| 1 in 250 | Sewers and mains | } minima usual. |
| 1 in 50 | Pipes and drains | |
| 1 in 25 | House drains | |
| 1 in 600 | } | limits for sewers generally. |
| to | | |
| 1 in 200 | | |

MAXIMUM GRADIENTS.

| | |
|---------|-----------------------|
| 1 in 50 | Ordinary railways. |
| 1 in 30 | Turnpike road. |
| 1 in 20 | Public road. |
| 1 in 16 | Private road. |
| 1 in 8 | For wheeled vehicles. |
| 1 in 4 | Beasts of burden. |
| 1 in 1½ | Hill-walking. |

ANGLES OF REPOSE.

| | | | | |
|--------|------|------|------|--------------------------------------|
| ½ to 1 | to 1 | to 1 | to 1 | Chalk ; dry clay. |
| 1 | to 1 | | | { Compact earth, dry set, rubble. |
| 1¼ | to 1 | | | { Gravel, shingle, dry sand. |
| 1½ | to 1 | | | { Average mixed earth, dry. |
| 1¾ | to 1 | | | { Vegetable earth, dry. |
| 2 | to 1 | | | Sand, dry. |
| 3 | to 1 | to 4 | to 1 | Wet clay, peat. |

N.B.—Wetted soil requires a less slope than dry soil generally.

VARIOUS SLOPES.

| | |
|---------|---|
| 2 to 1 | Minimum for slated and tiled roofs. |
| 2½ to 1 | Maximum for back slopes of rammed earthen dams. |
| ½ to 1 | Maximum for breast slopes of rammed earthen dams. |

TABLE VII.—CANALS AND CHANNELS.

Approximate velocities of discharge for canals, channels, and straight regular reaches of rivers, for various hydraulic mean radii (R) and slopes (S) according to the formula—

$$V = c \times 100 (R \cdot S)^{\frac{1}{2}} \text{ when } c = 1.$$

Part 1. When the hydraulic slope is represented by a ratio in the old form of a fall of unity in a certain length.

Part 2. When the hydraulic slope is represented by S , the sine of the slope ; and S per 1000 is the fall in 1000 feet.

Part 3. Conditions and dimensions of equal-discharging channels of trapezoidal section, with side slopes of 1 to one, under a coefficient of rugosity $n = 0.025$.

N.B.—For the use of co-efficients (c) and (n), see Table XII.

PART I.—Values of the expression $100 \sqrt{RS}$.

| R in feet | For hydraulic slopes of one in | | | | | | | | | |
|--------------|--|-------|-------|-------|-------|-------|-------|-------|-------|--------|
| | 1000 | 2000 | 3000 | 4000 | 5000 | 6000 | 7000 | 8000 | 9000 | 10 000 |
| | Approximate velocities of discharge in feet per second | | | | | | | | | |
| .05 | .707 | .5 | .409 | .353 | .316 | .289 | .267 | .25 | .236 | .224 |
| .1 | 1. | .707 | .577 | .5 | .447 | .408 | .378 | .353 | .333 | .316 |
| .15 | 1.225 | .866 | .707 | .612 | .547 | .5 | .463 | .433 | .408 | .387 |
| .20 | 1.414 | 1. | .816 | .707 | .632 | .577 | .534 | .5 | .471 | .477 |
| .25 | 1.581 | 1.118 | .913 | .790 | .707 | .645 | .597 | .559 | .527 | .5 |
| .3 | 1.732 | 1.225 | .999 | .866 | .775 | .706 | .655 | .612 | .577 | .548 |
| .35 | 1.871 | 1.323 | 1.801 | .935 | .837 | .764 | .707 | .661 | .624 | .592 |
| .4 | 2. | 1.414 | 1.154 | 1. | .894 | .816 | .756 | .707 | .666 | .632 |
| .45 | 2.121 | 1.500 | 1.224 | 1.060 | .949 | .865 | .802 | .750 | .707 | .671 |
| .5 | 2.236 | 1.581 | 1.290 | 1.118 | 1. | .912 | .845 | .790 | .745 | .707 |
| .55 | 2.345 | 1.658 | 1.354 | 1.172 | 1.049 | .957 | .886 | .829 | .782 | .742 |
| .6 | 2.449 | 1.732 | 1.414 | 1.224 | 1.095 | 1. | .926 | .866 | .816 | .775 |
| .65 | 2.550 | 1.803 | 1.472 | 1.275 | 1.140 | 1.041 | .964 | .901 | .850 | .806 |
| .7 | 2.646 | 1.871 | 1.528 | 1.323 | 1.183 | 1.080 | 1. | .935 | .882 | .837 |
| .75 | 2.739 | 1.936 | 1.581 | 1.369 | 1.225 | 1.118 | 1.035 | .968 | .913 | .866 |
| .8 | 2.828 | 2. | 1.633 | 1.414 | 1.265 | 1.155 | 1.069 | 1. | .943 | .894 |
| .85 | 2.915 | 2.062 | 1.683 | 1.457 | 1.304 | 1.190 | 1.101 | 1.031 | .972 | .922 |
| .9 | 3. | 2.121 | 1.732 | 1.5 | 1.342 | 1.225 | 1.132 | 1.060 | 1. | .949 |
| .95 | 3.082 | 2.179 | 1.779 | 1.541 | 1.378 | 1.257 | 1.164 | 1.089 | 1.027 | .975 |
| 1.00 | 3.162 | 2.236 | 1.826 | 1.581 | 1.414 | 1.283 | 1.195 | 1.118 | 1.054 | 1. |
| 1.1 | 3.317 | 2.345 | 1.915 | 1.658 | 1.483 | 1.354 | 1.254 | 1.172 | 1.106 | 1.049 |
| 1.2 | 3.464 | 2.449 | 2. | 1.732 | 1.549 | 1.414 | 1.310 | 1.224 | 1.155 | 1.095 |
| 1.3 | 3.606 | 2.550 | 2.082 | 1.803 | 1.612 | 1.472 | 1.363 | 1.275 | 1.202 | 1.140 |
| 1.4 | 3.742 | 2.646 | 2.160 | 1.871 | 1.673 | 1.527 | 1.414 | 1.323 | 1.247 | 1.183 |
| 1.5 | 3.873 | 2.739 | 2.236 | 1.936 | 1.732 | 1.581 | 1.464 | 1.366 | 1.291 | 1.225 |
| 1.6 | 4. | 2.828 | 2.309 | 2. | 1.789 | 1.632 | 1.511 | 1.414 | 1.333 | 1.265 |
| 1.7 | 4.123 | 2.915 | 2.380 | 2.061 | 1.844 | 1.683 | 1.558 | 1.457 | 1.374 | 1.304 |
| 1.8 | 4.243 | 3. | 2.449 | 2.121 | 1.897 | 1.731 | 1.604 | 1.5 | 1.414 | 1.342 |
| 1.9 | 4.359 | 3.082 | 2.517 | 2.179 | 1.949 | 1.779 | 1.648 | 1.541 | 1.453 | 1.378 |
| 2. | 4.472 | 3.162 | 2.582 | 2.236 | 2. | 1.825 | 1.691 | 1.581 | 1.461 | 1.414 |
| 2.1 | 4.583 | 3.240 | 2.646 | 2.291 | 2.049 | 1.871 | 1.732 | 1.620 | 1.528 | 1.449 |
| 2.2 | 4.690 | 3.317 | 2.707 | 2.345 | 2.098 | 1.914 | 1.773 | 1.658 | 1.563 | 1.483 |
| 2.3 | 4.796 | 3.391 | 2.769 | 2.398 | 2.145 | 1.958 | 1.812 | 1.695 | 1.599 | 1.517 |
| 2.4 | 4.899 | 3.464 | 2.828 | 2.449 | 2.191 | 1.999 | 1.852 | 1.732 | 1.633 | 1.549 |
| 2.5 | 5. | 3.536 | 2.886 | 2.5 | 2.236 | 2.040 | 1.889 | 1.768 | 1.666 | 1.581 |
| 2.6 | 5.099 | 3.606 | 2.943 | 2.549 | 2.280 | 2.081 | 1.927 | 1.803 | 1.699 | 1.612 |
| 2.7 | 5.196 | 3.674 | 3. | 2.598 | 2.324 | 2.121 | 1.964 | 1.837 | 1.732 | 1.643 |
| 2.8 | 5.292 | 3.742 | 3.055 | 2.646 | 2.366 | 2.160 | 2. | 1.871 | 1.764 | 1.673 |
| 2.9 | 5.385 | 3.808 | 3.109 | 2.692 | 2.408 | 2.198 | 2.035 | 1.904 | 1.795 | 1.703 |
| 3. | 5.477 | 3.873 | 3.163 | 2.738 | 2.449 | 2.236 | 2.070 | 1.936 | 1.826 | 1.732 |

PART I (continued).

| R in feet | For hydraulic slopes of one in | | | | | | | | | |
|--------------|--|-------|-------|-------|-------|-------|-------|-------|-------|--------|
| | 1000 | 2000 | 3000 | 4000 | 5000 | 6000 | 7000 | 8000 | 9000 | 10 000 |
| | Approximate velocities of discharge in feet per second | | | | | | | | | |
| 3.1 | 5.568 | 3.937 | 3.215 | 2.784 | 2.490 | 2.273 | 2.105 | 1.968 | 1.856 | 1.761 |
| 3.2 | 5.657 | 4. | 3.266 | 2.828 | 2.530 | 2.309 | 2.138 | 2. | 1.886 | 1.789 |
| 3.3 | 5.745 | 4.062 | 3.317 | 2.872 | 2.569 | 2.345 | 2.172 | 2.031 | 1.915 | 1.817 |
| 3.4 | 5.831 | 4.123 | 3.367 | 2.915 | 2.608 | 2.382 | 2.204 | 2.061 | 1.944 | 1.844 |
| 3.5 | 5.916 | 4.183 | 3.416 | 2.958 | 2.646 | 2.415 | 2.236 | 2.092 | 1.972 | 1.871 |
| 3.6 | 6. | 4.243 | 3.464 | 3. | 2.683 | 2.449 | 2.267 | 2.121 | 2. | 1.897 |
| 3.7 | 6.083 | 4.301 | 3.512 | 3.041 | 2.720 | 2.483 | 2.299 | 2.150 | 2.028 | 1.924 |
| 3.8 | 6.164 | 4.359 | 3.559 | 3.082 | 2.757 | 2.516 | 2.330 | 2.179 | 2.055 | 1.949 |
| 3.9 | 6.245 | 4.416 | 3.606 | 3.122 | 2.793 | 2.548 | 2.360 | 2.208 | 2.082 | 1.975 |
| 4. | 6.325 | 4.472 | 3.651 | 3.162 | 2.828 | 2.581 | 2.390 | 2.236 | 2.108 | 2. |
| 4.1 | 6.403 | 4.528 | 3.696 | 3.202 | 2.864 | 2.613 | 2.421 | 2.264 | 2.134 | 2.025 |
| 4.2 | 6.481 | 4.583 | 3.741 | 3.240 | 2.898 | 2.645 | 2.450 | 2.291 | 2.160 | 2.049 |
| 4.3 | 6.557 | 4.637 | 3.786 | 3.278 | 2.933 | 2.680 | 2.480 | 2.318 | 2.186 | 2.074 |
| 4.4 | 6.633 | 4.690 | 3.829 | 3.316 | 2.966 | 2.707 | 2.507 | 2.345 | 2.211 | 2.098 |
| 4.5 | 6.708 | 4.743 | 3.873 | 3.354 | 3. | 2.738 | 2.535 | 2.371 | 2.236 | 2.121 |
| 4.6 | 6.782 | 4.796 | 3.916 | 3.391 | 3.033 | 2.769 | 2.564 | 2.398 | 2.261 | 2.145 |
| 4.7 | 6.856 | 4.848 | 3.958 | 3.428 | 3.066 | 2.798 | 2.591 | 2.424 | 2.285 | 2.168 |
| 4.8 | 6.928 | 4.899 | 4. | 3.464 | 3.098 | 2.828 | 2.619 | 2.449 | 2.309 | 2.191 |
| 4.9 | 7. | 4.950 | 4.041 | 3.5 | 3.130 | 2.857 | 2.646 | 2.475 | 2.333 | 2.214 |
| 5. | 7.071 | 5. | 4.082 | 3.535 | 3.162 | 2.886 | 2.672 | 2.5 | 2.357 | 2.236 |
| 5.1 | 7.141 | 5.050 | 4.122 | 3.570 | 3.194 | 2.914 | 2.699 | 2.525 | 2.380 | 2.258 |
| 5.2 | 7.211 | 5.099 | 4.164 | 3.605 | 3.225 | 2.944 | 2.725 | 2.549 | 2.404 | 2.280 |
| 5.3 | 7.280 | 5.148 | 4.204 | 3.640 | 3.258 | 2.972 | 2.751 | 2.574 | 2.427 | 2.302 |
| 5.4 | 7.348 | 5.196 | 4.242 | 3.674 | 3.286 | 2.999 | 2.777 | 2.598 | 2.449 | 2.324 |
| 5.5 | 7.416 | 5.244 | 4.282 | 3.708 | 3.317 | 3.027 | 2.803 | 2.622 | 2.472 | 2.345 |
| 5.6 | 7.483 | 5.292 | 4.320 | 3.742 | 3.347 | 3.054 | 2.828 | 2.646 | 2.494 | 2.366 |
| 5.7 | 7.550 | 5.339 | 4.359 | 3.775 | 3.376 | 3.080 | 2.854 | 2.669 | 2.517 | 2.387 |
| 5.8 | 7.616 | 5.385 | 4.397 | 3.808 | 3.406 | 3.109 | 2.878 | 2.692 | 2.539 | 2.408 |
| 5.9 | 7.681 | 5.431 | 4.434 | 3.840 | 3.435 | 3.135 | 2.903 | 2.715 | 2.560 | 2.429 |
| 6. | 7.746 | 5.477 | 4.472 | 3.873 | 3.464 | 3.162 | 2.928 | 2.738 | 2.582 | 2.449 |
| 6.1 | 7.810 | 5.523 | 4.508 | 3.905 | 3.493 | 3.187 | 2.952 | 2.761 | 2.603 | 2.470 |
| 6.2 | 7.874 | 5.568 | 4.546 | 3.937 | 3.521 | 3.214 | 2.977 | 2.784 | 2.625 | 2.490 |
| 6.3 | 7.937 | 5.612 | 4.583 | 3.968 | 3.550 | 3.240 | 3. | 2.806 | 2.646 | 2.510 |
| 6.4 | 8. | 5.657 | 4.619 | 4. | 3.578 | 3.264 | 3.024 | 2.828 | 2.666 | 2.530 |
| 6.5 | 8.062 | 5.701 | 4.654 | 4.031 | 3.606 | 3.290 | 3.048 | 2.850 | 2.687 | 2.550 |
| 6.6 | 8.124 | 5.745 | 4.690 | 4.062 | 3.633 | 3.316 | 3.071 | 2.872 | 2.708 | 2.570 |
| 6.7 | 8.185 | 5.788 | 4.725 | 4.093 | 3.661 | 3.340 | 3.093 | 2.894 | 2.728 | 2.588 |
| 6.8 | 8.246 | 5.831 | 4.761 | 4.123 | 3.688 | 3.366 | 3.117 | 2.915 | 2.749 | 2.608 |
| 6.9 | 8.307 | 5.874 | 4.796 | 4.153 | 3.715 | 3.391 | 3.138 | 2.937 | 2.769 | 2.627 |
| 7. | 8.367 | 5.916 | 4.830 | 4.184 | 3.742 | 3.415 | 3.162 | 2.957 | 2.789 | 2.646 |

PART I (continued).

| R in feet | For hydraulic slopes of one in | | | | | | | | | |
|--------------|--|--------|--------|--------|--------|--------|-------|-------|-------|--------|
| | 1000 | 2000 | 3000 | 4000 | 5000 | 6000 | 7000 | 8000 | 9000 | 10 000 |
| | Approximate velocities of discharge in feet per second | | | | | | | | | |
| 7.1 | 8.426 | 5.958 | 4.865 | 4.213 | 3.768 | 3.439 | 3.185 | 2.979 | 2.809 | 2.665 |
| 7.2 | 8.485 | 6. | 4.898 | 4.242 | 3.795 | 3.463 | 3.207 | 3. | 2.828 | 2.683 |
| 7.3 | 8.544 | 6.042 | 4.933 | 4.272 | 3.821 | 3.483 | 3.229 | 3.021 | 2.848 | 2.702 |
| 7.4 | 8.602 | 6.083 | 4.966 | 4.301 | 3.847 | 3.511 | 3.252 | 3.041 | 2.867 | 2.720 |
| 7.5 | 8.660 | 6.124 | 5. | 4.330 | 3.873 | 3.535 | 3.273 | 3.062 | 2.887 | 2.739 |
| 7.6 | 8.718 | 6.164 | 5.033 | 4.359 | 3.899 | 3.558 | 3.296 | 3.082 | 2.906 | 2.757 |
| 7.7 | 8.775 | 6.205 | 5.066 | 4.387 | 3.924 | 3.582 | 3.317 | 3.102 | 2.925 | 2.775 |
| 7.8 | 8.832 | 6.245 | 5.099 | 4.416 | 3.950 | 3.605 | 3.339 | 3.122 | 2.944 | 2.793 |
| 7.9 | 8.888 | 6.285 | 5.132 | 4.444 | 3.975 | 3.628 | 3.360 | 3.142 | 2.963 | 2.811 |
| 8. | 8.944 | 6.325 | 5.163 | 4.472 | 4. | 3.650 | 3.380 | 3.162 | 2.981 | 2.828 |
| 8.1 | 9. | 6.364 | 5.196 | 4.5 | 4.025 | 3.674 | 3.400 | 3.182 | 3. | 2.846 |
| 8.2 | 9.055 | 6.403 | 5.229 | 4.527 | 4.050 | 3.697 | 3.423 | 3.201 | 3.018 | 2.863 |
| 8.3 | 9.110 | 6.442 | 5.261 | 4.555 | 4.074 | 3.719 | 3.443 | 3.221 | 3.037 | 2.881 |
| 8.4 | 9.165 | 6.481 | 5.291 | 4.582 | 4.099 | 3.741 | 3.464 | 3.240 | 3.055 | 2.898 |
| 8.5 | 9.220 | 6.519 | 5.322 | 4.610 | 4.123 | 3.763 | 3.485 | 3.259 | 3.073 | 2.915 |
| 8.6 | 9.274 | 6.557 | 5.354 | 4.637 | 4.147 | 3.785 | 3.505 | 3.278 | 3.091 | 2.933 |
| 8.7 | 9.327 | 6.595 | 5.385 | 4.663 | 4.171 | 3.807 | 3.525 | 3.297 | 3.109 | 2.950 |
| 8.8 | 9.381 | 6.633 | 5.416 | 4.690 | 4.195 | 3.829 | 3.545 | 3.316 | 3.127 | 2.966 |
| 8.9 | 9.434 | 6.671 | 5.447 | 4.717 | 4.219 | 3.851 | 3.566 | 3.335 | 3.145 | 2.983 |
| 9. | 9.487 | 6.708 | 5.477 | 4.743 | 4.243 | 3.872 | 3.586 | 3.354 | 3.162 | 3. |
| 9.1 | 9.539 | 6.745 | 5.506 | 4.769 | 4.266 | 3.893 | 3.606 | 3.372 | 3.179 | 3.017 |
| 9.2 | 9.592 | 6.782 | 5.537 | 4.796 | 4.290 | 3.915 | 3.625 | 3.391 | 3.197 | 3.033 |
| 9.3 | 9.644 | 6.819 | 5.568 | 4.822 | 4.313 | 3.936 | 3.645 | 3.409 | 3.215 | 3.050 |
| 9.4 | 9.695 | 6.856 | 5.599 | 4.847 | 4.336 | 3.958 | 3.665 | 3.428 | 3.232 | 3.066 |
| 9.5 | 9.747 | 6.892 | 5.630 | 4.873 | 4.359 | 3.980 | 3.685 | 3.446 | 3.249 | 3.082 |
| 9.6 | 9.798 | 6.928 | 5.658 | 4.899 | 4.382 | 4. | 3.704 | 3.464 | 3.266 | 3.098 |
| 9.7 | 9.849 | 6.964 | 5.686 | 4.924 | 4.405 | 4.020 | 3.723 | 3.482 | 3.283 | 3.114 |
| 9.8 | 9.899 | 7. | 5.714 | 4.949 | 4.427 | 4.039 | 3.741 | 3.5 | 3.299 | 3.130 |
| 9.9 | 9.950 | 7.036 | 5.745 | 4.975 | 4.450 | 4.060 | 3.751 | 3.518 | 3.317 | 3.146 |
| 10 | 10. | 7.071 | 5.773 | 5. | 4.472 | 4.082 | 3.778 | 3.535 | 3.333 | 3.162 |
| 15 | 12.247 | 8.660 | 7.070 | 6.123 | 5.477 | 4.998 | 4.629 | 4.330 | 4.082 | 3.873 |
| 20 | 14.142 | 10. | 8.165 | 7.071 | 6.325 | 5.773 | 5.346 | 5. | 4.714 | 4.472 |
| 25 | 15.811 | 11.180 | 9.128 | 7.905 | 7.071 | 6.453 | 5.975 | 5.590 | 5.270 | 5. |
| 30 | 17.321 | 12.247 | 10. | 8.660 | 7.746 | 7.070 | 6.546 | 6.123 | 5.773 | 5.477 |
| 35 | 18.708 | 13.229 | 10.801 | 9.354 | 8.367 | 7.636 | 7.071 | 6.614 | 6.236 | 5.916 |
| 40 | 20. | 14.412 | 11.545 | 10. | 8.944 | 8.162 | 7.559 | 7.071 | 6.666 | 6.325 |
| 45 | 21.213 | 15. | 12.247 | 10.606 | 9.487 | 8.659 | 8.017 | 7.5 | 7.071 | 6.708 |
| 50 | 22.361 | 15.811 | 12.910 | 11.180 | 10. | 9.127 | 8.456 | 7.905 | 7.454 | 7.071 |
| 60 | 24.495 | 17.321 | 14.142 | 12.242 | 10.954 | 10. | 9.258 | 8.660 | 8.165 | 7.746 |
| 70 | 26.458 | 18.708 | 15.275 | 13.229 | 11.832 | 10.799 | 10. | 9.354 | 8.819 | 8.367 |

For true velocities, apply the correct value of c . See Table XII.

PART 2.—Values of the Expression $100\sqrt{RS}$.

| R in feet | For values of S per thousand of | | | | | | |
|--------------|--|--------|--------|--------|--------|--------|--------|
| | 5.0 | 4.5 | 4.0 | 3.5 | 3.0 | 2.5 | 2.0 |
| | Approximate velocities of discharge in feet per second | | | | | | |
| 1 | 7.071 | 6.708 | 6.325 | 5.916 | 5.476 | 5. | 4.472 |
| 1.25 | 7.906 | 7.5 | 7.071 | 6.614 | 6.123 | 5.590 | 5. |
| 1.5 | 8.660 | 8.216 | 7.746 | 7.246 | 6.708 | 6.123 | 5.477 |
| 1.75 | 9.354 | 8.874 | 8.367 | 7.826 | 7.246 | 6.614 | 5.916 |
| 2 | 10. | 9.487 | 8.944 | 8.367 | 7.746 | 7.071 | 6.325 |
| 2.25 | 10.606 | 10.062 | 9.487 | 8.874 | 8.216 | 7.5 | 6.708 |
| 2.5 | 11.180 | 10.606 | 10. | 9.354 | 8.660 | 7.906 | 7.071 |
| 2.75 | 11.726 | 11.124 | 10.488 | 9.810 | 9.083 | 8.291 | 7.416 |
| 3 | 12.247 | 11.619 | 10.954 | 10.247 | 9.487 | 8.660 | 7.746 |
| 3.25 | 12.747 | 12.093 | 11.402 | 10.665 | 9.874 | 9.014 | 8.062 |
| 3.5 | 13.229 | 12.550 | 11.832 | 11.068 | 10.247 | 9.354 | 8.367 |
| 3.75 | 13.697 | 12.990 | 12.248 | 11.456 | 10.611 | 9.682 | 8.660 |
| 4 | 14.142 | 13.416 | 12.650 | 11.832 | 10.954 | 10. | 8.944 |
| 4.25 | 14.577 | 13.829 | 13.038 | 12.196 | 11.292 | 10.308 | 9.220 |
| 4.5 | 15. | 14.230 | 13.416 | 12.550 | 11.619 | 10.606 | 9.487 |
| 4.75 | 15.411 | 14.620 | 13.784 | 12.894 | 11.937 | 10.897 | 9.747 |
| 5 | 15.811 | 15. | 14.142 | 13.229 | 12.247 | 11.180 | 10. |
| 5.25 | 16.201 | 15.375 | 14.492 | 13.555 | 12.550 | 11.456 | 10.246 |
| 5.5 | 16.583 | 15.732 | 14.832 | 13.874 | 12.845 | 11.726 | 10.488 |
| 5.75 | 16.956 | 16.086 | 15.166 | 14.186 | 13.134 | 11.989 | 10.724 |
| 6 | 17.321 | 16.432 | 15.492 | 14.491 | 13.416 | 12.247 | 10.954 |
| 6.5 | 18.028 | 17.103 | 16.124 | 15.083 | 13.964 | 12.747 | 11.402 |
| 7 | 18.708 | 17.748 | 16.734 | 15.652 | 14.491 | 13.229 | 11.832 |
| 7.5 | 19.365 | 18.371 | 17.320 | 16.202 | 15. | 13.697 | 12.247 |
| 8 | 20. | 18.974 | 17.888 | 16.733 | 15.492 | 14.142 | 12.649 |
| 8.5 | 20.616 | 19.558 | 18.440 | 17.248 | 15.969 | 14.577 | 13.038 |
| 9 | 21.213 | 20.125 | 19.974 | 17.748 | 16.432 | 15. | 13.416 |
| 10 | 22.361 | 21.213 | 20. | 18.708 | 17.321 | 15.811 | 14.142 |
| 11 | 22.452 | 22.249 | 20.976 | 19.621 | 18.166 | 16.583 | 14.832 |
| 12 | 24.495 | 23.238 | 21.908 | 20.494 | 18.974 | 17.321 | 15.492 |
| 13 | 25.494 | 24.187 | 22.804 | 21.331 | 19.748 | 18.028 | 16.124 |
| 14 | 26.458 | 25.100 | 23.664 | 22.136 | 20.494 | 18.708 | 16.734 |
| 15 | 27.394 | 25.981 | 24.495 | 22.913 | 21.213 | 19.365 | 17.320 |
| 16 | 28.284 | 26.833 | 25.298 | 23.664 | 21.909 | 20. | 17.888 |
| 20 | 31.613 | 30. | 28.284 | 26.458 | 24.495 | 22.361 | 20. |

PART 2 (cont.).—Values of the Expression

| <i>R</i> in feet | For values of <i>S</i> per thousand of | | | | | | |
|---------------------|--|--------|--------|--------|--------|--------|--------|
| | 1·5 | 1·0 | 0·95 | 0·90 | 0·85 | 0·80 | 0·75 |
| | Approximate velocities of discharge in feet per second | | | | | | |
| 1 | 3·873 | 3·162 | 3·082 | 3· | 2·915 | 2·828 | 2·738 |
| 1·25 | 4·330 | 3·536 | 3·446 | 3·354 | 3·259 | 3·162 | 3·062 |
| 1·5 | 4·743 | 3·873 | 3·775 | 3·674 | 3·571 | 3·464 | 3·354 |
| 1·75 | 5·123 | 4·183 | 4·077 | 3·969 | 3·857 | 3·742 | 3·623 |
| 2 | 5·476 | 4·472 | 4·359 | 4·243 | 4·123 | 4· | 3·873 |
| 2·25 | 5·809 | 4·743 | 4·623 | 4·5 | 4·373 | 4·242 | 4·108 |
| 2·5 | 6·123 | 5· | 4·873 | 4·743 | 4·610 | 4·472 | 4·330 |
| 2·75 | 6·423 | 5·244 | 5·111 | 4·975 | 4·835 | 4·690 | 4·542 |
| 3 | 6·708 | 5·477 | 5·339 | 5·196 | 5·050 | 4·898 | 4·743 |
| 3·25 | 6·982 | 5·701 | 5·556 | 5·408 | 5·256 | 5·098 | 4·937 |
| 3·5 | 7·246 | 5·916 | 5·766 | 5·612 | 5·454 | 5·292 | 5·123 |
| 3·75 | 7·5 | 6·124 | 5·969 | 5·809 | 5·646 | 5·477 | 5·303 |
| 4 | 7·746 | 6·325 | 6·164 | 6· | 5·831 | 5·657 | 5·476 |
| 4·25 | 7·984 | 6·519 | 6·354 | 6·185 | 6·010 | 5·830 | 5·646 |
| 4·5 | 8·216 | 6·708 | 6·538 | 6·364 | 6·185 | 6· | 5·809 |
| 4·75 | 8·441 | 6·892 | 6·718 | 6·538 | 6·354 | 6·164 | 5·969 |
| 5 | 8·660 | 7·071 | 6·892 | 6·708 | 6·519 | 6·325 | 6·123 |
| 5·25 | 8·874 | 7·246 | 7·062 | 6·874 | 6·680 | 6·482 | 6·275 |
| 5·5 | 9·083 | 7·416 | 7·228 | 7·036 | 6·827 | 6·633 | 6·423 |
| 5·75 | 9·287 | 7·583 | 7·391 | 7·194 | 6·991 | 6·782 | 6·567 |
| 6 | 9·487 | 7·746 | 7·550 | 7·348 | 7·141 | 6·928 | 6·708 |
| 6·5 | 9·874 | 8·062 | 7·858 | 7·649 | 7·433 | 7·211 | 6·982 |
| 7 | 10·247 | 8·367 | 8·155 | 7·937 | 7·714 | 7·484 | 7·246 |
| 7·5 | 10·611 | 8·660 | 8·441 | 8·216 | 7·984 | 7·746 | 7·5 |
| 8 | 10·954 | 8·944 | 8·718 | 8·485 | 8·246 | 8· | 7·746 |
| 8·5 | 11·292 | 9·220 | 8·986 | 8·746 | 8·500 | 8·246 | 7·984 |
| 9 | 11·619 | 9·487 | 9·247 | 9· | 8·746 | 8·486 | 8·216 |
| 10 | 12·247 | 10· | 9·747 | 9·487 | 9·220 | 8·944 | 8·660 |
| 11 | 12·845 | 10·488 | 10·223 | 9·950 | 9·670 | 9·381 | 9·083 |
| 12 | 13·416 | 10·954 | 10·667 | 10·392 | 10·100 | 9·797 | 9·487 |
| 13 | 13·964 | 11·402 | 11·113 | 10·817 | 10·512 | 10·198 | 9·874 |
| 14 | 14·491 | 11·832 | 11·533 | 11·225 | 10·909 | 10·583 | 10·247 |
| 15 | 15· | 12·247 | 11·938 | 11·619 | 11·292 | 10·954 | 10·611 |
| 16 | 15·492 | 12·649 | 12·329 | 12· | 11·662 | 11·314 | 10·954 |
| 20 | 17·321 | 14·142 | 13·784 | 13·416 | 13·038 | 12·650 | 12·247 |

$100\sqrt{RS}$, suitable to Canals and Channels.

| R in feet | For values of S per thousand of | | | | | | |
|----------------|--|--------|--------|--------|-------|-------|-------|
| | 0.70 | 0.65 | 0.60 | 0.55 | 0.50 | 0.45 | 0.40 |
| | Approximate velocities of discharge in feet per second | | | | | | |
| 1 | 2.646 | 2.550 | 2.449 | 2.345 | 2.236 | 2.121 | 2. |
| 1.25 | 2.958 | 2.850 | 2.739 | 2.622 | 2.5 | 2.372 | 2.236 |
| 1.5 | 3.240 | 3.122 | 3. | 2.872 | 2.739 | 2.598 | 2.449 |
| 1.75 | 3.500 | 3.372 | 3.240 | 3.102 | 2.958 | 2.806 | 2.646 |
| 2. | 3.742 | 3.606 | 3.464 | 3.317 | 3.162 | 3. | 2.828 |
| 2.25 | 3.969 | 3.824 | 3.674 | 3.518 | 3.354 | 3.182 | 3. |
| 2.5 | 4.183 | 4.031 | 3.873 | 3.708 | 3.536 | 3.354 | 3.162 |
| 2.75 | 4.387 | 4.228 | 4.062 | 3.889 | 3.708 | 3.518 | 3.317 |
| 3. | 4.583 | 4.416 | 4.243 | 4.062 | 3.873 | 3.674 | 3.464 |
| 3.25 | 4.770 | 4.596 | 4.416 | 4.228 | 4.031 | 3.824 | 3.606 |
| 3.5 | 4.950 | 4.769 | 4.583 | 4.387 | 4.183 | 3.969 | 3.742 |
| 3.75 | 5.123 | 4.937 | 4.743 | 4.541 | 4.330 | 4.108 | 3.873 |
| 4. | 5.292 | 5.099 | 4.899 | 4.690 | 4.472 | 4.243 | 4. |
| 4.25 | 5.454 | 5.256 | 5.050 | 4.835 | 4.610 | 4.373 | 4.124 |
| 4.5 | 5.612 | 5.408 | 5.196 | 4.975 | 4.743 | 4.5 | 4.243 |
| 4.75 | 5.766 | 5.557 | 5.339 | 5.111 | 4.873 | 4.623 | 4.358 |
| 5. | 5.916 | 5.701 | 5.477 | 5.244 | 5. | 4.743 | 4.472 |
| 5.25 | 6.062 | 5.842 | 5.612 | 5.374 | 5.123 | 4.861 | 4.572 |
| 5.5 | 6.205 | 5.979 | 5.744 | 5.500 | 5.244 | 4.975 | 4.690 |
| 5.75 | 6.344 | 6.114 | 5.874 | 5.624 | 5.362 | 5.087 | 4.796 |
| 6. | 6.481 | 6.245 | 6. | 5.745 | 5.477 | 5.196 | 4.898 |
| 6.5 | 6.745 | 6.5 | 6.245 | 5.979 | 5.701 | 5.408 | 5.098 |
| 7. | 7. | 6.745 | 6.480 | 6.205 | 5.916 | 5.612 | 5.292 |
| 7.5 | 7.246 | 6.982 | 6.708 | 6.423 | 6.124 | 5.809 | 5.477 |
| 8. | 7.483 | 7.211 | 6.928 | 6.633 | 6.325 | 6. | 5.657 |
| 8.5 | 7.714 | 7.433 | 7.141 | 6.837 | 6.519 | 6.185 | 5.830 |
| 9. | 7.937 | 7.649 | 7.348 | 7.036 | 6.708 | 6.364 | 6. |
| 10. | 8.367 | 8.062 | 7.746 | 7.416 | 7.071 | 6.708 | 6.325 |
| 11. | 8.775 | 8.456 | 8.124 | 7.778 | 7.416 | 7.036 | 6.633 |
| 12. | 9.165 | 8.832 | 8.486 | 8.124 | 7.746 | 7.348 | 6.928 |
| 13. | 9.539 | 9.192 | 8.832 | 8.456 | 8.062 | 7.649 | 7.211 |
| 14. | 9.899 | 9.539 | 9.165 | 8.775 | 8.367 | 7.937 | 7.484 |
| 15. | 10.247 | 9.874 | 9.486 | 9.083 | 8.660 | 8.216 | 7.746 |
| 16. | 10.583 | 10.198 | 9.798 | 9.381 | 8.944 | 8.485 | 8. |
| 20. | 11.832 | 11.402 | 10.954 | 10.488 | 10. | 9.487 | 8.944 |

For true velocities, apply the correct value of e . See Table XII.

PART 2 (continued).—Values of the Expression $100\sqrt{RS}$.

| R in feet | For values of S per thousand of | | | | | | |
|--------------|--|-------|-------|-------|-------|-------|-------|
| | 0·35 | 0·30 | 0·25 | 0·20 | 0·15 | 0·10 | 0·05 |
| | Approximate velocities of discharge in feet per second | | | | | | |
| 1 | 1·871 | 1·732 | 1·581 | 1·414 | 1·225 | 1· | 0·707 |
| 1·25 | 2·092 | 1·936 | 1·767 | 1·581 | 1·369 | 1·118 | 0·790 |
| 1·5 | 2·291 | 2·121 | 1·936 | 1·732 | 1·5 | 1·225 | 0·866 |
| 1·75 | 2·475 | 2·291 | 2·092 | 1·871 | 1·620 | 1·323 | 0·935 |
| 2 | 2·646 | 2·449 | 2·236 | 2· | 1·732 | 1·414 | 1· |
| 2·25 | 2·806 | 2·598 | 2·371 | 2·121 | 1·837 | 1·5 | 1·060 |
| 2·5 | 2·958 | 2·739 | 2·5 | 2·236 | 1·936 | 1·581 | 1·118 |
| 2·75 | 3·102 | 2·872 | 2·622 | 2·345 | 2·031 | 1·658 | 1·172 |
| 3 | 3·240 | 3· | 2·739 | 2·449 | 2·121 | 1·732 | 1·225 |
| 3·25 | 3·373 | 3·122 | 2·850 | 2·549 | 2·208 | 1·803 | 1·275 |
| 3·5 | 3·5 | 3·240 | 2·958 | 2·646 | 2·291 | 1·871 | 1·323 |
| 3·75 | 3·623 | 3·354 | 3·062 | 2·738 | 2·371 | 1·937 | 1·369 |
| 4 | 3·742 | 3·464 | 3·102 | 2·828 | 2·449 | 2· | 1·414 |
| 4·25 | 3·857 | 3·571 | 3·259 | 2·915 | 2·525 | 2·062 | 1·458 |
| 4·5 | 3·969 | 3·674 | 3·354 | 3· | 2·598 | 2·121 | 1·5 |
| 4·75 | 4·077 | 3·775 | 3·446 | 3·082 | 2·669 | 2·179 | 1·541 |
| 5 | 4·183 | 3·873 | 3·536 | 3·162 | 2·739 | 2·236 | 1·581 |
| 5·25 | 4·287 | 3·969 | 3·623 | 3·241 | 2·806 | 2·291 | 1·620 |
| 5·5 | 4·387 | 4·062 | 3·708 | 3·317 | 2·872 | 2·345 | 1·658 |
| 5·75 | 4·486 | 4·153 | 3·791 | 3·391 | 2·937 | 2·398 | 1·696 |
| 6 | 4·583 | 4·243 | 3·873 | 3·464 | 3· | 2·449 | 1·732 |
| 6·5 | 4·770 | 4·416 | 4·031 | 3·606 | 3·122 | 2·549 | 1·803 |
| 7 | 4·950 | 4·583 | 4·183 | 3·742 | 3·240 | 2·646 | 1·871 |
| 7·5 | 5·123 | 4·743 | 4·330 | 3·874 | 3·354 | 2·738 | 1·937 |
| 8 | 5·292 | 4·899 | 4·472 | 4· | 3·464 | 2·828 | 2· |
| 8·5 | 5·454 | 5·050 | 4·610 | 4·124 | 3·571 | 2·915 | 2·062 |
| 9 | 5·612 | 5·196 | 4·743 | 4·243 | 3·674 | 3· | 2·121 |
| 10 | 5·916 | 5·477 | 5· | 4·472 | 3·873 | 3·162 | 2·236 |
| 11 | 6·205 | 5·744 | 5·244 | 4·690 | 4·062 | 3·317 | 2·345 |
| 12 | 6·481 | 6· | 5·477 | 4·898 | 4·243 | 3·464 | 2·449 |
| 13 | 6·745 | 6·245 | 5·701 | 5·098 | 4·416 | 3·606 | 2·549 |
| 14 | 7· | 6·480 | 5·916 | 5·292 | 4·583 | 3·742 | 2·646 |
| 15 | 7·246 | 6·708 | 6·124 | 5·477 | 4·743 | 3·873 | 2·738 |
| 16 | 7·483 | 6·928 | 6·325 | 5·657 | 4·899 | 4· | 2·828 |
| 20 | 8·367 | 7·746 | 7·071 | 6·325 | 5·477 | 4·472 | 3·162 |

For true velocities, apply the correct value of c . See Table XII.

ART 3.—Conditions of equal discharging channels, with low mean velocities suited to earth, for Trapezoidal Sections having side-slopes of one to one, the channel being in earth, and in good average order, with a co-efficient of rugosity and irregularity, $n=0.025$.

Q is the quantity discharged; V , the mean velocity in feet per second; S per 1 000 is the fall in 1 000; b is the bed-width; d is the depth of water in feet.

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 1.0 | 1.0 | 1.5 | 1.5 | 1.5 | 2. | 2. | 2. | 3. | 3. |
| d | . . . | 1.0 | 1.5 | 0.5 | 1.0 | 1.5 | 0.5 | 0.75 | 1. | 0.5 | 0.75 |
| S per 1 000 | | 0.26 | 0.05 | 1.90 | 0.13 | 0.04 | 1.04 | 0.26 | 0.09 | 0.48 | 0.13 |
| V | . . . | 0.50 | 0.27 | 1.00 | 0.40 | 0.22 | 0.80 | 0.49 | 0.33 | 0.57 | 0.36 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 1.5 | 1.5 | 2. | 2. | 2. | 2. | 3. | 3. | 3. | 4. |
| d | . . . | 1.0 | 1.5 | 0.5 | 0.75 | 1. | 1.25 | 0.5 | 0.75 | 1. | 1. |
| S per 1 000 | | 0.47 | 0.11 | 3.74 | 0.92 | 0.33 | 0.15 | 1.9 | 0.44 | 0.17 | 0.10 |
| V | . . . | 0.80 | 0.44 | 1.60 | 0.97 | 0.67 | 0.49 | 1.14 | 0.71 | 0.50 | 0.40 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 1.5 | 2. | 2. | 2. | 2. | 3. | 3. | 3. | 4. | 5. |
| d | . . . | 2.0 | 0.75 | 1. | 1.25 | 1.5 | 0.5 | 0.75 | 1. | 1. | 1. |
| S per 1 000 | | 0.09 | 2.06 | 0.69 | 0.31 | 0.16 | 4.21 | 0.95 | 0.35 | 0.20 | 0.14 |
| V | . . . | 0.43 | 1.46 | 1. | 0.74 | 0.57 | 1.72 | 1.07 | 0.75 | 0.60 | 0.50 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 2. | 2. | 3. | 3. | 3. | 4. | 5. | 6. |
| d | . . . | 0.75 | 1. | 1.25 | 1.75 | 0.75 | 1. | 1.5 | 1. | 1. | 1. |
| S per 1 000 | | 3.73 | 1.24 | 0.52 | 0.15 | 1.71 | 0.60 | 0.15 | 0.36 | 0.24 | 0.17 |
| V | . . . | 1.94 | 1.33 | 0.99 | 0.61 | 1.42 | 1. | 0.59 | 0.80 | 0.67 | 0.57 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 2. | 3. | 3. | 3. | 4. | 4. | 5. | 6. |
| d | . . . | 1. | 1.5 | 2. | 0.75 | 1. | 1.5 | 1. | 1.5 | 1. | 1. |
| S per 1 000 | | 1.91 | 0.40 | 0.14 | 2.70 | 0.92 | 0.35 | 0.54 | 0.14 | 0.36 | 0.26 |
| V | . . . | 1.67 | 0.95 | 0.63 | 1.78 | 1.25 | 0.74 | 1.00 | 0.61 | 0.83 | 0.71 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 2. | 3. | 3. | 3. | 4. | 4. | 5. | 6. |
| d | . . . | 1. | 1.5 | 2. | 1. | 1.5 | 2. | 1. | 1.5 | 1. | 1. |
| S per 1 000 | | 2.72 | 0.58 | 0.18 | 1.35 | 0.30 | 0.10 | 0.77 | 0.19 | 0.50 | 0.36 |
| V | . . . | 2. | 1.48 | 0.75 | 1.50 | 0.89 | 0.60 | 1.20 | 0.73 | 1.00 | 0.86 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 2. | 3. | 3. | 3. | 4. | 4. | 5. | 6. |
| d | . . . | 1.25 | 1.5 | 2. | 1. | 1.5 | 2. | 1. | 1.5 | 1. | 1. |
| S per 1 000 | | 1.51 | 0.76 | 0.25 | 1.80 | 0.41 | 0.14 | 1.04 | 0.25 | 0.68 | 0.48 |
| V | . . . | 1.72 | 1.33 | 0.88 | 1.75 | 1.04 | 0.70 | 1.40 | 0.85 | 1.17 | 1.00 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 3. | 3. | 3. | 4. | 4. | 4. | 5. | 6. |
| d | . . . | 1.25 | 2. | 1. | 1.5 | 2. | 1. | 1.5 | 2. | 1. | 1. |
| S per 1 000 | | 2.04 | 0.32 | 2.35 | 0.52 | 0.18 | 1.39 | 0.33 | 0.12 | 0.88 | 0.62 |
| V | . . . | 1.97 | 1.0 | 2. | 1.18 | 0.80 | 1.60 | 0.97 | 0.67 | 1.33 | 1.14 |

| | | | | | | | | | | | |
|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| b | . . . | 2. | 2. | 3. | 3. | 4. | 4. | 4. | 5. | 6. | 8. |
| d | . . . | 1.5 | 2. | 1.5 | 2. | 1. | 1.5 | 2. | 1. | 1. | 1. |
| S per 1 000 | | 1.28 | 0.39 | 0.65 | 0.23 | 1.74 | 0.40 | 0.15 | 1.12 | 0.77 | 0.45 |
| V | . . . | 1.72 | 1.13 | 1.33 | 0.90 | 1.80 | 1.09 | 0.75 | 1.33 | 1.29 | 1.00 |

PART 3 (continued).

Q in cubic feet per second

Q is the quantity discharged; V , the mean velocity in feet per second; S per 1 000 is the fall in 1 000; b is the bed-width; d is the depth of water in feet.

| | | | | | | | | | | | | |
|----|---------------|-------|------|------|------|------|------|------|------|------|------|------|
| 10 | b | . . . | 2' | 3' | 3' | 4' | 4' | 5' | 5' | 6' | 8' | 10' |
| | d | . . . | 1.75 | 2' | 2.5 | 1.5 | 2' | 1' | 2' | 1' | 1' | 1' |
| | S per 1 000 | | 0.83 | 0.28 | 0.12 | 0.49 | 0.18 | 1.40 | 0.12 | 0.95 | 0.54 | 0.3 |
| | V | . . . | 1.52 | 1.00 | 0.73 | 1.21 | 0.83 | 1.67 | 0.71 | 1.43 | 1.11 | 0.9 |
| 12 | b | . . . | 3' | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 10' |
| | d | . . . | 2' | 2.5 | 1.5 | 2' | 1' | 2' | 1' | 2' | 1' | 1' |
| | S per 1 000 | | 0.39 | 0.17 | 0.70 | 0.25 | 1.97 | 0.17 | 1.40 | 0.13 | 0.78 | 0.5 |
| | V | . . . | 1.20 | 0.87 | 1.45 | 1.00 | 2.00 | 0.86 | 1.71 | 0.75 | 1.33 | 1.09 |
| 14 | b | . . . | 3' | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 10' |
| | d | . . . | 2' | 3' | 2' | 2.5 | 2' | 2.5 | 1' | 2' | 1' | 1' |
| | S per 1 000 | | 0.52 | 0.11 | 0.34 | 0.15 | 0.23 | 0.10 | 1.87 | 0.17 | 1.05 | 0.68 |
| | V | . . . | 1.40 | 0.78 | 1.17 | 0.86 | 1.00 | 0.75 | 2.00 | 0.88 | 1.56 | 1.27 |
| 16 | b | . . . | 3' | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 10' |
| | d | . . . | 2' | 3' | 2' | 2.5 | 2' | 2.5 | 2' | 2.5 | 1' | 1' |
| | S per 1 000 | | 0.67 | 0.14 | 0.43 | 0.19 | 0.30 | 0.13 | 0.21 | 0.10 | 1.41 | 0.88 |
| | V | . . . | 1.60 | 0.89 | 1.33 | 0.98 | 1.14 | 0.85 | 1.00 | 0.75 | 1.78 | 1.45 |
| 18 | b | . . . | 3' | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 10' |
| | d | . . . | 2' | 3' | 2' | 3' | 2' | 2.5 | 2' | 2.5 | 1' | 1' |
| | S per 1 000 | | 0.85 | 0.18 | 0.54 | 0.12 | 0.37 | 0.17 | 0.27 | 0.12 | 1.76 | 1.12 |
| | V | . . . | 1.80 | 1.00 | 1.50 | 0.86 | 1.29 | 0.96 | 1.13 | 0.85 | 2.00 | 1.64 |
| 20 | b | . . . | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 8' | 10' |
| | d | . . . | 3' | 2' | 3' | 2' | 3' | 2' | 2.5 | 1' | 2' | 1' |
| | S per 1 000 | | 0.22 | 0.66 | 0.15 | 0.45 | 0.10 | 0.33 | 0.15 | 2.14 | 0.19 | 1.88 |
| | V | . . . | 1.11 | 1.67 | 0.95 | 1.43 | 0.83 | 1.25 | 0.94 | 2.22 | 1.00 | 1.82 |
| 25 | b | . . . | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 8' | 10' |
| | d | . . . | 3' | 2' | 3' | 2' | 3' | 2' | 3' | 2' | 2.5 | 2' |
| | S per 1 000 | | 0.34 | 1.02 | 0.22 | 0.70 | 0.16 | 0.50 | 0.12 | 0.30 | 0.14 | 0.19 |
| | V | . . . | 1.39 | 2.08 | 1.19 | 1.82 | 1.04 | 1.56 | 0.93 | 1.25 | 0.95 | 1.04 |
| 30 | b | . . . | 3' | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 8' | 10' |
| | d | . . . | 3' | 3' | 4' | 2.5 | 3.5 | 2' | 3' | 2' | 3' | 2' |
| | S per 1 000 | | 0.47 | 0.32 | 0.10 | 0.44 | 0.13 | 0.72 | 0.17 | 0.42 | 0.10 | 0.28 |
| | V | . . . | 1.67 | 1.43 | 0.94 | 1.60 | 1.01 | 1.88 | 1.11 | 1.50 | 0.91 | 1.25 |
| 35 | b | . . . | 4' | 4' | 5' | 5' | 6' | 6' | 8' | 8' | 10' | 10' |
| | d | . . . | 3' | 4' | 2.5 | 4' | 2.5 | 3.5 | 2' | 3' | 2' | 3' |
| | S per 1 000 | | 0.42 | 0.14 | 0.59 | 0.10 | 0.42 | 0.13 | 0.57 | 0.14 | 0.37 | 0.09 |
| | V | . . . | 1.67 | 1.09 | 1.87 | 0.97 | 1.65 | 1.05 | 1.75 | 1.06 | 1.46 | 0.90 |
| 40 | b | . . . | 4' | 4' | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' |
| | d | . . . | 3' | 4' | 3' | 4' | 2' | 3' | 2' | 3' | 2' | 3' |
| | S per 1 000 | | 0.55 | 0.18 | 0.31 | 0.10 | 0.74 | 0.18 | 0.49 | 0.12 | 0.34 | 0.09 |
| | V | . . . | 1.90 | 1.25 | 1.48 | 1.00 | 2.00 | 1.21 | 1.67 | 1.03 | 1.43 | 0.89 |

PART 3 (continued).

Q is the quantity discharged ; V , the mean velocity in feet per second ; S per 1 000 is the fall in 1 000 ; b is the bed-width ; d is the depth of water in feet.

| | | | | | | | | | | | |
|-----|---------------|------|------|------|------|------|------|------|------|------|------|
| 50 | b | 4' | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' |
| | d | 4' | 3' | 4' | 3' | 4' | 3' | 4' | 3' | 4' | 2' |
| | S per 1 000 | 0·28 | 0·48 | 0·16 | 0·28 | 0·09 | 0·19 | 0·07 | 0·17 | 0·05 | 0·39 |
| | V | 1·56 | 1·85 | 1·25 | 1·52 | 1·04 | 1·28 | 0·89 | 1·11 | 0·78 | 1·56 |
| 60 | b | 4' | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' |
| | d | 4' | 3·5 | 5' | 3' | 4' | 3' | 4·5 | 3' | 4' | 2' |
| | S per 1 000 | 0·39 | 0·37 | 0·09 | 0·39 | 0·14 | 0·29 | 0·06 | 0·19 | 0·07 | 0·56 |
| | V | 1·88 | 1·80 | 1·09 | 1·82 | 1·25 | 1·54 | 0·92 | 1·33 | 0·94 | 1·88 |
| 70 | b | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' | 14' |
| | d | 3·5 | 5' | 3' | 4' | 3' | 5' | 3' | 4·5 | 3' | 4' |
| | S per 1 000 | 0·49 | 0·15 | 0·53 | 0·19 | 0·35 | 0·06 | 0·25 | 0·06 | 0·19 | 0·07 |
| | V | 2·11 | 1·27 | 2·12 | 1·46 | 1·80 | 0·93 | 1·56 | 0·94 | 1·37 | 0·97 |
| 80 | b | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' | 14' |
| | d | 4' | 5' | 3·5 | 5' | 3·5 | 5' | 3' | 4·5 | 3' | 4' |
| | S per 1 000 | 0·39 | 0·19 | 0·39 | 0·10 | 0·27 | 0·07 | 0·33 | 0·08 | 0·25 | 0·09 |
| | V | 2·00 | 1·45 | 1·99 | 1·23 | 1·69 | 1·07 | 1·78 | 1·08 | 1·57 | 1·11 |
| 90 | b | 6' | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' | 14' |
| | d | 4' | 5' | 4' | 5' | 3·5 | 5' | 3' | 5' | 3' | 5' |
| | S per 1 000 | 0·48 | 0·20 | 0·31 | 0·13 | 0·33 | 0·09 | 0·41 | 0·07 | 0·31 | 0·05 |
| | V | 2·25 | 1·64 | 1·88 | 1·38 | 1·90 | 1·20 | 2·00 | 1·06 | 1·76 | 0·95 |
| 100 | b | 6' | 8' | 8' | 10' | 10' | 12' | 12' | 14' | 14' | 16' |
| | d | 5' | 4' | 5' | 4' | 5' | 3·5 | 5' | 3' | 5' | 3' |
| | S per 1 000 | 0·26 | 0·38 | 0·17 | 0·27 | 0·11 | 0·29 | 0·08 | 0·38 | 0·06 | 0·29 |
| | V | 1·82 | 2·08 | 1·54 | 1·79 | 1·33 | 1·84 | 1·18 | 1·96 | 1·05 | 1·75 |
| 150 | b | 8' | 10' | 12' | 12' | 14' | 14' | 16' | 16' | 18' | 18' |
| | d | 5' | 5' | 4·5 | 5' | 4' | 6' | 4' | 6' | 4' | 5' |
| | S per 1 000 | 0·37 | 0·25 | 0·27 | 0·19 | 0·31 | 0·07 | 0·24 | 0·06 | 0·19 | 0·09 |
| | V | 2·31 | 2·00 | 2·02 | 1·76 | 2·08 | 1·25 | 1·88 | 1·14 | 1·70 | 1·30 |
| 200 | b | 12' | 14' | 14' | 16' | 16' | 18' | 18' | 20' | 20' | 25' |
| | d | 5' | 5' | 7' | 5' | 7' | 5' | 6' | 4' | 6' | 4' |
| | S per 1 000 | 0·33 | 0·25 | 0·07 | 0·19 | 0·06 | 0·16 | 0·08 | 0·28 | 0·07 | 0·18 |
| | V | 2·35 | 2·11 | 1·36 | 1·90 | 1·24 | 1·74 | 1·39 | 2·08 | 1·28 | 1·72 |
| 300 | b | 14' | 16' | 18' | 20' | 20' | 25' | 25' | 30' | 30' | 35' |
| | d | 7' | 7' | 7' | 6' | 7' | 5' | 7' | 5' | 6' | 4' |
| | S per 1 000 | 0·16 | 0·13 | 0·10 | 0·15 | 0·09 | 0·19 | 0·06 | 0·14 | 0·07 | 0·21 |
| | V | 2·04 | 1·86 | 1·71 | 1·92 | 1·59 | 2·00 | 0·89 | 1·71 | 1·39 | 1·92 |
| 400 | b | 20' | 25' | 25' | 30' | 30' | 35' | 35' | 40' | 40' | 50' |
| | d | 7' | 6' | 8' | 6' | 7' | 5' | 7' | 4' | 6' | 4' |
| | S per 1 000 | 0·16 | 0·18 | 0·06 | 0·12 | 0·07 | 0·17 | 0·05 | 0·29 | 0·07 | 0·18 |
| | V | 2·12 | 2·15 | 1·52 | 1·85 | 1·54 | 2·00 | 1·36 | 2·27 | 1·45 | 1·82 |

PART 3 (continued).

Q in cubic feet per second

Q is the quantity discharged; V , the mean velocity in feet per second; S per 1 000 is the fall in 1 000; b is the bed-width; d is the depth of water in feet.

| | | | | | | | | | | | |
|------|---------------|------|------|------|------|------|------|------|------|------|------|
| 500 | b | 25° | 30° | 30° | 35° | 35° | 40° | 40° | 50° | 50° | 60° |
| | d | 8° | 7° | 8° | 7° | 8° | 5° | 7° | 4·5 | 6·5 | 4° |
| | S per 1 000 | 0·10 | 0·11 | 0·07 | 0·08 | 0·05 | 0·20 | 0·06 | 0·19 | 0·05 | 0·20 |
| | V | 1·89 | 1·93 | 1·64 | 1·70 | 1·45 | 2·22 | 1·52 | 2·04 | 1·36 | 1·95 |
| 600 | b | 30° | 35° | 35° | 40° | 40° | 50° | 50° | 60° | 60° | 70° |
| | d | 8° | 7° | 8° | 7° | 8° | 5° | 7° | 4·5 | 6·5 | 4·5 |
| | S per 1 000 | 0·10 | 0·12 | 0·08 | 0·09 | 0·06 | 0·19 | 0·06 | 0·19 | 0·05 | 0·14 |
| | V | 1·97 | 2·04 | 1·70 | 1·82 | 1·56 | 2·18 | 1·50 | 2·07 | 1·39 | 1·79 |
| 700 | b | 35° | 40° | 50° | 50° | 60° | 60° | 70° | 70° | 80° | 80° |
| | d | 8° | 8° | 6° | 7·5 | 5·5 | 7° | 5° | 6·5 | 4·5 | 6° |
| | S per 1 000 | 0·10 | 0·08 | 0·14 | 0·06 | 0·13 | 0·05 | 0·13 | 0·05 | 0·15 | 0·05 |
| | V | 2·04 | 1·82 | 2·08 | 1·62 | 1·94 | 1·49 | 1·87 | 1·41 | 1·84 | 1·36 |
| 800 | b | 40° | 50° | 50° | 60° | 60° | 70° | 70° | 80° | 80° | 90° |
| | d | 8° | 6·5 | 8° | 6·5 | 7·5 | 5·5 | 7° | 5° | 6·5 | 5° |
| | S per 1 000 | 0·10 | 0·14 | 0·07 | 0·09 | 0·06 | 0·12 | 0·05 | 0·13 | 0·05 | 0·10 |
| | V | 2·08 | 2·18 | 1·72 | 1·85 | 1·58 | 1·93 | 1·68 | 1·88 | 1·42 | 1·68 |
| 900 | b | 50° | 50° | 50° | 60° | 70° | 70° | 80° | 80° | 90° | 90° |
| | d | 8° | 9° | 7° | 8° | 6° | 7·5 | 5·5 | 7° | 5° | 6·5 |
| | S per 1 000 | 0·08 | 0·05 | 0·09 | 0·06 | 0·11 | 0·05 | 0·12 | 0·05 | 0·13 | 0·05 |
| | V | 1·94 | 1·69 | 1·92 | 1·65 | 1·97 | 1·55 | 1·91 | 1·48 | 1·89 | 1·43 |
| 1000 | b | 50° | 60° | 60° | 70° | 70° | 80° | 80° | 90° | 90° | 100° |
| | d | 9° | 7·5 | 8° | 6·5 | 7·5 | 6° | 7° | 5·5 | 6·5 | 5·5 |
| | S per 1 000 | 0·07 | 0·09 | 0·07 | 0·11 | 0·06 | 0·11 | 0·06 | 0·12 | 0·06 | 0·09 |
| | V | 1·88 | 1·98 | 1·84 | 2·01 | 1·72 | 1·94 | 1·64 | 1·90 | 1·59 | 1·72 |
| 2000 | b | 100° | 120° | 120° | 140° | 140° | 160° | 160° | 180° | 180° | 200° |
| | d | 9° | 7·5 | 8° | 7·5 | 8° | 7° | 7·5 | 6° | 7° | 6° |
| | S per 1 000 | 0·09 | 0·09 | 0·07 | 0·06 | 0·05 | 0·07 | 0·05 | 0·08 | 0·05 | 0·07 |
| | V | 2·04 | 2·09 | 1·95 | 1·81 | 1·69 | 1·71 | 1·59 | 1·79 | 1·53 | 1·62 |
| 3000 | b | 140° | 160° | 160° | 180° | 180° | 200° | 200° | 220° | 220° | 240° |
| | d | 10° | 8° | 9° | 7° | 8° | 7° | 8° | 6·5 | 7·5 | 6·5 |
| | S per 1 000 | 0·05 | 0·09 | 0·05 | 0·11 | 0·07 | 0·09 | 0·05 | 0·10 | 0·06 | 0·08 |
| | V | 2·00 | 2·23 | 1·97 | 2·29 | 1·99 | 2·07 | 1·80 | 2·04 | 1·76 | 1·87 |
| 4000 | b | 180° | 200° | 220° | 220° | 240° | 240° | 260° | 260° | 280° | 300° |
| | d | 10° | 9° | 8° | 9° | 7·5 | 8° | 7° | 8° | 7° | 7° |
| | S per 1 000 | 0·06 | 0·07 | 0·09 | 0·05 | 0·09 | 0·07 | 0·10 | 0·06 | 0·08 | 0·07 |
| | V | 2·11 | 2·13 | 2·19 | 1·94 | 2·15 | 2·02 | 2·14 | 1·87 | 1·99 | 1·86 |
| 5000 | b | 240° | 260° | 280° | 300° | 6000 | | 280° | 300° | 7000 | |
| | d | 10° | 9° | 8° | 8° | | | 10° | 9° | 10° | |
| | S per 1 000 | 0·05 | 0·05 | 0·08 | 0·07 | | | 0·05 | 0·07 | 0·06 | |
| | V | 2·00 | 2·07 | 2·17 | 2·03 | | | 2·07 | 2·16 | 2·26 | |

These numbers serve for purposes of interpolation, but for accuracy in any special case more figures should be used in the values of S .

EXPLANATORY EXAMPLES TO TABLE VII.

EXAMPLE I.

An old canal has a hydraulic radius of 5·2 feet, a hydraulic slope of 1 in 5000 and a cross section of 1000 square feet, required the discharge, assuming a co-efficient of rugosity of ·03.

By Part 1 of Table VII. the unmodified mean velocity of discharge = 3·225 feet per second, and by Table XII. the value of c the co-efficient suitable to this radius and slope is ·66, hence the true discharge = $c \times A \times V = \cdot 66 \times 1000 \times 3 \cdot 225 = 2128$ cubic feet per second.

EXAMPLE II.

Suppose the canal mentioned in the last example to have a hydraulic slope of ·0015, the remaining data being as before, required the discharge.

In this case the fall per 1000 is 1·5, and by interpolating Part 2 of Table VII. to the hydraulic radius, 5·2 feet, an unmodified mean velocity of discharge 8·83 feet per second is obtained. Taking the suitable co-efficient c from Table XII., the true discharge = $c \times A \times V = \cdot 65 \times 1000 \times 8 \cdot 83 = 5740$ cubic feet per second.

EXAMPLE III.

A canal in earth is of trapezoidal section with side slopes of 1 to one, its bed-width is 40 feet, its depth of water 5 feet; it is to discharge 500 cubic feet per second, when in moderate average order, with a co-efficient of rugosity $n = 0 \cdot 025$. What hydraulic slope must it have?

By Part 3, Table VII., the hydraulic slope is 0·00020, or 0·20 per 1000.

EXAMPLE IV.

What will be the discharge and the mean velocity in the canal mentioned in the last example, when it has deteriorated to a condition when $n = 0 \cdot 030$?

By sectional data, Part 2, Table V., page 45, $A = 225$, and $R = 4 \cdot 155$; S remains 0·20 per 1000. Also from Part 2, Table VII., we interpolate to obtain $100 \sqrt{RS} = 2 \cdot 88$; from Table XII. we obtain, when $n = 0 \cdot 030$ for given values of R and S , $c = 0 \cdot 63$; hence $V = 0 \cdot 63 \times 2 \cdot 88 = 1 \cdot 814$ feet per second, and $Q = 1 \cdot 814 \times 225 = 408$ cubic feet per second.

For tables of velocity and discharge in canals under various values of n , see 'Canal and Culvert Tables' (London: Allen, 1878).

TABLE VIII.—PIPES AND CULVERTS, JUST FULL.

Part 1. Approximate velocities in feet per second, when $c = 1$, formula—

$$V = c \cdot 100 \sqrt{HS}.$$

Part 2. Approximate discharges in cubic feet per second, when $c = 1$, formula—

$$Q = A \cdot c \cdot 100 \sqrt{HS} = c \cdot 39 \cdot 27 \sqrt{d^5 S}.$$

Part 3. Approximate diameters in feet, when $c = 1$, formula—

$$d = \frac{1}{c^{\frac{2}{5}}} \times 0 \cdot 23 \left(\frac{Q^2}{S} \right)^{\frac{1}{5}}$$

Part 4. Approximate heads in feet for a length of 1 000 feet, when $c = 1$, formula—

$$h = \frac{1}{c^2} \cdot 0 \cdot 648 \frac{Q^5}{d^5}.$$

Part 5. Conditions of equal-discharging culverts and drain-pipes, running just full, under a co-efficient of rugosity $n = 0 \cdot 013$.

NOTE.—For correct results, apply values of c from Table XII. in Parts 1, 2, 3, and 4.

For the use of co-efficients (c) and (n) see Table XII.

PART I.—*Approximate velocities
or, Values of the Expression*

| <i>R</i> in feet | <i>S</i> per thousand | | | | | | |
|---------------------|-----------------------|--------|--------|--------|--------|--------|--------|
| | 20 | 18 | 17 | 16 | 15 | 14 | 13 |
| 0.05 | 3.162 | 3. | 2.915 | 2.828 | 2.738 | 2.648 | 2.549 |
| 0.10 | 4.472 | 4.243 | 4.123 | 4. | 3.873 | 3.742 | 3.606 |
| 0.15 | 5.477 | 5.196 | 5.050 | 4.898 | 4.743 | 4.583 | 4.416 |
| 0.20 | 6.325 | 6. | 5.831 | 5.656 | 5.476 | 5.292 | 5.099 |
| 0.25 | 7.071 | 6.708 | 6.519 | 6.325 | 6.123 | 5.916 | 5.701 |
| 0.30 | 7.746 | 7.348 | 7.141 | 6.928 | 6.708 | 6.481 | 6.245 |
| 0.35 | 8.367 | 7.937 | 7.713 | 7.484 | 7.246 | 7. | 6.745 |
| 0.40 | 8.944 | 8.485 | 8.246 | 8. | 7.746 | 7.483 | 7.211 |
| 0.45 | 9.487 | 9. | 8.746 | 8.486 | 8.216 | 7.937 | 7.647 |
| 0.50 | 10. | 9.487 | 9.220 | 8.944 | 8.660 | 8.367 | 8.062 |
| 0.6 | 10.954 | 10.392 | 10.100 | 9.797 | 9.487 | 9.165 | 8.832 |
| 0.7 | 11.832 | 11.225 | 10.909 | 10.583 | 10.247 | 9.899 | 9.539 |
| 0.8 | 12.649 | 12. | 11.662 | 11.314 | 10.954 | 10.583 | 10.198 |
| 0.9 | 13.416 | 12.728 | 12.369 | 12. | 11.619 | 11.225 | 10.817 |
| 1.0 | 14.142 | 13.416 | 13.038 | 12.649 | 12.247 | 11.832 | 11.402 |
| 1.1 | 14.832 | 14.071 | 13.675 | 13.266 | 12.845 | 12.410 | 11.958 |
| 1.2 | 15.492 | 14.697 | 14.283 | 13.856 | 13.416 | 12.961 | 12.490 |
| 1.3 | 16.125 | 15.297 | 14.866 | 14.422 | 13.964 | 13.491 | 13. |
| 1.4 | 16.733 | 15.875 | 15.427 | 14.967 | 14.491 | 14. | 13.491 |
| 1.5 | 17.321 | 16.432 | 15.969 | 15.492 | 15. | 14.491 | 13.964 |
| 1.6 | 17.889 | 16.971 | 16.492 | 16. | 15.492 | 14.967 | 14.422 |
| 1.7 | 18.439 | 17.493 | 17. | 16.492 | 15.969 | 15.427 | 14.866 |
| 1.8 | 18.974 | 18. | 17.493 | 16.971 | 16.432 | 15.875 | 15.294 |
| 1.9 | 19.494 | 18.493 | 17.972 | 17.436 | 16.882 | 16.310 | 15.716 |
| 2.0 | 20. | 18.974 | 18.439 | 17.889 | 17.321 | 16.733 | 16.125 |
| 2.1 | | | | | | | |
| 2.2 | | | | | | | |
| 2.3 | | | | | | | |
| 2.4 | | | | | | | |
| 2.5 | | | | | | | |
| 2.6 | | | | | | | |
| 2.7 | | | | | | | |
| 2.8 | | | | | | | |
| 2.9 | | | | | | | |
| 3.0 | | | | | | | |

N. B.—For correct velocity, apply the

in feet per second, $100\sqrt{RS}$, suitable to Culverts and Pipes.

| <i>R</i> in feet | S per thousand | | | | | | |
|---------------------|----------------|--------|--------|--------|--------|--------|--------|
| | 12 | 11 | 10 | 9 | 8 | 7 | 6 |
| 0.05 | 2.449 | 2.345 | 2.236 | 2.121 | 2. | 1.871 | 1.732 |
| 0.10 | 3.464 | 3.317 | 3.162 | 3. | 2.828 | 2.648 | 2.449 |
| 0.15 | 4.243 | 4.062 | 3.873 | 3.674 | 3.464 | 3.240 | 3. |
| 0.20 | 4.899 | 4.690 | 4.472 | 4.243 | 4. | 3.742 | 3.464 |
| 0.25 | 5.477 | 5.244 | 5. | 4.743 | 4.472 | 4.183 | 3.873 |
| 0.30 | 6. | 5.745 | 5.477 | 5.196 | 4.898 | 4.583 | 4.243 |
| 0.35 | 6.480 | 6.205 | 5.916 | 5.612 | 5.292 | 4.950 | 4.583 |
| 0.40 | 6.928 | 6.633 | 6.325 | 6. | 5.656 | 5.292 | 4.899 |
| 0.45 | 7.348 | 7.035 | 6.708 | 6.364 | 6. | 5.612 | 5.196 |
| 0.50 | 7.746 | 7.416 | 7.071 | 6.708 | 6.325 | 5.916 | 5.477 |
| 0.6 | 8.486 | 8.124 | 7.746 | 7.348 | 6.928 | 6.481 | 6. |
| 0.7 | 9.165 | 8.775 | 8.367 | 7.937 | 7.484 | 7. | 6.480 |
| 0.8 | 9.798 | 9.381 | 8.944 | 8.485 | 8. | 7.483 | 6.928 |
| 0.9 | 10.392 | 9.950 | 9.487 | 9. | 8.486 | 7.937 | 7.348 |
| 1.0 | 10.954 | 10.488 | 10. | 9.487 | 8.944 | 8.367 | 7.746 |
| 1.1 | 11.489 | 11. | 10.488 | 9.950 | 9.381 | 8.775 | 8.124 |
| 1.2 | 12. | 11.489 | 10.954 | 10.392 | 9.797 | 9.165 | 8.486 |
| 1.3 | 12.490 | 11.958 | 11.402 | 10.817 | 10.198 | 9.539 | 8.832 |
| 1.4 | 12.961 | 12.410 | 11.832 | 11.225 | 10.583 | 9.899 | 9.165 |
| 1.5 | 13.416 | 12.845 | 12.247 | 11.619 | 10.954 | 10.247 | 9.487 |
| 1.6 | 13.856 | 13.266 | 12.649 | 12. | 11.314 | 10.583 | 9.798 |
| 1.7 | 14.283 | 13.675 | 13.038 | 12.369 | 11.662 | 10.909 | 10.100 |
| 1.8 | 14.697 | 14.071 | 13.416 | 12.728 | 12. | 11.225 | 10.392 |
| 1.9 | 15.100 | 14.457 | 13.784 | 13.077 | 12.329 | 11.533 | 10.677 |
| 2.0 | 15.492 | 14.832 | 14.142 | 13.416 | 12.650 | 11.832 | 10.954 |
| 2.1 | 15.875 | 15.199 | 14.491 | 13.748 | 12.961 | 12.124 | 11.225 |
| 2.2 | 16.248 | 15.556 | 14.832 | 14.071 | 13.266 | 12.410 | 11.489 |
| 2.3 | 16.613 | 15.906 | 15.166 | 14.387 | 13.565 | 12.689 | 11.747 |
| 2.4 | 16.971 | 16.248 | 15.492 | 14.697 | 13.856 | 12.961 | 12. |
| 2.5 | 17.321 | 16.583 | 15.811 | 15. | 14.142 | 13.229 | 12.247 |
| 2.6 | 17.664 | 16.912 | 16.125 | 15.297 | 14.422 | 13.491 | 12.490 |
| 2.7 | 18. | 17.234 | 16.432 | 15.588 | 14.697 | 13.748 | 12.689 |
| 2.8 | 18.330 | 17.550 | 16.733 | 15.875 | 14.967 | 14. | 12.961 |
| 2.9 | 18.655 | 17.861 | 17.029 | 16.155 | 15.232 | 14.248 | 13.191 |
| 3.0 | 18.974 | 18.166 | 17.321 | 16.432 | 15.492 | 14.491 | 13.416 |

suitable value of *c* from Table XII.

PART I (continued).—Approximate
or, Values of the Expression

| R in feet | S per thousand | | | | | | |
|--------------|----------------|--------|--------|--------|-------|-------|-------|
| | 5· | 4·5 | 4· | 3·5 | 3· | 2·5 | 2· |
| 0·05 | 1·581 | 1·5 | 1·414 | 1·324 | 1·225 | 1·118 | 1· |
| 0·10 | 2·236 | 2·121 | 2· | 1·871 | 1·732 | 1·581 | 1·414 |
| 0·15 | 2·739 | 2·598 | 2·449 | 2·291 | 2·121 | 1·936 | 1·732 |
| 0·20 | 3·162 | 3· | 2·828 | 2·648 | 2·449 | 2·236 | 2· |
| 0·25 | 3·536 | 3·354 | 3·162 | 2·958 | 2·739 | 2·5 | 2·236 |
| 0·30 | 3·873 | 3·674 | 3·464 | 3·240 | 3· | 2·739 | 2·449 |
| 0·35 | 4·183 | 3·963 | 3·742 | 3·5 | 3·240 | 2·958 | 2·646 |
| 0·40 | 4·472 | 4·243 | 4· | 3·742 | 3·464 | 3·162 | 2·828 |
| 0·45 | 4·743 | 4·5 | 4·243 | 3·969 | 3·674 | 3·354 | 3· |
| 0·50 | 5· | 4·743 | 4·472 | 4·183 | 3·873 | 3·536 | 3·162 |
| 0·6 | 5·477 | 5·196 | 4·898 | 4·583 | 4·243 | 3·873 | 3·464 |
| 0·7 | 5·916 | 5·612 | 5·292 | 4·950 | 4·583 | 4·183 | 3·742 |
| 0·8 | 6·325 | 6· | 5·656 | 5·292 | 4·899 | 4·472 | 4· |
| 0·9 | 6·708 | 6·364 | 6· | 5·612 | 5·196 | 4·743 | 4·243 |
| 1·0 | 7·071 | 6·708 | 6·325 | 5·916 | 5·477 | 5· | 4·472 |
| 1·1 | 7·416 | 7·036 | 6·633 | 6·205 | 5·744 | 5·244 | 4·690 |
| 1·2 | 7·746 | 7·348 | 6·928 | 6·481 | 6· | 5·477 | 4·898 |
| 1·3 | 8·062 | 7·649 | 7·211 | 6·745 | 6·245 | 5·701 | 5·098 |
| 1·4 | 8·367 | 7·937 | 7·484 | 7· | 6·480 | 5·916 | 5·292 |
| 1·5 | 8·660 | 8·216 | 7·746 | 7·246 | 6·708 | 6·123 | 5·477 |
| 1·6 | 8·944 | 8·485 | 8· | 7·483 | 6·928 | 6·325 | 5·656 |
| 1·7 | 9·220 | 8·746 | 8·246 | 7·714 | 7·141 | 6·519 | 5·831 |
| 1·8 | 9·487 | 9· | 8·486 | 7·937 | 7·348 | 6·708 | 6· |
| 1·9 | 9·747 | 9·247 | 8·718 | 8·155 | 7·550 | 6·892 | 6·164 |
| 2·0 | 10· | 9·487 | 8·944 | 8·367 | 7·746 | 7·071 | 6·325 |
| 2·1 | 10·247 | 9·721 | 9·165 | 8·631 | 7·937 | 7·246 | 6·481 |
| 2·2 | 10·488 | 9·950 | 9·381 | 8·775 | 8·124 | 7·416 | 6·633 |
| 2·3 | 10·724 | 10·174 | 9·592 | 8·972 | 8·307 | 7·583 | 6·782 |
| 2·4 | 10·954 | 10·392 | 9·797 | 9·165 | 8·486 | 7·746 | 6·928 |
| 2·5 | 11·180 | 10·606 | 10· | 9·354 | 8·660 | 7·905 | 7·071 |
| 2·6 | 11·402 | 10·817 | 10·198 | 9·539 | 8·832 | 8·062 | 7·211 |
| 2·7 | 11·619 | 11·023 | 10·392 | 9·670 | 9· | 8·216 | 7·348 |
| 2·8 | 11·832 | 11·225 | 10·583 | 9·899 | 9·165 | 8·367 | 7·484 |
| 2·9 | 12·042 | 11·424 | 10·770 | 10·075 | 9·327 | 8·515 | 7·616 |
| 3·0 | 12·247 | 11·619 | 10·954 | 10·247 | 9·487 | 8·660 | 7·746 |

N. B.—For correct velocity, apply the

velocities in feet per second,

100 \sqrt{RS} , suitable to Culverts and Pipes.

| R in feet | S per thousand | | | | | | |
|--------------|----------------|-------|-------|-------|-------|-------|-------|
| | 1.5 | 1. | 0.95 | 0.90 | 0.85 | 0.80 | 0.75 |
| 0.05 | 0.866 | 0.707 | 0.689 | 0.671 | 0.652 | 0.632 | 0.612 |
| 0.10 | 1.225 | 1. | 0.975 | 0.949 | 0.922 | 0.894 | 0.866 |
| 0.15 | 1.5 | 1.225 | 1.193 | 1.162 | 1.129 | 1.095 | 1.060 |
| 0.20 | 1.732 | 1.414 | 1.378 | 1.342 | 1.304 | 1.265 | 1.225 |
| 0.25 | 1.936 | 1.581 | 1.541 | 1.5 | 1.457 | 1.414 | 1.369 |
| 0.30 | 2.121 | 1.732 | 1.688 | 1.643 | 1.592 | 1.549 | 1.5 |
| 0.35 | 2.291 | 1.871 | 1.823 | 1.775 | 1.725 | 1.673 | 1.620 |
| 0.40 | 2.449 | 2. | 1.949 | 1.897 | 1.844 | 1.789 | 1.732 |
| 0.45 | 2.598 | 2.121 | 2.07 | 2.012 | 1.956 | 1.897 | 1.837 |
| 0.50 | 2.739 | 2.236 | 2.179 | 2.121 | 2.061 | 2. | 1.936 |
| 0.6 | 3. | 2.449 | 2.387 | 2.324 | 2.258 | 2.191 | 2.121 |
| 0.7 | 3.240 | 2.646 | 2.579 | 2.510 | 2.439 | 2.366 | 2.291 |
| 0.8 | 3.464 | 2.828 | 2.757 | 2.683 | 2.608 | 2.530 | 2.449 |
| 0.9 | 3.674 | 3. | 2.924 | 2.846 | 2.766 | 2.683 | 2.598 |
| 1.0 | 3.873 | 3.162 | 3.082 | 3. | 2.915 | 2.828 | 2.739 |
| 1.1 | 4.062 | 3.317 | 3.233 | 3.146 | 3.058 | 2.966 | 2.872 |
| 1.2 | 4.243 | 3.464 | 3.376 | 3.286 | 3.194 | 3.098 | 3. |
| 1.3 | 4.416 | 3.606 | 3.514 | 3.421 | 3.324 | 3.225 | 3.122 |
| 1.4 | 4.583 | 3.742 | 3.647 | 3.550 | 3.450 | 3.347 | 3.240 |
| 1.5 | 4.743 | 3.873 | 3.775 | 3.674 | 3.571 | 3.464 | 3.354 |
| 1.6 | 4.899 | 4. | 3.899 | 3.795 | 3.688 | 3.578 | 3.464 |
| 1.7 | 5.050 | 4.123 | 4.019 | 3.912 | 3.801 | 3.688 | 3.571 |
| 1.8 | 5.196 | 4.243 | 4.135 | 4.025 | 3.912 | 3.795 | 3.674 |
| 1.9 | 5.339 | 4.359 | 4.249 | 4.135 | 4.019 | 3.899 | 3.775 |
| 2.0 | 5.477 | 4.472 | 4.359 | 4.243 | 4.123 | 4. | 3.873 |
| 2.1 | 5.612 | 4.583 | 4.467 | 4.347 | 4.225 | 4.099 | 3.969 |
| 2.2 | 5.744 | 4.690 | 4.572 | 4.450 | 4.324 | 4.195 | 4.062 |
| 2.3 | 5.874 | 4.796 | 4.674 | 4.550 | 4.422 | 4.313 | 4.153 |
| 2.4 | 6. | 4.898 | 4.775 | 4.648 | 4.517 | 4.382 | 4.243 |
| 2.5 | 6.123 | 5. | 4.873 | 4.743 | 4.610 | 4.472 | 4.330 |
| 2.6 | 6.245 | 5.098 | 4.970 | 4.837 | 4.701 | 4.561 | 4.416 |
| 2.7 | 6.364 | 5.196 | 5.065 | 4.930 | 4.791 | 4.648 | 4.500 |
| 2.8 | 6.480 | 5.292 | 5.158 | 5.020 | 4.879 | 4.733 | 4.583 |
| 2.9 | 6.595 | 5.385 | 5.249 | 5.109 | 4.965 | 4.817 | 4.664 |
| 3.0 | 6.708 | 5.477 | 5.339 | 5.196 | 5.050 | 4.898 | 4.743 |

suitable value of c from Table XII.

PART I (continued).—Approximate
or, Values of the Expression

| R in feet | S per thousand | | | | | | |
|--------------|----------------|-------|-------|-------|-------|-------|-------|
| | 0.70 | 0.65 | 0.60 | 0.55 | 0.50 | 0.45 | 0.40 |
| 0.05 | 0.592 | 0.570 | 0.548 | 0.524 | 0.5 | 0.474 | 0.447 |
| 0.10 | 0.837 | 0.806 | 0.775 | 0.742 | 0.707 | 0.671 | 0.632 |
| 0.15 | 1.025 | 0.987 | 0.949 | 0.908 | 0.866 | 0.822 | 0.775 |
| 0.20 | 1.183 | 1.140 | 1.095 | 1.049 | 1. | 0.949 | 0.894 |
| 0.25 | 1.323 | 1.275 | 1.225 | 1.172 | 1.118 | 1.061 | 1. |
| 0.30 | 1.449 | 1.396 | 1.342 | 1.284 | 1.225 | 1.162 | 1.095 |
| 0.35 | 1.565 | 1.508 | 1.449 | 1.387 | 1.323 | 1.255 | 1.183 |
| 0.40 | 1.673 | 1.612 | 1.549 | 1.483 | 1.414 | 1.342 | 1.265 |
| 0.45 | 1.775 | 1.710 | 1.643 | 1.573 | 1.5 | 1.423 | 1.342 |
| 0.50 | 1.871 | 1.803 | 1.732 | 1.658 | 1.581 | 1.5 | 1.414 |
| 0.6 | 2.049 | 1.975 | 1.897 | 1.816 | 1.732 | 1.643 | 1.549 |
| 0.7 | 2.214 | 2.133 | 2.049 | 1.962 | 1.871 | 1.775 | 1.673 |
| 0.8 | 2.366 | 2.280 | 2.191 | 2.098 | 2. | 1.897 | 1.789 |
| 0.9 | 2.510 | 2.419 | 2.324 | 2.225 | 2.121 | 2.012 | 1.897 |
| 1.0 | 2.646 | 2.550 | 2.449 | 2.345 | 2.236 | 2.121 | 2. |
| 1.1 | 2.775 | 2.674 | 2.569 | 2.460 | 2.345 | 2.225 | 2.098 |
| 1.2 | 2.898 | 2.793 | 2.683 | 2.569 | 2.449 | 2.324 | 2.191 |
| 1.3 | 3.017 | 2.907 | 2.793 | 2.674 | 2.549 | 2.419 | 2.280 |
| 1.4 | 3.130 | 3.017 | 2.898 | 2.775 | 2.646 | 2.510 | 2.366 |
| 1.5 | 3.240 | 3.122 | 3. | 2.872 | 2.739 | 2.598 | 2.449 |
| 1.6 | 3.347 | 3.225 | 3.098 | 2.966 | 2.828 | 2.683 | 2.530 |
| 1.7 | 3.450 | 3.324 | 3.194 | 3.058 | 2.915 | 2.766 | 2.608 |
| 1.8 | 3.550 | 3.421 | 3.286 | 3.146 | 3. | 2.846 | 2.683 |
| 1.9 | 3.647 | 3.514 | 3.376 | 3.233 | 3.082 | 2.924 | 2.757 |
| 2.0 | 3.742 | 3.606 | 3.464 | 3.317 | 3.162 | 3. | 2.828 |
| 2.1 | 3.834 | 3.695 | 3.550 | 3.399 | 3.240 | 3.074 | 2.898 |
| 2.2 | 3.924 | 3.782 | 3.633 | 3.479 | 3.317 | 3.146 | 2.966 |
| 2.3 | 4.012 | 3.867 | 3.715 | 3.557 | 3.391 | 3.217 | 3.033 |
| 2.4 | 4.099 | 3.950 | 3.795 | 3.633 | 3.464 | 3.286 | 3.098 |
| 2.5 | 4.183 | 4.031 | 3.873 | 3.708 | 3.536 | 3.354 | 3.162 |
| 2.6 | 4.266 | 4.111 | 3.950 | 3.782 | 3.606 | 3.421 | 3.225 |
| 2.7 | 4.347 | 4.189 | 4.025 | 3.854 | 3.674 | 3.486 | 3.286 |
| 2.8 | 4.427 | 4.266 | 4.099 | 3.924 | 3.742 | 3.550 | 3.347 |
| 2.9 | 4.506 | 4.342 | 4.171 | 3.994 | 3.808 | 3.612 | 3.406 |
| 3.0 | 4.583 | 4.416 | 4.243 | 4.062 | 3.873 | 3.674 | 3.464 |

N. E.—For correct velocity, apply the

velocities in feet per second,

 $100\sqrt{RS}$, suitable to Culverts and Pipes.

| <i>R</i> in feet | <i>S</i> per thousand | | | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|-------|-------|
| | 0.35 | 0.30 | 0.25 | 0.20 | 0.15 | 0.10 | 0.05 |
| 0.05 | 0.418 | 0.387 | 0.354 | 0.316 | 0.274 | 0.224 | 0.158 |
| 0.10 | 0.592 | 0.548 | 0.500 | 0.447 | 0.387 | 0.316 | 0.224 |
| 0.15 | 0.725 | 0.671 | 0.612 | 0.548 | 0.474 | 0.387 | 0.274 |
| 0.20 | 0.837 | 0.775 | 0.707 | 0.632 | 0.548 | 0.447 | 0.316 |
| 0.25 | 0.935 | 0.866 | 0.790 | 0.707 | 0.612 | 0.500 | 0.354 |
| 0.30 | 1.025 | 0.949 | 0.866 | 0.775 | 0.671 | 0.548 | 0.387 |
| 0.35 | 1.107 | 1.025 | 0.935 | 0.837 | 0.725 | 0.592 | 0.418 |
| 0.40 | 1.183 | 1.095 | 1. | 0.894 | 0.775 | 0.632 | 0.447 |
| 0.45 | 1.255 | 1.162 | 1.061 | 0.949 | 0.822 | 0.671 | 0.474 |
| 0.50 | 1.323 | 1.225 | 1.118 | 1. | 0.866 | 0.707 | 0.500 |
| 0.6 | 1.449 | 1.342 | 1.225 | 1.095 | 0.949 | 0.775 | 0.548 |
| 0.7 | 1.565 | 1.449 | 1.323 | 1.183 | 1.025 | 0.837 | 0.592 |
| 0.8 | 1.673 | 1.549 | 1.414 | 1.265 | 1.095 | 0.894 | 0.632 |
| 0.9 | 1.775 | 1.643 | 1.5 | 1.342 | 1.162 | 0.949 | 0.671 |
| 1.0 | 1.871 | 1.732 | 1.581 | 1.414 | 1.225 | 1. | 0.707 |
| 1.1 | 1.962 | 1.817 | 1.658 | 1.483 | 1.285 | 1.049 | 0.742 |
| 1.2 | 2.049 | 1.897 | 1.732 | 1.549 | 1.342 | 1.095 | 0.775 |
| 1.3 | 2.133 | 1.975 | 1.803 | 1.612 | 1.396 | 1.140 | 0.806 |
| 1.4 | 2.214 | 2.049 | 1.871 | 1.673 | 1.449 | 1.183 | 0.837 |
| 1.5 | 2.291 | 2.121 | 1.936 | 1.732 | 1.500 | 1.225 | 0.866 |
| 1.6 | 2.366 | 2.191 | 2. | 1.789 | 1.549 | 1.265 | 0.894 |
| 1.7 | 2.439 | 2.258 | 2.062 | 1.844 | 1.597 | 1.304 | 0.922 |
| 1.8 | 2.510 | 2.324 | 2.121 | 1.897 | 1.643 | 1.342 | 0.949 |
| 1.9 | 2.579 | 2.387 | 2.179 | 1.949 | 1.688 | 1.378 | 0.975 |
| 2.0 | 2.646 | 2.449 | 2.236 | 2. | 1.732 | 1.414 | 1. |
| 2.1 | 2.711 | 2.510 | 2.291 | 2.049 | 1.775 | 1.449 | 1.025 |
| 2.2 | 2.775 | 2.569 | 2.345 | 2.098 | 1.817 | 1.483 | 1.049 |
| 2.3 | 2.837 | 2.627 | 2.398 | 2.145 | 1.857 | 1.517 | 1.072 |
| 2.4 | 2.898 | 2.683 | 2.449 | 2.191 | 1.897 | 1.549 | 1.095 |
| 2.5 | 1.958 | 2.739 | 2.500 | 2.236 | 1.936 | 1.581 | 1.118 |
| 2.6 | 3.017 | 2.793 | 2.549 | 2.280 | 1.975 | 1.612 | 1.140 |
| 2.7 | 3.074 | 2.847 | 2.598 | 2.324 | 2.012 | 1.643 | 1.162 |
| 2.8 | 3.130 | 2.898 | 2.646 | 2.366 | 2.049 | 1.673 | 1.183 |
| 2.9 | 3.186 | 2.950 | 2.693 | 2.408 | 2.086 | 1.703 | 1.204 |
| 3.0 | 3.240 | 3. | 2.739 | 2.449 | 2.121 | 1.732 | 1.225 |

suitable value of *c* from Table XII.

PART 2.—Approximate Discharges through full cylindrical tubes, Pipes, Culverts, &c.

| For diameters in feet | For slopes of one in | | | | | | | Tabular No. to be multiplied by \sqrt{S} for other slopes |
|-----------------------|---|--------|--------|--------|--------|--------|--------|---|
| | 100 | 150 | 200 | 300 | 400 | 500 | 1000 | |
| | Approximate discharges in cubic feet per second | | | | | | | |
| (1") .083 | .008 | .006 | .006 | .005 | .004 | .004 | .003 | .079 |
| (2") .166 | .04 | .04 | .03 | .03 | .02 | .02 | .01 | .445 |
| (3") .25 | .12 | .10 | .09 | .07 | .06 | .05 | .04 | 1.227 |
| (4") .33 | .25 | .21 | .18 | .15 | .13 | .11 | .08 | 2.519 |
| (5") .416 | .44 | .36 | .31 | .25 | .22 | .20 | .14 | 4.401 |
| (6") .5 | .69 | .57 | .49 | .40 | .35 | .31 | .22 | 6.939 |
| (7") .583 | 1.02 | .83 | .72 | .59 | .51 | .46 | .32 | 10.206 |
| (8") .66 | 1.43 | 1.16 | 1.01 | .82 | .71 | .64 | .45 | 14.251 |
| (9") .75 | 1.91 | 1.56 | 1.35 | 1.10 | .97 | .86 | .61 | 19.128 |
| (10") .83 | 2.49 | 2.03 | 1.76 | 1.44 | 1.25 | 1.11 | .79 | 24.895 |
| (11") .916 | 3.16 | 2.58 | 2.23 | 1.82 | 1.58 | 1.41 | 1.00 | 31.594 |
| (12") 1.00 | 3.93 | 3.28 | 2.78 | 2.27 | 1.96 | 1.76 | 1.24 | 39.27 |
| 1.25 | 6.86 | 5.60 | 4.85 | 3.96 | 3.43 | 3.07 | 2.16 | 68.601 |
| 1.5 | 10.82 | 8.82 | 7.55 | 6.25 | 5.41 | 4.84 | 3.42 | 108.216 |
| 1.75 | 15.91 | 12.99 | 11.25 | 9.18 | 7.95 | 7.11 | 5.03 | 159.095 |
| 2. | 22.21 | 18.14 | 15.71 | 12.83 | 11.11 | 9.93 | 7.02 | 222.140 |
| 2.25 | 29.82 | 24.35 | 21.08 | 17.22 | 14.91 | 13.34 | 9.43 | 298.505 |
| 2.5 | 38.81 | 31.69 | 27.44 | 22.41 | 19.40 | 17.35 | 12.27 | 388.078 |
| 2.75 | 49.25 | 40.22 | 34.82 | 28.43 | 24.62 | 22.02 | 15.57 | 492.489 |
| 3. | 61.21 | 49.99 | 43.28 | 35.34 | 30.61 | 27.37 | 19.35 | 612.105 |
| 3.25 | 74.77 | 61.04 | 52.87 | 43.18 | 37.38 | 33.44 | 23.64 | 747.744 |
| 3.5 | 88.99 | 73.49 | 63.63 | 51.96 | 44.99 | 40.25 | 28.46 | 899.990 |
| 3.75 | 106.94 | 87.33 | 75.61 | 61.74 | 53.46 | 47.82 | 33.81 | 1069.397 |
| 4. | 125.66 | 102.63 | 88.84 | 72.55 | 62.83 | 56.20 | 39.73 | 1256.640 |
| 4.25 | 146.23 | 119.42 | 103.38 | 84.32 | 73.11 | 65.39 | 46.24 | 1462.262 |
| 4.5 | 168.69 | 137.76 | 119.26 | 97.39 | 84.34 | 75.44 | 53.34 | 1686.886 |
| 4.75 | 193.10 | 157.70 | 136.52 | 111.48 | 96.55 | 86.36 | 61.06 | 1931.028 |
| 5. | 219.54 | 179.26 | 155.24 | 126.75 | 109.77 | 99.18 | 69.43 | 2195.436 |
| 5.5 | 278.61 | 227.48 | 197.00 | 160.85 | 139.30 | 124.60 | 88.10 | 2786.060 |
| 6. | 346.31 | 282.76 | 244.88 | 199.94 | 173.16 | 154.88 | 109.51 | 3463.130 |
| 6.5 | 423.03 | 345.40 | 299.13 | 244.23 | 211.51 | 189.18 | 133.77 | 4230.262 |
| 7. | 509.13 | 415.70 | 360.01 | 293.65 | 254.57 | 227.69 | 161.00 | 5091.322 |

N. B.—For correct discharge, apply the suitable value of c from Table XII.

PART 2 (continued).

| For diameters in feet | For hydraulic slopes of one in | | | | | | |
|---|--------------------------------|-------|-------|-------|-------|-------|-------|
| | 1250 | 1500 | 2000 | 2500 | 3000 | 4000 | 5000 |
| Approximate discharges in cubic feet per second | | | | | | | |
| 1" | 0'002 | 0'002 | 0'002 | 0'002 | 0'001 | 0'001 | 0'001 |
| 2" | 0'013 | 0'011 | 0'010 | 0'009 | 0'008 | 0'007 | 0'006 |
| 3" | 0'035 | 0'032 | 0'028 | 0'025 | 0'022 | 0'019 | 0'017 |
| 4" | 0'071 | 0'065 | 0'056 | 0'050 | 0'046 | 0'040 | 0'036 |
| 5" | 0'124 | 0'114 | 0'099 | 0'088 | 0'080 | 0'070 | 0'062 |
| 6" | 0'196 | 0'179 | 0'155 | 0'139 | 0'127 | 0'110 | 0'098 |
| 7" | 0'289 | 0'264 | 0'228 | 0'204 | 0'186 | 0'161 | 0'144 |
| 8" | 0'403 | 0'368 | 0'319 | 0'285 | 0'260 | 0'225 | 0'202 |
| 9" | 0'541 | 0'494 | 0'428 | 0'383 | 0'349 | 0'302 | 0'271 |
| 10" | 0'704 | 0'643 | 0'557 | 0'498 | 0'455 | 0'394 | 0'352 |
| 11" | 0'894 | 0'816 | 0'706 | 0'632 | 0'577 | 0'500 | 0'447 |
| 12" | 1'111 | 1'014 | 0'878 | 0'785 | 0'717 | 0'621 | 0'555 |
| 1'25 | 1'940 | 1'771 | 1'534 | 1'372 | 1'252 | 1'085 | 0'970 |
| 1'5 | 3'060 | 2'794 | 2'420 | 2'164 | 1'976 | 1'711 | 1'530 |
| 1'75 | 4'500 | 4'108 | 3'558 | 3'182 | 2'905 | 2'516 | 2'250 |
| 2' | 6'284 | 5'736 | 4'968 | 4'442 | 4'056 | 3'513 | 3'142 |
| 2'25 | 8'444 | 7'708 | 6'675 | 5'964 | 5'450 | 4'720 | 4'222 |
| 2'5 | 10'98 | 10'02 | 8'678 | 7'762 | 7'086 | 6'136 | 5'489 |
| 2'75 | 13'93 | 12'72 | 11'01 | 9'850 | 8'991 | 7'786 | 6'965 |
| 3' | 17'31 | 15'80 | 13'69 | 12'24 | 11'18 | 9'679 | 8'657 |
| 3'25 | 21'16 | 19'31 | 16'72 | 15'95 | 13'65 | 11'82 | 10'58 |
| 3'5 | 25'46 | 23'24 | 20'13 | 18'00 | 16'43 | 14'23 | 12'73 |
| 3'75 | 30'24 | 27'61 | 23'91 | 21'38 | 19'52 | 16'91 | 15'12 |
| 4' | 35'54 | 32'45 | 28'10 | 25'14 | 22'94 | 19'87 | 17'77 |
| 4'25 | 41'36 | 37'76 | 32'70 | 29'24 | 26'70 | 23'12 | 20'68 |
| 4'5 | 47'72 | 43'55 | 37'73 | 33'74 | 30'80 | 26'67 | 23'86 |
| 4'75 | 55'88 | 49'86 | 43'18 | 38'62 | 35'25 | 30'53 | 27'94 |
| 5' | 62'10 | 56'69 | 49'10 | 43'90 | 40'08 | 34'71 | 31'05 |
| 5'5 | 78'80 | 71'94 | 62'30 | 55'72 | 50'87 | 44'05 | 39'40 |
| 6' | 97'96 | 89'42 | 77'44 | 69'26 | 63'23 | 54'76 | 48'98 |
| 6'5 | 119'8 | 109'2 | 94'59 | 84'60 | 77'24 | 66'89 | 59'83 |
| 7' | 144'0 | 131'5 | 113'9 | 101'8 | 92'96 | 80'50 | 72'01 |

N. B.—For correct discharge, apply the suitable value of *c* from Table XII.

PART 2 (continued).

| Diam. of Pipe | For hydraulic slopes S per thousand | | | | | | |
|------------------|---|-------|--------|-------|-------|-------|-------|
| | 20 | 15 | 10 | 9 | 8 | 7 | 6 |
| | Approximate discharges in cubic feet per second | | | | | | |
| 1" | 0·011 | 0·010 | 0·008 | 0·007 | 0·007 | 0·007 | 0·006 |
| 2" | 0·063 | 0·055 | 0·045 | 0·042 | 0·040 | 0·037 | 0·034 |
| 3" | 0·174 | 0·150 | 0·123 | 0·116 | 0·110 | 0·103 | 0·095 |
| 4" | 0·356 | 0·309 | 0·252 | 0·239 | 0·225 | 0·211 | 0·195 |
| 5" | 0·622 | 0·539 | 0·440 | 0·418 | 0·394 | 0·368 | 0·341 |
| 6" | 0·981 | 0·850 | 0·694 | 0·658 | 0·621 | 0·581 | 0·537 |
| 7" | 1·443 | 1·250 | 1·021 | 0·968 | 0·913 | 0·854 | 0·790 |
| 8" | 2·015 | 1·745 | 1·425 | 1·352 | 1·275 | 1·192 | 1·104 |
| 9" | 2·705 | 2·343 | 1·913 | 1·815 | 1·711 | 1·600 | 1·481 |
| 10" | 3·521 | 3·049 | 2·490 | 2·362 | 2·227 | 2·083 | 1·928 |
| 11" | 4·468 | 3·869 | 3·159 | 2·997 | 2·825 | 2·643 | 2·447 |
| 12" | 5·554 | 4·810 | 3·927 | 3·726 | 3·512 | 3·285 | 3·041 |
| 1·25 | 9·702 | 8·402 | 6·860 | 6·509 | 6·136 | 5·740 | 5·313 |
| 1·5 | 15·30 | 13·25 | 10·82 | 10·27 | 9·679 | 9·054 | 8·381 |
| 1·75 | 22·50 | 19·49 | 15·91 | 15·09 | 14·23 | 13·31 | 12·32 |
| 2· | 31·42 | 27·21 | 22·21 | 21·07 | 19·87 | 18·59 | 17·20 |
| 2·25 | 42·22 | 36·56 | 29·82 | 28·32 | 26·70 | 24·97 | 23·12 |
| 2·5 | 54·89 | 47·53 | 38·81 | 36·82 | 34·71 | 32·47 | 30·06 |
| 2·75 | 69·65 | 60·31 | 49·25 | 46·72 | 44·05 | 41·20 | 38·14 |
| 3· | 86·57 | 74·97 | 61·21 | 58·07 | 54·75 | 51·21 | 47·40 |
| 3·25 | 105·8 | 91·58 | 74·77 | 70·94 | 66·88 | 62·56 | 57·90 |
| 3·5 | 127·3 | 110·2 | 89·99 | 85·38 | 80·50 | 75·30 | 69·70 |
| 3·75 | 151·2 | 131·0 | 106·94 | 101·5 | 95·65 | 89·47 | 82·82 |
| 4· | 177·7 | 153·9 | 125·66 | 119·2 | 112·4 | 105·1 | 97·32 |
| 4·25 | 206·8 | 171·0 | 146·23 | 138·7 | 130·8 | 122·3 | 113·2 |
| 4·5 | 238·6 | 201·9 | 168·69 | 160·0 | 150·9 | 141·1 | 130·6 |
| 4·75 | 279·4 | 236·5 | 193·10 | 183·2 | 172·7 | 161·5 | 149·5 |
| 5· | 310·5 | 268·9 | 219·54 | 208·3 | 196·4 | 183·7 | 170·0 |
| 5·5 | 394·0 | 341·2 | 278·61 | 264·3 | 249·2 | 233·1 | 215·8 |
| 6· | 489·8 | 424·1 | 346·31 | 328·6 | 309·8 | 289·7 | 268·2 |
| 6·5 | 598·3 | 518·1 | 423·03 | 401·3 | 378·4 | 353·9 | 327·6 |
| 7· | 720·1 | 623·6 | 509·13 | 483·0 | 455·4 | 426·0 | 394·3 |

N.B.—For correct discharge, apply the

PART 2 (continued).

| Diam. of Pipe | For hydraulic slopes S per thousand | | | | | | |
|------------------|---|-------|-------|--------|--------|-------|-------|
| | 5' | 4' | 3' | 2.5 | 2' | 1.75 | 1.5 |
| | Approximate discharges in cubic feet per second | | | | | | |
| 1" | 0.006 | 0.005 | 0.004 | 0.004 | 0.004 | 0.003 | 0.003 |
| 2" | 0.032 | 0.028 | 0.024 | 0.022 | 0.020 | 0.019 | 0.017 |
| 3" | 0.087 | 0.078 | 0.067 | 0.061 | 0.055 | 0.052 | 0.048 |
| 4" | 0.178 | 0.159 | 0.138 | 0.126 | 0.113 | 0.106 | 0.098 |
| 5" | 0.311 | 0.278 | 0.241 | 0.220 | 0.197 | 0.184 | 0.171 |
| 6" | 0.491 | 0.439 | 0.380 | 0.347 | 0.311 | 0.291 | 0.269 |
| 7" | 0.722 | 0.646 | 0.559 | 0.510 | 0.457 | 0.427 | 0.395 |
| 8" | 1.008 | 0.901 | 0.781 | 0.713 | 0.638 | 0.596 | 0.552 |
| 9" | 1.353 | 1.210 | 1.148 | 0.956 | 0.856 | 0.800 | 0.741 |
| 10" | 1.761 | 1.575 | 1.364 | 1.250 | 1.113 | 1.042 | 0.964 |
| 11" | 2.234 | 1.998 | 1.731 | 1.580 | 1.413 | 1.322 | 1.224 |
| 12" | 2.777 | 2.484 | 2.151 | 1.964 | 1.756 | 1.643 | 1.521 |
| 1.25 | 4.851 | 4.339 | 3.758 | 3.430 | 3.068 | 2.870 | 2.657 |
| 1.5 | 7.650 | 6.844 | 5.928 | 5.411 | 4.840 | 4.527 | 4.191 |
| 1.75 | 11.250 | 10.06 | 8.714 | 7.955 | 7.115 | 6.655 | 6.160 |
| 2' | 15.71 | 14.05 | 12.17 | 11.11 | 9.935 | 9.295 | 8.600 |
| 2.25 | 21.11 | 18.88 | 16.39 | 14.91 | 13.35 | 12.49 | 11.56 |
| 2.5 | 27.44 | 24.55 | 21.26 | 19.40 | 17.36 | 16.24 | 15.03 |
| 2.75 | 34.82 | 31.15 | 26.98 | 24.62 | 22.03 | 20.60 | 19.07 |
| 3' | 43.28 | 38.72 | 33.53 | 30.61 | 27.37 | 25.61 | 23.70 |
| 3.25 | 52.87 | 47.30 | 40.96 | 37.38 | 33.44 | 31.28 | 28.95 |
| 3.5 | 63.63 | 56.92 | 49.30 | 44.99 | 40.25 | 37.65 | 34.85 |
| 3.75 | 75.61 | 67.64 | 58.58 | 53.46 | 47.82 | 44.74 | 41.41 |
| 4' | 88.84 | 79.48 | 68.84 | 62.83 | 56.20 | 52.55 | 48.66 |
| 4.25 | 103.38 | 92.49 | 80.10 | 73.11 | 65.40 | 61.15 | 56.60 |
| 4.5 | 119.26 | 106.7 | 92.39 | 84.34 | 75.45 | 70.55 | 65.30 |
| 4.75 | 136.52 | 122.1 | 105.8 | 96.55 | 86.35 | 80.75 | 74.75 |
| 5' | 155.24 | 138.9 | 120.3 | 109.77 | 98.20 | 91.85 | 85.00 |
| 5.5 | 197.00 | 176.2 | 152.6 | 139.30 | 124.60 | 116.6 | 107.9 |
| 6' | 244.88 | 219.0 | 189.7 | 173.16 | 154.88 | 144.9 | 134.1 |
| 6.5 | 299.13 | 267.5 | 231.7 | 211.51 | 189.18 | 177.0 | 163.8 |
| 7' | 360.01 | 322.0 | 278.9 | 254.57 | 227.69 | 213.0 | 197.2 |

suitable value of c from Table XII.

PART 2 (continued).

| Diam of Pipe | For hydraulic slopes S per thousand | | | | | | |
|-----------------|---|--------|-------|-------|-------|-------|-------|
| | 1.25 | 1. | 0.9 | 0.8 | 0.7 | 0.6 | 0.5 |
| | Approximate discharges in cubic feet per second | | | | | | |
| 1" | 0.003 | 0.002 | 0.002 | 0.002 | 0.002 | 0.002 | 0.002 |
| 2" | 0.016 | 0.014 | 0.014 | 0.013 | 0.012 | 0.011 | 0.010 |
| 3" | 0.044 | 0.039 | 0.037 | 0.035 | 0.032 | 0.030 | 0.028 |
| 4" | 0.089 | 0.080 | 0.076 | 0.071 | 0.067 | 0.062 | 0.056 |
| 5" | 0.156 | 0.139 | 0.132 | 0.124 | 0.116 | 0.108 | 0.099 |
| 6" | 0.246 | 0.219 | 0.208 | 0.196 | 0.184 | 0.170 | 0.155 |
| 7" | 0.361 | 0.323 | 0.306 | 0.289 | 0.270 | 0.250 | 0.228 |
| 8" | 0.504 | 0.451 | 0.428 | 0.403 | 0.377 | 0.349 | 0.319 |
| 9" | 0.677 | 0.605 | 0.574 | 0.541 | 0.506 | 0.469 | 0.428 |
| 10" | 0.881 | 0.787 | 0.747 | 0.704 | 0.659 | 0.610 | 0.557 |
| 11" | 1.117 | 0.999 | 0.948 | 0.894 | 0.836 | 0.774 | 0.706 |
| 12" | 1.389 | 1.242 | 1.178 | 1.111 | 1.039 | 0.962 | 0.878 |
| 1.25 | 2.426 | 2.170 | 2.058 | 1.940 | 1.815 | 1.680 | 1.534 |
| 1.5 | 3.825 | 3.422 | 3.247 | 3.060 | 2.863 | 2.650 | 2.420 |
| 1.75 | 5.625 | 5.031 | 4.773 | 4.500 | 4.209 | 3.898 | 3.558 |
| 2. | 7.855 | 7.025 | 6.665 | 6.284 | 5.877 | 5.442 | 4.968 |
| 2.25 | 10.55 | 9.440 | 8.956 | 8.444 | 7.898 | 7.312 | 6.675 |
| 2.5 | 13.72 | 12.27 | 11.64 | 10.98 | 10.27 | 9.506 | 8.678 |
| 2.75 | 17.41 | 15.57 | 14.77 | 13.93 | 13.03 | 12.06 | 11.01 |
| 3. | 21.64 | 19.36 | 18.37 | 17.3 | 16.20 | 14.99 | 13.69 |
| 3.25 | 26.44 | 23.65 | 22.43 | 21.16 | 19.78 | 18.32 | 16.72 |
| 3.5 | 31.82 | 28.46 | 27.00 | 25.46 | 23.81 | 22.04 | 20.13 |
| 3.75 | 37.81 | 33.82 | 32.08 | 30.24 | 28.29 | 26.20 | 23.91 |
| 4. | 44.42 | 39.74 | 37.70 | 35.54 | 33.25 | 30.78 | 28.10 |
| 4.25 | 51.69 | 46.24 | 43.87 | 41.36 | 38.69 | 34.20 | 32.70 |
| 4.5 | 59.63 | 53.34 | 50.61 | 47.72 | 44.63 | 40.38 | 37.73 |
| 4.75 | 68.26 | 61.06 | 57.93 | 55.88 | 51.09 | 47.30 | 43.18 |
| 5. | 77.62 | 69.43 | 65.87 | 62.10 | 58.09 | 53.78 | 49.10 |
| 5.5 | 98.50 | 88.10 | 83.58 | 78.80 | 73.71 | 68.24 | 62.30 |
| 6. | 122.4 | 109.51 | 103.9 | 97.96 | 91.62 | 84.82 | 77.44 |
| 6.5 | 149.6 | 133.77 | 126.9 | 119.8 | 111.9 | 103.6 | 94.59 |
| 7. | 180.0 | 161.00 | 152.8 | 144.0 | 134.7 | 124.7 | 113.9 |

N.B.—For correct discharge, apply the

PART 2 (continued).

| Diam. of Pipe | For hydraulic slopes S per thousand of | | | | | | |
|------------------|--|-------|-------|-------|-------|-------|-------|
| | 0.4 | 0.3 | 0.25 | 0.2 | 0.15 | 0.1 | 0.05 |
| | Approximate discharges in cubic feet per second] | | | | | | |
| 1" | 0.002 | 0.001 | 0.001 | 0.001 | 0.001 | 0.001 | 0.001 |
| 2" | 0.009 | 0.008 | 0.007 | 0.006 | 0.006 | 0.004 | 0.003 |
| 3" | 0.025 | 0.021 | 0.019 | 0.017 | 0.015 | 0.012 | 0.009 |
| 4" | 0.050 | 0.044 | 0.040 | 0.036 | 0.031 | 0.025 | 0.018 |
| 5" | 0.088 | 0.076 | 0.070 | 0.062 | 0.054 | 0.044 | 0.031 |
| 6" | 0.139 | 0.120 | 0.110 | 0.098 | 0.085 | 0.069 | 0.049 |
| 7" | 0.204 | 0.177 | 0.161 | 0.144 | 0.125 | 0.102 | 0.072 |
| 8" | 0.285 | 0.247 | 0.225 | 0.202 | 0.175 | 0.143 | 0.101 |
| 9" | 0.383 | 0.331 | 0.302 | 0.271 | 0.234 | 0.191 | 0.135 |
| 10" | 0.498 | 0.431 | 0.394 | 0.352 | 0.305 | 0.249 | 0.176 |
| 11" | 0.632 | 0.547 | 0.500 | 0.447 | 0.387 | 0.316 | 0.223 |
| 12" | 0.785 | 0.608 | 0.621 | 0.555 | 0.481 | 0.393 | 0.278 |
| 1.25 | 1.372 | 1.188 | 1.085 | 0.970 | 0.840 | 0.686 | 0.485 |
| 1.5 | 2.164 | 1.874 | 1.711 | 1.530 | 1.325 | 1.082 | 0.765 |
| 1.75 | 3.182 | 2.756 | 2.516 | 2.250 | 1.949 | 1.591 | 1.125 |
| 2. | 4.442 | 3.848 | 3.513 | 3.142 | 2.721 | 2.221 | 1.571 |
| 2.25 | 5.964 | 5.182 | 4.720 | 4.222 | 3.656 | 2.982 | 2.108 |
| 2.5 | 7.762 | 6.722 | 6.136 | 5.489 | 4.753 | 3.881 | 2.744 |
| 2.75 | 9.850 | 8.531 | 7.786 | 6.965 | 6.031 | 4.925 | 3.482 |
| 3. | 12.24 | 10.60 | 9.679 | 8.657 | 7.497 | 6.121 | 4.328 |
| 3.25 | 15.95 | 12.95 | 11.82 | 10.58 | 9.158 | 7.477 | 5.287 |
| 3.5 | 18.00 | 15.59 | 14.23 | 12.73 | 11.02 | 8.999 | 6.363 |
| 3.75 | 21.38 | 18.52 | 16.91 | 15.12 | 13.10 | 10.69 | 7.561 |
| 4. | 25.14 | 21.77 | 19.87 | 17.77 | 15.39 | 12.57 | 8.884 |
| 4.25 | 29.24 | 25.33 | 23.12 | 20.68 | 17.10 | 14.62 | 10.34 |
| 4.5 | 33.74 | 29.22 | 26.67 | 23.86 | 20.19 | 16.87 | 11.93 |
| 4.75 | 38.62 | 33.45 | 30.53 | 27.94 | 23.65 | 19.31 | 13.65 |
| 5. | 43.90 | 38.03 | 34.71 | 31.05 | 26.89 | 21.95 | 15.52 |
| 5.5 | 55.72 | 48.26 | 44.05 | 39.40 | 34.12 | 27.86 | 19.70 |
| 6. | 69.26 | 59.98 | 54.76 | 48.98 | 42.41 | 34.63 | 24.49 |
| 6.5 | 84.60 | 73.27 | 66.89 | 59.83 | 51.81 | 42.30 | 29.91 |
| 7. | 101.8 | 88.19 | 80.50 | 72.01 | 62.36 | 50.91 | 30.00 |

suitable value of c from Table XII.

PART 3.—Approximate diameters of full Pipes

| Discharges in cubic feet per second | For slopes of one in | | | | | | | Tabular No. to be multiplied by $(\frac{1}{S})^{\frac{1}{2}}$ for other slopes |
|--|-------------------------------|-----|------|------|------|------|------|--|
| | 100 | 150 | 200 | 300 | 400 | 500 | 1000 | |
| | Approximate diameters in feet | | | | | | | |
| ·1 | ·23 | ·25 | ·26 | ·29 | ·30 | ·32 | ·36 | ·0916 |
| ·2 | ·30 | ·33 | ·35 | ·38 | ·40 | ·42 | ·48 | ·1208 |
| ·3 | ·36 | ·39 | ·41 | ·44 | ·47 | ·49 | ·57 | ·1421 |
| ·4 | ·40 | ·43 | ·46 | ·50 | ·53 | ·55 | ·63 | ·1594 |
| ·5 | ·44 | ·47 | ·50 | ·55 | ·58 | ·60 | ·69 | ·1743 |
| ·6 | ·47 | ·51 | ·54 | ·59 | ·62 | ·65 | ·75 | ·1875 |
| ·7 | ·50 | ·54 | ·58 | ·62 | ·66 | ·69 | ·79 | ·1994 |
| ·8 | ·53 | ·57 | ·61 | ·66 | ·70 | ·73 | ·84 | ·2104 |
| ·9 | ·56 | ·60 | ·64 | ·69 | ·73 | ·77 | ·88 | ·2215 |
| 1 | ·58 | ·63 | ·66 | ·72 | ·76 | ·80 | ·92 | ·2300 |
| 1·1 | ·60 | ·65 | ·69 | ·75 | ·79 | ·83 | ·95 | ·2385 |
| 1·2 | ·62 | ·67 | ·71 | ·77 | ·82 | ·86 | ·99 | ·2474 |
| 1·3 | ·64 | ·70 | ·74 | ·80 | ·85 | ·89 | 1·02 | ·2556 |
| 1·4 | ·66 | ·72 | ·76 | ·82 | ·87 | ·91 | 1·05 | ·2631 |
| 1·5 | ·68 | ·74 | ·78 | ·85 | ·90 | ·94 | 1·08 | ·2705 |
| 1·6 | ·70 | ·76 | ·80 | ·87 | ·92 | ·96 | 1·11 | ·2776 |
| 1·7 | ·71 | ·77 | ·82 | ·89 | ·94 | ·99 | 1·13 | ·2844 |
| 1·8 | ·73 | ·79 | ·84 | ·91 | ·96 | 1·01 | 1·16 | ·2910 |
| 1·9 | ·75 | ·81 | ·86 | ·93 | ·99 | 1·03 | 1·18 | ·2973 |
| 2·0 | ·76 | ·83 | ·88 | ·95 | 1·01 | 1·05 | 1·21 | ·3035 |
| 2·1 | ·78 | ·84 | ·89 | ·97 | 1·03 | 1·07 | 1·23 | ·3095 |
| 2·2 | ·79 | ·86 | ·91 | ·99 | 1·04 | 1·09 | 1·26 | ·3153 |
| 2·3 | ·81 | ·87 | ·93 | 1·01 | 1·06 | 1·11 | 1·28 | ·3209 |
| 2·4 | ·82 | ·89 | ·94 | 1·02 | 1·08 | 1·13 | 1·30 | ·3265 |
| 2·5 | ·83 | ·90 | ·96 | 1·04 | 1·10 | 1·15 | 1·32 | ·3318 |
| 2·6 | ·85 | ·92 | ·97 | 1·05 | 1·12 | 1·17 | 1·34 | ·3368 |
| 2·7 | ·86 | ·93 | ·99 | 1·07 | 1·13 | 1·19 | 1·36 | ·3422 |
| 2·8 | ·87 | ·95 | 1·00 | 1·09 | 1·15 | 1·20 | 1·38 | ·3472 |
| 2·9 | ·88 | ·96 | 1·02 | 1·10 | 1·17 | 1·22 | 1·40 | ·3521 |
| 3·0 | ·90 | ·97 | 1·03 | 1·12 | 1·18 | 1·24 | 1·42 | ·3569 |

Modify the discharge by a co-efficient (c) before applying it to the table, to find the correct diameter. See Table XII.



of small discharge and high inclination.

| Discharges in cubic feet per second | For slopes of one in | | | | | | | Tabular No. to be multiplied by $(\frac{1}{S})^{\frac{1}{2}}$ for other slopes |
|--|-------------------------------|------|------|-------|-------|-------|-------|--|
| | 500 | 1000 | 1500 | 2000 | 2500 | 3000 | 4000 | |
| | Approximate diameters in feet | | | | | | | |
| 1 | .80 | .92 | .99 | 1.05 | 1.10 | 1.14 | 1.21 | .23 |
| 2 | 1.05 | 1.21 | 1.31 | 1.39 | 1.45 | 1.51 | 1.59 | .30348 |
| 3 | 1.24 | 1.42 | 1.54 | 1.63 | 1.71 | 1.77 | 1.88 | .35692 |
| 4 | 1.39 | 1.59 | 1.73 | 1.83 | 1.91 | 1.99 | 2.10 | .40045 |
| 5 | 1.52 | 1.74 | 1.89 | 2.00 | 2.09 | 2.17 | 2.30 | .43780 |
| 6 | 1.63 | 1.87 | 2.03 | 2.15 | 2.25 | 2.34 | 2.47 | .47096 |
| 7 | 1.74 | 1.99 | 2.16 | 2.29 | 2.40 | 2.48 | 2.63 | .50092 |
| 8 | 1.83 | 2.10 | 2.28 | 2.42 | 2.53 | 2.62 | 2.78 | .52840 |
| 9 | 1.92 | 2.21 | 2.39 | 2.53 | 2.65 | 2.75 | 2.91 | .55389 |
| 10 | 2.00 | 2.30 | 2.49 | 2.64 | 2.76 | 2.87 | 3.04 | .57773 |
| 11 | 2.08 | 2.39 | 2.59 | 2.74 | 2.87 | 2.98 | 3.15 | .60018 |
| 12 | 2.15 | 2.47 | 2.68 | 2.84 | 2.97 | 3.08 | 3.27 | .62144 |
| 13 | 2.22 | 2.55 | 2.76 | 2.93 | 3.07 | 3.18 | 3.37 | .64166 |
| 14 | 2.29 | 2.63 | 2.85 | 3.02 | 3.16 | 3.28 | 3.47 | .66096 |
| 15 | 2.36 | 2.71 | 2.93 | 3.11 | 3.25 | 3.37 | 3.57 | .67946 |
| 16 | 2.42 | 2.78 | 3.01 | 3.19 | 3.32 | 3.46 | 3.66 | .69723 |
| 17 | 2.48 | 2.84 | 3.08 | 3.27 | 3.42 | 3.54 | 3.75 | .71434 |
| 18 | 2.53 | 2.91 | 3.16 | 3.34 | 3.50 | 3.63 | 3.84 | .73086 |
| 19 | 2.59 | 2.97 | 3.22 | 3.42 | 3.57 | 3.70 | 3.92 | .74684 |
| 20 | 2.64 | 3.03 | 3.29 | 3.49 | 3.65 | 3.78 | 4.00 | .76232 |
| 30 | 3.11 | 3.57 | 3.87 | 4.10 | 4.29 | 4.45 | 4.71 | .89655 |
| 40 | 3.49 | 4.00 | 4.34 | 4.60 | 4.81 | 4.99 | 5.28 | 1.0059 |
| 50 | 3.81 | 4.38 | 4.75 | 5.03 | 5.26 | 5.45 | 5.78 | 1.0998 |
| 60 | 4.10 | 4.71 | 5.11 | 5.41 | 5.66 | 5.87 | 6.21 | 1.1830 |
| 70 | 4.36 | 5.01 | 5.43 | 5.75 | 6.02 | 6.24 | 6.61 | 1.2582 |
| 80 | 4.60 | 5.28 | 5.73 | 6.07 | 6.35 | 6.58 | 6.97 | 1.3273 |
| 90 | 4.82 | 5.54 | 6.00 | 6.36 | 6.65 | 6.90 | 7.31 | 1.3913 |
| 100 | 5.03 | 5.78 | 6.26 | 6.64 | 6.94 | 7.20 | 7.62 | 1.4512 |
| 200 | 6.64 | 7.62 | 8.27 | 8.76 | 9.16 | 9.50 | 10.06 | 1.9149 |
| 300 | 7.81 | 8.97 | 9.72 | 10.30 | 10.77 | 11.16 | 11.83 | 2.2520 |

Modify the discharge by a co-efficient (c) before applying it to the table, to find the correct diameter. See Table XII.

PART 4.—Pipes. Approximate

| For discharges in cubic feet per second | For diameters in feet | | | | | Tabular No. to be divided by d^5 for other diameters |
|---|-----------------------------------|--------------|-------------|--------------|--------------|--|
| | ·083 (1") | ·166 (2") | ·25 (3") | ·333 (4") | ·416 (5") | |
| | Approximate head of water in feet | | | | | |
| 0·01 | 16·1 | 0·504 | 0·066 | 0·016 | 0·005 | 0·00006 |
| 0·02 | 64·5 | 2·016 | 0·265 | 0·063 | 0·021 | 0·00026 |
| 0·03 | 145·1 | 4·535 | 0·597 | 0·142 | 0·046 | 0·00058 |
| 0·04 | 258·0 | 8·062 | 1·062 | 0·253 | 0·083 | 0·00104 |
| 0·05 | 403·1 | 12·597 | 1·659 | 0·394 | 0·129 | 0·00162 |
| 0·06 | 580·4 | 18·056 | 2·389 | 0·567 | 0·186 | 0·00233 |
| 0·07 | 790·0 | 24·690 | 3·251 | 0·772 | 0·253 | 0·00318 |
| 0·08 | 1031·8 | 32·248 | 4·247 | 1·008 | 0·330 | 0·00415 |
| 0·09 | 1306·1 | 40·815 | 5·375 | 1·275 | 0·418 | 0·00525 |
| ·1 | 1612· | 50·4 | 6·64 | 1·57 | 0·516 | 0·00648 |
| ·2 | 6450· | 201·6 | 26·54 | 6·30 | 2·064 | 0·02592 |
| ·3 | 14512· | 453·5 | 59·72 | 14·17 | 4·644 | 0·05832 |
| ·4 | | 806·2 | 106·17 | 25·25 | 8·256 | 0·10368 |
| ·5 | | 1259·7 | 165·89 | 39·37 | 12·899 | 0·162 |
| ·6 | | 1805·6 | 238·88 | 56·69 | 18·575 | 0·23328 |
| ·7 | | 2469·0 | 325·14 | 77·16 | 25·283 | 0·31752 |
| ·8 | | 3224·8 | 424·67 | 100·78 | 33·023 | 0·41472 |
| ·9 | | 4081·5 | 537·48 | 127·54 | 41·794 | 0·52488 |
| 1·0 | | 5038·9 | 663·55 | 157·46 | 51·598 | 0·648 |
| 1·1 | | | 802·90 | 190·53 | 62·433 | 0·78408 |
| 1·2 | | | 955·51 | 226·75 | 74·301 | 0·93312 |
| 1·3 | | | 1121·40 | 266·11 | 87·200 | 1·09512 |
| 1·4 | | | 1300·56 | 308·63 | 101·132 | 1·27008 |
| 1·5 | | | 1492·99 | 354·29 | 116·095 | 1·458 |
| 1·6 | | | 1698·69 | 403·11 | 132·090 | 1·65888 |
| 1·7 | | | 1917·67 | 455·07 | 149·118 | 1·87272 |
| 1·8 | | | 2149·92 | 510·18 | 167·177 | 2·09952 |
| 1·9 | | | 2395·42 | 568·44 | 186·268 | 2·33928 |
| 2·0 | | | 2654·21 | 629·86 | 206·391 | 2·592 |

For special cases modify the discharge by a co-efficient (c) before applying it, to find the correct head.

Head for a length of 1 000 feet.

| For discharges in cubic feet per second | For diameters in feet | | | | | Tabular number to be divided by d^5 for other diameters |
|---|-----------------------------------|---------------|---------------|--------------|----------------|---|
| | 0.5 (6") | 0.583 (7") | 0.666 (8") | 0.75 (9") | 0.833 (10") | |
| | Approximate head of water in feet | | | | | |
| 0.1 | 0.207 | 0.096 | 0.049 | 0.027 | 0.016 | 0.00648 |
| 0.2 | 0.829 | 0.384 | 0.197 | 0.107 | 0.064 | 0.02592 |
| 0.3 | 1.866 | 0.863 | 0.443 | 0.246 | 0.145 | 0.05832 |
| 0.4 | 3.318 | 1.535 | 0.787 | 0.437 | 0.258 | 0.10368 |
| 0.5 | 5.184 | 2.398 | 1.230 | 0.683 | 0.403 | 0.162 |
| 0.6 | 7.465 | 3.454 | 1.772 | 0.989 | 0.580 | 0.23328 |
| 0.7 | 10.163 | 4.701 | 2.411 | 1.338 | 0.790 | 0.31752 |
| 0.8 | 13.271 | 6.140 | 3.149 | 1.748 | 1.032 | 0.41472 |
| 0.9 | 16.796 | 7.753 | 3.995 | 2.212 | 1.306 | 0.52488 |
| 1.0 | 20.736 | 9.594 | 4.921 | 2.731 | 1.612 | 0.648 |
| 1.1 | 25.091 | 11.608 | 5.954 | 3.304 | 1.951 | 0.78408 |
| 1.2 | 29.860 | 13.815 | 7.086 | 3.932 | 2.322 | 0.93312 |
| 1.3 | 35.044 | 16.213 | 8.316 | 4.615 | 2.725 | 1.09512 |
| 1.4 | 40.643 | 18.804 | 9.645 | 5.352 | 3.160 | 1.27008 |
| 1.5 | 46.656 | 21.586 | 11.072 | 6.144 | 3.628 | 1.458 |
| 1.6 | 53.084 | 24.560 | 12.597 | 6.991 | 4.128 | 1.65888 |
| 1.7 | 59.927 | 27.726 | 14.221 | 7.892 | 4.660 | 1.87272 |
| 1.8 | 67.185 | 31.084 | 15.943 | 8.847 | 5.224 | 2.09952 |
| 1.9 | 74.857 | 34.633 | 17.764 | 9.858 | 5.821 | 2.33928 |
| 2.0 | 82.944 | 38.375 | 19.683 | 10.974 | 6.450 | 2.592 |
| 2.1 | 91.446 | 42.309 | 21.701 | 12.042 | 7.111 | 2.85768 |
| 2.2 | 100.362 | 45.377 | 23.816 | 13.216 | 7.804 | 3.13632 |
| 2.3 | 109.693 | 50.751 | 26.031 | 14.445 | 8.530 | 3.42792 |
| 2.4 | 119.439 | 55.260 | 28.343 | 15.729 | 9.288 | 3.73248 |
| 2.5 | 129.600 | 59.961 | 30.755 | 17.067 | 10.078 | 4.050 |
| 2.6 | 140.175 | 64.854 | 33.264 | 18.459 | 10.900 | 4.38048 |
| 2.7 | 151.165 | 69.939 | 35.872 | 19.906 | 11.755 | 4.72392 |
| 2.8 | 162.570 | 75.215 | 38.579 | 21.408 | 12.642 | 5.08032 |
| 2.9 | 174.390 | 80.683 | 41.383 | 22.965 | 13.561 | 5.44968 |
| 3.0 | 186.624 | 86.344 | 44.287 | 24.576 | 14.512 | 5.832 |

Modify the discharge by a co-efficient (e) before applying it, to find the correct head.

PART 4 (cont.).—Cylindrical Pipes and Culverts running full.

| For discharges in cubic feet per second | For diameters in feet | | | | | Tabular number to be divided by d^2 for other diameters |
|---|-----------------------------------|--------|--------|-------|-------|---|
| | 1 | 1.5 | 2 | 2.5 | 3.0 | |
| | Approximate head of water in feet | | | | | |
| 1 | 0.648 | 0.085 | 0.020 | 0.007 | 0.003 | 0.648 |
| 2 | 2.592 | 0.341 | 0.081 | 0.027 | 0.011 | 2.592 |
| 3 | 5.832 | 0.768 | 0.182 | 0.060 | 0.024 | 5.832 |
| 4 | 10.368 | 1.365 | 0.324 | 0.106 | 0.043 | 10.368 |
| 5 | 16.2 | 2.133 | 0.506 | 0.166 | 0.067 | 16.2 |
| 6 | 23.328 | 3.072 | 0.729 | 0.239 | 0.096 | 23.328 |
| 7 | 31.752 | 4.181 | 0.992 | 0.325 | 0.131 | 31.752 |
| 8 | 41.472 | 5.461 | 1.296 | 0.425 | 0.167 | 41.472 |
| 9 | 52.488 | 6.912 | 1.640 | 0.537 | 0.216 | 52.488 |
| 10 | 64.8 | 8.533 | 2.025 | 0.664 | 0.267 | 64.8 |
| 11 | 78.408 | 10.325 | 2.450 | 0.803 | 0.323 | 78.408 |
| 12 | 93.312 | 12.288 | 2.916 | 0.956 | 0.384 | 93.312 |
| 13 | 109.512 | 14.421 | 3.422 | 1.121 | 0.451 | 109.512 |
| 14 | 127.008 | 16.725 | 3.969 | 1.301 | 0.523 | 127.008 |
| 15 | 145.8 | 19.2 | 4.556 | 1.493 | 0.600 | 145.8 |
| 16 | 165.888 | 21.845 | 5.184 | 1.699 | 0.667 | 165.888 |
| 17 | 187.272 | 24.661 | 5.852 | 1.918 | 0.771 | 187.272 |
| 18 | 209.952 | 27.648 | 6.501 | 2.150 | 0.864 | 209.952 |
| 19 | 233.928 | 30.805 | 7.310 | 2.395 | 0.963 | 233.928 |
| 20 | 259.2 | 34.133 | 8.1 | 2.654 | 1.067 | 259.2 |
| 21 | | 37.632 | 8.930 | 2.926 | 1.176 | 285.768 |
| 22 | | 41.301 | 9.801 | 3.212 | 1.291 | 313.632 |
| 23 | | 45.141 | 10.712 | 3.510 | 1.411 | 342.792 |
| 24 | | 49.152 | 11.664 | 3.822 | 1.536 | 373.248 |
| 25 | | 53.333 | 12.656 | 4.147 | 1.667 | 405.0 |
| 26 | | 57.685 | 13.689 | 4.486 | 1.803 | 438.048 |
| 27 | | 62.208 | 14.762 | 4.837 | 1.944 | 472.392 |
| 28 | | 66.901 | 15.876 | 5.202 | 2.091 | 508.032 |
| 29 | | 71.765 | 17.030 | 5.580 | 2.243 | 544.968 |
| 30 | | 76.8 | 18.225 | 5.972 | 2.400 | 583.2 |

Modify the discharge by a co-efficient (c) before applying it, to find the correct head.

Approximate Head for a length of 1 000 feet.

| For discharges in cubic feet per second | For diameters in feet | | | | | Tabular numbers to be divided by d^5 for other diameters |
|---|-----------------------------------|---------|---------|---------|---------|--|
| | 3 | 4 | 5 | 6 | 7 | |
| | Approximate head of water in feet | | | | | |
| 1 | 0·003 | 0·0006 | 0·0002 | 0·00008 | 0·00004 | 0·6 |
| 2 | 0·011 | 0·0025 | 0·0008 | 0·00033 | 0·00015 | 2·6 |
| 3 | 0·024 | 0·0057 | 0·0018 | 0·00075 | 0·00035 | 5·8 |
| 4 | 0·043 | 0·0101 | 0·0033 | 0·00133 | 0·00062 | 10·4 |
| 5 | 0·067 | 0·0158 | 0·0052 | 0·00208 | 0·00096 | 16·2 |
| 6 | 0·096 | 0·0228 | 0·0074 | 0·00300 | 0·00139 | 23·3 |
| 7 | 0·131 | 0·0310 | 0·0102 | 0·00408 | 0·00189 | 31·8 |
| 8 | 0·167 | 0·0405 | 0·0133 | 0·00533 | 0·00247 | 41·5 |
| 9 | 0·216 | 0·0513 | 0·0168 | 0·00675 | 0·00312 | 52·5 |
| 10 | 0·267 | 0·0633 | 0·0207 | 0·00833 | 0·00386 | 64·8 |
| 15 | 0·600 | 0·1424 | 0·0466 | 0·01875 | 0·00868 | 145·8 |
| 20 | 1·067 | 0·2531 | 0·0829 | 0·03333 | 0·01542 | 259·2 |
| 25 | 1·667 | 0·3955 | 0·1296 | 0·05208 | 0·02410 | 405·0 |
| 30 | 2·400 | 0·5695 | 0·1866 | 0·07500 | 0·03470 | 583·2 |
| 35 | 3·267 | 0·7752 | 0·2540 | 0·10208 | 0·04723 | 793·8 |
| 40 | 4·267 | 1·0132 | 0·3318 | 0·13333 | 0·06169 | 1036·8 |
| 45 | 5·400 | 1·2815 | 0·4199 | 0·16875 | 0·07875 | 1312·2 |
| 50 | 6·667 | 1·5823 | 0·5184 | 0·20833 | 0·09639 | 1620·0 |
| 55 | 8·067 | 1·9143 | 0·6273 | 0·25208 | 0·11663 | 1960·2 |
| 60 | 9·600 | 2·2781 | 0·7465 | 0·30000 | 0·13880 | 2332·8 |
| 65 | 11·267 | 2·6736 | 0·8761 | 0·35208 | 0·16289 | 2737·8 |
| 70 | 13·067 | 3·1008 | 1·0161 | 0·40833 | 0·18892 | 3175·2 |
| 75 | 15·000 | 3·5596 | 1·1664 | 0·46875 | 0·21687 | 3645·0 |
| 80 | 16·678 | 4·0500 | 1·3271 | 0·53333 | 0·24675 | 4117·2 |
| 85 | 19·267 | 4·5721 | 1·4982 | 0·60208 | 0·27856 | 4681·8 |
| 90 | 21·600 | 5·1258 | 1·6796 | 0·67500 | 0·31230 | 5248·8 |
| 95 | 24·067 | 5·7112 | 1·8714 | 0·75208 | 0·34796 | 5848·2 |
| 100 | | 6·3281 | 2·0736 | 0·83333 | 0·38555 | 6480· |
| 200 | | 25·3120 | 18·2944 | 3·33333 | 1·54222 | 25920· |
| 300 | | 56·9530 | 18·6624 | 7·50000 | 3·46998 | 58320· |

Modify the discharge by a co-efficient (c) before applying it, to find the correct head.

PART 5.—Conditions of drain pipes and culverts of equal discharge (Q) running per 1 000, V the mean velocity

CYLINDRICAL CULVERTS, with a co-efficient of rugosity $n = 0.013$.

| Q in cubic feet per second | d | S per 1 000 | c | V | d | S per 1 000 | c | V | Q in cubic feet per second | d | S per 1 000 | c | V | |
|------------------------------|---------------|---------------|------|------|------|---------------|-----|-----|------------------------------|---------------|---------------|------|------|------|
| 1 | d | 0.66 | 0.75 | 0.83 | 1. | | | | 40 | d | 3. | 3.5 | 4. | 4.5 |
| | S per 1 000 | 8.51 | 4.36 | 2.48 | 0.87 | | | | | S per 1 000 | 3.52 | 1.54 | 0.75 | 0.41 |
| | c | 0.76 | 0.79 | 0.81 | 0.85 | | | | | c | 1.10 | 1.14 | 1.16 | 1.18 |
| 2 | d | 0.83 | 1. | 1.25 | 1.50 | | | | 50 | d | 3.5 | 4. | 4.5 | 5. |
| | S per 1 000 | 9.83 | 3.58 | 1.05 | 0.40 | | | | | S per 1 000 | 2.40 | 1.18 | 0.63 | 0.36 |
| | c | 0.81 | 0.85 | 0.90 | 0.92 | | | | | c | 1.14 | 1.17 | 1.19 | 1.20 |
| 4 | d | 1. | 1.25 | 1.5 | 1.75 | | | | 60 | d | 4. | 4.5 | 5. | 5.5 |
| | S per 1 000 | 14.3 | 4.17 | 1.57 | 0.67 | | | | | S per 1 000 | 1.69 | 0.90 | 0.52 | 0.31 |
| | c | 0.85 | 0.90 | 0.95 | 0.98 | | | | | c | 1.17 | 1.19 | 1.20 | 1.22 |
| 6 | d | 1.25 | 1.50 | 1.75 | 2.0 | | | | 80 | d | 5. | 5.5 | 6. | 6.5 |
| | S per 1 000 | 9.37 | 3.43 | 1.48 | 0.72 | | | | | S per 1 000 | 0.95 | 0.55 | 0.34 | 0.23 |
| | c | 0.90 | 0.95 | 0.98 | 1.01 | | | | | c | 1.21 | 1.23 | 1.24 | 1.24 |
| 8 | d | 1.25 | 1.50 | 1.75 | 2.0 | | | | 100 | d | 5.5 | 6. | 6.5 | 7. |
| | S per 1 000 | 16.7 | 6.11 | 2.63 | 1.28 | | | | | S per 1 000 | 0.85 | 0.54 | 0.36 | 0.24 |
| | c | 0.90 | 0.95 | 0.98 | 1.01 | | | | | c | 1.23 | 1.24 | 1.26 | 1.26 |
| 10 | d | 1.5 | 1.75 | 2. | 2.25 | | | | 120 | d | 6. | 6.5 | 7. | 7.5 |
| | S per 1 000 | 9.57 | 4.10 | 1.98 | 1.05 | | | | | S per 1 000 | 0.77 | 0.51 | 0.39 | 0.24 |
| | c | 0.95 | 0.98 | 1.01 | 1.04 | | | | | c | 1.25 | 1.26 | 1.27 | 1.28 |
| 15 | d | 2. | 2.25 | 2.5 | 2.75 | | | | 140 | d | 6.5 | 7. | 7.5 | 8. |
| | S per 1 000 | 4.46 | 2.35 | 1.34 | 0.79 | | | | | S per 1 000 | 0.69 | 0.47 | 0.33 | 0.23 |
| | c | 1.01 | 1.04 | 1.06 | 1.08 | | | | | c | 1.26 | 1.27 | 1.28 | 1.29 |
| 20 | d | 2.25 | 2.5 | 2.75 | 3. | | | | 160 | d | 7. | 7.5 | 8. | |
| | S per 1 000 | 4.18 | 2.36 | 1.41 | 0.88 | | | | | S per 1 000 | 0.61 | 0.42 | 0.30 | |
| | c | 1.04 | 1.06 | 1.08 | 1.10 | | | | | c | 1.28 | 1.29 | 1.30 | |
| 25 | d | 2.5 | 2.75 | 3. | 3.5 | | | | 180 | d | 7. | 7.5 | 8. | |
| | S per 1 000 | 3.68 | 2.20 | 1.39 | 0.60 | | | | | S per 1 000 | 0.77 | 0.53 | 0.38 | |
| | c | 1.06 | 1.08 | 1.10 | 1.13 | | | | | c | 1.28 | 1.29 | 1.30 | |
| 30 | d | 2.75 | 3. | 3.5 | 4. | | | | 200 | d | 7. | 8. | | |
| | S per 1 000 | 3.17 | 1.98 | 0.86 | 0.42 | | | | | S per 1 000 | 0.94 | 0.47 | | |
| | c | 1.08 | 1.10 | 1.14 | 1.16 | | | | | c | 1.28 | 1.30 | | |
| | V | 5.05 | 4.24 | 3.12 | 2.39 | | | | | V | 5.20 | 3.98 | | |

For materials see values of n in Table XII. V is generally above 2.5 feet per second to pre-

just full ; d being the transverse diameter in feet, S per 1 000 the fall in feet per second.

HAWKSLEY'S OVOID CULVERT, with a co-efficient of rugosity, $n = 0.013$.

| Q in cubic feet per second | Diameter (d) | | | | Q in cubic feet per second | Diameter (d) | | | | |
|----------------------------|---------------|-------|-------|-------|----------------------------|---------------|-------|-------|-------|------|
| | 1' 0" | 1' 2" | 1' 4" | 1' 6" | | 3' 6" | 4' 0" | 4' 4" | 4' 8" | |
| 4 | S per 1 000 | 7.59 | 3.26 | 1.60 | 0.77 | S per 1 000 | 1.87 | 0.90 | 0.60 | 0.41 |
| | c | 0.88 | 0.91 | 0.94 | 0.97 | c | 1.16 | 1.19 | 1.20 | 1.21 |
| | V | 4.02 | 2.95 | 2.26 | 1.78 | V | 4.92 | 3.77 | 3.21 | 2.77 |
| 6 | S per 1 000 | 17.07 | 7.31 | 3.52 | 1.86 | S per 1 000 | 1.24 | 0.81 | 0.55 | 0.38 |
| | c | 0.88 | 0.91 | 0.94 | 0.97 | c | 1.20 | 1.20 | 1.21 | 1.22 |
| | V | 6.03 | 4.43 | 3.39 | 2.68 | V | 4.39 | 3.75 | 3.23 | 2.81 |
| 8 | S per 1 000 | 13.00 | 6.24 | 3.29 | 1.86 | S per 1 000 | 1.05 | 0.72 | 0.50 | 0.36 |
| | c | 0.91 | 0.94 | 0.97 | 0.99 | c | 1.20 | 1.22 | 1.23 | 1.23 |
| | V | 5.90 | 4.52 | 3.57 | 2.89 | V | 4.28 | 3.69 | 3.21 | 2.83 |
| 10 | S per 1 000 | 9.75 | 5.11 | 2.89 | 1.76 | S per 1 000 | 0.90 | 0.53 | 0.45 | 0.3 |
| | c | 0.94 | 0.97 | 0.99 | 1.01 | c | 1.22 | 1.23 | 1.24 | 1.25 |
| | V | 5.65 | 4.46 | 3.62 | 2.99 | V | 4.15 | 3.62 | 3.18 | 2.85 |
| 15 | S per 1 000 | 6.48 | 3.86 | 2.43 | 1.60 | S per 1 000 | 0.77 | 0.55 | 0.46 | 0.41 |
| | c | 0.99 | 1.01 | 1.03 | 1.05 | c | 1.23 | 1.24 | 1.24 | 1.25 |
| | V | 5.42 | 4.48 | 3.76 | 3.21 | V | 4.02 | 3.53 | 3.32 | 3.17 |
| 20 | S per 1 000 | 6.86 | 4.28 | 2.47 | 2.89 | S per 1 000 | 0.79 | 0.66 | 0.59 | 0.43 |
| | c | 1.01 | 1.03 | 1.07 | 1.10 | c | 1.24 | 1.25 | 1.25 | 1.26 |
| | V | 5.97 | 5.02 | 3.69 | 2.83 | V | 4.24 | 3.99 | 3.80 | 3.35 |
| 25 | S per 1 000 | 6.67 | 2.90 | 1.41 | 0.75 | S per 1 000 | 1.06 | 0.90 | 0.80 | 0.58 |
| | c | 1.03 | 1.07 | 1.10 | 1.12 | c | 1.25 | 1.25 | 1.26 | 1.27 |
| | V | 6.23 | 4.61 | 3.53 | 2.78 | V | 4.94 | 4.65 | 4.44 | 3.91 |
| 30 | S per 1 000 | 4.16 | 2.01 | 1.07 | 0.61 | S per 1 000 | 1.17 | 1.03 | 0.75 | |
| | c | 1.07 | 1.10 | 1.13 | 1.14 | c | 1.25 | 1.26 | 1.27 | |
| | V | 5.54 | 4.24 | 3.34 | 2.71 | V | 5.31 | 3.04 | 4.46 | |
| 40 | S per 1 000 | 3.58 | 1.90 | 1.09 | 0.65 | S per 1 000 | 1.28 | 0.91 | | |
| | c | 1.10 | 1.13 | 1.15 | 1.16 | c | 1.26 | 1.27 | | |
| | V | 5.65 | 4.46 | 3.62 | 2.99 | V | 5.70 | 5.02 | | |
| 50 | S per 1 000 | 3.06 | 1.69 | 1.01 | 0.64 | S per 1 000 | 1.15 | | | |
| | c | 1.13 | 1.15 | 1.17 | 1.18 | c | 1.27 | | | |
| | V | 5.57 | 4.52 | 3.74 | 3.14 | V | 5.58 | | | |

vent deposition of sediment. For long diameter and sectional data, see Table V. Part 4.

PART 5 (cont.).—Conditions of drain pipes and culverts of equal discharge (Q) per 1000, V the mean

METROPOLITAN OVOID CULVERT, with a co-efficient of rugosity, $n = 0.013$.

| Q in cubic feet per second | | | | Q in cubic feet per second | | | | | | | |
|----------------------------|-------------|--------|--------|----------------------------|--------|-----|-------------|-------|-------|-------|-------|
| 4 | d | 1' 0 | 1' 2" | 1' 4" | 1' 6" | 60 | d | 3' 6" | 4' 0" | 4' 4" | 4' 8" |
| | S per 1 000 | 5' 31 | 2' 30 | 1' 12 | 0' 94 | | S per 1 000 | 1' 33 | 0' 65 | 0' 43 | 0' 29 |
| | c | 0' 89 | 0' 92 | 0' 95 | 0' 98 | | c | 1' 17 | 1' 19 | 1' 20 | 1' 21 |
| 6 | V | 3' 48 | 2' 56 | 1' 96 | 1' 55 | 70 | d | 4' 26 | 3' 27 | 2' 78 | 2' 40 |
| | S per 1 000 | 11' 92 | 5' 12 | 2' 49 | 1' 33 | | S per 1 000 | 0' 86 | 0' 58 | 0' 39 | 0' 27 |
| | c | 0' 89 | 0' 92 | 0' 95 | 0' 98 | | c | 1' 19 | 1' 21 | 1' 22 | 1' 23 |
| 8 | V | 5' 22 | 3' 84 | 2' 94 | 2' 32 | 80 | d | 3' 81 | 3' 25 | 2' 80 | 2' 36 |
| | S per 1 000 | 1' 2" | 1' 4" | 1' 6" | 1' 8" | | S per 1 000 | 4' 4" | 4' 8" | 5' 0" | 5' 4" |
| | c | 0' 92 | 0' 95 | 0' 98 | 1' 00 | | c | 1' 21 | 1' 22 | 1' 23 | 1' 24 |
| 10 | V | 5' 12 | 3' 92 | 3' 10 | 2' 51 | 90 | d | 3' 71 | 3' 20 | 2' 69 | 2' 45 |
| | S per 1 000 | 1' 4" | 1' 6" | 1' 8" | 1' 10" | | S per 1 000 | 4' 8" | 5' 0" | 5' 4" | 5' 8" |
| | c | 0' 95 | 0' 98 | 1' 00 | 1' 03 | | c | 1' 22 | 1' 23 | 1' 24 | 1' 24 |
| 15 | V | 4' 90 | 3' 87 | 3' 13 | 2' 59 | 100 | d | 3' 60 | 3' 03 | 2' 75 | 2' 44 |
| | S per 1 000 | 1' 8" | 1' 10" | 2' 0" | 2' 2" | | S per 1 000 | 5' 0" | 5' 4" | 5' 6" | 5' 8" |
| | c | 1' 00 | 1' 03 | 1' 05 | 1' 06 | | c | 1' 24 | 1' 25 | 1' 25 | 1' 25 |
| 20 | V | 4' 70 | 3' 89 | 3' 27 | 2' 78 | 120 | d | 3' 37 | 3' 06 | 2' 88 | 2' 77 |
| | S per 1 000 | 1' 10" | 2' 0" | 2' 4" | 2' 8" | | S per 1 000 | 5' 4" | 5' 6" | 5' 8" | 6' 0" |
| | c | 1' 03 | 1' 05 | 1' 08 | 1' 10 | | c | 1' 25 | 1' 25 | 1' 26 | 1' 26 |
| 25 | V | 5' 18 | 4' 35 | 3' 20 | 2' 45 | 140 | d | 3' 67 | 3' 45 | 3' 25 | 2' 99 |
| | S per 1 000 | 2' 0" | 2' 4" | 2' 8" | 3' 0" | | S per 1 000 | 5' 4" | 5' 6" | 5' 8" | 6' 0" |
| | c | 1' 05 | 1' 08 | 1' 11 | 1' 13 | | c | 1' 25 | 1' 26 | 1' 26 | 1' 26 |
| 30 | V | 5' 44 | 3' 99 | 3' 06 | 2' 42 | 160 | d | 4' 29 | 4' 03 | 3' 80 | 3' 33 |
| | S per 1 000 | 2' 4" | 2' 8" | 3' 0" | 3' 4" | | S per 1 000 | 5' 4" | 5' 6" | 5' 8" | 6' 0" |
| | c | 1' 08 | 1' 11 | 1' 13 | 1' 15 | | c | 1' 26 | 1' 26 | 1' 26 | 1' 26 |
| 40 | V | 4' 80 | 3' 67 | 2' 90 | 2' 35 | 180 | d | 4' 00 | 4' 60 | 4' 34 | 3' 88 |
| | S per 1 000 | 2' 8" | 3' 0" | 3' 4" | 3' 8" | | S per 1 000 | 5' 6" | 5' 8" | 6' 0" | 6' 0" |
| | c | 1' 10 | 1' 14 | 1' 16 | 1' 17 | | c | 1' 26 | 1' 27 | 1' 28 | 1' 28 |
| 50 | V | 4' 90 | 3' 87 | 3' 13 | 2' 60 | 200 | d | 5' 18 | 4' 88 | 4' 35 | |
| | S per 1 000 | 3' 0" | 3' 4" | 3' 8" | 4' 0" | | S per 1 000 | 5' 8" | 6' 0" | | |
| | c | 1' 14 | 1' 16 | 1' 17 | 1' 19 | | c | 1' 27 | 1' 28 | | |
| | V | 4' 84 | 3' 92 | 3' 24 | 2' 72 | | d | 5' 42 | 4' 84 | | |

For long diameter and sectional data see Table V. Part 4.

running just full; d being the transverse diameter in feet, S per 1000 the fall velocity in feet per second.

JACKSON'S OVOID (PEG-TOP SECTION) CULVERT, with a co-efficient of rugosity, $n = 0.013$.

| Q in cubic feet per second | | | | | Q in cubic feet per second | | | | |
|----------------------------|--------|---------------|-------|--------|----------------------------|-------|---------------|-------|-------|
| | d | S per 1 000 | c | V | | d | S per 1 000 | c | V |
| 4 | 1' 0" | 1' 2" | 1' 4" | 1' 6" | 60 | 3' 6" | 4' 0" | 4' 4" | 4' 8" |
| | 0.87 | 0.90 | 0.93 | 0.96 | | 1.15 | 1.18 | 1.19 | 1.20 |
| | 3.85 | 2.83 | 2.17 | 1.71 | | 4.72 | 3.61 | 3.08 | 2.65 |
| 6 | 1' 0" | 1' 2" | 1' 4" | 1' 6" | 70 | 4' 0" | 4' 4" | 4' 8" | 5' 0" |
| | 16.50 | 7.06 | 3.41 | 1.81 | | 1.17 | 0.78 | 0.53 | 0.37 |
| | 0.87 | 0.90 | 0.93 | 0.96 | | 1.18 | 1.19 | 1.21 | 1.21 |
| 8 | 1' 2" | 1' 4" | 1' 6" | 1' 8" | 80 | 4' 4" | 4' 8" | 5' 0" | 5' 4" |
| | 12.56 | 6.04 | 3.04 | 1.80 | | 1.13 | 0.69 | 0.46 | 0.34 |
| | 0.90 | 0.93 | 0.96 | 0.99 | | 1.20 | 1.21 | 1.22 | 1.23 |
| 10 | 1' 4" | 1' 6" | 1' 8" | 1' 10" | 90 | 4' 8" | 5' 0" | 5' 4" | 5' 8" |
| | 9.43 | 4.93 | 2.78 | 1.68 | | 0.87 | 0.60 | 0.43 | 0.31 |
| | 0.93 | 0.96 | 0.99 | 1.01 | | 1.21 | 1.22 | 1.23 | 1.24 |
| 15 | 1' 8" | 1' 10" | 2' 0" | 2' 2" | 100 | 5' 0" | 5' 4" | 5' 8" | 6' 0" |
| | 6.23 | 3.70 | 2.33 | 1.54 | | 0.74 | 0.53 | 0.45 | 0.39 |
| | 0.99 | 1.01 | 1.03 | 1.05 | | 1.22 | 1.23 | 1.24 | 1.24 |
| 20 | 1' 10" | 2' 0" | 2' 4" | 2' 8" | 120 | 5' 4" | 5' 8" | 6' 0" | 6' 4" |
| | 6.56 | 4.11 | 1.81 | 0.87 | | 0.76 | 0.64 | 0.55 | 0.41 |
| | 1.01 | 1.03 | 1.06 | 1.09 | | 1.24 | 1.24 | 1.25 | 1.25 |
| 25 | 2' 0" | 2' 4" | 2' 8" | 3' 0" | 140 | 5' 8" | 6' 0" | 6' 4" | 6' 8" |
| | 6.42 | 2.79 | 1.36 | 0.72 | | 1.02 | 0.87 | 0.75 | 0.55 |
| | 1.03 | 1.06 | 1.09 | 1.11 | | 1.24 | 1.24 | 1.25 | 1.26 |
| 30 | 2' 4" | 2' 8" | 3' 0" | 3' 4" | 160 | 6' 0" | 6' 4" | 6' 8" | 7' 0" |
| | 4.00 | 1.94 | 1.03 | 0.59 | | 1.13 | 0.97 | 0.72 | |
| | 1.06 | 1.09 | 1.12 | 1.13 | | 1.25 | 1.25 | 1.26 | |
| 40 | 2' 8" | 3' 0" | 3' 4" | 3' 8" | 180 | 6' 4" | 6' 8" | 7' 0" | 7' 4" |
| | 3.45 | 1.84 | 1.04 | 0.63 | | 1.20 | 0.91 | | |
| | 1.09 | 1.12 | 1.14 | 1.16 | | 1.25 | 1.25 | 1.26 | |
| 50 | 3' 0" | 3' 4" | 3' 8" | 4' 0" | 200 | 6' 8" | 7' 0" | 7' 4" | 7' 8" |
| | 2.86 | 1.62 | 0.97 | 0.61 | | 1.11 | | | |
| | 1.12 | 1.14 | 1.16 | 1.17 | | 1.26 | | | |
| | 5' 35 | 4' 33 | 3' 58 | 3' 01 | | 5' 35 | | | |

The values of V in sewers should exceed 2.5 feet per second to prevent deposit.

EXPLANATORY EXAMPLES TO TABLE VIII.

EXAMPLE I.

What is the discharge of a new glazed 3-inch pipe having a hydraulic slope of 1 in 400; and what would be its least full discharge when old, irrespectively of sectional obstruction?

By Table VIII., Part 2, the approximate discharge is $\cdot 06$ cubic feet per second; and by the Table of Co-efficients (Table XII., Part 3), for very smooth surfaces, including smooth plaster, and enamelled or glazed pipes, the co-efficient c for a pipe having this slope and a hydraulic radius, which for cylindrical pipes running full is one-fourth of the diameter, is $\cdot 84$; hence the discharge when new is $= \cdot 84 \times \cdot 06 = \cdot 05$ cubic feet per second.

If preferred in any other unit, refer to Table II., Part 4, p. 12, by inspecting which we find this to be 18 gallons per minute.

When the pipe is rather old its surface will be as rough as that of ordinary metal, and taking the co-efficient for metal with this slope and radius to be $\cdot 61$, the discharge is then $= \cdot 61 \times \cdot 06 = \cdot 037$ cubic feet per second or 14 gallons per minute.

NOTE.—In this example, the co-efficient adopted for roughness (n) of glazed surfaces is $0\cdot 010$, and that for unglazed metal surfaces is $0\cdot 013$; the corresponding co-efficients of velocity will be found under them in Table XII.

EXAMPLE II.

A cylindrical masonry culvert has a diameter of 42 inches, and a fall of 5 in 1 000, what is its discharge when just running full?

By Part 2, Table VIII., the approximate discharge is $63\cdot 63$ cubic feet per second, and the co-efficient for this slope and a hydraulic radius of $\cdot 875$ feet will according to Table XII. be $1\cdot 10$; hence the actual discharge will be $1\cdot 10 \times 63\cdot 63 = 70$ cubic feet per second.

NOTE.—The co-efficient of roughness (n) for new ashlar masonry is $0\cdot 013$, the required velocity co-efficients (c) will be found under it in Table XII.

EXAMPLE III.

What must be the diameter of a cylindrical cast-iron pipe to discharge 20 cubic feet per second with a slope of one in 500?

By Part 3, Table VIII., the approximate diameter will be $2\cdot 64$ feet; and hence the hydraulic radius is $0\cdot 66$ feet; from the table of co-efficients (Table XII., Part 3), take $c = 1\cdot 03$; and assuming a modified discharge $\frac{Q}{c} = 19\cdot 4$, refer again to Part 3, Table VIII., and obtain a true diameter $= 2\cdot 62$ feet.

EXAMPLE 4.

A series of glazed pipes has a total head of 30 feet, and consists of 3 600 feet of 8-inch pipe, 2 100 feet of 6-inch, and 600 feet of 5-inch; required the discharge and head necessary for each pipe.

Assume any discharge as 1 cubic foot per second, and obtaining the separate tabular heads due to it in Part 4, Table VIII., divide the total head in the same proportion.

| | |
|---|---|
| $4\cdot921 \times 3\cdot6 = 17\cdot72$ | $17\cdot72 \times 30 \div 92 = 5\cdot77$ feet |
| $20\cdot736 \times 2\cdot1 = 43\cdot55$ | $43\cdot55 \times 30 \div 92 = 14\cdot15$,, |
| $51\cdot598 \times 0\cdot6 = 30\cdot95$ | $30\cdot95 \times 30 \div 92 = 10\cdot08$,, |
| Total = 92·22 | Total = 30· feet. |

Modifying these by the squares of the suitable co-efficients, obtain actual heads for a first approximation, and reduce them by proportion.

| | |
|--------------------------------------|--|
| $5\cdot77 \div (.95)^2 = 6\cdot41$ | $6\cdot41 \times 30 \div 39\cdot22 = 4\cdot90$ feet in 3 600 |
| $14\ 15 \div (.87)^2 = 18\cdot92$ | $18\cdot62 \times 30 \div 39\cdot22 = 14\cdot24$,, in 2 100 |
| $10\cdot08 \div (.84)^2 = 14\cdot19$ | $14\cdot19 \times 30 \div 39\cdot22 = 10\cdot86$,, in 600 |
| Total = 39·22 | Total = 30· feet. |

The discharge = $\frac{1 \times \sqrt{30}}{\sqrt{92}} = 0\cdot57$ cubic feet per second: and this by Table II., Part 4, is 213 gallons per minute.

EXAMPLE 5.

A discharge of 300 gallons per minute is required through a series of ordinary iron pipes composed of 800 yards of 7-inch, 300 yards of 6-inch, and 100 yards of 5-inch; what is the head required for each pipe?

By Tables of Equivalentents (Part 4, Table II.), 300 gallons per minute = 0·8 cubic feet per second. Taking the corresponding tabular heads in Part 4, Table VIII., as first approximations, and modifying these by the squares of the suitable co-efficients given in Table XII., we get the true heads thus:—

| Length. | Heads. | True Heads. |
|---------|---|---|
| 7 inch | $6\cdot140 \times 2\cdot4 = 14\cdot74$ | $14\cdot74 \div (.66)^2 = 33\cdot50$ feet |
| 6 inch | $13\cdot271 \times 0\cdot9 = 11\cdot94$ | $11\cdot94 \div (.63)^2 = 29\cdot85$,, |
| 5 inch | $33\cdot023 \times 0\cdot3 = 9\cdot91$ | $9\cdot91 \div (.61)^2 = 26\cdot78$,, |
| | 36·59 feet. | Total 90·13 feet. |

NOTE.—The squares and the reciprocals required with co-efficients can be obtained through the Table of Powers and Roots in the Miscellaneous Tables.

EXAMPLE 6.

Required the dimensions and conditions of a brickwork sewer, the section Metropolitan Ovoid, to discharge 50 cubic feet per second; with a hydraulic slope, or fall per 1 000 of 1·40 feet, when running just full.

By inspecting page 102, Part 5, Table VIII.; the mean velocity will be 3·92 feet per second, and the transverse diameter will be 3 feet 4 inches; referring to Table V., Part 4, page 58, its long diameter is 5 feet, and its sectional area 12·76 square feet.

EXAMPLE 7.

What will be the mean velocity in the sewer last mentioned, when its supply is reduced so that it runs one-third full, that is, the depth of liquid is one-third the depth of the sewer?

By Table V., Part 4, page 58, the section of flow will be 3·156 square feet, and the hydraulic radius 0·689 feet, and the fall per 1 000 is still 1·40 feet. By interpolating Part 1 of Table VIII. at page 85, the approximate velocity is 3·098 feet per second; and obtaining from Table XII. the co-efficient suitable to these values of R and S , which is 1·08; we obtain the true velocity = $3·098 \times 1·08 = 3·35$ feet per second; also $Q = AV = 3·16 \times 3·35 = 10·59$ cubic feet per second.

EXAMPLE 8.

What will be the discharge in the same sewer when it is running two-thirds full, or filled to two-thirds its depth? the remaining conditions being as before.

By Table V., part 4, page 58, the section of flow will be 8·4 square feet and the hydraulic radius 1·052 feet; the fall per 1 000 is still 1·40 feet. By Table XII. the co-efficient of velocity under $n = 0·013$ will be 1·18. By interpolating Part 1, Table VIII., page 85, the approximate velocity is 3·822; hence $Q = A . c . 100 \sqrt{RS} = 8·4 \times 1·18 \times 3·822 = 37·88$ cubic feet per second.

NOTE.—Many of these calculations may be abbreviated by using accented four-figure logarithms. For tables of velocity and discharge in culverts and pipes of various sections under different values of n , see 'Canal and Culvert Tables' (London: Allen, 1878).

TABLE IX.—BENDS AND OBSTRUCTIONS.

Part 1. Giving loss of head in feet due to bends of 90° in pipes corresponding to certain discharges.—(Weisbach formula.)

$$h' = \frac{v^2}{2g} \cdot \frac{d}{2R}; \quad R = \text{radius of bend.}$$

Part 2. Giving loss of head due to bends in channels corresponding to certain velocities.—(Mississippi formula.)

$$h' = N \cdot V^2 \times 0.001865.$$

Part 3. Giving approximate rise of water in feet due to obstructions, bridges, weirs, &c. :—(the whole section of water being = 1), and corresponding to certain velocities.—(Dubuat formula.)

$$h' = \frac{v^2}{2g \cdot \sigma^2} \left(\frac{A^2}{a^2} - 1 \right), \quad \text{when } \sigma = 0.96.$$

NOTE.—This table does not allow for variable co-efficients, and hence is merely generally correct for ordinary purposes.

PART I.—Table giving loss of head due to one bend of 90°

| Diameter of pipe | Radius of bend | Loss of head of water in feet | | | | | |
|------------------|----------------|---|------|-------|-------|-------|-------|
| | | 0·01 | 0·05 | 0·1 | 0·2 | 0·3 | |
| | Feet | Corresponding to discharges in cubic feet per second of | | | | | |
| (1") | ·083 | ·5 | ·02 | ·04 | ·05 | ·06 | ·09 |
| (2") | ·166 | ·5 | ·07 | ·15 | ·22 | ·30 | ·37 |
| (3") | ·25 | 1' | ·15 | ·34 | ·49 | ·69 | ·84 |
| (4") | ·33 | 1' | ·26 | ·59 | ·84 | 1·18 | 1·45 |
| (5") | ·416 | 1·5 | ·43 | ·96 | 1·33 | 1·93 | 2·36 |
| (6") | ·5 | 1·5 | ·61 | 1·35 | 1·92 | 2·71 | 3·32 |
| (7") | ·583 | 1·5 | ·81 | 1·81 | 2·56 | 3·62 | 4·34 |
| (8") | ·66 | 1·5 | 1·06 | 2·38 | 3·34 | 4·76 | 5·83 |
| (9") | ·75 | 1·5 | 1·32 | 2·94 | 4·16 | 5·89 | 7·21 |
| (10") | ·83 | 1·5 | 1·57 | 3·52 | 4·98 | 7·04 | 8·63 |
| (11") | ·916 | 1·5 | 1·89 | 4·22 | 5·96 | 8·43 | 10·33 |
| (12") | 1·0 | 1·75 | 2·27 | 5·08 | 7·18 | 10·16 | 12·44 |
| | 1·25 | 2' | 3·4 | 7·7 | 10·9 | 15·4 | 18·9 |
| | 1·5 | 2·5 | 5·0 | 11·3 | 15·9 | 22·6 | 27·6 |
| | 1·75 | 3' | 6·9 | 15·5 | 21·9 | 30·9 | 37·9 |
| | 2' | 4' | 9·4 | 20·9 | 29·6 | 41·9 | 51·3 |
| | 2·25 | 4·5 | 11·8 | 26·5 | 38·5 | 53·0 | 64·9 |
| | 2·5 | 5·0 | 14·6 | 32·7 | 46·2 | 65·4 | 80·1 |
| | 2·75 | 5·5 | 17·7 | 39·6 | 56·0 | 79·2 | 97·0 |
| | 3' | 6' | 21·1 | 47·1 | 66·6 | 94·2 | 115·4 |
| | 3·25 | 6·5 | 24·7 | 55·3 | 78·2 | 110·6 | 135·4 |
| | 3·5 | 7' | 28·7 | 64·1 | 90·7 | 128·2 | 157·1 |
| | 3·75 | 7·5 | 32·9 | 73·6 | 104·1 | 147·2 | 180·3 |
| | 4' | 8' | 37·3 | 83·9 | 118·7 | 167·9 | 205·6 |
| | 4·5 | 9' | 47·4 | 106·0 | 149·9 | 212·0 | 259·6 |
| | 5' | 10' | 58·5 | 130·9 | 185·1 | 261·7 | 320·5 |
| | 5·5 | 11' | 71· | 158· | 224· | 317· | 388· |
| | 6' | 12' | 84· | 188· | 266· | 377· | 462· |
| | 6·5 | 13' | 99· | 221· | 313· | 442· | 542· |
| | 7' | 14' | 115· | 336· | 363· | 513· | 628· |

NOTE.—Interpolate the given discharge in the horizontal line corresponding to line. See example following the table.

for cylindrical pipes with different discharges.

| Loss of head of water in feet | | | | | | | |
|---|-------|-------|-------|-------|-------|-------|-------|
| 0·4 | 0·5 | 0·6 | 0·7 | 0·8 | 0·9 | 1 | 2 |
| Corresponding to discharges in cubic feet per second of | | | | | | | |
| ·10 | ·12 | ·13 | ·14 | ·15 | ·16 | ·17 | ·23 |
| ·43 | ·48 | ·53 | ·57 | ·61 | ·64 | ·68 | ·96 |
| ·97 | 1·08 | 1·19 | 1·28 | 1·37 | 1·46 | 1·53 | 2·17 |
| 1·66 | 1·87 | 2·05 | 2·21 | 2·36 | 2·51 | 2·64 | 3·74 |
| 2·66 | 3·05 | 3·34 | 3·61 | 3·86 | 4·00 | 4·31 | 6·09 |
| 3·83 | 4·29 | 4·69 | 5·07 | 5·42 | 5·75 | 6·06 | 8·55 |
| 5·12 | 5·73 | 6·28 | 6·78 | 7·25 | 7·69 | 8·10 | 11·46 |
| 6·69 | 7·52 | 8·24 | 8·90 | 9·52 | 10·03 | 10·64 | 15·05 |
| 8·33 | 9·31 | 10·20 | 11·02 | 11·78 | 12·49 | 13·17 | 18·58 |
| 9·96 | 11·13 | 12·20 | 13·18 | 14·09 | 14·94 | 15·75 | 22·27 |
| 11·92 | 13·34 | 14·61 | 15·78 | 16·87 | 17·89 | 18·86 | 26·67 |
| 14·36 | 16·06 | 17·60 | 19·01 | 20·32 | 21·55 | 22·71 | 32·12 |
| 21·8 | 24·4 | 26·7 | 28·9 | 30·9 | 32·7 | 34·5 | 48·1 |
| 31·9 | 35·7 | 39·1 | 42·2 | 45·1 | 47·9 | 50·4 | 71·4 |
| 43·7 | 48·9 | 53·5 | 57·8 | 61·8 | 65·6 | 69·1 | 97·7 |
| 59·2 | 66·2 | 72·5 | 78·3 | 83·7 | 88·8 | 93·6 | 132·4 |
| 74·9 | 83·8 | 91·8 | 99·2 | 106·0 | 112·4 | 118·5 | 167·6 |
| 92·4 | 103·4 | 113·3 | 122·4 | 130·9 | 138·8 | 146·3 | 206·9 |
| 112·0 | 125·2 | 137·1 | 148·1 | 158·3 | 167·9 | 177·0 | 250·3 |
| 133·2 | 148·9 | 163·2 | 176·3 | 188·4 | 199·9 | 210·7 | 297·9 |
| 156·4 | 174·8 | 191·5 | 206·9 | 221·1 | 234·6 | 247·2 | 349·7 |
| 181·3 | 202·8 | 222·1 | 239·9 | 256·4 | 272·0 | 286·7 | 405·5 |
| 208·2 | 232·8 | 255·0 | 275·4 | 294·4 | 311·1 | 329·2 | 465·5 |
| 237·4 | 265·4 | 290·8 | 314·1 | 335·8 | 356·1 | 375·4 | 530·9 |
| 299·8 | 335·2 | 367·2 | 396·6 | 424·0 | 449·7 | 474·0 | 670·4 |
| 370·1 | 413·8 | 453·3 | 489·6 | 523·4 | 555·2 | 585·2 | 827·6 |
| 448· | 501· | 548· | 592· | 633· | 672· | 708· | 1001· |
| 533· | 596· | 653· | 705· | 754· | 800· | 843· | 1192· |
| 625· | 699· | 766· | 827· | 885· | 938· | 989· | 1399· |
| 725· | 811· | 888· | 960· | 1026· | 1088· | 1147· | 1622· |

the given diameter of pipe, and obtain the loss of head by interpolation in the head-

PART 2.—Loss of head due to Bends of Channels.

| Velocity in feet per second | Arc of bend | | | | | |
|-----------------------------|----------------------|-------|-------|-------|-------|--------|
| | 10° | 15° | 30° | 60° | 90° | 180° |
| | Loss of head in feet | | | | | |
| 0·25 | ·0000 | ·0001 | ·0001 | ·0002 | ·0003 | ·0007 |
| 0·5 | ·0002 | ·0002 | ·0005 | ·0009 | ·0014 | ·0028 |
| 0·75 | ·0003 | ·0005 | ·0011 | ·0021 | ·0031 | ·0063 |
| 1 | ·0006 | ·0009 | ·0019 | ·0037 | ·0056 | ·0112 |
| 1·25 | ·0009 | ·0015 | ·0029 | ·0058 | ·0087 | ·0175 |
| 1·5 | ·0014 | ·0021 | ·0042 | ·0084 | ·0126 | ·0252 |
| 1·75 | ·0019 | ·0029 | ·0057 | ·0113 | ·0171 | ·0343 |
| 2 | ·0025 | ·0050 | ·0075 | ·0149 | ·0224 | ·0448 |
| 2·25 | ·0031 | ·0047 | ·0094 | ·0189 | ·0284 | ·0566 |
| 2·5 | ·0039 | ·0058 | ·0117 | ·0233 | ·0349 | ·0699 |
| 2·75 | ·0047 | ·0071 | ·0141 | ·0282 | ·0423 | ·0846 |
| 3 | ·0056 | ·0084 | ·0168 | ·0336 | ·0504 | ·1007 |
| 3·25 | ·0066 | ·0099 | ·0197 | ·0394 | ·0591 | ·1182 |
| 3·5 | ·0076 | ·0114 | ·0227 | ·0457 | ·0685 | ·1371 |
| 3·75 | ·0087 | ·0131 | ·0262 | ·0524 | ·0786 | ·1573 |
| 4 | ·0099 | ·0149 | ·0298 | ·0597 | ·0895 | ·1790 |
| 4·25 | ·0103 | ·0164 | ·0327 | ·0674 | ·1011 | ·2021 |
| 4·5 | ·0126 | ·0189 | ·0378 | ·0755 | ·1133 | ·2266 |
| 4·75 | ·0140 | ·0210 | ·0421 | ·0842 | ·1262 | ·2525 |
| 5 | ·0155 | ·0233 | ·0466 | ·0933 | ·1399 | ·2798 |
| 5·5 | ·0188 | ·0282 | ·0564 | ·1128 | ·1692 | ·3385 |
| 6 | ·0224 | ·0336 | ·0671 | ·1343 | ·2014 | ·4028 |
| 6·5 | ·0263 | ·0394 | ·0788 | ·1576 | ·2364 | ·4728 |
| 7 | ·0305 | ·0457 | ·0914 | ·1828 | ·2742 | ·5483 |
| 7·5 | ·0350 | ·0525 | ·1049 | ·2098 | ·3147 | ·6294 |
| 8 | ·0398 | ·0597 | ·1194 | ·2387 | ·3581 | ·7162 |
| 8·5 | ·0449 | ·0674 | ·1347 | ·2695 | ·4042 | ·8085 |
| 9 | ·0503 | ·0756 | ·1511 | ·3021 | ·4532 | ·9064 |
| 9·5 | ·0561 | ·0842 | ·1683 | ·3366 | ·5049 | 1·0099 |
| 10 | ·0622 | ·0933 | ·1865 | ·3730 | ·5595 | 1·1190 |

PART 3.—*Rise from Obstructions in Channels.*

| Antecedent velocity in feet per second | Proportion of Section Obstructed | | | | | |
|---|----------------------------------|-------|-------|-------|-------|-------|
| | 0·1 | 0·2 | 0·3 | 0·4 | 0·5 | 0·6 |
| | Rise in feet | | | | | |
| 0·25 | 0·000 | 0·001 | 0·001 | 0·002 | 0·003 | 0·006 |
| 0·5 | 0·001 | 0·002 | 0·004 | 0·008 | 0·013 | 0·022 |
| 0·75 | 0·002 | 0·005 | 0·010 | 0·017 | 0·028 | 0·050 |
| 1 | 0·004 | 0·009 | 0·018 | 0·031 | 0·051 | 0·089 |
| 1·25 | 0·006 | 0·015 | 0·028 | 0·047 | 0·079 | 0·136 |
| 1·5 | 0·009 | 0·022 | 0·040 | 0·068 | 0·114 | 0·199 |
| 1·75 | 0·012 | 0·029 | 0·054 | 0·092 | 0·155 | 0·272 |
| 2 | 0·015 | 0·038 | 0·070 | 0·120 | 0·203 | 0·355 |
| 2·25 | 0·020 | 0·048 | 0·089 | 0·152 | 0·257 | 0·499 |
| 2·5 | 0·024 | 0·060 | 0·110 | 0·188 | 0·317 | 0·555 |
| 2·75 | 0·029 | 0·072 | 0·133 | 0·227 | 0·383 | 0·671 |
| 3 | 0·035 | 0·086 | 0·158 | 0·271 | 0·456 | 0·798 |
| 3·25 | 0·041 | 0·101 | 0·192 | 0·318 | 0·536 | 0·937 |
| 3·5 | 0·048 | 0·117 | 0·215 | 0·369 | 0·621 | 1·087 |
| 3·75 | 0·055 | 0·134 | 0·247 | 0·423 | 0·713 | 1·248 |
| 4 | 0·062 | 0·153 | 0·281 | 0·481 | 0·811 | 1·420 |
| 4·25 | 0·070 | 0·173 | 0·318 | 0·543 | 0·916 | 1·603 |
| 4·5 | 0·079 | 0·194 | 0·356 | 0·609 | 1·027 | 1·797 |
| 4·75 | 0·088 | 0·216 | 0·394 | 0·679 | 1·144 | 2·002 |
| 5 | 0·097 | 0·239 | 0·439 | 0·752 | 1·268 | 2·218 |
| 5·5 | 0·118 | 0·289 | 0·532 | 0·910 | 1·534 | 2·684 |
| 6 | 0·140 | 0·344 | 0·633 | 1·083 | 1·825 | 3·194 |
| 6·5 | 0·164 | 0·404 | 0·768 | 1·271 | 2·142 | 3·748 |
| 7 | 0·191 | 0·468 | 0·861 | 1·474 | 2·484 | 4·347 |
| 7·5 | 0·219 | 0·538 | 0·989 | 1·692 | 2·852 | 4·991 |
| 8 | 0·249 | 0·611 | 1·125 | 1·925 | 3·245 | 5·678 |
| 8·5 | 0·281 | 0·691 | 1·270 | 2·173 | 3·663 | 6·410 |
| 9 | 0·315 | 0·774 | 1·424 | 2·436 | 4·107 | 7·186 |
| 9·5 | 0·351 | 0·863 | 1·587 | 2·715 | 4·576 | 8·007 |
| 10 | 0·389 | 0·956 | 1·758 | 3·008 | 5·070 | 8·872 |

EXPLANATORY EXAMPLES TO TABLE IX.

EXAMPLE 1.

A series of pipes have to discharge 5 gallons per second; there are 7 bends in the portion that consists of 5-inch pipe, 4 in that of 6-inch pipe, and 8 in that of 7-inch pipe; what is the total loss of head on account of these bends?

From Table II. Part 4, page 13, 5 gallons per second = 0.8 cubic feet per second. Taking the heads separately from Table IX. Part 1,

$$\begin{aligned} 7 \text{ bends in 5-inch give } & 7 \times 0.045 = 0.315 \text{ feet.} \\ 4 \text{ " " 6 " " } & 4 \times 0.030 = 0.120 \text{ " " } \\ 8 \text{ " " 7 " " } & 8 \times 0.010 = 0.080 \text{ " " } \\ \text{Total loss of head} & = 0.515 \text{ feet.} \end{aligned}$$

The head on the pipes must therefore not only be sufficient to force 0.8 cubic feet per second through the pipes under ordinary conditions, but must also be increased by 0.515 feet on account of bends.

EXAMPLE 2.

A channel has one bend of 15° , two of 30° , and one of 90° , what is the total loss of head expended in overcoming these bends, when the velocity is 5 feet per second?

From Part 2, Table IX.

$$\begin{aligned} 1 \text{ bend of } 15^\circ \text{ gives } & 1 \times 0.0233 = 0.0233 \text{ feet.} \\ 2 \text{ " " } 30^\circ \text{ " " } & 2 \times 0.0466 = 0.0932 \text{ " " } \\ 1 \text{ " " } 90^\circ \text{ " " } & 1 \times 0.1399 = 0.1399 \text{ " " } \\ \text{Total head expended} & = 0.2564 \text{ feet.} \end{aligned}$$

EXAMPLE 3.

A channel having a hydraulic slope less than 0.001 has its section obstructed by the piers and abutments of a bridge to the extent of one-fifth, the normal velocity being 3.5 per second, what is the rise caused by the bridge?

By Part 3, Table IX., the rise will be 0.117 feet.

NOTE.—For channels having steeper hydraulic slopes, that is, falls of more than 1 foot in 1 000, apply a correction according to the formula given in the text, page 106.

TABLE X.—ORIFICES AND OVERFALLS.

Velocities of discharge in feet per second for sluices, and orifices, due to various heads for certain co-efficients, also theoretical velocities to which any co-efficient may be applied; being an application of the formula

$$V = o \times 8.025 \sqrt{H},$$

where for orifices H = depth of centre of motion of orifice.

The same table also applies to overfalls, weirs, and notches, but in this case using the same general formula, H is the depth from still water to sill-level; but the velocity given in the table must be reduced by one-third to obtain velocity of discharge for any overfall, as by formula

$$V = \frac{2}{3} \cdot 0 \times 8.025 \sqrt{H}.$$

For values of (o) the co-efficient, see Parts 5 and 6, Table XII.

This table can also be used for the converse purpose.

To obtain the discharge (Q) in either case

$$Q = A V,$$

where A is the hydraulic section, see text, page 115.

TABLE X.—*Orifices and Overfalls.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|-------------------------|----------------------------|--------------------------|-------------------|----------------------|------------------------|
| | For natural velocity | For narrow bridge-openings | For velocity of approach | For special weirs | For special orifices | For broad crested dams |
| | 1. | 9. | 8. | 7. | 6. | 5. |
| | Velocities of discharge | | | | | |
| ·01 | ·803 | ·722 | ·642 | ·562 | ·482 | ·401 |
| ·02 | 1·135 | 1·021 | ·908 | ·794 | ·681 | ·567 |
| ·03 | 1·390 | 1·251 | 1·112 | ·973 | ·834 | ·695 |
| ·04 | 1·605 | 1·445 | 1·284 | 1·123 | ·963 | ·803 |
| ·05 | 1·794 | 1·615 | 1·435 | 1·256 | 1·076 | ·897 |
| ·06 | 1·966 | 1·769 | 1·573 | 1·376 | 1·180 | ·983 |
| ·07 | 2·123 | 1·911 | 1·698 | 1·486 | 1·274 | 1·062 |
| ·08 | 2·270 | 2·043 | 1·816 | 1·589 | 1·362 | 1·135 |
| ·09 | 2·408 | 2·167 | 1·926 | 1·686 | 1·445 | 1·204 |
| ·1 | 2·538 | 2·284 | 2·030 | 1·777 | 1·523 | 1·269 |
| ·2 | 3·589 | 3·230 | 2·871 | 2·512 | 2·153 | 1·794 |
| ·3 | 4·395 | 3·956 | 3·516 | 3·078 | 2·637 | 2·198 |
| ·4 | 5·075 | 4·568 | 4·060 | 3·553 | 3·045 | 2·538 |
| ·5 | 5·675 | 5·108 | 4·540 | 3·973 | 3·405 | 2·837 |
| ·6 | 6·216 | 5·594 | 4·973 | 4·351 | 3·730 | 3·108 |
| ·7 | 6·714 | 6·043 | 5·371 | 4·700 | 4·028 | 3·352 |
| ·8 | 7·178 | 6·460 | 5·742 | 5·025 | 4·307 | 3·589 |
| ·9 | 7·613 | 6·852 | 6·090 | 5·329 | 4·568 | 3·807 |
| 1. | 8·025 | 7·223 | 6·420 | 5·618 | 4·815 | 4·013 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|--------------------------|------------------|-------------------|---------------------|------------------------|----------------------|
| | For wide bridge-openings | For lock sluices | For special weirs | For weirs generally | For orifices generally | For special orifices |
| | ·96 | ·84 | ·727 | ·666 | ·62 | ·55 |
| | Velocities of discharge | | | | | |
| ·01 | ·770 | ·674 | ·584 | ·535 | ·498 | ·441 |
| ·02 | 1·089 | ·953 | ·825 | ·756 | ·704 | ·624 |
| ·03 | 1·334 | 1·168 | 1·011 | ·926 | ·862 | ·765 |
| ·04 | 1·541 | 1·348 | 1·167 | 1·069 | ·995 | ·883 |
| ·05 | 1·722 | 1·507 | 1·304 | 1·185 | 1·112 | ·987 |
| ·06 | 1·887 | 1·651 | 1·429 | 1·309 | 1·219 | 1·081 |
| ·07 | 2·038 | 1·783 | 1·543 | 1·414 | 1·316 | 1·169 |
| ·08 | 2·179 | 1·907 | 1·650 | 1·512 | 1·407 | 1·249 |
| ·09 | 2·311 | 2·023 | 1·751 | 1·604 | 1·493 | 1·324 |
| ·1 | 2·436 | 2·132 | 1·845 | 1·690 | 1·574 | 1·396 |
| ·2 | 3·445 | 3·014 | 2·609 | 2·390 | 2·225 | 1·973 |
| ·3 | 4·219 | 3·694 | 3·195 | 2·927 | 2·725 | 2·418 |
| ·4 | 4·872 | 4·264 | 3·689 | 3·380 | 3·147 | 2·792 |
| ·5 | 5·448 | 4·768 | 4·126 | 3·780 | 3·519 | 3·121 |
| ·6 | 5·968 | 5·221 | 4·519 | 4·140 | 3·854 | 3·419 |
| ·7 | 6·445 | 5·640 | 4·881 | 4·471 | 4·163 | 3·687 |
| ·8 | 6·890 | 6·030 | 5·218 | 4·781 | 4·450 | 3·948 |
| ·9 | 7·308 | 6·395 | 5·535 | 5·070 | 4·720 | 4·187 |
| 1 | 7·704 | 6·742 | 5·834 | 5·345 | 4·976 | 4·414 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|-------------------------|----------------------------|--------------------------|-------------------|----------------------|------------------------|
| | For natural velocity | For narrow bridge-openings | For velocity of approach | For special weirs | For special orifices | For broad-crested dam; |
| | 1 | 9 | 8 | 7 | 6 | 5 |
| | Velocities of discharge | | | | | |
| 1 | 8.0250 | 7.223 | 6.420 | 5.618 | 4.815 | 4.013 |
| 1.25 | 8.9722 | 8.075 | 7.178 | 6.281 | 5.383 | 4.486 |
| 1.5 | 9.8286 | 8.846 | 7.863 | 6.880 | 5.897 | 4.915 |
| 1.75 | 19.6161 | 9.554 | 8.943 | 7.431 | 6.370 | 5.308 |
| 2 | 11.3491 | 10.214 | 9.079 | 7.944 | 6.809 | 5.675 |
| 2.25 | 12.0375 | 10.834 | 9.630 | 8.426 | 7.223 | 6.019 |
| 2.5 | 12.6886 | 11.420 | 10.151 | 8.882 | 7.613 | 6.345 |
| 2.75 | 13.3079 | 11.977 | 10.646 | 9.316 | 7.985 | 6.654 |
| 3 | 13.8997 | 12.510 | 11.120 | 9.730 | 8.340 | 6.950 |
| 3.25 | 14.4673 | 13.020 | 11.574 | 10.127 | 8.680 | 7.234 |
| 3.5 | 15.0134 | 13.512 | 12.010 | 10.509 | 9.008 | 7.507 |
| 3.75 | 15.5403 | 13.986 | 12.432 | 10.878 | 9.324 | 7.770 |
| 4 | 16.0500 | 14.445 | 12.840 | 11.235 | 9.630 | 8.025 |
| 4.25 | 16.5439 | 14.890 | 13.235 | 11.581 | 9.926 | 8.272 |
| 4.5 | 17.0235 | 15.322 | 13.619 | 11.916 | 10.214 | 8.512 |
| 4.75 | 17.4901 | 15.741 | 13.992 | 12.243 | 10.494 | 8.745 |
| 5 | 17.9444 | 16.150 | 14.355 | 12.561 | 10.767 | 8.972 |
| 5.25 | 18.3876 | 16.549 | 14.710 | 12.871 | 11.033 | 9.194 |
| 5.5 | 18.8203 | 16.938 | 15.056 | 13.174 | 11.292 | 9.410 |
| 5.75 | 19.2433 | 17.319 | 15.395 | 13.470 | 11.546 | 9.622 |
| 6 | 19.6572 | 17.691 | 15.726 | 13.760 | 11.794 | 9.829 |
| 6.25 | 20.0625 | 18.057 | 16.050 | 14.044 | 12.038 | 10.032 |
| 6.5 | 20.4598 | 18.414 | 16.368 | 14.322 | 12.276 | 10.230 |
| 6.75 | 20.8496 | 18.765 | 16.680 | 14.595 | 12.510 | 10.425 |
| 7 | 21.2322 | 19.109 | 16.986 | 14.863 | 12.739 | 10.616 |
| 7.25 | 21.6079 | 19.447 | 17.286 | 15.126 | 12.965 | 10.804 |
| 7.5 | 21.9774 | 19.779 | 17.582 | 15.384 | 13.186 | 10.989 |
| 7.75 | 22.3406 | 20.107 | 17.873 | 15.638 | 13.404 | 11.171 |
| 8 | 22.6981 | 20.428 | 18.158 | 15.889 | 13.619 | 11.349 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|--------------------------|------------------|-------------------|---------------------|------------------------|----------------------|
| | For wide bridge-openings | For lock sluices | For special weirs | For weirs generally | For orifices generally | For special orifices |
| | ·96 | ·84 | ·727 | ·666 | ·62 | ·55 |
| | Velocities of discharge | | | | | |
| 1 | 7·704 | 6·741 | 5·836 | 5·345 | 4·975 | 4·413 |
| 1·25 | 8·614 | 7·537 | 6·525 | 5·976 | 5·562 | 4·934 |
| 1·50 | 9·436 | 8·256 | 7·147 | 6·546 | 6·109 | 5·420 |
| 1·75 | 10·192 | 8·918 | 7·720 | 7·071 | 6·582 | 5·839 |
| 2 | 10·895 | 9·533 | 8·253 | 7·558 | 6·936 | 6·241 |
| 2·25 | 11·556 | 10·112 | 8·754 | 8·017 | 7·461 | 6·621 |
| 2·50 | 12·181 | 10·659 | 9·227 | 8·451 | 7·867 | 6·978 |
| 2·75 | 12·776 | 11·179 | 9·678 | 8·863 | 8·251 | 7·319 |
| 3 | 13·344 | 11·676 | 10·108 | 9·257 | 8·618 | 7·645 |
| 3·25 | 13·879 | 12·153 | 10·521 | 9·635 | 8·825 | 7·957 |
| 3·50 | 14·413 | 12·612 | 10·918 | 9·999 | 9·308 | 8·258 |
| 3·75 | 14·919 | 13·054 | 11·301 | 10·350 | 9·635 | 8·547 |
| 4 | 15·408 | 13·482 | 11·672 | 10·689 | 9·951 | 8·827 |
| 4·25 | 15·882 | 13·897 | 12·027 | 11·018 | 10·257 | 9·099 |
| 4·50 | 16·343 | 14·300 | 12·380 | 11·338 | 10·554 | 9·363 |
| 4·75 | 16·800 | 14·695 | 12·718 | 11·651 | 10·846 | 9·622 |
| 5 | 17·227 | 15·074 | 13·049 | 11·952 | 10·121 | 9·865 |
| 5·25 | 17·652 | 15·446 | 13·372 | 12·247 | 11·400 | 10·113 |
| 5·50 | 18·068 | 15·809 | 13·686 | 12·534 | 11·669 | 10·351 |
| 5·75 | 18·474 | 16·165 | 13·994 | 12·817 | 11·931 | 10·584 |
| 6 | 18·871 | 16·512 | 14·295 | 13·092 | 12·188 | 10·812 |
| 6·25 | 19·260 | 16·853 | 14·590 | 13·362 | 12·439 | 11·034 |
| 6·50 | 19·642 | 17·187 | 14·879 | 13·627 | 12·685 | 11·253 |
| 6·75 | 20·016 | 17·514 | 15·162 | 13·886 | 12·927 | 11·467 |
| 7 | 20·383 | 17·835 | 15·440 | 14·141 | 13·164 | 11·688 |
| 7·25 | 20·744 | 18·181 | 15·714 | 14·391 | 13·402 | 11·889 |
| 7·50 | 21·099 | 18·481 | 15·982 | 14·637 | 13·626 | 12·082 |
| 7·75 | 21·447 | 18·767 | 16·246 | 14·879 | 13·851 | 12·287 |
| 8 | 21·791 | 19·067 | 16·506 | 15·117 | 14·073 | 12·484 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|-------------------------|----------------------------|--------------------------|-------------------|----------------------|------------------------|
| | For natural velocity | For narrow bridge-openings | For velocity of approach | For special weirs | For special orifices | For broad-crested dams |
| | 1 | ·9 | ·8 | ·7 | ·6 | ·5 |
| | Velocities of discharge | | | | | |
| 8·25 | 23·051 | 20·746 | 18·441 | 16·135 | 13·831 | 11·525 |
| 8·50 | 23·397 | 21·057 | 18·717 | 16·377 | 14·032 | 11·698 |
| 8·75 | 23·739 | 21·365 | 18·992 | 16·617 | 14·243 | 11·869 |
| 9 | 24·076 | 21·668 | 19·261 | 16·853 | 14·445 | 12·038 |
| 9·25 | 24·408 | 21·996 | 19·526 | 17·085 | 14·645 | 12·204 |
| 9·50 | 24·735 | 22·261 | 19·788 | 17·316 | 14·841 | 12·367 |
| 9·75 | 25·059 | 22·553 | 20·047 | 17·541 | 15·035 | 12·529 |
| 10 | 25·378 | 22·840 | 20·302 | 17·764 | 15·227 | 12·689 |
| 10·5 | 26·005 | 23·404 | 20·804 | 18·203 | 15·603 | 13·002 |
| 11 | 26·617 | 23·955 | 21·293 | 18·631 | 15·970 | 13·308 |
| 11·5 | 27·215 | 24·493 | 21·772 | 19·050 | 16·329 | 13·607 |
| 12 | 27·800 | 25·020 | 22·240 | 19·460 | 16·680 | 13·900 |
| 12·5 | 28·373 | 25·535 | 22·698 | 19·861 | 17·024 | 14·186 |
| 13 | 28·935 | 26·041 | 23·148 | 20·254 | 17·361 | 14·467 |
| 13·5 | 29·486 | 26·545 | 23·596 | 20·646 | 17·697 | 14·747 |
| 14 | 30·027 | 27·024 | 24·021 | 21·019 | 18·016 | 15·013 |
| 14·5 | 30·559 | 27·503 | 24·447 | 21·391 | 18·335 | 15·279 |
| 15 | 31·081 | 27·973 | 24·864 | 21·756 | 18·648 | 15·540 |
| 15 | 31·594 | 28·434 | 25·275 | 22·115 | 18·956 | 15·797 |
| 16·5 | 32·101 | 28·891 | 25·681 | 22·470 | 19·261 | 16·050 |
| 16·5 | 32·598 | 29·338 | 26·078 | 22·818 | 19·555 | 16·299 |
| 17 | 33·089 | 29·780 | 26·471 | 23·162 | 19·853 | 16·544 |
| 17·5 | 33·572 | 30·214 | 26·857 | 23·500 | 20·143 | 16·786 |
| 18 | 34·048 | 30·643 | 27·238 | 23·833 | 20·429 | 17·024 |
| 18·5 | 34·518 | 31·066 | 27·614 | 24·162 | 20·711 | 17·259 |
| 19 | 34·981 | 31·483 | 27·985 | 24·486 | 20·988 | 17·490 |
| 19·5 | 35·438 | 31·894 | 28·350 | 24·806 | 21·283 | 17·719 |
| 20 | 35·889 | 32·300 | 29·711 | 25·122 | 21·533 | 17·944 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|--------------------------|------------------|-------------------|---------------------|------------------------|----------------------|
| | For wide bridge-openings | For lock sluices | For special weirs | For weirs generally | For orifices generally | For special orifices |
| | 96 | 84 | 727 | 666 | 62 | 55 |
| | Velocities of discharge | | | | | |
| 8.25 | 22.129 | 19.362 | 16.762 | 15.352 | 14.292 | 12.677 |
| 8.50 | 22.461 | 19.654 | 17.014 | 15.582 | 14.506 | 12.867 |
| 8.75 | 22.789 | 19.941 | 17.263 | 15.810 | 14.718 | 13.056 |
| 9. | 23.112 | 20.223 | 17.508 | 16.034 | 14.927 | 13.242 |
| 9.25 | 23.431 | 20.502 | 17.749 | 16.256 | 15.133 | 13.424 |
| 9.50 | 23.746 | 20.778 | 17.987 | 16.473 | 15.336 | 13.604 |
| 9.75 | 24.056 | 28.049 | 18.223 | 16.689 | 15.536 | 13.782 |
| 10. | 24.363 | 21.317 | 18.455 | 16.902 | 15.734 | 13.958 |
| 10.5 | 24.964 | 21.844 | 18.910 | 17.112 | 16.123 | 14.302 |
| 11. | 25.552 | 22.358 | 19.355 | 17.727 | 16.502 | 14.639 |
| 11.5 | 26.126 | 22.860 | 19.791 | 18.125 | 16.873 | 14.968 |
| 12. | 26.688 | 23.352 | 20.216 | 18.515 | 17.236 | 15.290 |
| 12.5 | 27.238 | 23.834 | 20.633 | 18.897 | 17.591 | 15.605 |
| 13. | 27.778 | 24.306 | 21.042 | 19.271 | 17.940 | 15.914 |
| 13.5 | 28.307 | 24.769 | 21.442 | 19.637 | 18.287 | 16.222 |
| 14. | 28.826 | 25.223 | 21.836 | 19.998 | 18.617 | 16.514 |
| 14.5 | 29.337 | 25.670 | 22.222 | 20.352 | 18.946 | 16.807 |
| 15. | 29.838 | 26.108 | 22.602 | 20.700 | 19.270 | 17.094 |
| 15.5 | 30.331 | 26.540 | 22.976 | 21.042 | 19.588 | 17.377 |
| 16. | 30.817 | 26.965 | 23.344 | 21.379 | 19.903 | 17.655 |
| 16.5 | 31.294 | 27.383 | 23.706 | 21.711 | 20.207 | 17.929 |
| 17. | 31.765 | 27.794 | 24.062 | 22.037 | 20.515 | 18.198 |
| 17.5 | 32.229 | 28.200 | 24.413 | 22.358 | 20.815 | 18.465 |
| 18. | 32.686 | 28.600 | 24.760 | 22.676 | 21.110 | 18.726 |
| 18.5 | 33.137 | 28.995 | 25.101 | 22.988 | 21.391 | 18.985 |
| 19. | 33.582 | 29.384 | 25.438 | 23.298 | 21.688 | 19.239 |
| 19.5 | 34.021 | 29.768 | 25.771 | 23.602 | 21.991 | 19.491 |
| 20. | 34.454 | 30.147 | 26.091 | 23.902 | 22.251 | 19.739 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—continued.

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|-------------------------|----------------------------|--------------------------|-------------------|----------------------|------------------------|
| | For natural velocity | For narrow bridge-openings | For velocity of approach | For special weirs | For special orifices | For broad-crested dams |
| | 1 | ·9 | ·8 | ·7 | ·6 | ·5 |
| | Velocities of discharge | | | | | |
| 20·5 | 36·336 | 32·702 | 29·068 | 25·435 | 21·801 | 18·168 |
| 21· | 36·776 | 33·098 | 29·420 | 25·743 | 22·066 | 18·388 |
| 21·5 | 37·211 | 33·490 | 29·768 | 26·047 | 22·327 | 18·605 |
| 22· | 37·641 | 33·877 | 30·112 | 26·348 | 22·585 | 18·820 |
| 22·5 | 38·067 | 34·260 | 30·453 | 26·646 | 22·840 | 19·033 |
| 23· | 38·487 | 34·647 | 30·797 | 26·948 | 23·098 | 19·298 |
| 23·5 | 38·903 | 35·012 | 31·122 | 27·232 | 23·342 | 19·451 |
| 24· | 39·315 | 35·383 | 31·452 | 27·520 | 23·589 | 19·657 |
| 24·5 | 39·723 | 35·750 | 31·778 | 27·806 | 23·834 | 19·861 |
| 25· | 40·126 | 36·113 | 32·100 | 28·088 | 24·075 | 20·063 |
| 25·5 | 40·525 | 36·472 | 32·420 | 28·367 | 24·315 | 20·262 |
| 26· | 40·921 | 36·837 | 32·737 | 28·644 | 24·553 | 20·460 |
| 26·5 | 41·312 | 37·180 | 33·049 | 28·918 | 24·787 | 20·656 |
| 27· | 41·700 | 37·530 | 33·360 | 29·190 | 25·020 | 20·850 |
| 27·5 | 42·084 | 37·875 | 33·667 | 29·458 | 25·250 | 21·042 |
| 28· | 42·465 | 38·218 | 33·972 | 29·725 | 25·479 | 21·232 |
| 28·5 | 42·843 | 38·558 | 34·275 | 29·990 | 25·706 | 21·421 |
| 29· | 43·216 | 38·890 | 34·569 | 30·248 | 25·927 | 21·606 |
| 29·5 | 43·588 | 39·229 | 34·870 | 30·511 | 26·153 | 21·794 |
| 30· | 43·956 | 39·560 | 35·164 | 30·779 | 26·374 | 21·978 |
| 30·5 | 44·320 | 39·888 | 35·456 | 31·024 | 26·592 | 22·160 |
| 31· | 44·682 | 40·213 | 35·745 | 31·277 | 26·809 | 22·340 |
| 31·5 | 45·041 | 40·537 | 36·032 | 31·528 | 27·025 | 22·520 |
| 32· | 45·397 | 40·857 | 36·317 | 31·778 | 27·238 | 22·698 |
| 32·5 | 45·751 | 41·176 | 36·601 | 32·025 | 27·451 | 22·875 |
| 33· | 46·101 | 41·491 | 36·880 | 32·270 | 27·660 | 23·050 |
| 33·5 | 46·449 | 41·804 | 37·159 | 32·514 | 27·869 | 23·224 |
| 34· | 46·794 | 42·114 | 37·435 | 32·755 | 28·076 | 23·397 |
| 34·5 | 47·137 | 42·423 | 37·709 | 32·996 | 28·282 | 23·568 |
| 35· | 47·478 | 42·730 | 37·982 | 33·234 | 28·487 | 23·739 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE X.—*continued.*

| Effective head in feet | CO-EFFICIENTS | | | | | |
|------------------------|--------------------------|------------------|-------------------|---------------------|------------------------|----------------------|
| | For wide bridge-openings | For lock sluices | For special weirs | For weirs generally | For orifices generally | For special orifices |
| | ·96 | ·84 | ·727 | ·666 | ·62 | ·55 |
| | Velocities of discharge | | | | | |
| 20·5 | 34·882 | 30·522 | 26·423 | 24·199 | 22·528 | 19·985 |
| 21· | 35·305 | 30·892 | 26·737 | 24·493 | 22·701 | 20·227 |
| 21·5 | 35·723 | 31·257 | 27·060 | 24·783 | 22·971 | 20·465 |
| 22· | 36·136 | 31·619 | 27·373 | 25·069 | 23·337 | 20·702 |
| 22·5 | 36·544 | 31·976 | 27·682 | 25·353 | 23·601 | 20·936 |
| 23· | 36·948 | 32·329 | 27·998 | 25·633 | 23·868 | 21·228 |
| 23·5 | 37·347 | 32·679 | 28·291 | 25·910 | 24·120 | 21·396 |
| 24· | 37·743 | 33·025 | 28·590 | 26·184 | 24·375 | 21·623 |
| 24·5 | 38·134 | 33·367 | 28·886 | 26·455 | 24·628 | 21·847 |
| 25· | 38·521 | 33·706 | 29·180 | 26·724 | 24·878 | 22·069 |
| 25·5 | 38·904 | 34·041 | 29·470 | 26·990 | 25·125 | 22·288 |
| 26· | 39·284 | 34·373 | 29·757 | 27·253 | 25·371 | 22·506 |
| 26·5 | 39·660 | 34·702 | 30·042 | 27·514 | 25·613 | 22·722 |
| 27· | 40·032 | 35·028 | 30·324 | 27·761 | 25·854 | 22·935 |
| 27·5 | 40·401 | 35·351 | 30·604 | 28·028 | 26·092 | 23·146 |
| 28· | 40·767 | 35·671 | 30·881 | 28·282 | 26·328 | 23·355 |
| 28·5 | 41·129 | 35·988 | 31·155 | 28·533 | 26·563 | 23·563 |
| 29· | 41·488 | 36·302 | 31·427 | 28·782 | 26·891 | 23·766 |
| 29·5 | 41·844 | 36·614 | 31·697 | 29·029 | 27·024 | 23·973 |
| 30· | 42·197 | 36·923 | 31·956 | 29·274 | 27·253 | 24·176 |
| 30·5 | 42·548 | 37·229 | 32·230 | 29·517 | 27·478 | 24·376 |
| 31· | 42·895 | 37·533 | 32·493 | 29·758 | 27·703 | 24·574 |
| 31·5 | 43·240 | 37·835 | 32·754 | 29·997 | 27·925 | 24·772 |
| 32· | 43·581 | 38·134 | 33·013 | 30·234 | 28·146 | 24·968 |
| 32·5 | 43·920 | 38·430 | 33·270 | 30·470 | 28·365 | 25·162 |
| 33· | 44·257 | 38·725 | 33·525 | 30·703 | 28·582 | 25·355 |
| 33·5 | 44·591 | 39·017 | 33·778 | 30·935 | 28·798 | 25·546 |
| 34· | 44·923 | 39·307 | 34·029 | 31·165 | 29·012 | 25·737 |
| 34·5 | 45·252 | 39·595 | 34·278 | 31·393 | 29·225 | 25·925 |
| 35· | 45·578 | 39·881 | 34·526 | 31·620 | 29·436 | 26·113 |

N.B.—For overfalls, reduce the tabular velocity by one-third.

EXPLANATORY EXAMPLES TO TABLE X.

EXAMPLE 1.

An orifice 6 inches in diameter, has its centre under a head of 5 feet of water; required its discharge.

For a circular orifice using $\cdot62$ for a co-efficient, the velocity of discharge is 11.121 feet per second, and the sectional area, according to Part 7, Table XII., being $\cdot1963$, the discharge = $\cdot1963 \times 11.121 = 2.1836$ cubic feet per second.

EXAMPLE 2.

A rectangular orifice is 8 inches broad and 4 inches deep, and is under an effective head of 4 feet 3 inches; required its discharge.

Since the breadth is greater than the depth, a special co-efficient is required. (See Co-efficients in Table XII.)

$$\text{Here } \frac{H}{L} = \frac{4.25}{.66} = 7 \text{ approximately, and } \frac{D}{L} = \frac{.33}{.66} = 0.5.$$

These require a co-efficient $\cdot612$, which must hence be applied to the tabular discharge for natural velocity due to the co-efficient 1.00; the discharge = $16.544 \times .22 \times \cdot612 = 2.227$ cubic feet per second.

EXAMPLE 3.

The fall of water through a bridge, having a sectional area of 500 square feet, is 0.05 feet; required the discharge.

Take $\cdot96$ as a co-efficient for a wide opening, and we get the discharge = $1.758 \times 500 = 879$ cubic feet per second.

EXAMPLE 4.

The difference of level between the upper and lower ponds of a canal is 6 feet, and the communicating sluice is 2 feet square; required its discharge.

Using the co-efficient $\cdot84$ and height 6, for a constant head of 6 feet, the discharge is $16.512 \times 4 = 66.048$ cubic feet per second.

The effective head gradually decreasing, the mean discharge due to the height is 33.024 cubic feet per second.

If the lock is 60 long and 20 broad, it will hold 7 200 cubic feet of

water, and at the above rate will be filled in 218 seconds, or about three minutes and a half.

EXAMPLE 5.

Required the diameter of a vertical pipe to discharge 2 cubic feet per second from a reservoir under a head of 30 feet.

Using the co-efficient $\cdot 84$, we obtain from the Table 36.923 as velocity of discharge.

$$\text{The section will then} = \frac{2}{36.923} = 0.05417 \text{ square feet} = 5.42 \text{ square}$$

inches; which will require a diameter of 3 inches, or 4 inches, for the pipe.

EXAMPLE 6.

Required the length of a weir to discharge 5696 cubic feet per second, at a depth or head from still water to sill of 4 feet.

With a co-efficient $\cdot 666$, the tabular velocity of discharge is 10.689, from which one-third has to be deducted to obtain the mean velocity of discharge over a weir.

$$\text{Hence } V = 10.689 - 3.563 = 7.126 \text{ feet per second,}$$

$$\text{and the section} = \frac{5696}{7.126} = \text{nearly 800 feet;}$$

$$\text{hence the length} = \frac{\text{section}}{\text{depth}} = \text{nearly 200 feet.}$$

EXAMPLE 7.

A river passes over a drowned weir: the upper level of water is 3 feet above the lower level, and is 4 feet above the sill of the weir, which is 100 feet long; required the discharge.

The upper portion may be considered as a simple overfall with a head $H=3$, and with a co-efficient $\cdot 666$; the lower portion as an orifice, with the same head, but a co-efficient $\cdot 62$.

According to the Table the velocity of discharge for the one is $9.257 - 3.086 = 6.171$ feet per second; and that for the other is 8.618 feet per second. Hence the discharge:

$$\begin{aligned} &= 50 (6.171 \times 3 + 8.618 \times 1) = 50 \times 27.131 \\ &= 1356 \text{ cubic feet per second.} \end{aligned}$$

EXAMPLE 8.

It is required to raise the upper portion of a river 1.5 feet by means of a drowned weir across it. The river has a discharge of 812 cubic feet per

second, and a width of 70 feet; what must be the height of the dam—1st, neglecting velocity of approach; 2nd, taking it at 2.5 feet per second?

1st. Let d = depth of sill of dam below the lower water.

Then V = velocity of upper portion, or true overfall;
 = $\frac{2}{3}$ velocity for head 1.5 to a co-efficient .666;
 = 4.364 feet per second (from Table);

and V^1 = velocity of lower portion of orifice;
 = velocity for a head 1.5 to a co-efficient .62;
 = 6.109 feet per second (from Table).

Then the total discharge 812, is as in the last Example

$$= 70 \left\{ V \times 1.5 + V^1 \times d \right\} = 70 (6.546 + d \times 6.109)$$

$$\text{hence } d = \frac{5.054}{6.109} = 0.827 \text{ feet.}$$

2nd. Taking into consideration the velocity of approach and modifying the co-efficients (*vide* Table XII.) accordingly.

The head due to velocity of approach 2.5 feet per second, for a co-efficient .8, is from Table IX. about .15 feet.

Hence the modified co-efficient for overfall will be

$$o \left\{ \left\{ 1 + \frac{h^{\frac{3}{2}}}{H} \right\} - \left\{ \frac{h^{\frac{3}{2}}}{H} \right\} \right\} = .666 \left\{ \left\{ 1 + \frac{.15^{\frac{3}{2}}}{1.5} \right\} - \left\{ \frac{.15^{\frac{3}{2}}}{1.5} \right\} \right\}$$

$$= .666 \left\{ (1.1)^{\frac{3}{2}} - (.1)^{\frac{3}{2}} \right\} = .745$$

and the modified co-efficient for orifice will be

$$o \sqrt{1 + \frac{.15}{1.5}} = o \sqrt{1.1} = .62 \times 1.049 = .648.$$

Making use of these two co-efficients instead of .666 and .62 as in the first portion of the Example, we obtain other values.

$$V = 4.894; \text{ and } V^1 = 6.385;$$

$$\text{hence } \frac{812}{70} = 11.6 = 1.5 V + d V^1 = 7.341 + d \times 6.385$$

$$\text{and } d = \frac{4.259}{6.385} = 0.667 \text{ feet.}$$

TABLE XI.

Mean velocities of discharge in feet per second,
in small channels of rectangular section
corresponding to observed maximum velocities (V_x) and to co-efficients
(c), of mean velocity ;
calculated according to the Bazin formula—

$$V_m = \frac{c \cdot V_x}{c + 0.2535}$$

Also a table of Limiting Velocities for Culverts and Canals.

Mean Velocities of Discharge corresponding to

| c | Maximum Velocities | | | | | | | |
|------|--------------------|-------|-------|-------|-------|-------|-------|-------|
| | 0·5 | 1· | 1·5 | 2· | 2·5 | 3· | 3·5 | 4· |
| 0·25 | 0·248 | 0·497 | 0·745 | 0·994 | 1·241 | 1·490 | 1·738 | 1·986 |
| 0·30 | 0·271 | 0·542 | 0·813 | 1·084 | 1·355 | 1·626 | 1·897 | 2·168 |
| 0·35 | 0·290 | 0·580 | 0·870 | 1·160 | 1·450 | 1·740 | 2·030 | 2·320 |
| 0·40 | 0·306 | 0·612 | 0·918 | 1·224 | 1·530 | 1·836 | 2·142 | 2·448 |
| 0·45 | 0·320 | 0·640 | 0·959 | 1·279 | 1·599 | 1·919 | 2·239 | 2·568 |
| 0·50 | 0·332 | 0·664 | 0·995 | 1·327 | 1·659 | 1·991 | 2·323 | 2·654 |
| 0·55 | 0·342 | 0·685 | 1·027 | 1·370 | 1·711 | 2·054 | 2·396 | 2·738 |
| 0·60 | 0·352 | 0·703 | 1·055 | 1·406 | 1·758 | 2·109 | 2·461 | 2·812 |
| 0·65 | 0·360 | 0·719 | 1·079 | 1·439 | 1·799 | 2·158 | 2·518 | 2·878 |
| 0·70 | 0·367 | 0·734 | 1·102 | 1·469 | 1·836 | 2·203 | 2·570 | 2·937 |
| 0·75 | 0·374 | 0·747 | 1·121 | 1·495 | 1·869 | 2·242 | 2·616 | 2·990 |
| 0·80 | 0·380 | 0·759 | 1·139 | 1·519 | 1·899 | 2·278 | 2·658 | 3·038 |
| 0·85 | 0·385 | 0·770 | 1·156 | 1·541 | 1·926 | 2·311 | 2·696 | 3·081 |
| 0·90 | 0·390 | 0·780 | 1·171 | 1·561 | 1·951 | 2·341 | 2·731 | 3·121 |
| 0·95 | 0·395 | 0·789 | 1·184 | 1·579 | 1·974 | 2·368 | 2·763 | 3·158 |
| 1·00 | 0·399 | 0·798 | 1·197 | 1·596 | 1·995 | 2·393 | 2·792 | 3·192 |
| 1·05 | 0·403 | 0·806 | 1·208 | 1·611 | 2·014 | 2·416 | 2·819 | 3·222 |
| 1·10 | 0·406 | 0·813 | 1·219 | 1·626 | 2·032 | 2·438 | 2·845 | 3·251 |
| 1·15 | 0·410 | 0·819 | 1·229 | 1·639 | 2·049 | 2·459 | 2·869 | 3·278 |
| 1·20 | 0·413 | 0·826 | 1·238 | 1·651 | 2·064 | 2·477 | 2·890 | 3·302 |
| 1·25 | 0·416 | 0·831 | 1·247 | 1·663 | 2·079 | 2·495 | 2·911 | 3·326 |
| 1·30 | 0·418 | 0·837 | 1·255 | 1·674 | 2·092 | 2·510 | 2·929 | 3·347 |
| 1·35 | 0·421 | 0·842 | 1·263 | 1·684 | 2·105 | 2·526 | 2·947 | 3·368 |
| 1·40 | 0·423 | 0·847 | 1·270 | 1·694 | 2·117 | 2·540 | 2·964 | 3·387 |
| 1·45 | 0·426 | 0·851 | 1·277 | 1·702 | 2·128 | 2·554 | 2·979 | 3·405 |
| 1·50 | 0·428 | 0·855 | 1·283 | 1·711 | 2·139 | 2·567 | 2·995 | 3·422 |
| 1·55 | 0·430 | 0·860 | 1·289 | 1·719 | 2·149 | 2·579 | 3·009 | 3·438 |
| 1·60 | 0·432 | 0·863 | 1·295 | 1·726 | 2·158 | 2·590 | 3·021 | 3·453 |
| 1·65 | 0·433 | 0·867 | 1·300 | 1·734 | 2·167 | 2·600 | 3·034 | 3·467 |
| 1·70 | 0·435 | 0·870 | 1·306 | 1·741 | 2·176 | 2·611 | 3·046 | 3·482 |
| 1·75 | 0·437 | 0·873 | 1·310 | 1·747 | 2·184 | 2·621 | 3·058 | 3·494 |
| 1·80 | 0·438 | 0·877 | 1·315 | 1·754 | 2·191 | 2·630 | 3·069 | 3·507 |
| 1·85 | 0·440 | 0·880 | 1·319 | 1·759 | 2·199 | 2·639 | 3·079 | 3·518 |
| 1·90 | 0·441 | 0·882 | 1·324 | 1·765 | 2·206 | 2·647 | 3·088 | 3·530 |
| 1·95 | 0·442 | 0·885 | 1·327 | 1·770 | 2·212 | 2·654 | 3·097 | 3·539 |
| 2·00 | 0·444 | 0·888 | 1·331 | 1·775 | 2·219 | 2·663 | 3·107 | 3·550 |
| 2·10 | 0·446 | 0·892 | 1·339 | 1·785 | 2·231 | 2·677 | 3·123 | 3·570 |
| 2·20 | 0·448 | 0·897 | 1·345 | 1·794 | 2·242 | 2·690 | 3·139 | 3·587 |

observed Maximum Velocities and Co-efficients (c).

| c | Maximum Velocities | | | | | | | |
|------|--------------------|-------|-------|-------|-------|-------|-------|-------|
| | 4.5 | 5. | 5.5 | 6. | 0.5 | 7. | 7.5 | 8. |
| 0.25 | 2.235 | 2.483 | 2.732 | 2.980 | 3.228 | 3.476 | 3.724 | 3.972 |
| 0.30 | 2.439 | 2.710 | 2.982 | 3.252 | 3.524 | 3.794 | 4.066 | 4.336 |
| 0.35 | 2.610 | 2.900 | 3.190 | 3.480 | 3.770 | 4.060 | 4.350 | 4.640 |
| 0.40 | 2.754 | 3.060 | 3.366 | 3.672 | 3.978 | 4.284 | 4.590 | 4.896 |
| 0.45 | 2.878 | 3.198 | 3.518 | 3.838 | 4.158 | 4.478 | 4.798 | 5.118 |
| 0.50 | 2.986 | 3.318 | 3.650 | 3.982 | 4.314 | 4.646 | 4.978 | 5.308 |
| 0.55 | 3.081 | 3.423 | 3.766 | 4.108 | 4.450 | 4.792 | 5.134 | 5.476 |
| 0.60 | 3.164 | 3.515 | 3.866 | 4.218 | 4.570 | 4.922 | 5.272 | 5.624 |
| 0.65 | 3.217 | 3.597 | 3.957 | 4.316 | 4.676 | 5.036 | 5.396 | 5.756 |
| 0.70 | 3.304 | 3.671 | 4.038 | 4.406 | 4.772 | 5.140 | 5.506 | 5.874 |
| 0.75 | 3.363 | 3.737 | 4.110 | 4.484 | 4.858 | 5.232 | 5.606 | 5.980 |
| 0.80 | 3.417 | 3.797 | 4.176 | 4.556 | 4.936 | 5.316 | 5.696 | 6.076 |
| 0.85 | 3.466 | 3.851 | 4.236 | 4.622 | 5.006 | 5.392 | 5.776 | 6.162 |
| 0.90 | 3.511 | 3.901 | 4.292 | 4.682 | 5.072 | 5.462 | 5.852 | 6.242 |
| 0.95 | 3.552 | 3.947 | 4.342 | 4.736 | 5.132 | 5.526 | 5.920 | 6.316 |
| 1.00 | 3.590 | 3.989 | 4.388 | 4.786 | 5.186 | 5.584 | 5.984 | 6.384 |
| 1.05 | 3.624 | 4.027 | 4.430 | 4.832 | 5.236 | 5.638 | 6.040 | 6.444 |
| 1.10 | 3.658 | 4.064 | 4.470 | 4.876 | 5.283 | 5.690 | 6.096 | 6.502 |
| 1.15 | 3.688 | 4.097 | 4.508 | 4.918 | 5.327 | 5.737 | 6.147 | 6.557 |
| 1.20 | 3.715 | 4.128 | 4.541 | 4.954 | 5.366 | 5.779 | 6.192 | 6.605 |
| 1.25 | 3.742 | 4.157 | 4.574 | 4.990 | 5.405 | 5.821 | 6.237 | 6.653 |
| 1.30 | 3.766 | 4.184 | 4.602 | 5.021 | 5.439 | 5.858 | 6.276 | 6.694 |
| 1.35 | 3.789 | 4.210 | 4.631 | 5.052 | 5.473 | 5.894 | 6.315 | 6.736 |
| 1.40 | 3.811 | 4.234 | 4.657 | 5.081 | 5.504 | 5.928 | 6.351 | 6.774 |
| 1.45 | 3.830 | 4.256 | 4.682 | 5.107 | 5.533 | 5.958 | 6.384 | 6.810 |
| 1.50 | 3.850 | 4.277 | 4.706 | 5.134 | 5.561 | 5.989 | 6.407 | 6.845 |
| 1.55 | 3.868 | 4.297 | 4.728 | 5.158 | 5.587 | 6.017 | 6.447 | 6.877 |
| 1.60 | 3.884 | 4.316 | 4.748 | 5.179 | 5.611 | 6.042 | 6.474 | 6.906 |
| 1.65 | 3.904 | 4.334 | 4.767 | 5.201 | 5.634 | 6.068 | 6.501 | 6.934 |
| 1.70 | 3.917 | 4.351 | 4.787 | 5.222 | 5.658 | 6.093 | 6.528 | 6.963 |
| 1.75 | 3.931 | 4.367 | 4.805 | 5.242 | 5.678 | 6.115 | 6.552 | 6.989 |
| 1.80 | 3.944 | 4.383 | 4.820 | 5.258 | 5.697 | 6.135 | 6.573 | 7.011 |
| 1.85 | 3.958 | 4.397 | 4.838 | 5.278 | 5.717 | 6.157 | 6.597 | 7.037 |
| 1.90 | 3.971 | 4.412 | 4.853 | 5.294 | 5.736 | 6.177 | 6.618 | 7.059 |
| 1.95 | 3.982 | 4.425 | 4.866 | 5.309 | 5.751 | 6.194 | 6.636 | 7.078 |
| 2.00 | 3.994 | 4.438 | 4.882 | 5.326 | 5.769 | 6.213 | 6.657 | 7.101 |
| 2.10 | 4.016 | 4.462 | 4.903 | 5.354 | 5.801 | 6.247 | 6.693 | 7.139 |
| 2.20 | 4.036 | 4.484 | 4.932 | 5.381 | 5.829 | 6.278 | 6.726 | 7.174 |

Mean Velocities of Discharge corresponding to

| c | Maximum Velocities | | | | | | | |
|------|--------------------|-------|-------|-------|------|------|-------|-------|
| | 8·5 | 9 | 9·5 | 10 | 10·5 | 11 | 11·5 | 12 |
| 0·25 | 4·222 | 4·470 | 4·718 | 4·965 | 5·22 | 5·46 | 5·71 | 5·96 |
| 0·30 | 4·608 | 4·878 | 5·150 | 5·420 | 5·70 | 5·97 | 6·24 | 6·50 |
| 0·35 | 4·930 | 5·220 | 5·510 | 5·800 | 6·09 | 6·38 | 6·67 | 6·96 |
| 0·40 | 5·202 | 5·508 | 5·814 | 6·120 | 6·43 | 6·73 | 7·04 | 7·34 |
| 0·45 | 5·436 | 5·756 | 6·076 | 6·395 | 6·72 | 7·04 | 7·36 | 7·68 |
| 0·50 | 5·640 | 5·972 | 6·304 | 6·636 | 6·97 | 7·30 | 7·64 | 7·97 |
| 0·55 | 5·820 | 6·162 | 6·504 | 6·845 | 7·19 | 7·53 | 7·88 | 8·22 |
| 0·60 | 5·976 | 6·328 | 6·678 | 7·030 | 7·38 | 7·73 | 8·09 | 8·44 |
| 0·65 | 6·114 | 6·474 | 6·834 | 7·194 | 7·56 | 7·97 | 8·28 | 8·64 |
| 0·70 | 6·240 | 6·608 | 6·976 | 7·342 | 7·71 | 8·08 | 8·45 | 8·81 |
| 0·75 | 6·352 | 6·726 | 7·100 | 7·474 | 7·85 | 8·23 | 8·60 | 8·98 |
| 0·80 | 6·454 | 6·834 | 7·214 | 7·594 | 7·98 | 8·36 | 8·74 | 9·12 |
| 0·85 | 6·546 | 6·932 | 7·318 | 7·703 | 8·09 | 8·47 | 8·86 | 9·25 |
| 0·90 | 6·632 | 7·022 | 7·412 | 7·802 | 8·19 | 8·58 | 8·97 | 9·36 |
| 0·95 | 6·710 | 7·104 | 7·498 | 7·894 | 8·29 | 8·69 | 9·08 | 9·48 |
| 1·00 | 6·782 | 7·180 | 7·580 | 7·978 | 8·38 | 8·78 | 9·18 | 9·58 |
| 1·05 | 6·846 | 7·248 | 7·652 | 8·055 | 8·46 | 8·86 | 9·26 | 9·67 |
| 1·10 | 6·908 | 7·316 | 7·722 | 8·128 | 8·53 | 8·94 | 9·35 | 9·75 |
| 1·15 | 6·967 | 7·376 | 7·786 | 8·194 | 8·61 | 9·02 | 9·43 | 9·84 |
| 1·20 | 7·018 | 7·430 | 7·843 | 8·256 | 8·67 | 9·08 | 9·49 | 9·90 |
| 1·25 | 7·069 | 7·484 | 7·900 | 8·314 | 8·74 | 9·15 | 9·57 | 9·98 |
| 1·30 | 7·113 | 7·531 | 7·950 | 8·368 | 8·79 | 9·24 | 9·62 | 10·06 |
| 1·35 | 7·157 | 7·578 | 7·999 | 8·419 | 8·85 | 9·27 | 9·69 | 10·11 |
| 1·40 | 7·198 | 7·621 | 8·045 | 8·467 | 8·89 | 9·32 | 9·74 | 10·17 |
| 1·45 | 7·235 | 7·661 | 8·086 | 8·512 | 8·94 | 9·36 | 9·80 | 10·21 |
| 1·50 | 7·273 | 7·700 | 8·128 | 8·554 | 8·99 | 9·42 | 9·84 | 10·27 |
| 1·55 | 7·307 | 7·736 | 8·166 | 8·595 | 9·03 | 9·46 | 9·89 | 10·32 |
| 1·60 | 7·337 | 7·769 | 8·200 | 8·633 | 9·06 | 9·50 | 9·93 | 10·36 |
| 1·65 | 7·368 | 7·801 | 8·235 | 8·668 | 9·11 | 9·54 | 9·97 | 10·41 |
| 1·70 | 7·398 | 7·834 | 8·269 | 8·702 | 9·14 | 9·57 | 10·02 | 10·45 |
| 1·75 | 7·426 | 7·862 | 8·299 | 8·734 | 9·17 | 9·61 | 10·05 | 10·48 |
| 1·80 | 7·449 | 7·888 | 8·326 | 8·765 | 9·21 | 9·65 | 10·08 | 10·52 |
| 1·85 | 7·477 | 7·916 | 8·356 | 8·795 | 9·24 | 9·68 | 10·12 | 10·56 |
| 1·90 | 7·500 | 7·942 | 8·383 | 8·823 | 9·27 | 9·71 | 10·15 | 10·59 |
| 1·95 | 7·521 | 7·963 | 8·406 | 8·849 | 9·29 | 9·73 | 10·18 | 10·62 |
| 2·00 | 7·545 | 7·988 | 8·432 | 8·875 | 9·32 | 9·77 | 10·21 | 10·66 |
| 2·10 | 7·585 | 8·032 | 8·478 | 8·923 | 9·37 | 9·82 | 10·27 | 10·71 |
| 2·20 | 7·623 | 8·071 | 8·520 | 8·967 | 9·42 | 9·87 | 10·31 | 10·76 |

observed Maximum Velocities and Co-efficients (c).

| c | Maximum Velocities | | | | | | | |
|------|--------------------|-------|-------|-------|-------|-------|-------|-------|
| | 13. | 14. | 15. | 16. | 17. | 18. | 19. | 20. |
| 0.25 | 6.46 | 6.95 | 7.45 | 7.94 | 8.43 | 8.94 | 9.43 | 9.93 |
| 0.30 | 7.05 | 7.59 | 8.13 | 8.67 | 9.21 | 9.76 | 10.30 | 10.84 |
| 0.35 | 7.54 | 8.12 | 8.70 | 9.28 | 9.86 | 10.44 | 11.02 | 11.60 |
| 0.40 | 7.96 | 8.57 | 9.18 | 9.79 | 10.40 | 11.02 | 11.63 | 12.24 |
| 0.45 | 8.31 | 8.95 | 9.59 | 10.23 | 10.87 | 11.51 | 12.15 | 12.79 |
| 0.50 | 8.63 | 9.29 | 9.95 | 10.62 | 11.28 | 11.94 | 12.61 | 13.27 |
| 0.55 | 8.90 | 9.58 | 10.27 | 10.95 | 11.63 | 12.32 | 13.01 | 13.69 |
| 0.60 | 9.14 | 9.84 | 10.55 | 11.25 | 11.95 | 12.65 | 13.36 | 14.06 |
| 0.65 | 9.35 | 10.07 | 10.79 | 11.51 | 12.23 | 12.95 | 13.67 | 14.39 |
| 0.70 | 9.54 | 10.28 | 11.02 | 11.73 | 12.48 | 13.21 | 13.95 | 14.68 |
| 0.75 | 9.65 | 10.40 | 11.21 | 11.88 | 12.62 | 13.37 | 14.11 | 14.95 |
| 0.80 | 9.87 | 10.63 | 11.39 | 12.15 | 12.91 | 13.67 | 14.43 | |
| 0.85 | 10.02 | 10.79 | 11.56 | 12.33 | 13.10 | 13.87 | 14.64 | |
| 0.90 | 10.14 | 10.92 | 11.71 | 12.48 | 13.26 | 14.04 | 14.82 | |
| 0.95 | 10.26 | 11.05 | 11.84 | 12.63 | 13.42 | 14.21 | 15.00 | |
| 1.00 | 10.44 | 11.24 | 11.97 | 12.85 | 13.65 | 14.45 | | |
| 1.05 | 10.47 | 11.27 | 12.08 | 12.89 | 13.69 | 14.50 | | |
| 1.10 | 10.57 | 11.38 | 12.19 | 13.01 | 13.82 | 14.63 | | |
| 1.15 | 10.65 | 11.47 | 12.29 | 13.11 | 13.93 | 14.75 | | |
| 1.20 | 10.73 | 11.56 | 12.38 | 13.21 | 14.03 | 14.86 | | |
| 1.25 | 10.81 | 11.64 | 12.47 | 13.30 | 14.13 | 14.97 | | |
| 1.30 | 10.88 | 11.72 | 12.55 | 13.39 | 14.23 | | | |
| 1.35 | 10.95 | 11.79 | 12.63 | 13.47 | 14.31 | | | |
| 1.40 | 11.01 | 11.85 | 12.70 | 13.54 | 14.39 | | | |
| 1.45 | 11.06 | 11.91 | 12.77 | 13.62 | 14.47 | | | |
| 1.50 | 11.12 | 11.98 | 12.83 | 13.69 | 14.54 | | | |
| 1.55 | 11.17 | 12.03 | 12.89 | 13.75 | 14.61 | | | |
| 1.60 | 11.23 | 12.09 | 12.95 | 13.82 | 14.68 | | | |
| 1.65 | 11.27 | 12.14 | 13.00 | 13.87 | 14.74 | | | |
| 1.70 | 11.31 | 12.18 | 13.06 | 13.92 | 14.79 | | | |
| 1.75 | 11.36 | 12.23 | 13.10 | 13.98 | 14.85 | | | |
| 1.80 | 11.40 | 12.27 | 13.15 | 14.02 | 14.90 | | | |
| 1.85 | 11.43 | 12.31 | 13.19 | 14.07 | 14.95 | | | |
| 1.90 | 11.47 | 12.36 | 13.24 | 14.12 | 15.00 | | | |
| 1.95 | 11.51 | 12.39 | 13.28 | 14.16 | | | | |
| 2.00 | 11.54 | 12.43 | 13.31 | 14.20 | | | | |
| 2.10 | 11.62 | 12.51 | 13.40 | 14.30 | | | | |
| 2.20 | 11.66 | 12.55 | 13.48 | 14.34 | | | | |

Various limiting velocities.

MAXIMA IN OPEN CANALS.

| | Feet per second. |
|---|------------------|
| For the worst or most sandy soil | 2·5 |
| For sandy soil generally | 2·75 |
| For ordinary loam | 3· |
| For firm gravel and hard soil | 4· |
| For brickwork, ashlar or rubble in cement | 5·5 to 7·5 |
| For hard sound stratified rock | 10· |
| For very hard homogeneous rock | 14 or 15 |

MINIMA FOR DRAINAGE IN CYLINDRICAL PIPES AND CULVERTS.¹

| | |
|---|-----|
| Small drain-pipes under 6" in diameter | 3·5 |
| Drain-pipes, 6" to 18" in diameter | 3· |
| Culverts from 1·5 to 4 feet in diameter | 2·5 |
| Larger cylindrical culverts | 2· |
| For ovoidal culverts, &c., compare with cylindrical culverts of equal hydraulic radius. | |

LIMITING VELOCITIES.

| | |
|--|-----------|
| Limits usual for canals | 1 to 4 |
| Limits for rivers and canals just navigable | 3 to 4½ |
| Limits for irrigating channels | 1 to 3 |
| Limits for sewers and brick conduits | 1 to 4½ |
| Limits for self-cleansing sewers and drainage pipes | 2·5 to 4½ |
| Limiting velocities for water-pipes, so as to get a maximum discharge under pressure | 25 to 35 |

NATURAL CHANNELS.

| | |
|--|-------------|
| Slow rivers, from | 0·25 to 1·5 |
| Ordinary rivers, from | 1·5 to 3·0 |
| Rapid rivers and torrents from | 3·0 to 12·0 |
| Maximum tidal current measured | 15 |

¹ Working minima are 0·5 higher than these, which are extreme minima.

TABLE XII.—HYDRAULIC CO-EFFICIENTS.

- Part 1. Co-efficients of flood-discharge (k) from catchment areas.
- Part 2. Formulæ connecting the co-efficients of velocity (v) with those of rugosity (n).
- Part 3. General values of co-efficients (n) of roughness in channels and culverts.
Local values of n for various canals and rivers.
- Part 4. Velocity co-efficients (v) for channels, culverts, and pipes.
Under grouped values of (n) for two fixed extreme values of S .
Under separate values of n , in separate tables.
- Part 5. Co-efficients of discharge (c) for orifices and outlets.
- Part 6. Co-efficients of discharge (c) for overfalls.

PART I.—*General and Local Co-efficients of flood-discharge from catchment areas.*

For the formula in Table IV., Part I, also given in the text.

$$Q = k \times 100 (K)^{\frac{2}{3}}$$

The value of this co-efficient (k) can be determined and made use of within local limits only, as it depends on the average maximum local downpour, the evaporation, the quality, inclination, and disposition of the surface of the area under consideration; it has hitherto been determined for very few districts, and not sufficiently satisfactory for some of those. In some cases, unfortunately, doubtful flood-marks have been used to obtain the flood gradient, and the velocities calculated according to very varied formulæ; in others, the obstructions caused by bridges and embankments have vitiated all the bases of calculation of discharge.

| | Values of k |
|--|-----------------|
| For very large Indian rivers near their mouths . . . | 0·3 to 2 |
| For catchment areas in Oudh generally . . . | 1 to 2 |
| The Madras Presidency, the whole Kaveri | } about 2· |
| The Godavery, Kistna, Tumbaddra, Pennair, Vigay | |
| The Chittaur, Palaur, Manjilanthi, Varhazanthi below | 5· |
| For the Kanhan River, Central Provinces, according to the highest flood yet known, less than . . . | 5· |
| For Bengal and Bahar, rainfall 2 to 4 feet—Col. Dickens gives a co-efficient of . . . | 8·25 |
| The Upper Kaveri, Tambrapurni, Gadanamatti. } | 12, 16, and 22· |
| For some rivers in Berar and the Central Provinces, according to calculated velocities only . . . | 16· to 24· |

Some further data for Indian rivers will be found in the 'Hydraulic Statistics' of the Author.

PART 2.—*Formulae connecting the Co-efficients of Velocity (c) with those of Rugosity (n).*

$$Q = \left\{ \frac{1.811}{n} + 41.6 + \frac{0.00281}{S} \right\} A \sqrt{RS}$$

$$Q = \left\{ \frac{1.811}{1 + \left(41.6 + \frac{0.00281}{S} \right) \frac{n}{\sqrt{R}}} \right\} A \sqrt{RS}$$

where Q is the mean discharge in cubic feet per second,
 A is the sectional area of water-way in square feet,
 R is the hydraulic radius of the section in feet,
 S is the sine of the hydraulic slope of the water surface,
 n is the co-efficient of roughness.

This may be reduced into the more convenient form,

$$Q = \frac{\sqrt{R} \left(\frac{m + 1.811}{m + \sqrt{R}} \right)}{n} \cdot A \sqrt{RS},$$

where m is a variable dependent on S and n alone,

$$\text{and } m = n \left(41.6 + \frac{0.00281}{S} \right);$$

or may be further modified into the form,

$$Q = cA \cdot 100 \sqrt{RS},$$

where c is the co-efficient of mean velocity,

$$\text{and } c = \frac{\sqrt{R} \left(\frac{m + 1.811}{m + \sqrt{R}} \right)}{100n};$$

or, into its most simple form, $Q = AV$,

where V is the mean velocity of discharge in feet per second,

and $V = c \cdot 100 \sqrt{RS}$; c being a variable quantity.

NOTE.

The values of S , the sine of the hydraulic slope, are more generally expressed for conciseness in the form of S per thousand in the Tables, thus, S per thousand = 0.4 instead of $S = 0.0004$; and S per thousand = 20, instead of $S = 0.02$.

PART 3.—*General or Average Values of Co-efficients (n)
as applied by the*

AQUEDUCTS, CANALS,

- ⁿ
 0·010 Pure cement in England and Europe generally; also
 Glazed materials of every sort; glazed, coated, or
 0·013 Brickwork and ashlar, in aqueducts, canals, and culverts }
 Ordinary cast and wrought iron. Unglazed stoneware }
 Materials mentioned under 0·010 when in bad order and
 0·017 Rubble in cement, in good order. Also, earth in highly
 Materials mentioned under 0·013 when in bad order and
 0·020 Coarse rubble, set dry. Rubble in cement in bad condi-
 0·0225 Dry coarse rubble in bad order. Rubble in cement,

CANALS IN NATURAL

- 0·020 Class I.—Very firm, regular gravel, carefully trimmed and
 0·0225 Class II.—Earth. Canals and channels. (Based on
 0·0250 Class III.—Earth. Canals and channels. (Based on
 0·0275 Class IV.—Earth. Canals and channels. (Based on
 0·030 Class V.—Earth. Canals in bad order, rather damaged,

General values of n for Temporary

- 0·009 Well-planed timber, in perfect order and alignment, and
 0·012 Unplaned timber, when perfectly continuous on the inside.
 0·015 { Wooden frames covered with canvas.
 { Rectangular wooden troughs, with battens on the inside,
 0·020 Rectangular wooden troughs, with battens on the inside,

NOTE.

The local values of *n*, suitable to rivers and natural channels
 perimentally determined for other rivers, or may be deduced
 nexion with other data and conditions. They vary between

*of Roughness, for various Materials, and Conditions of Surface ;
Author in the Tables.*

CULVERTS, AND PIPES.

Indian cement-plaster, with worked surface.
enamelled stoneware and iron.

in good order.

condition.

regular cases.

condition.

tion. Ruined brickwork and masonry.

in a ruinous condition.

UNWORKED MATERIAL.

punned in defective places. Trimmed earth in perfect order.

various data by the Author) ; above the average.

various data by the Author) ; in good average order.

various data by the Author) ; below the average.

slightly overgrown with weeds, or obstructed by detritus.

Constructions, determined by Kutter.

perfectly straight ; otherwise perhaps 0.010 would be suitable.

Flumes.

0.5 inch apart.

2 inches apart.

NOTE.

generally, may be obtained by comparison with those already ex-
from a consideration of the observed maximum velocities in con-
the limits of 0.020 and 0.035. See Kutter's local values, p. 136.

PART 3 (cont.).—*Local Values of the Co-efficient n of Roughness and Irregularity, according to Kutter.*

NATURAL CHANNELS.

| | | |
|--------------|--|------------------------------------|
| ⁿ | | |
| 0·0200 | Bayou Lafourche. | } Generally free from obstruction. |
| 0·0210 | Ohio, Point Pleasant. | |
| 0·0220 | Lech. ¹ | |
| 0·0227 | Rhine at Gernersheim. ¹ | |
| 0·0228 | Tiber at Rome. | |
| 0·0232 | Weser. | |
| 0·0237 | Hübengraben. | |
| 0·0243 | Hockenbach. | |
| 0·0243 | Rhine in Holland. | |
| 0·0250 | Seine at Paris. | |
| 0·0252 | Newka. | |
| 0·0260 | Speyerbach. | |
| 0·0260 | Seine at Poissy. | |
| 0·0260 | Haine. | |
| 0·0260 | Rhine at Speyer. ¹ | |
| 0·0262 | Newa. | |
| 0·0270 | Mississippi. | |
| 0·0270 | Saalach. ¹ | |
| 0·0270 | Plessur. ¹ | |
| 0·0280 | Saône at Raconnay. | |
| 0·0280 | Salzach. ¹ | |
| 0·0285 | Elbe. | |
| 0·0294 | Bayou Plaquemine. | |
| 0·0300 | Rhine at Basle. ¹ | } Obstructed by detritus. |
| 0·0305 | Isaar. ¹ | |
| 0·0310 | Meuse at Misox. ¹ | |
| 0·0310 | Rhine at Rheinwald. ¹ | |
| 0·0345 | Simme at Lenk. ¹ | |
| 0·0350 | Rhine at Domleschgerthal. ¹ | |

¹ Obstructed by detritus.

PART 3 (*cont.*)—*Local Values of the Co-efficient n of Roughness and Irregularity, selected from Bazin and Kutter.*

ARTIFICIAL CHANNELS.

In Cement.

- ⁿ
 0·0100 Series No. 24 of D'Arcy and Bazin, semicircular.
 0·0104 Series No. 2 of D'Arcy and Bazin, rectangular.
 0·0111 Series No. 25, D. & B., with one-third sand, semicircular.

In Ashlar and Brickwork.

- 0·0129 Series No. 3, D'Arcy and Bazin, brickwork, rectangular.
 0·0129 Series No. 39, D'Arcy and Bazin, ashlar, rectangular.
 0·0133 Series Nos. 1 & 2, D'Arcy and Bazin, ashlar, rectangular.

In Rubble.

- 0·0145 Gontenbachschale, new, dry, semicircular.
 0·0167 Series No. 32, D'Arcy and Bazin, rather damaged, rectangular.
 0·0170 Series No. 33, D'Arcy and Bazin, rather damaged, rectangular.
 0·0175 Grunnbachschale, damaged, dry, semicircular.
 0·0185 Gerbebachschale, damaged, dry, semicircular.
 0·0180 Series No. 1·4, D'Arcy and Bazin, rough.
 0·0182 Series No. 1·3, D'Arcy and Bazin, rough.
 0·0184 Series No. 1·6, D'Arcy and Bazin, rough.
 0·0192 Series No. 1·5, D'Arcy and Bazin, rough.
 0·0204 Series No. 44, D'Arcy and Bazin, with deposits, rectangular.
 0·0210 Series No. 46, D'Arcy and Bazin, with deposits, rectangular.
 0·0220 Series No. 35, D'Arcy and Bazin, damaged, trapezoidal.
 0·0230 Alpbachschale, much damaged, semicircular.

In Rammed Gravel.

- 0·0163 Series No. 27, D'Arcy and Bazin, $\frac{3}{4}$ -inch thick, semicircular.
 0·0170 Series No. 4, D'Arcy and Bazin, $\frac{3}{4}$ -inch thick, rectangular.
 0·0190 Series No. 5, D'Arcy and Bazin, $1\frac{1}{2}$ -inch thick, rectangular.

In Earth.

- 0·0184 A Canal in England.
 0·0222 Linth Canal, trapezoidal.
 0·0244 Marseilles Canal, rounded.
 0·0254 Pannerden Canal, Holland.
 0·0255 Jard Canal.
 0·0262 Lauter Canal, Neuberg.
 0·0300 Escher Canal (detritus).
 0·0301 Marmels Canal.
 0·0330 Chesapeake-Ohio Canal, rounded.

PART 4.—*Co-efficients of mean velocity suited to various materials, calculated for a fixed value of $S=0\cdot001$.*

| R in feet | Values of n | | | | | | | |
|----------------|---------------|-------|-------|-------|-------|--------|-------|-------|
| | ·010 | ·013 | ·017 | ·020 | ·0225 | ·0250 | ·0275 | ·0300 |
| | (1) | (2) | (3) | (I.) | (II.) | (III.) | (IV.) | (V.) |
| 0·5 | 1·385 | 1·011 | 0·730 | 0·598 | 0·518 | 0·455 | 0·404 | 0·363 |
| 1 | 1·562 | 1·615 | 0·860 | 0·715 | 0·625 | 0·554 | 0·496 | 0·449 |
| 1·25 | 1·614 | 1·212 | 0·901 | 0·752 | 0·660 | 0·586 | 0·527 | 0·478 |
| 1·5 | 1·655 | 1·249 | 0·933 | 0·782 | 0·688 | 0·613 | 0·552 | 0·502 |
| 1·75 | 1·688 | 1·279 | 0·961 | 0·808 | 0·712 | 0·635 | 0·573 | 0·522 |
| 2 | 1·716 | 1·305 | 0·984 | 0·829 | 0·732 | 0·655 | 0·592 | 0·450 |
| 2·25 | 1·740 | 1·327 | 1·004 | 0·848 | 0·750 | 0·672 | 0·608 | 0·555 |
| 2·5 | 1·761 | 1·346 | 1·021 | 0·864 | 0·765 | 0·687 | 0·622 | 0·569 |
| 2·75 | 1·779 | 1·363 | 1·037 | 0·879 | 0·779 | 0·700 | 0·635 | 0·581 |
| 3 | 1·795 | 1·378 | 1·051 | 0·892 | 0·792 | 0·712 | 0·647 | 0·592 |
| 3·25 | 1·809 | 1·392 | 1·063 | 0·904 | 0·804 | 0·723 | 0·657 | 0·603 |
| 3·5 | 1·823 | 1·404 | 1·075 | 0·915 | 0·814 | 0·733 | 0·667 | 0·612 |
| 4 | 1·845 | 1·426 | 1·095 | 0·935 | 0·833 | 0·751 | 0·685 | 0·629 |
| 4·5 | 1·865 | 1·444 | 1·113 | 0·951 | 0·849 | 0·767 | 0·700 | 0·644 |
| 5 | 1·881 | 1·460 | 1·128 | 0·966 | 0·863 | 0·781 | 0·713 | 0·657 |
| 5·5 | 1·896 | 1·474 | 1·141 | 0·979 | 0·876 | 0·793 | 0·725 | 0·668 |
| 6 | 1·909 | 1·487 | 1·153 | 0·991 | 0·887 | 0·804 | 0·736 | 0·679 |
| 6·5 | 1·921 | 1·498 | 1·164 | 1·001 | 0·897 | 0·814 | 0·746 | 0·688 |
| 7 | 1·931 | 1·508 | 1·174 | 1·010 | 0·907 | 0·823 | 0·754 | 0·697 |
| 7·5 | 1·940 | 1·517 | 1·183 | 1·019 | 0·915 | 0·831 | 0·763 | 0·705 |
| 8 | 1·949 | 1·526 | 1·191 | 1·027 | 0·923 | 0·839 | 0·770 | 0·712 |
| 8·5 | 1·957 | 1·534 | 1·198 | 1·034 | 0·930 | 0·846 | 0·777 | 0·719 |
| 9 | 1·964 | 1·541 | 1·205 | 1·041 | 0·937 | 0·853 | 0·784 | 0·726 |
| 10 | 1·977 | 1·554 | 1·218 | 1·054 | 0·949 | 0·865 | 0·795 | 0·737 |
| 15 | 2·023 | 1·599 | 1·263 | 1·068 | 0·993 | 0·908 | 0·838 | 0·780 |
| 20 | 2·051 | 1·627 | 1·291 | 1·126 | 1·021 | 0·936 | 0·866 | 0·807 |

PART 4 (*cont.*).—*Co-efficients of mean velocity suited to various materials, calculated for a fixed value of $S=0.0001$.*

| <i>R</i> in feet | Values of <i>n</i> | | | | | | | |
|---------------------|--------------------|-------|-------|--------|-------|--------|-------|-------|
| | ·010 | ·013 | ·017 | ·020 | ·0225 | ·0250 | ·0275 | ·0300 |
| | (1) | (2) | (3) | (I.) | (II.) | (III.) | (IV.) | (V.) |
| 0.5 | 1.263 | 0.916 | 0.658 | 0.539 | 0.467 | 0.410 | 0.365 | 0.329 |
| 1. | 1.478 | 1.097 | 0.806 | 0.669 | 0.585 | 0.518 | 0.465 | 0.421 |
| 1.25 | 1.545 | 1.155 | 0.855 | 0.713? | 0.625 | 0.556 | 0.499 | 0.453 |
| 1.5 | 1.598 | 1.201 | 0.895 | 0.750? | 0.659 | 0.587 | 0.529 | 0.480 |
| 1.75 | 1.643 | 1.240 | 0.929 | 0.780 | 0.687 | 0.613 | 0.554 | 0.504 |
| 2. | 1.680 | 1.274 | 0.959 | 0.807 | 0.712 | 0.637 | 0.576 | 0.525 |
| 2.25 | 1.712 | 1.303 | 0.984 | 0.831 | 0.734 | 0.658 | 0.595 | 0.543 |
| 2.5 | 1.741 | 1.329 | 1.007 | 0.852 | 0.754 | 0.676 | 0.613 | 0.560 |
| 2.75 | 1.766 | 1.352 | 1.028 | 0.871 | 0.772 | 0.693 | 0.629 | 0.575 |
| 3. | 1.788 | 1.372 | 1.046 | 0.888 | 0.788 | 0.709 | 0.643 | 0.589 |
| 3.25 | 1.809 | 1.391 | 1.063 | 0.904 | 0.803 | 0.723 | 0.657 | 0.602 |
| 3.5 | 1.827 | 1.408 | 1.079 | 0.918 | 0.817 | 0.736 | 0.670 | 0.614 |
| 4. | 1.860 | 1.438 | 1.106 | 0.944 | 0.842 | 0.760 | 0.692 | 0.636 |
| 4.5 | 1.888 | 1.465 | 1.130 | 0.967 | 0.864 | 0.780 | 0.712 | 0.655 |
| 5. | 1.912 | 1.487 | 1.152 | 0.987 | 0.883 | 0.799 | 0.730 | 0.672 |
| 5.5 | 1.933 | 1.508 | 1.170 | 1.005 | 0.900 | 0.816 | 0.746 | 0.688 |
| 6 | 1.952 | 1.526 | 1.187 | 1.021 | 0.916 | 0.831 | 0.760 | 0.702 |
| 7 | 1.985 | 1.557 | 1.217 | 1.050 | 0.943 | 0.857 | 0.786 | 0.727 |
| 8 | 2.012 | 1.583 | 1.242 | 1.073 | 0.966 | 0.880 | 0.808 | 0.748 |
| 9 | 2.035 | 1.605 | 1.263 | 1.094 | 0.986 | 0.899 | 0.827 | 0.767 |
| 10 | 2.055 | 1.625 | 1.282 | 1.112 | 1.004 | 0.916 | 0.844 | 0.783 |
| 11 | 2.073 | 1.642 | 1.298 | 1.128 | 1.020 | 0.932 | 0.859 | 0.798 |
| 12 | 2.088 | 1.657 | 1.313 | 1.143 | 1.034 | 0.946 | 0.873 | 0.811 |
| 13 | 2.102 | 1.670 | 1.326 | 1.156 | 1.047 | 0.958 | 0.885 | 0.823 |
| 14 | 2.114 | 1.683 | 1.338 | 1.168 | 1.058 | 0.970 | 0.896 | 0.834 |
| 15 | 2.126 | 1.694 | 1.349 | 1.178 | 1.069 | 0.980 | 0.907 | 0.845 |
| 20 | 2.170 | 1.738 | 1.393 | 1.222 | 1.112 | 1.023 | 0.949 | 0.886 |

PART 4 (cont.).—Co-efficient (c) of Mean Velocity
Corresponding to Values of R , the Hydraulic

$$n = 0.010$$

| R in feet | S per thousand | | | | |
|----------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.1 | 0.938 | 0.932 | 0.923 | 0.916 | 0.905 |
| 0.2 | 1.132 | 1.126 | 1.117 | 1.111 | 1.101 |
| 0.3 | 1.245 | 1.241 | 1.233 | 1.226 | 1.217 |
| 0.4 | 1.325 | 1.320 | 1.313 | 1.307 | 1.299 |
| 0.5 | 1.385 | 1.381 | 1.374 | 1.369 | 1.361 |
| 0.6 | 1.433 | 1.430 | 1.423 | 1.419 | 1.411 |
| 0.7 | 1.473 | 1.470 | 1.464 | 1.460 | 1.453 |
| 0.8 | 1.507 | 1.504 | 1.499 | 1.494 | 1.488 |
| 0.9 | 1.536 | 1.533 | 1.528 | 1.524 | 1.519 |
| 1. | 1.562 | 1.559 | 1.554 | 1.551 | 1.546 |
| 1.5 | 1.655 | 1.653 | 1.650 | 1.648 | 1.644 |
| 2. | 1.716 | 1.715 | 1.713 | 1.712 | 1.710 |
| 2.5 | 1.761 | 1.760 | 1.759 | 1.758 | 1.757 |
| 3. | 1.795 | 1.795 | 1.794 | 1.794 | 1.794 |
| 3.5 | 1.823 | 1.823 | 1.823 | 1.823 | 1.823 |
| 4. | 1.845 | 1.846 | 1.847 | 1.847 | 1.848 |
| 4.5 | 1.865 | 1.865 | 1.867 | 1.867 | 1.869 |
| 5. | 1.881 | 1.882 | 1.884 | 1.885 | 1.887 |
| 5.5 | 1.896 | 1.897 | 1.899 | 1.900 | 1.903 |
| 6. | 1.909 | 1.910 | 1.913 | 1.914 | 1.917 |
| 7. | 1.931 | 1.933 | 1.935 | 1.937 | 1.941 |
| 8. | 1.949 | 1.951 | 1.954 | 1.957 | 1.960 |
| 9. | 1.964 | 1.966 | 1.970 | 1.973 | 1.977 |
| 10. | 1.977 | 1.980 | 1.984 | 1.987 | 1.991 |
| 11. | 1.989 | 1.991 | 1.995 | 1.999 | 2.004 |
| 12. | 1.999 | 2.002 | 2.006 | 2.009 | 2.015 |
| 13. | 2.008 | 2.011 | 2.015 | 2.019 | 2.024 |
| 14. | 2.016 | 2.019 | 2.024 | 2.027 | 2.033 |
| 15. | 2.023 | 2.026 | 2.031 | 2.035 | 2.041 |
| 16. | 2.030 | 2.033 | 2.038 | 2.042 | 2.048 |
| 20. | 2.051 | 2.055 | 2.061 | 2.065 | 2.072 |

TABLE XII. PART 4] *HYDRAULIC COEFFICIENTS.*

*for Cement and Glazed Material (New),
Radius in feet, and of S per thousand.*

$n = 0.010$

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.1 | 0.889 | 0.858 | 0.830 | 0.783 | 0.682 |
| 0.2 | 1.085 | 1.055 | 1.028 | 0.980 | 0.875 |
| 0.3 | 1.202 | 1.174 | 1.149 | 1.104 | 1.001 |
| 0.4 | 1.285 | 1.259 | 1.236 | 1.193 | 1.095 |
| 0.5 | 1.349 | 1.325 | 1.303 | 1.263 | 1.170 |
| 0.6 | 1.400 | 1.378 | 1.357 | 1.320 | 1.233 |
| 0.7 | 1.442 | 1.422 | 1.403 | 1.368 | 1.286 |
| 0.8 | 1.478 | 1.460 | 1.442 | 1.410 | 1.332 |
| 0.9 | 1.510 | 1.492 | 1.476 | 1.446 | 1.373 |
| 1 | 1.537 | 1.521 | 1.506 | 1.478 | 1.410 |
| 1.5 | 1.639 | 1.628 | 1.618 | 1.598 | 1.551 |
| 2 | 1.706 | 1.699 | 1.692 | 1.680 | 1.649 |
| 2.5 | 1.755 | 1.751 | 1.748 | 1.741 | 1.723 |
| 3 | 1.793 | 1.792 | 1.791 | 1.788 | 1.783 |
| 3.5 | 1.824 | 1.825 | 1.826 | 1.827 | 1.832 |
| 4 | 1.849 | 1.852 | 1.855 | 1.860 | 1.873 |
| 4.5 | 1.871 | 1.875 | 1.880 | 1.888 | 1.909 |
| 5 | 1.890 | 1.896 | 1.901 | 1.912 | 1.940 |
| 5.5 | 1.906 | 1.914 | 1.920 | 1.933 | 1.968 |
| 6 | 1.921 | 1.929 | 1.937 | 1.952 | 1.993 |
| 7 | 1.946 | 1.956 | 1.966 | 1.985 | 2.036 |
| 8 | 1.966 | 1.978 | 1.990 | 2.012 | 2.072 |
| 9 | 1.984 | 1.997 | 2.010 | 2.035 | 2.103 |
| 10 | 1.999 | 2.013 | 2.028 | 2.055 | 2.130 |
| 11 | 2.012 | 2.027 | 2.043 | 2.073 | 2.154 |
| 12 | 2.023 | 2.040 | 2.056 | 2.088 | 2.175 |
| 13 | 2.033 | 2.051 | 2.068 | 2.102 | 2.194 |
| 14 | 2.042 | 2.061 | 2.079 | 2.114 | 2.211 |
| 15 | 2.051 | 2.070 | 2.089 | 2.126 | 2.227 |
| 16 | 2.058 | 2.078 | 2.098 | 2.136 | 2.241 |
| 20 | 2.083 | 2.106 | 2.127 | 2.170 | 2.229 |

PART 4 (cont.).—Co-efficients (c) of Mean Velocity for Brickwork,
Corresponding to values of R

$$n = 0.013$$

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.1 | 0.650 | 0.646 | 0.639 | 0.634 | 0.627 |
| 0.2 | 0.802 | 0.798 | 0.791 | 0.786 | 0.779 |
| 0.3 | 0.895 | 0.891 | 0.885 | 0.880 | 0.873 |
| 0.4 | 0.961 | 0.957 | 0.951 | 0.947 | 0.940 |
| 0.5 | 1.011 | 1.008 | 1.003 | 0.999 | 0.992 |
| 0.6 | 1.053 | 1.050 | 1.045 | 1.041 | 1.035 |
| 0.7 | 1.087 | 1.084 | 1.080 | 1.076 | 1.071 |
| 0.8 | 1.117 | 1.114 | 1.110 | 1.106 | 1.101 |
| 0.9 | 1.142 | 1.140 | 1.136 | 1.133 | 1.128 |
| 1.0 | 1.165 | 1.163 | 1.159 | 1.156 | 1.152 |
| 1.5 | 1.249 | 1.247 | 1.247 | 1.243 | 1.240 |
| 2 | 1.305 | 1.304 | 1.302 | 1.301 | 1.299 |
| 2.5 | 1.346 | 1.345 | 1.344 | 1.344 | 1.343 |
| 3 | 1.378 | 1.378 | 1.378 | 1.377 | 1.377 |
| 3.5 | 1.404 | 1.404 | 1.404 | 1.404 | 1.405 |
| 4 | 1.426 | 1.426 | 1.427 | 1.427 | 1.428 |
| 4.5 | 1.444 | 1.445 | 1.446 | 1.447 | 1.448 |
| 5 | 1.460 | 1.461 | 1.463 | 1.464 | 1.465 |
| 5.5 | 1.474 | 1.475 | 1.477 | 1.478 | 1.480 |
| 6 | 1.487 | 1.488 | 1.490 | 1.492 | 1.494 |
| 7 | 1.508 | 1.510 | 1.512 | 1.514 | 1.517 |
| 8 | 1.526 | 1.528 | 1.530 | 1.533 | 1.536 |
| 9 | 1.541 | 1.543 | 1.546 | 1.548 | 1.552 |
| 10 | 1.554 | 1.556 | 1.559 | 1.562 | 1.566 |
| 11 | 1.565 | 1.567 | 1.571 | 1.574 | 1.579 |
| 12 | 1.575 | 1.577 | 1.581 | 1.585 | 1.589 |
| 13 | 1.584 | 1.586 | 1.591 | 1.594 | 1.599 |
| 14 | 1.592 | 1.594 | 1.599 | 1.602 | 1.608 |
| 15 | 1.599 | 1.602 | 1.606 | 1.610 | 1.616 |
| 16 | 1.606 | 1.608 | 1.613 | 1.617 | 1.623 |
| 20 | 1.627 | 1.630 | 1.636 | 1.640 | 1.647 |

*Ashlar, New Cast and Wrought Iron, and Unglazed Stoneware
in feet and S per thousand.*

$$n = 0.013$$

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.1 | 0.615 | 0.593 | 0.574 | 0.541 | 0.472 |
| 0.2 | 0.767 | 0.745 | 0.725 | 0.691 | 0.617 |
| 0.3 | 0.861 | 0.840 | 0.821 | 0.788 | 0.714 |
| 0.4 | 0.929 | 0.910 | 0.891 | 0.859 | 0.788 |
| 0.5 | 0.932 | 0.964 | 0.947 | 0.916 | 0.847 |
| 0.6 | 1.026 | 1.008 | 0.992 | 0.963 | 0.898 |
| 0.7 | 1.062 | 1.046 | 1.031 | 1.003 | 0.941 |
| 0.8 | 1.093 | 1.078 | 1.064 | 1.038 | 0.979 |
| 0.9 | 1.120 | 1.106 | 1.093 | 1.069 | 1.013 |
| 1 | 1.145 | 1.131 | 1.119 | 1.097 | 1.044 |
| 1.5 | 1.235 | 1.226 | 1.217 | 1.201 | 1.163 |
| 2 | 1.296 | 1.290 | 1.284 | 1.274 | 1.249 |
| 2.5 | 1.341 | 1.338 | 1.335 | 1.329 | 1.314 |
| 3 | 1.376 | 1.375 | 1.374 | 1.372 | 1.367 |
| 3.5 | 1.405 | 1.406 | 1.407 | 1.408 | 1.412 |
| 4 | 1.429 | 1.432 | 1.434 | 1.438 | 1.450 |
| 4.5 | 1.450 | 1.454 | 1.458 | 1.465 | 1.483 |
| 5 | 1.468 | 1.473 | 1.478 | 1.487 | 1.512 |
| 5.5 | 1.484 | 1.490 | 1.496 | 1.508 | 1.538 |
| 6 | 1.498 | 1.505 | 1.512 | 1.526 | 1.561 |
| 7 | 1.522 | 1.531 | 1.540 | 1.557 | 1.602 |
| 8 | 1.542 | 1.553 | 1.563 | 1.583 | 1.636 |
| 9 | 1.559 | 1.571 | 1.583 | 1.605 | 1.665 |
| 10 | 1.573 | 1.587 | 1.600 | 1.625 | 1.691 |
| 11 | 1.586 | 1.601 | 1.615 | 1.642 | 1.714 |
| 12 | 1.597 | 1.613 | 1.628 | 1.657 | 1.735 |
| 13 | 1.607 | 1.624 | 1.640 | 1.670 | 1.753 |
| 14 | 1.617 | 1.634 | 1.650 | 1.683 | 1.770 |
| 15 | 1.625 | 1.643 | 1.660 | 1.694 | 1.785 |
| 16 | 1.632 | 1.651 | 1.669 | 1.704 | 1.799 |
| 20 | 1.657 | 1.678 | 1.699 | 1.738 | 1.846 |

PART 3 (cont.).—Co-efficients (c) of Mean Velocity for New Rubble,
Corresponding to values of R

$$n = 0.017$$

| R n feet | S per thousand | | | | |
|-------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.1 | 0.445 | 0.443 | 0.438 | 0.434 | 0.429 |
| 0.2 | 0.561 | 0.558 | 0.554 | 0.550 | 0.545 |
| 0.3 | 0.634 | 0.632 | 0.627 | 0.623 | 0.618 |
| 0.4 | 0.688 | 0.685 | 0.681 | 0.677 | 0.672 |
| 0.7 | 0.730 | 0.727 | 0.723 | 0.720 | 0.715 |
| 0.6 | 0.764 | 0.762 | 0.758 | 0.755 | 0.750 |
| 0.7 | 0.793 | 0.791 | 0.787 | 0.784 | 0.780 |
| 0.8 | 0.818 | 0.816 | 0.813 | 0.810 | 0.806 |
| 0.9 | 0.840 | 0.838 | 0.835 | 0.833 | 0.829 |
| 1 | 0.860 | 0.858 | 0.855 | 0.853 | 0.849 |
| 1.5 | 0.933 | 0.932 | 0.930 | 0.928 | 0.926 |
| 2 | 0.984 | 0.983 | 0.982 | 0.980 | 0.979 |
| 2.5 | 1.021 | 1.021 | 1.020 | 1.019 | 1.019 |
| 3 | 1.051 | 1.051 | 1.050 | 1.050 | 1.050 |
| 3.5 | 1.075 | 1.075 | 1.075 | 1.076 | 1.076 |
| 4 | 1.095 | 1.096 | 1.096 | 1.097 | 1.097 |
| 4.5 | 1.113 | 1.113 | 1.114 | 1.115 | 1.116 |
| 5 | 1.128 | 1.129 | 1.130 | 1.131 | 1.132 |
| 5.5 | 1.141 | 1.142 | 1.144 | 1.145 | 1.147 |
| 6 | 1.153 | 1.154 | 1.156 | 1.158 | 1.160 |
| 7 | 1.174 | 1.175 | 1.177 | 1.179 | 1.182 |
| 8 | 1.191 | 1.193 | 1.195 | 1.197 | 1.200 |
| 9 | 1.205 | 1.207 | 1.210 | 1.212 | 1.216 |
| 10 | 1.218 | 1.220 | 1.223 | 1.226 | 1.230 |
| 11 | 1.229 | 1.231 | 1.235 | 1.237 | 1.242 |
| 12 | 1.239 | 1.241 | 1.245 | 1.248 | 1.252 |
| 13 | 1.248 | 1.250 | 1.254 | 1.257 | 1.262 |
| 14 | 1.256 | 1.258 | 1.262 | 1.265 | 1.270 |
| 15 | 1.263 | 1.265 | 1.270 | 1.273 | 1.278 |
| 16 | 1.269 | 1.272 | 1.276 | 1.280 | 1.285 |
| 20 | 1.291 | 1.294 | 1.299 | 1.303 | 1.309 |

Old Brickwork or Ashlar, and Old Iron and Unglazed Stoneware.
in feet and S per thousand.

$$n = 0.017$$

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.1 | 0.421 | 0.406 | 0.393 | 0.371 | 0.326 |
| 0.2 | 0.536 | 0.520 | 0.507 | 0.483 | 0.433 |
| 0.3 | 0.610 | 0.594 | 0.581 | 0.557 | 0.506 |
| 0.4 | 0.664 | 0.650 | 0.636 | 0.613 | 0.563 |
| 0.5 | 0.708 | 0.693 | 0.681 | 0.658 | 0.610 |
| 0.6 | 0.743 | 0.730 | 0.718 | 0.696 | 0.649 |
| 0.7 | 0.773 | 0.761 | 0.749 | 0.729 | 0.684 |
| 0.8 | 0.800 | 0.788 | 0.777 | 0.758 | 0.715 |
| 0.9 | 0.823 | 0.812 | 0.802 | 0.783 | 0.742 |
| 1 | 0.844 | 0.833 | 0.824 | 0.806 | 0.767 |
| 1.5 | 0.922 | 0.915 | 0.908 | 0.895 | 0.867 |
| 2 | 0.976 | 0.971 | 0.967 | 0.959 | 0.939 |
| 2.5 | 1.017 | 1.014 | 1.012 | 1.007 | 0.996 |
| 3 | 1.049 | 1.049 | 1.048 | 1.046 | 1.042 |
| 3.5 | 1.076 | 1.077 | 1.077 | 1.079 | 1.082 |
| 4 | 1.099 | 1.101 | 1.103 | 1.106 | 1.115 |
| 4.5 | 1.118 | 1.121 | 1.124 | 1.130 | 1.145 |
| 5 | 1.135 | 1.139 | 1.144 | 1.152 | 1.172 |
| 5.5 | 1.150 | 1.155 | 1.161 | 1.170 | 1.195 |
| 6 | 1.163 | 1.170 | 1.176 | 1.187 | 1.217 |
| 7 | 1.186 | 1.194 | 1.202 | 1.217 | 1.254 |
| 8 | 1.205 | 1.215 | 1.224 | 1.242 | 1.287 |
| 9 | 1.222 | 1.233 | 1.243 | 1.263 | 1.314 |
| 10 | 1.236 | 1.248 | 1.260 | 1.282 | 1.339 |
| 11 | 1.248 | 1.261 | 1.274 | 1.298 | 1.361 |
| 12 | 1.259 | 1.273 | 1.287 | 1.313 | 1.380 |
| 13 | 1.269 | 1.284 | 1.299 | 1.326 | 1.398 |
| 14 | 1.278 | 1.294 | 1.309 | 1.338 | 1.414 |
| 15 | 1.287 | 1.303 | 1.319 | 1.349 | 1.429 |
| 16 | 1.294 | 1.311 | 1.328 | 1.359 | 1.443 |
| 20 | 1.319 | 1.338 | 1.357 | 1.393 | 1.489 |

PART 4 (cont.).—Coefficients (c) of Mean Velocity for damaged Rubble, or for Earthwork in Class I. of the best order corresponding to values of R in feet, and of S per thousand, when $n=0.020$.

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.4 | 0.561 | 0.559 | 0.555 | 0.553 | 0.549 |
| 0.6 | 0.629 | 0.627 | 0.623 | 0.621 | 0.617 |
| 0.8 | 0.677 | 0.675 | 0.672 | 0.670 | 0.667 |
| 1 | 0.715 | 0.713 | 0.711 | 0.709 | 0.706 |
| 1.5 | 0.782 | 0.781 | 0.779 | 0.778 | 0.776 |
| 2 | 0.829 | 0.828 | 0.827 | 0.826 | 0.825 |
| 2.5 | 0.864 | 0.864 | 0.863 | 0.863 | 0.862 |
| 3 | 0.892 | 0.892 | 0.892 | 0.892 | 0.891 |
| 4 | 0.935 | 0.935 | 0.935 | 0.936 | 0.936 |
| 5 | 0.966 | 0.967 | 0.968 | 0.969 | 0.970 |
| 6 | 0.991 | 0.991 | 0.993 | 0.994 | 0.996 |
| 7 | 1.010 | 1.012 | 1.014 | 1.015 | 1.018 |
| 8 | 1.027 | 1.029 | 1.031 | 1.033 | 1.036 |
| 9 | 1.041 | 1.043 | 1.046 | 1.048 | 1.051 |
| 10 | 1.054 | 1.056 | 1.059 | 1.061 | 1.064 |
| 11 | 1.065 | 1.067 | 1.070 | 1.072 | 1.076 |
| 12 | 1.074 | 1.076 | 1.080 | 1.083 | 1.087 |
| 13 | 1.083 | 1.085 | 1.089 | 1.092 | 1.096 |
| 14 | 1.091 | 1.093 | 1.097 | 1.100 | 1.105 |
| 16 | 1.104 | 1.107 | 1.111 | 1.115 | 1.120 |
| 20 | 1.126 | 1.129 | 1.134 | 1.138 | 1.143 |

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.4 | 0.542 | 0.530 | 0.519 | 0.500 | 0.460 |
| 0.6 | 0.611 | 0.600 | 0.590 | 0.572 | 0.534 |
| 0.8 | 0.661 | 0.651 | 0.642 | 0.626 | 0.591 |
| 1 | 0.701 | 0.692 | 0.684 | 0.669 | 0.637 |
| 1.5 | 0.772 | 0.766 | 0.760 | 0.750 | 0.725 |
| 2 | 0.822 | 0.818 | 0.814 | 0.807 | 0.790 |
| 2.5 | 0.861 | 0.858 | 0.856 | 0.852 | 0.842 |
| 3 | 0.891 | 0.890 | 0.889 | 0.888 | 0.885 |
| 4 | 0.937 | 0.939 | 0.941 | 0.944 | 0.952 |
| 5 | 0.972 | 0.976 | 0.980 | 0.987 | 1.005 |
| 6 | 0.999 | 1.005 | 1.011 | 1.021 | 1.047 |
| 7 | 1.022 | 1.029 | 1.036 | 1.050 | 1.083 |
| 8 | 1.040 | 1.049 | 1.058 | 1.074 | 1.114 |
| 9 | 1.056 | 1.066 | 1.076 | 1.094 | 1.140 |
| 10 | 1.070 | 1.081 | 1.092 | 1.112 | 1.164 |
| 11 | 1.083 | 1.095 | 1.106 | 1.128 | 1.185 |
| 12 | 1.093 | 1.107 | 1.119 | 1.143 | 1.204 |
| 13 | 1.103 | 1.117 | 1.131 | 1.156 | 1.221 |
| 14 | 1.112 | 1.127 | 1.141 | 1.168 | 1.237 |
| 16 | 1.128 | 1.144 | 1.159 | 1.188 | 1.265 |
| 20 | 1.153 | 1.171 | 1.188 | 1.222 | 1.310 |

PART 4 (cont.).—*Coefficients (c) of Mean Velocity for Earth-work in Class II. in above-average order, corresponding to values of R in feet, and of S per thousand, when $n=0.0225$.*

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.4 | 0.484 | 0.482 | 0.479 | 0.477 | 0.473 |
| 0.6 | 0.545 | 0.544 | 0.541 | 0.539 | 0.535 |
| 0.8 | 0.590 | 0.588 | 0.586 | 0.584 | 0.581 |
| 1 | 0.625 | 0.623 | 0.621 | 0.619 | 0.617 |
| 1.5 | 0.688 | 0.687 | 0.685 | 0.684 | 0.682 |
| 2 | 0.732 | 0.731 | 0.730 | 0.729 | 0.728 |
| 2.5 | 0.765 | 0.765 | 0.764 | 0.764 | 0.763 |
| 3 | 0.792 | 0.792 | 0.792 | 0.792 | 0.791 |
| 4 | 0.832 | 0.833 | 0.834 | 0.834 | 0.835 |
| 5 | 0.863 | 0.864 | 0.865 | 0.866 | 0.867 |
| 6 | 0.887 | 0.888 | 0.890 | 0.891 | 0.893 |
| 7 | 0.907 | 0.908 | 0.910 | 0.911 | 0.913 |
| 8 | 0.923 | 0.924 | 0.926 | 0.928 | 0.931 |
| 9 | 0.937 | 0.939 | 0.941 | 0.943 | 0.946 |
| 10 | 0.949 | 0.951 | 0.954 | 0.956 | 0.959 |
| 11 | 0.960 | 0.962 | 0.965 | 0.967 | 0.971 |
| 12 | 0.969 | 0.971 | 0.975 | 0.977 | 0.981 |
| 13 | 0.978 | 0.980 | 0.984 | 0.987 | 0.991 |
| 14 | 0.986 | 0.988 | 0.992 | 0.995 | 0.999 |
| 16 | 0.999 | 1.002 | 1.006 | 1.009 | 1.014 |
| 20 | 1.021 | 1.024 | 1.028 | 1.032 | 1.037 |

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.4 | 0.467 | 0.457 | 0.448 | 0.432 | 0.398 |
| 0.6 | 0.530 | 0.520 | 0.512 | 0.497 | 0.464 |
| 0.8 | 0.576 | 0.567 | 0.559 | 0.546 | 0.515 |
| 1 | 0.612 | 0.605 | 0.597 | 0.585 | 0.557 |
| 1.5 | 0.679 | 0.673 | 0.668 | 0.659 | 0.638 |
| 2 | 0.726 | 0.722 | 0.719 | 0.712 | 0.698 |
| 2.5 | 0.762 | 0.760 | 0.758 | 0.754 | 0.746 |
| 3 | 0.791 | 0.790 | 0.790 | 0.788 | 0.785 |
| 4 | 0.836 | 0.837 | 0.839 | 0.842 | 0.849 |
| 5 | 0.869 | 0.873 | 0.876 | 0.883 | 0.899 |
| 6 | 0.895 | 0.901 | 0.906 | 0.916 | 0.939 |
| 7 | 0.917 | 0.924 | 0.931 | 0.943 | 0.973 |
| 8 | 0.935 | 0.944 | 0.952 | 0.966 | 1.003 |
| 9 | 0.951 | 0.961 | 0.970 | 0.986 | 1.029 |
| 10 | 0.965 | 0.975 | 0.985 | 1.004 | 1.051 |
| 11 | 0.977 | 0.988 | 0.999 | 1.020 | 1.072 |
| 12 | 0.988 | 1.000 | 1.012 | 1.034 | 1.090 |
| 13 | 0.997 | 1.011 | 1.023 | 1.047 | 1.107 |
| 14 | 1.006 | 1.020 | 1.033 | 1.058 | 1.123 |
| 16 | 1.022 | 1.037 | 1.051 | 1.079 | 1.150 |
| 20 | 1.046 | 1.064 | 1.080 | 1.112 | 1.195 |

PART 4 (cont.).—Co-efficients (c) of Mean Velocity, for Earthwork in Class III., in good average order, corresponding to values of R in feet, and of S per thousand, when $n=0.025$.

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.4 | 0.424 | 0.422 | 0.420 | 0.418 | 0.414 |
| 0.6 | 0.480 | 0.479 | 0.476 | 0.474 | 0.471 |
| 0.8 | 0.521 | 0.520 | 0.518 | 0.516 | 0.513 |
| 1 | 0.554 | 0.553 | 0.550 | 0.549 | 0.546 |
| 1.5 | 0.613 | 0.612 | 0.611 | 0.609 | 0.608 |
| 2 | 0.655 | 0.654 | 0.653 | 0.652 | 0.651 |
| 2.5 | 0.687 | 0.686 | 0.686 | 0.685 | 0.684 |
| 3 | 0.712 | 0.712 | 0.712 | 0.712 | 0.711 |
| 4 | 0.751 | 0.752 | 0.752 | 0.753 | 0.753 |
| 5 | 0.781 | 0.781 | 0.782 | 0.783 | 0.784 |
| 6 | 0.804 | 0.805 | 0.806 | 0.808 | 0.809 |
| 6 | 0.823 | 0.824 | 0.826 | 0.827 | 0.830 |
| 8 | 0.839 | 0.840 | 0.843 | 0.844 | 0.847 |
| 9 | 0.853 | 0.854 | 0.857 | 0.859 | 0.862 |
| 10 | 0.865 | 0.867 | 0.869 | 0.871 | 0.875 |
| 11 | 0.876 | 0.877 | 0.880 | 0.883 | 0.886 |
| 12 | 0.885 | 0.887 | 0.890 | 0.893 | 0.896 |
| 13 | 0.893 | 0.895 | 0.899 | 0.902 | 0.905 |
| 14 | 0.901 | 0.903 | 0.907 | 0.910 | 0.914 |
| 16 | 0.915 | 0.917 | 0.921 | 0.924 | 0.929 |
| 20 | 0.936 | 0.939 | 0.943 | 0.947 | 0.952 |

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.4 | 0.409 | 0.400 | 0.392 | 0.379 | 0.350 |
| 0.6 | 0.467 | 0.458 | 0.451 | 0.437 | 0.410 |
| 0.8 | 0.509 | 0.501 | 0.494 | 0.482 | 0.456 |
| 1 | 0.543 | 0.536 | 0.529 | 0.518 | 0.494 |
| 1.5 | 0.605 | 0.600 | 0.595 | 0.587 | 0.568 |
| 2 | 0.649 | 0.646 | 0.643 | 0.637 | 0.624 |
| 2.5 | 0.683 | 0.681 | 0.680 | 0.676 | 0.669 |
| 3 | 0.711 | 0.710 | 0.710 | 0.709 | 0.706 |
| 4 | 0.754 | 0.755 | 0.757 | 0.760 | 0.766 |
| 5 | 0.786 | 0.790 | 0.793 | 0.799 | 0.813 |
| 6 | 0.812 | 0.817 | 0.822 | 0.831 | 0.852 |
| 7 | 0.833 | 0.840 | 0.846 | 0.857 | 0.885 |
| 8 | 0.851 | 0.859 | 0.866 | 0.880 | 0.913 |
| 9 | 0.866 | 0.875 | 0.884 | 0.899 | 0.938 |
| 10 | 0.880 | 0.890 | 0.899 | 0.916 | 0.960 |
| 11 | 0.892 | 0.902 | 0.913 | 0.932 | 0.980 |
| 12 | 0.902 | 0.914 | 0.925 | 0.946 | 0.998 |
| 13 | 0.912 | 0.924 | 0.936 | 0.958 | 1.015 |
| 14 | 0.921 | 0.934 | 0.946 | 0.970 | 1.030 |
| 16 | 0.936 | 0.950 | 0.964 | 0.990 | 1.057 |
| 20 | 0.961 | 0.977 | 0.993 | 1.023 | 1.101 |

PART 4 (cont.).—*Co-efficients (c) of Mean Velocity, for Earthwork in Class IV. in below-average order, corresponding to values of R in feet, and of S per thousand, when $n=0.0275$.*

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.4 | 0.376 | 0.375 | 0.372 | 0.370 | 0.368 |
| 0.6 | 0.428 | 0.427 | 0.424 | 0.423 | 0.420 |
| 0.8 | 0.466 | 0.465 | 0.463 | 0.461 | 0.459 |
| 1. | 0.496 | 0.495 | 0.493 | 0.492 | 0.490 |
| 1.5 | 0.552 | 0.551 | 0.550 | 0.549 | 0.547 |
| 2. | 0.592 | 0.591 | 0.590 | 0.589 | 0.588 |
| 2.5 | 0.622 | 0.622 | 0.621 | 0.621 | 0.620 |
| 3. | 0.647 | 0.647 | 0.646 | 0.646 | 0.646 |
| 4. | 0.685 | 0.685 | 0.685 | 0.686 | 0.686 |
| 5. | 0.713 | 0.714 | 0.715 | 0.715 | 0.716 |
| 6. | 0.736 | 0.737 | 0.738 | 0.739 | 0.741 |
| 7. | 0.754 | 0.755 | 0.757 | 0.758 | 0.760 |
| 8. | 0.770 | 0.771 | 0.773 | 0.775 | 0.777 |
| 9. | 0.784 | 0.785 | 0.787 | 0.789 | 0.792 |
| 10. | 0.795 | 0.797 | 0.800 | 0.802 | 0.805 |
| 11. | 0.806 | 0.808 | 0.810 | 0.813 | 0.816 |
| 12. | 0.815 | 0.817 | 0.820 | 0.822 | 0.826 |
| 13. | 0.824 | 0.826 | 0.829 | 0.831 | 0.835 |
| 14. | 0.831 | 0.833 | 0.837 | 0.839 | 0.843 |
| 16. | 0.845 | 0.847 | 0.851 | 0.854 | 0.858 |
| 20. | 0.866 | 0.869 | 0.873 | 0.876 | 0.881 |

| <i>R</i> in feet | <i>S</i> per thousand | | | | |
|---------------------|-----------------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.4 | 0.363 | 0.355 | 0.348 | 0.336 | 0.312 |
| 0.6 | 0.416 | 0.408 | 0.402 | 0.390 | 0.366 |
| 0.8 | 0.455 | 0.448 | 0.442 | 0.431 | 0.408 |
| 1. | 0.486 | 0.480 | 0.475 | 0.465 | 0.444 |
| 1.5 | 0.545 | 0.540 | 0.536 | 0.529 | 0.512 |
| 2. | 0.587 | 0.584 | 0.581 | 0.576 | 0.564 |
| 2.5 | 0.619 | 0.617 | 0.616 | 0.613 | 0.606 |
| 3. | 0.646 | 0.645 | 0.645 | 0.643 | 0.641 |
| 4. | 0.687 | 0.688 | 0.690 | 0.692 | 0.698 |
| 5. | 0.718 | 0.721 | 0.724 | 0.730 | 0.743 |
| 6. | 0.743 | 0.748 | 0.752 | 0.760 | 0.780 |
| 7. | 0.764 | 0.770 | 0.776 | 0.786 | 0.812 |
| 8. | 0.781 | 0.788 | 0.795 | 0.808 | 0.839 |
| 9. | 0.796 | 0.805 | 0.813 | 0.827 | 0.863 |
| 10. | 0.809 | 0.819 | 0.828 | 0.844 | 0.884 |
| 11. | 0.821 | 0.831 | 0.841 | 0.859 | 0.904 |
| 12. | 0.832 | 0.843 | 0.853 | 0.873 | 0.921 |
| 13. | 0.841 | 0.853 | 0.864 | 0.885 | 0.937 |
| 14. | 0.850 | 0.862 | 0.874 | 0.896 | 0.952 |
| 16. | 0.865 | 0.879 | 0.892 | 0.916 | 0.979 |
| 20. | 0.889 | 0.905 | 0.920 | 0.949 | 1.022 |

PART 4 (cont.).—Co-efficients (c) of Mean Velocity, for Earthwork in Class V., in bad order, partly overgrown, or partly impeded by detritus, when $n=0.030$.

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 1.0 | 0.8 | 0.6 | 0.5 | 0.4 |
| 0.4 | 0.337 | 0.336 | 0.334 | 0.332 | 0.330 |
| 0.6 | 0.385 | 0.384 | 0.382 | 0.380 | 0.378 |
| 0.8 | 0.421 | 0.420 | 0.418 | 0.416 | 0.414 |
| 1 | 0.449 | 0.448 | 0.447 | 0.445 | 0.443 |
| 1.5 | 0.502 | 0.501 | 0.500 | 0.499 | 0.498 |
| 2 | 0.540 | 0.539 | 0.538 | 0.538 | 0.537 |
| 2.5 | 0.569 | 0.568 | 0.568 | 0.568 | 0.567 |
| 3 | 0.592 | 0.592 | 0.592 | 0.592 | 0.592 |
| 4 | 0.629 | 0.629 | 0.630 | 0.630 | 0.630 |
| 5 | 0.657 | 0.657 | 0.658 | 0.659 | 0.660 |
| 6 | 0.679 | 0.679 | 0.681 | 0.682 | 0.683 |
| 7 | 0.697 | 0.698 | 0.699 | 0.701 | 0.703 |
| 8 | 0.712 | 0.713 | 0.715 | 0.717 | 0.719 |
| 9 | 0.726 | 0.727 | 0.729 | 0.731 | 0.733 |
| 10 | 0.737 | 0.739 | 0.741 | 0.743 | 0.746 |
| 11 | 0.748 | 0.749 | 0.752 | 0.754 | 0.757 |
| 12 | 0.757 | 0.759 | 0.761 | 0.764 | 0.767 |
| 13 | 0.765 | 0.767 | 0.770 | 0.772 | 0.776 |
| 14 | 0.773 | 0.775 | 0.778 | 0.780 | 0.784 |
| 15 | 0.786 | 0.788 | 0.782 | 0.795 | 0.799 |
| 20 | 0.807 | 0.810 | 0.814 | 0.817 | 0.822 |

| R in feet | S per thousand | | | | |
|--------------|----------------|-------|-------|-------|-------|
| | 0.3 | 0.2 | 0.15 | 0.1 | 0.05 |
| 0.4 | 0.326 | 0.319 | 0.313 | 0.302 | 0.281 |
| 0.6 | 0.374 | 0.368 | 0.362 | 0.352 | 0.330 |
| 0.8 | 0.411 | 0.405 | 0.399 | 0.390 | 0.370 |
| 1 | 0.440 | 0.435 | 0.430 | 0.421 | 0.402 |
| 1.5 | 0.495 | 0.491 | 0.487 | 0.480 | 0.466 |
| 2 | 0.535 | 0.532 | 0.529 | 0.525 | 0.514 |
| 2.5 | 0.566 | 0.564 | 0.563 | 0.560 | 0.554 |
| 3 | 0.591 | 0.591 | 0.590 | 0.589 | 0.587 |
| 4 | 0.631 | 0.632 | 0.634 | 0.636 | 0.641 |
| 5 | 0.661 | 0.664 | 0.667 | 0.672 | 0.684 |
| 6 | 0.686 | 0.690 | 0.694 | 0.702 | 0.720 |
| 7 | 0.706 | 0.711 | 0.717 | 0.727 | 0.750 |
| 8 | 0.723 | 0.730 | 0.736 | 0.748 | 0.776 |
| 9 | 0.738 | 0.745 | 0.753 | 0.767 | 0.800 |
| 10 | 0.751 | 0.759 | 0.768 | 0.783 | 0.821 |
| 11 | 0.762 | 0.772 | 0.781 | 0.798 | 0.839 |
| 12 | 0.772 | 0.783 | 0.793 | 0.811 | 0.856 |
| 13 | 0.782 | 0.793 | 0.804 | 0.823 | 0.872 |
| 14 | 0.790 | 0.802 | 0.814 | 0.834 | 0.887 |
| 16 | 0.806 | 0.818 | 0.831 | 0.854 | 0.912 |
| 20 | 0.830 | 0.845 | 0.859 | 0.886 | 0.955 |

PART 5.—*Co-efficients of Discharge for Orifices, being values of σ for the formula in Table X., and given in the Text.*

$$V = \sigma \times 8.025 \sqrt{H}$$

| Applied in the Table. | According to Experiment. | |
|-----------------------|--------------------------|---|
| .55 | .572 | } Rectangular, width 7 depth, ($W \ 7 \ D$); see next page. |
| .6 | .709 | |
| .62 | .62 | } Orifices generally. |
| .66 | .66 | |
| .7 | .7 | } Sluices without side walls. |
| .727 | .62 | |
| .84 | .83 | } Canal lock gates and dock gates. |
| .84 | .84 | |
| .9 | .9 | } Undershot wheel gates. |
| .96 | .94 | |
| .96 | .96 | } Sluices in lock gates. |
| .96 | .96 | |
| .96 | .96 | } Large vertical pipes. |
| .96 | .96 | |
| 1 | 1 | } Narrow bridge openings. |
| 1.3 | 1.3 | |
| | | } Large sluices. |
| | | |
| | | } Wide openings from reservoirs. |
| | | |
| | | } Wide bridge openings. |
| | | |
| | | } Orifices with converging mouth-pieces. |
| | | |
| | | } Large orifices with diverging mouth-pieces. |
| | | |
| | | } Attached diverging mill channels. |
| | | |

Modification of the co-efficient so as to include the effect due to velocity of approach ;

Let h = head due to this velocity only,

$$\text{then } \sigma_1 = \sigma \sqrt{1 + \frac{h}{H}}$$

and σ_1 is the new co-efficient to be used.

PART 5 (cont.).—Co-efficients of Discharge for Orifices.

Table of Co-efficients of Velocity or Discharge for Rectangular Orifices, when the depth (D) is less than the width (W) for a head (H).

| $\frac{H}{W}$ | $\frac{D}{W}$ 1.0 | $\frac{D}{W}$ 0.5 | $\frac{D}{W}$ 0.25 | $\frac{D}{W}$ 0.15 | $\frac{D}{W}$ 0.1 | $\frac{D}{W}$ 0.05 |
|---------------|----------------------|----------------------|-----------------------|-----------------------|----------------------|-----------------------|
| | Values of σ | | | | | |
| .05 | | | | | | .709 |
| .10 | | | | | .660 | .698 |
| .15 | | | | .638 | .660 | .691 |
| .20 | | | .612 | .640 | .659 | .685 |
| .25 | | | .617 | .640 | .659 | .682 |
| .30 | | | .622 | .640 | .658 | .678 |
| .40 | | .600 | .626 | .639 | .657 | .671 |
| .50 | | .605 | .628 | .638 | .655 | .667 |
| .60 | .572 | .609 | .630 | .637 | .654 | .664 |
| .75 | .585 | .611 | .631 | .635 | .653 | .660 |
| 1.00 | .592 | .613 | .634 | .634 | .650 | .655 |
| 1.50 | .598 | .616 | .632 | .632 | .645 | .650 |
| 2.00 | .400 | .617 | .631 | .631 | .642 | .647 |
| 2.50 | .602 | .617 | .631 | .630 | .640 | .643 |
| 3.50 | .604 | .616 | .629 | .629 | .637 | .638 |
| 4.00 | .605 | .615 | .627 | .627 | .632 | .627 |
| 6.00 | .604 | .613 | .623 | .623 | .625 | .621 |
| 8.00 | .602 | .611 | .619 | .619 | .618 | .616 |
| 10.00 | .601 | .607 | .613 | .613 | .613 | .613 |

The above was deduced by Rankine from results of experiments by Poncelet and Lesbros.

N.B.—When $H \geq 3D$, the centre of figure may be considered the centre of motion.

PART 6.—*Co-efficients of Discharge for Overfalls, being values of σ for the formula applied in Table X., and given in the Text.*¹

$$V = \frac{2}{3}\sigma \times 8 \cdot 025 \sqrt{H}$$

Here l = length of weir sill : L = length of dam, or breadth of channel :
 H = head on sill : D = depth of notch.

| In Table. | By Experimentalists. | |
|------------|----------------------|---|
| ·5 | ·5 | { Broad-crested or flat-topped dams Dams with a channel attached |
| ·55 | ·595 | { Weirs with 1-inch crests when $l =$ or $7 \frac{L}{4}$; the exact value of σ being = $\cdot 57 \times \frac{l}{10L}$ |
| | ·662 | |
| ·6 | ·6 | { Overfalls when $l > \frac{L}{4}$ and $< \frac{L}{3}$ V-shaped notch, when $l = \frac{D}{2}$ |
| 62 | ·62 | V-shaped notch, when $l = \frac{D}{4}$ |
| ·666 | ·552 | { Weirs when $l = L$, and $H > \frac{1}{3}$ height of the barrier; in this case the velocity of approach must be considered in addition. |
| ·7 ·727 | ·666 | Weirs generally when $l = L$ and $H < \frac{1}{3}$ the height of the barrier. |

To modify the co-efficient σ so as to include the effect due to velocity of approach,

Let h = head due to velocity of approach only :—

$$\text{then } \sigma_1 = \sigma \left\{ \left(1 + \frac{h}{H} \right)^{\frac{3}{2}} - \left(\frac{h}{H} \right)^{\frac{3}{2}} \right\}$$

and σ_1 is the new co-efficient to be used.

¹ In using Table X. for overfalls, always diminish the velocity of discharge there given by one-third; this alone admits of the use of the same table for discharges both of orifices and overfalls.



APPENDIX
OF
MISCELLANEOUS TABLES AND DATA.

MASONRY DAMS.
RETAINING WALLS.
WEIGHT OF MATERIALS.
THICKNESS AND WEIGHT OF
WATER-PIPES.
ABSORPTION AND STRENGTH
OF STONEWARE PIPES.
OVOID CULVERT-SECTIONS.

TABLE OF ARCS AND SECTORS.
TABLES OF POWERS, ROOTS,
AND RECIPROCAL.
DUTY OF HYDRAULIC MACHINES
AND CONTRIVANCES.
CONSTANTS OF LABOUR AND
CARTAGE.

Masonry Dams.

(By the Author.)

Dimensions of Trapezoidal Masonry Dams, having both faces battering, for heights up to 40 feet.

| | Good rubble. | Inferior rubble. | Brickwork. |
|-------------------------------|----------------|------------------|--------------|
| Height of dam | H | H | H |
| Thickness at top | $\frac{1}{8}H$ | $\cdot 2H$ | $\cdot 3H$ |
| Thickness at bottom | $\cdot 5H$ | $\cdot 6H$ | $\cdot 7H$ |
| Front batter | 1 in 24 | 1 in 15 | 1 in 15 |
| Back batter | 1 in 3 | 1 in 3 | 1 in 3 |
| Sectional area | $\cdot 3H^2$ | $\cdot 4H^2$ | $\cdot 5H^2$ |

Dimensions of Trapezoidal Masonry Dams, having the water face vertical, for heights up to 40 feet.

| | Good rubble. | Inferior rubble. | Brickwork. |
|--|---------------|------------------|---------------|
| Weight of masonry per cubic foot | 140 lbs. | 120 lbs. | 100 lbs. |
| Height of dam | H | H | H |
| Thickness at top | $\cdot 24H$ | $\cdot 25H$ | $\cdot 28H$ |
| Thickness at bottom | $\cdot 48H$ | $\cdot 51H$ | $\cdot 56H$ |
| Water face | Vertical | Vertical | Vertical |
| Outer face | 1 in 4·25 | 1 in 4 | 1 in 3·57 |
| Sectional area | $\cdot 36H^2$ | $\cdot 375H^2$ | $\cdot 42H^2$ |
| Weight per unit of length | $50H^2$ | $45H^2$ | $42H^2$ |
| Mean pressure | $104H$ | $90H$ | $75H$ |
| Maximum pressure | $416H$ | $360H$ | $300H$ |

These data apply to the same limiting value of g , the ratio to the breadth of the base of the distance along it from the foot at which the direction of the resultant pressure cuts, which is taken at one-third. A slight modification of the above section may be used for heights up to 50 feet. For lofty dams with curved profiles, the mode of Delocre, or its modifications, require adaptation to the individual case and conditions in order to obtain the correct dimensions.

Lofty Dams.

The Delocre polygonal section (No. 25) applies to masonry having twice the density of water, or weighing 2 footweight per cubic foot, and capable of resisting a pressure of nearly 200 footweight per square foot. This latter is also assumed to be the limiting pressure allowed on the foundation. The co-efficient of friction for the sliding of the courses on each other is taken at 0.73; the effect of cohesion of the mortar being neglected.

The trace is polygonal on both faces, thus consisting of four rectilinear portions; and is thus a practical approximation to the theoretical double-curved dam without any top-thickness; the flatter curvature being on the water-face, the greater curvature on the rear or outer face. The following are the dimensions in feet.

| | Height. | Breadth. | Front offset. | Rear offset. |
|--------------------|---------|----------|---------------|--------------|
| At the top | 0 | 16.40 | 0 | 5. |
| | 39.36 | 21.38 | 0 | 33.27 |
| | 65.60 | 54.65 | 10.73 | 35.01 |
| | 124.64 | 100.39 | 18.83 | 39.76 |
| Extreme | 164. | 158.95 | | |

The Molesworth corresponding curved section is obtained by ordinates; and is very nearly thus:—

If P = limiting pressure in footweight per square foot

b = top width of dam

a = width of dam at any depth x from the top

x = depth from water surface

y = offset from vertical line to outer face at any depth x

z = offset from vertical line to inner face at any depth x

Then $y = 0.95 \left(\frac{x^3}{P} \right)^{\frac{1}{3}}$; $z = \frac{1}{10} y$; also $b = 0.55 y$; and

$a = 1.1 y$ when $x = \frac{1}{4} H$, the total height of the dam; but no value of y less than $0.6 x$ is admissible.

Also in very lofty dams the value of P should be diminished by substituting for it the term $P(1 - 0.0013 x)$.

Formulae and Data for Retaining Walls.

Extracted from various articles by J. H. E. Hart, C.E.

$$(1) \text{ General equation for breadth of base, } x = \sqrt{\left\{ \frac{nH}{3w(q \pm q^1)} \right\}}$$

Where H = total horizontal pressure against the back of the wall.

n = the ratio of its sectional area to that of a rectangle of equal height and breadth.

w = the weight of a cubic foot of the wall.

qx = the horizontal deviation of the centre of resistance of the base from the middle of the base.

q^1x = the horizontal deviation of the centre of gravity of the profile from the middle of the base.

$$\text{With vertical rectangular sections, } n=1, q^1=0, x = \sqrt{\left(\frac{H}{3wq} \right)}$$

With plumb-faced trapezoidal sections of a top thickness (t)

$$n = \frac{x+t}{2x} \text{ and } q^1 = \left(\frac{t-x}{6} \right) \left(\frac{x+2t}{x(x+t)} \right)$$

$$x = \sqrt{\left\{ \frac{2H - wt}{3w \left(q - \frac{1}{6} \right)} + \left(\frac{t}{2} \right)^2 \right\}} - \frac{t}{2}$$

With plumb-backed trapezoidal sections of a top thickness (t)

$$n = \frac{x+t}{2x} \text{ and } q^1 = \left(\frac{x-t}{6} \right) \left(\frac{x+2t}{x(x+t)} \right)$$

$$x = \sqrt{\left\{ \frac{2H + wt^2}{3w \left(q + \frac{1}{6} \right)} + \left(\frac{t}{2} \right)^2 \right\}} - \frac{t}{2}$$

(2) Modulus. The limiting value of q to avoid tension in the masonry is $\frac{1}{6}$, but its limiting value in actual practice is $\frac{1}{4}$. In special cases, since it must not be so great as to cause the maximum pressure (P) to exceed the safe resistance (C) to crushing of the material, its values correspond as follows to the values of $\frac{P}{p}$, where p = the mean pressure per unit of surface of base, = sum of the vertical forces \div area of the base; and P is less than C .

$$q = \frac{1}{12}, \frac{1}{11}, \frac{1}{10}, \frac{1}{9}, \frac{1}{8}, \frac{1}{7}, \frac{1}{6}; \frac{2}{11}, \frac{1}{5}, \frac{2}{9}, \frac{1}{4}$$

$$\text{when } \frac{P}{p} = \frac{3}{2}, \frac{17}{11}, \frac{8}{5}, \frac{5}{3}, \frac{7}{4}, \frac{13}{7}, 2; \frac{15}{7}, \frac{20}{9}, \frac{12}{5}, \frac{8}{3}.$$

(3) Surcharge. If x = thickness of a vertical rectangular wall to sustain a horizontal-topped bank,

x_1 = do. for an indefinite surcharge,

x = do. for a surcharge of a height c ,

$$x_2 = \frac{hx + 2cx_1}{h + 2c} \text{ where } h = \text{height of the wall.}$$

Additional formulæ for Retaining Walls.

(1) Horizontal thrust (H) for a section whose breadth is unity.

For walls having vertical backs, and for earth with various angles of repose.

Horizontal pressure $H = \text{co-efficient} \times \text{weight of 1 cubic foot of earth} \times h^2$.
 Angles of repose of 27° 30° 33° 36° 39° 42° 45° 48°
 Co-efficients of earth pressure.

| | | | | | | | | | |
|---|---|------|------|------|------|------|------|------|------|
| Earth horizontal at the level of the top | } | .188 | .167 | .147 | .130 | .114 | .099 | .085 | .073 |
| Indefinite Surcharge to angle of repose | } | .397 | .375 | .351 | .327 | .302 | .276 | .250 | .224 |

For walls with sloping backs, having determined the position of the plane of maximum pressure, and hence also the values of ϵ the inclination of that plane with the angle of repose, and A the sectional area of effective pressure, then $H = A \tan \epsilon \times \text{weight of 1 cubic foot of the earth}$.

For water pressure, $H = \frac{1}{2} w_1 h^2 = 31.25 \times h^2$, when $w_1 = 62.5$.

(2) Allowance for limiting resistance to crushing.

Having calculated x , the bottom thickness, in the ordinary way, obtain x_2 , additional bottom thickness necessary, as follows.

Let C = resistance, which is roughly 8 tons per square foot for brickwork and 40 tons per square foot for the heaviest masonry.

W = weight of wall per unit of length, also in tons.

Then for a brickwork wall of height h , and mean thickness t in feet,

$$x_2 = \frac{W}{2C} = \frac{ht}{20 \times 2 \times 8} = \frac{ht}{320} \text{ in feet.}$$

In this case the whole thickness $x = x_1 + x_2 = x_1 \left(1 + \frac{h}{320} \right)$.

The limits of weight of wall are from 80 to 160 lbs. per cubic foot; the mortar 100; granite rubble 140; basalt rubble 150; ashlar from 120 to 170 lbs.

(3) Allowance for the effect of batter in a wall.

Calculate as for a rectangular wall the suitable bottom thickness; but as in overhanging walls the horizontal thrust would be greater, and in reclining walls it would be less, the altered thickness may be obtained by constructing a diagram to scale, and allowing the plumb-face to revolve around a point at one-third of the height. Under that condition the breadth may be scaled; for the horizontal movement of the centre of gravity of the wall is not affected, nor its stability.

Weight of Materials for Dams and Walls.

(By Byrne, in Spon's 'Dictionary of Engineering.')

| | Specific gravity. | | Specific gravity. |
|--------------------------------|-------------------|-----------------------------------|-------------------|
| Clay, dry | 1.95 | Brickwork in new mortar | 1.87 |
| „ wet | 2.17 | „ „ in old mortar | 1.52 |
| Earth, common dry | 1.64 | Cement new | 1.61 |
| Earthy clay and sand | 1.5 to 1.7 | Flint masonry | 2.34 |
| Gravel | 1.5 to 1.9 | Granites | 3.05 to 2.25 |
| Mould, garden | 1.4 | Granite masonry | 2.75 |
| Sand, dry fine | 1.4 to 1.6 | Limestones | 2.54 to 1.86 |
| „ damp | 1.9 | Mortars, new | 1.9 |
| Shingle, loose | 2.2 | „ old | 1.42 |
| Basalts and traps | 3 to 2.4 | Sandstones | 2.67 to 1.38 |
| Bricks, red | 2.16 | Slates | 2.9 to 2.5 |
| „ common | 1.76 | | |
| „ stock (London) | 1.84 | | |
| Brickwork in cement | 1.92 | | |

NOTE.—Ashlar, weight = $\frac{7}{8}$ that of stone + $\frac{1}{8}$ that of mortar.Rubble, weight = $\frac{3}{4}$ to $\frac{2}{3}$ that of stone + $\frac{1}{4}$ to $\frac{1}{3}$ that of mortar.*Working Loads or safe units of pressure adopted in existing structures.*

(By Byrne, in Spon's 'Dictionary of Engineering.')

| | Tons on the square foot. |
|---|----------------------------------|
| Soft rock foundations | 9 |
| Concrete in lime mortar | 3 |
| Earth | $1\frac{1}{4}$ |
| Ashlar masonry, limestone, Britannia Bridge | 16 |
| „ „ granite, Saltash Bridge | 10 |
| „ backed with rubble, Peniston Viaduct | 6 |
| Rubble masonry, sandstone in Aberthaw lime, Pont y Pridd | $20\frac{3}{4}$ |
| „ „ limestone in chalk lime, Barentine Viaduct | $3\frac{1}{2}$ |
| „ „ in hydraulic lime, Almanza Dam | 12.8 |
| „ „ „ „ Ban | 7.3 |
| „ „ „ „ Furens | 6 |
| „ „ „ „ Tulsi | 8.9 to 6.9 |
| Brickwork, London paviers' in cement, Charing Cross Bridge | 12 |
| „ Staffordshire blue brick in cement, Clifton Suspension Bridge | 10 |
| „ red Birmingham in lias lime, Railway Viaduct | 7 |
| Cement mortar | 20 to 32 |
| Lime mortar | $2\frac{1}{2}$ to $5\frac{1}{4}$ |

NOTE.—The safe working load for masonry and brickwork is that for the mortar used; but in ordinary calculation, 5 tons per square foot for brickwork and rubble in lime, and 30 for ashlar in cement, is generally allowed.

Proportions of Sections of Ovoid Culverts.

(By the Author.)

| | Phillips | Hawksley | Pegtop |
|--|-------------|----------|--------|
| Transverse diameter or } extreme inside width } | 2 | 2 | 2 |
| Radius of top circle . | 1 | 1 | 1 |
| Total vertical depth . | 3 | 2.5858 | 3 |
| Radius of curved side . | 3 | 2 | x |
| Radius of invert . . . | 0.5 | 0.5858 | 0.375 |
| Length of side, or arc . | 36° 52' 14" | 45° | 1.5 |
| Arc of top circle . . . | 180° | 180° | 220° |
| Arc of invert | 106° 16' | 90° | 140° |
| Area of Full Section . | 4.594 | 3.9820 | 4.1542 |
| Area, filled to $\frac{2}{3}$ depth . | 3.023 | 2.6858 | 2.5834 |
| Area, filled to $\frac{1}{3}$ depth . | 1.136 | 1.0278 | 0.9687 |
| Perimeter of Full Section | 7.930 | 7.2034 | 7.7560 |
| ,, filled to $\frac{2}{3}$ depth | 4.788 | 4.3375 | 4.6144 |
| ,, filled to $\frac{1}{3}$ depth | 2.750 | 2.5957 | 2.5413 |
| Hyd. Rad. for Full Section | 0.579 | 0.553 | 0.536 |
| ,, filled to $\frac{2}{3}$ depth | 0.631 | 0.620 | 0.560 |
| ,, filled to $\frac{1}{3}$ depth | 0.413 | 0.396 | 0.381 |

The above comparison is based on an equal transverse diameter for each form of culvert.

If the culverts are assumed to be of equal section when completely filled, the relative diameters for the different forms of culvert are thus—

| | |
|-----------------------------------|-------------------|
| Cylindrical Section | 1.1286 |
| Phillips's Metropolitan | 1.0002 and 1.2930 |
| Hawksley's Ovoid | 0.9331 and 1.3996 |
| Jackson's Pegtop | 0.9813 and 1.4720 |

The Pegtop section flushes highest with the same quantity of liquid ; but its sides must be of slightly increased thickness, when subject to much lateral pressure.

*Cast Iron Water-pipes ; adopted in the Rio de Janeiro
Waterworks.*

| Diameter of pipe | | Thickness | Length without socket | Socket | Weight without ring or socket | | Total weight with ring and socket | | | |
|------------------|-----------------------|-------------------|-----------------------|-------------------|-------------------------------|------|-----------------------------------|-------|------|------|
| m. | inches | in. | feet | inches | cwts. | qrs. | lbs. | cwts. | qrs. | lbs. |
| 0·80 | or 31 $\frac{1}{2}$ | 1 $\frac{3}{16}$ | 12 | 5 $\frac{14}{16}$ | 40 | 2 | 4 | 43 | 3 | 23 |
| 0·80 | or 31 $\frac{1}{2}$ | 1 $\frac{3}{16}$ | 9 | 5 $\frac{14}{16}$ | 30 | 1 | 17 | 33 | 3 | 8 |
| 0·50 | or 19 $\frac{11}{16}$ | 1 | 12 | 5 $\frac{2}{16}$ | 21 | 1 | 13 | 22 | 3 | 27 |
| 0·50 | or 19 $\frac{11}{16}$ | 1 | 9 | 5 $\frac{2}{16}$ | 16 | 0 | 3 | 17 | 2 | 17 |
| 0·40 | or 15 $\frac{12}{16}$ | 1 $\frac{14}{16}$ | 12 | 5 $\frac{2}{16}$ | 15 | 0 | 13 | 16 | 1 | 14 |
| 0·40 | or 15 $\frac{12}{16}$ | 1 $\frac{14}{16}$ | 9 | 5 $\frac{2}{16}$ | 11 | 1 | 10 | 12 | 2 | 11 |
| 0·30 | or 11 $\frac{13}{16}$ | 1 $\frac{13}{16}$ | 9 | 4 $\frac{15}{16}$ | 7 | 3 | 7 | 8 | 2 | 15 |
| 0·30 | or 11 $\frac{13}{16}$ | 1 $\frac{11}{16}$ | 9 | 4 $\frac{15}{16}$ | 6 | 3 | 27 | 7 | 2 | 27 |
| 0·25 | or 9 $\frac{13}{16}$ | 1 $\frac{10}{16}$ | 9 | 4 $\frac{15}{16}$ | 5 | 0 | 23 | 5 | 3 | 5 |
| 0·20 | or 7 $\frac{14}{16}$ | 1 $\frac{9}{16}$ | 9 | 4 $\frac{5}{16}$ | 3 | 2 | 18 | 4 | 0 | 6 |
| 0·15 | or 5 $\frac{14}{16}$ | 1 $\frac{8}{16}$ | 9 | 4 $\frac{5}{16}$ | 2 | 2 | 10 | 2 | 3 | 14 |
| 0·10 | or 3 $\frac{14}{16}$ | 1 $\frac{8}{16}$ | 9 | 4 $\frac{5}{16}$ | 1 | 2 | 16 | 1 | 3 | 10 |

Testing pressure 15 atmospheres ; for 31 $\frac{1}{2}$ " pipes 20 atmospheres ; specific gravity of iron taken at 7·20.

Cast Iron Water-pipes adopted at Glasgow.

| Length | Thick-ness | Weight incl. Socket | | Working head | Length | Thick-ness | Weight incl. Socket | | Working head | | |
|--------|-------------------|---------------------|------|--------------|--------|------------|---------------------|------|--------------|------|-----|
| | | cwts. | qrs. | lbs. | feet | | cwts. | qrs. | lbs. | feet | |
| 33" | 1" | 39 | 1 | 25 | 210 | 14 | 11 $\frac{1}{16}$ | 8 | 3 | 25 | 290 |
| 30 | 1 $\frac{1}{4}$ | 44 | 0 | 3 | 300 | 14 | 10 $\frac{10}{16}$ | 8 | 0 | 25 | 250 |
| 30 | 1 | 35 | 3 | 5 | 230 | 14 | 9 $\frac{9}{16}$ | 7 | 2 | 0 | 200 |
| 24 | 1 | 28 | 1 | 23 | 300 | 12 | 5 $\frac{5}{8}$ | 6 | 3 | 13 | 290 |
| 20 | 7 $\frac{7}{8}$ | 16 | 0 | 4 | 270 | 12 | 9 $\frac{9}{16}$ | 6 | 0 | 26 | 240 |
| 20 | 3 $\frac{3}{4}$ | 13 | 3 | 25 | 240 | 10 | 9 $\frac{9}{16}$ | 5 | 0 | 16 | 300 |
| 18 | 1 $\frac{13}{16}$ | 13 | 1 | 12 | 300 | 9 | 9 $\frac{9}{16}$ | 4 | 2 | 24 | " |
| 18 | 3 $\frac{3}{4}$ | 12 | 1 | 19 | 260 | 8 | 1 $\frac{1}{2}$ | 3 | 2 | 23 | " |
| 18 | 1 $\frac{11}{16}$ | 11 | 1 | 27 | 230 | 7 | 1 $\frac{1}{2}$ | 3 | 1 | 1 | " |
| 16 | 3 $\frac{3}{4}$ | 10 | 3 | 27 | 300 | 6 | 7 $\frac{7}{16}$ | 2 | 1 | 27 | " |
| 16 | 1 $\frac{11}{16}$ | 10 | 0 | 18 | 250 | 5 | 1 $\frac{13}{32}$ | 1 | 3 | 24 | " |
| 16 | 5 $\frac{5}{8}$ | 9 | 1 | 9 | 200 | 4 | 3 $\frac{3}{8}$ | 1 | 1 | 20 | " |
| 15 | 1 $\frac{11}{16}$ | 9 | 2 | 3 | 270 | 3 | 3 $\frac{3}{8}$ | 1 | 0 | 10 | " |
| 15 | 9 $\frac{9}{16}$ | 7 | 3 | 25 | 180 | 2 | 3 $\frac{3}{8}$ | 0 | 2 | 4 | 300 |

Testing strain double the working pressure.

The lengths are 9 feet excluding socket ; but for 24" pipes and upwards the length is 12 feet ; and for 2" pipe 6 feet.

Absorption and Strength of Cylindrical Stoneware Pipes.

(By Baldwin Latham, C.E.)

| Maker and place | Diam. | Thick- ness " | Length | Weight when dry lbs. | Weight after 24 hours' im- mersion lbs. | Perce- ntage of absorp- tion |
|-------------------------|-------|---------------------|--------|----------------------------|---|---------------------------------------|
| Doulton . . . | 6" | 0.75 | 1' 11" | 31 | 31.25 | 0.806 |
| Doulton, London . . . | | 0.72 | 1 11 | 29.5 | 29.75 | 0.85 |
| Fisher . . . | | 0.63 | 2 0 | 28 | 28.75 | 2.68 |
| Wortley . . . | | 0.74 | 1 11 | 30.5 | 31.75 | 4.10 |
| Doulton, London . . . | 9" | 0.87 | 2 0 | 57.75 | 58.75 | 1.73 |
| Huddersfield . . . | | 0.92 | 2 4 | 73 | 73.75 | 1.03 |
| Cliff, Wortley . . . | | 0.81 | 2 4 | 60.5 | 63.25 | 4.54 |
| Aylesford . . . | 12" | 1.00 | 2 0 | 58 | 62 | 6.89 |
| Doulton, Stafford . . . | | 1.05 | 2 0 | 96.0 | 97.5 | 1.56 |
| Fisher . . . | | 1 | 1 11 | 84 | 88 | 4.76 |
| Stiff . . . | | 1.02 | 1 10 | 66.25 | 67.5 | 1.88 |
| Wilcox, Wortley . . . | | 1.03 | 1 11 | 79.5 | 82.5 | 3.77 |
| Doulton, London . . . | | 1.06 | 1 11 | 116.5 | 117.0 | 0.43 |
| Doulton, Stafford . . . | 15" | 1.26 | 2 6 | 132 | 139 | 5.30 |
| Wilcox . . . | | 1.72 | 1 10 | 130 | 137 | 5.38 |
| Ingham . . . | | .31 | 2 6 | 165 | 174.5 | 5.75 |
| Doulton . . . | 18" | 1.43 | 2 4 | 221 | 226 | 2.26 |
| „ . . . | | 1.38 | 2 5 | 210 | 217 | 3.33 |

| | Diam. | Thick- ness " | Length ' " | Bursting pressure B. | Tensile strength T. | Resistance to crushing C. |
|-------------------------|-------|---------------------|---------------|----------------------------|---------------------------|---------------------------------|
| Doulton, Stafford . . . | 6" | 0.65 | 1 11 | 50 | 230.7 | 1742 |
| „ London . . . | | 0.72 | 1 11 | 10 | 41.6 | |
| Ingham, Wortley . . . | | 0.48 | 1 11 | 4 | 25 | 2956 |
| „ „ . . . | | 0.69 | 1 11 | 70 | 304.3 | |
| Doulton, London . . . | 9" | 0.84 | 2 0 | 40 | 214.2 | 2470 |
| „ Stafford . . . | | 0.79 | 1 11 | 20 | 113.9 | |
| Aylesford . . . | | 1.00 | 2 0 | 45 | 202.5 | 3561 |
| Cliff, Wortley . . . | 0.84 | 2 4 | 60 | 321.4 | | |
| Doulton, Stafford . . . | 12" | 1.07 | 2 0 | 7 | 39.2 | 2834 |
| Wilcox, Wortley . . . | | 0.94 | 1 11 | 7 | 44.6 | |
| Doulton, London . . . | 15" | 1.19 | 2 5 | 33 | 207.9 | Not tested |
| „ Stafford . . . | | 1.10 | 1 10 | 20 | 136.3 | |
| Ingham, Wortley . . . | | 1.15 | 2 5 | 20 | 130.4 | |
| Seacombe, Ruabon . . . | | 1.10 | 1 10 | 63 | 429.5 | |

B, T, and C are all in lbs. per sq. inch.

*Arcs of Circles, having a Diameter=1;
or Areas of Sectors of Circles, having a Radius=1.*

| Deg. | Arc or Sector | Deg. | Arc or Sector | Deg. | Arc or Sector | Deg. | Arc or Sector | Deg. | Arc or Sector |
|------|---------------|------|---------------|------|---------------|------|---------------|------|---------------|
| 1 | ·00873 | 31 | ·27053 | 61 | ·53233 | 91 | ·79412 | 121 | 1·05592 |
| 2 | ·01745 | 32 | ·27925 | 62 | ·54105 | 92 | ·80206 | 122 | 1·06465 |
| 3 | ·02618 | 33 | ·28798 | 63 | ·54978 | 93 | ·81158 | 123 | 1·07338 |
| 4 | ·03491 | 34 | ·29671 | 64 | ·55851 | 94 | ·82030 | 124 | 1·08210 |
| 5 | ·04363 | 35 | ·30543 | 65 | ·56723 | 95 | ·82903 | 125 | 1·09083 |
| 6 | ·05236 | 36 | ·31416 | 66 | ·57596 | 96 | ·83776 | 126 | 1·09956 |
| 7 | ·06109 | 37 | ·32289 | 67 | ·58469 | 97 | ·84648 | 127 | 1·10828 |
| 8 | ·06981 | 38 | ·33161 | 68 | ·59341 | 98 | ·85521 | 128 | 1·11701 |
| 9 | ·07854 | 39 | ·34034 | 69 | ·60214 | 99 | ·86394 | 129 | 1·12574 |
| 10 | ·08727 | 40 | ·34907 | 70 | ·61087 | 100 | ·87266 | 130 | 1·13446 |
| 11 | ·09599 | 41 | ·35779 | 71 | ·61959 | 101 | ·88139 | 131 | 1·14319 |
| 12 | ·10472 | 42 | ·36652 | 72 | ·62832 | 102 | ·89012 | 132 | 1·15192 |
| 13 | ·11345 | 43 | ·37525 | 73 | ·63705 | 103 | ·89884 | 133 | 1·16064 |
| 14 | ·12217 | 44 | ·38397 | 74 | ·64577 | 104 | ·90757 | 134 | 1·16937 |
| 15 | ·13090 | 45 | ·39270 | 75 | ·65450 | 105 | ·91630 | 135 | 1·17810 |
| 16 | ·13963 | 46 | ·40143 | 76 | ·66323 | 106 | ·92502 | 136 | 1·18682 |
| 17 | ·14835 | 47 | ·41015 | 77 | ·67195 | 107 | ·93375 | 137 | 1·19555 |
| 18 | ·15708 | 48 | ·41888 | 78 | ·68068 | 108 | ·94248 | 138 | 1·20428 |
| 19 | ·16581 | 49 | ·42761 | 79 | ·68941 | 109 | ·95120 | 139 | 1·21300 |
| 20 | ·17453 | 50 | ·43633 | 80 | ·69813 | 110 | ·95993 | 140 | 1·22173 |
| 21 | ·18326 | 51 | ·44506 | 81 | ·70686 | 111 | ·96866 | 141 | 1·23046 |
| 22 | ·19199 | 52 | ·45379 | 82 | ·71559 | 112 | ·97738 | 142 | 1·23918 |
| 23 | ·20071 | 53 | ·46251 | 83 | ·72431 | 113 | ·98611 | 143 | 1·24791 |
| 24 | ·20944 | 54 | ·47124 | 84 | ·73304 | 114 | ·99484 | 144 | 1·25664 |
| 25 | ·21817 | 55 | ·47997 | 85 | ·74176 | 115 | 1·00356 | 145 | 1·26536 |
| 26 | ·22689 | 56 | ·48869 | 86 | ·75049 | 116 | 1·01229 | 146 | 1·27409 |
| 27 | ·23562 | 57 | ·49742 | 87 | ·75922 | 117 | 1·02102 | 147 | 1·28282 |
| 28 | ·24435 | 58 | ·50615 | 88 | ·76794 | 118 | 1·02974 | 148 | 1·29154 |
| 29 | ·25307 | 59 | ·51487 | 89 | ·77667 | 119 | 1·03847 | 149 | 1·30027 |
| 30 | ·26180 | 60 | ·52360 | 90 | ·78540 | 120 | 1·04720 | 150 | 1·30900 |

*Arcs of Circles, having a Diameter=1;
or Areas of Sectors of Circles, having a Radius=1.*

| Deg. | Arc or Sector | Min. | Arc or Sector | Min. | Arc or Sector | Sec. | Arc or Sector | Sec. | Arc or Sector |
|------|---------------|------|---------------|------|---------------|------|---------------|------|---------------|
| 151 | 1°31'772 | 1 | '00015 | 31 | '00451 | 1 | '000 002 | 31 | '000 075 |
| 152 | 1°32'645 | 2 | '00029 | 32 | '00465 | 2 | '000 005 | 32 | '000 078 |
| 153 | 1°33'518 | 3 | '00044 | 33 | '00480 | 3 | '000 007 | 33 | '000 080 |
| 154 | 1°34'390 | 4 | '00058 | 34 | '00494 | 4 | '000 010 | 34 | '000 082 |
| 155 | 1°35'263 | 5 | '00078 | 35 | '00509 | 5 | '000 012 | 35 | '000 085 |
| 156 | 1°36'136 | 6 | '00087 | 36 | '00524 | 6 | '000 015 | 36 | '000 087 |
| 157 | 1°37'008 | 7 | '00102 | 37 | '00538 | 7 | '000 017 | 37 | '000 090 |
| 158 | 1°37'881 | 8 | '00116 | 38 | '00553 | 8 | '000 019 | 38 | '000 092 |
| 159 | 1°38'754 | 9 | '00131 | 39 | '00567 | 9 | '000 022 | 39 | '000 095 |
| 160 | 1°39'626 | 10 | '00145 | 40 | '00582 | 10 | '000 024 | 40 | '000 097 |
| 161 | 1°40'499 | 11 | '00160 | 41 | '00596 | 11 | '000 026 | 41 | '000 099 |
| 162 | 1°41'372 | 12 | '00175 | 42 | '00611 | 12 | '000 029 | 42 | '000 102 |
| 163 | 1°42'244 | 13 | '00189 | 43 | '00625 | 13 | '000 031 | 43 | '000 104 |
| 164 | 1°43'117 | 14 | '00204 | 44 | '00640 | 14 | '000 034 | 44 | '000 107 |
| 165 | 1°43'990 | 15 | '00218 | 45 | '00655 | 15 | '000 036 | 45 | '000 109 |
| 166 | 1°44'862 | 16 | '00233 | 46 | '00669 | 16 | '000 039 | 46 | '000 112 |
| 167 | 1°45'735 | 17 | '00247 | 47 | '00684 | 17 | '000 041 | 47 | '000 114 |
| 168 | 1°46'608 | 18 | '00262 | 48 | '00698 | 18 | '000 044 | 48 | '000 116 |
| 169 | 1°47'480 | 19 | '00276 | 49 | '00713 | 19 | '000 046 | 49 | '000 119 |
| 170 | 1°48'353 | 20 | '00291 | 50 | '00727 | 20 | '000 049 | 50 | '000 121 |
| 171 | 1°49'226 | 21 | '00305 | 51 | '00742 | 21 | '000 051 | 51 | '000 124 |
| 172 | 1°50'098 | 22 | '00320 | 52 | '00756 | 22 | '000 053 | 52 | '000 126 |
| 173 | 1°50'971 | 23 | '00335 | 53 | '00771 | 23 | '000 056 | 53 | '000 129 |
| 174 | 1°51'844 | 24 | '00349 | 54 | '00785 | 24 | '000 058 | 54 | '000 131 |
| 175 | 1°52'716 | 25 | '00364 | 55 | '00800 | 25 | '000 061 | 55 | '000 133 |
| 176 | 1°53'589 | 26 | '00378 | 56 | '00814 | 26 | '000 063 | 56 | '000 136 |
| 177 | 1°54'462 | 27 | '00393 | 57 | '00829 | 27 | '000 065 | 57 | '000 138 |
| 178 | 1°55'334 | 28 | '00407 | 58 | '00844 | 28 | '000 068 | 58 | '000 141 |
| 179 | 1°56'207 | 29 | '00422 | 59 | '00858 | 29 | '000 070 | 59 | '000 143 |
| 180 | 1°57'080 | 30 | '00436 | 60 | '00873 | 30 | '000 073 | 60 | '000 145 |

Powers, Roots, and Reciprocals.

| Number | Square | Square Root | Cube Root | Fifth Root | Power of $\frac{1}{2}$ | Power of $\frac{1}{3}$ | Reciprocal |
|--------|--------|-------------|-----------|------------|------------------------|------------------------|------------|
| 0.01 | .0001 | .1 | .2154 | .3981 | .00001 | .1584 | 100. |
| 0.015 | .0002 | .1225 | .2466 | .4317 | .00003 | .1864 | 66.667 |
| 0.02 | .0004 | .1414 | .2714 | .4573 | .00006 | .2140 | 50. |
| 0.025 | .0006 | .1581 | .2924 | .4782 | .00010 | .2287 | 40. |
| 0.03 | .0009 | .1732 | .3107 | .4959 | .00017 | .2460 | 33.333 |
| 0.035 | .0012 | .1871 | .3271 | .5115 | .00023 | .2616 | 28.571 |
| 0.04 | .0016 | .2 | .3420 | .5253 | .00032 | .2759 | 25. |
| 0.045 | .0020 | .2121 | .3557 | .5378 | .00043 | .2893 | 22.222 |
| 0.05 | .0025 | .2236 | .3684 | .5493 | .00056 | .3017 | 20. |
| 0.055 | .0030 | .2345 | .3803 | .5599 | .00071 | .3134 | 18.182 |
| 0.06 | .0036 | .2449 | .3915 | .5697 | .00088 | .3245 | 16.667 |
| 0.065 | .0042 | .2550 | .4021 | .5789 | .00108 | .3351 | 15.385 |
| 0.07 | .0049 | .2646 | .4121 | .5875 | .00130 | .3452 | 14.286 |
| 0.075 | .0056 | .2739 | .4217 | .5957 | .00154 | .3548 | 13.333 |
| 0.08 | .0064 | .2828 | .4309 | .6034 | .00181 | .3641 | 12.5 |
| 0.085 | .0072 | .2915 | .4397 | .6108 | .00211 | .3731 | 11.764 |
| 0.09 | .0081 | .3 | .4481 | .6178 | .00243 | .3817 | 11.111 |
| 0.095 | .0090 | .3082 | .4563 | .6245 | .00278 | .3900 | 10.526 |
| 0.1 | .01 | .3162 | .4642 | .6310 | .0032 | .3981 | 10. |
| 0.15 | .0225 | .3873 | .5313 | .6843 | .0087 | .4682 | 6.6667 |
| 0.2 | .04 | .4472 | .5848 | .7248 | .0179 | .5253 | 5. |
| 0.25 | .0625 | .5 | .6300 | .7579 | .0313 | .5743 | 4. |
| 0.3 | .09 | .5477 | .6694 | .7860 | .0493 | .6178 | 3.3333 |
| 0.35 | .1225 | .5916 | .7047 | .8106 | .0769 | .6571 | 2.8571 |
| 0.4 | .1600 | .6324 | .7368 | .8326 | .1012 | .6931 | 2.5 |
| 0.45 | .2025 | .6708 | .7663 | .8524 | .1358 | .7266 | 2.2222 |
| 0.5 | .25 | .7071 | .7937 | .8706 | .1768 | .7579 | 2. |
| 0.52 | .2704 | .7211 | .8041 | .8774 | .1950 | .7698 | 1.9231 |
| 0.54 | .2916 | .7348 | .8143 | .8841 | .2143 | .7816 | 1.8519 |
| 0.56 | .3136 | .7483 | .8243 | .8905 | .2347 | .7930 | 1.7857 |
| 0.58 | .3364 | .7616 | .8340 | .8968 | .2562 | .8042 | 1.7241 |
| 0.6 | .36 | .7746 | .8434 | .9029 | .2788 | .8152 | 1.6667 |
| 0.62 | .3844 | .7874 | .8527 | .9088 | .3027 | .8260 | 1.6129 |
| 0.64 | .4096 | .8 | .8618 | .9146 | .3277 | .8365 | 1.5625 |
| 0.66 | .4356 | .8124 | .8707 | .9203 | .3539 | .8469 | 1.5152 |
| 0.68 | .4624 | .8246 | .8794 | .9258 | .3813 | .8570 | 1.4706 |
| 0.7 | .49 | .8366 | .8879 | .9312 | .4100 | .8670 | 1.4286 |
| 0.72 | .5184 | .8485 | .8963 | .9364 | .4399 | .8769 | 1.3889 |
| 0.74 | .5476 | .8602 | .9045 | .9416 | .4711 | .8865 | 1.3514 |
| 0.76 | .5776 | .8718 | .9126 | .9466 | .5036 | .8961 | 1.3158 |
| 0.78 | .6084 | .8832 | .9205 | .9515 | .5373 | .9054 | 1.2821 |
| 0.8 | .64 | .8944 | .9283 | .9564 | .5724 | .9146 | 1.25 |

Powers, Roots, and Reciprocals.

| Number | Square | Square Root | Cube Root | Fifth Root | Power of $\frac{1}{3}$ | Power of $\frac{2}{3}$ | Reciprocal |
|--------|--------|-------------|-----------|------------|------------------------|------------------------|------------|
| 0·82 | ·6724 | ·9055 | ·9360 | ·9611 | ·6089 | ·9237 | 1·2195 |
| 0·84 | ·7056 | ·9165 | ·9435 | ·9657 | ·6467 | ·9327 | 1·1905 |
| 0·86 | ·7396 | ·9274 | ·9510 | ·9702 | ·6859 | ·9415 | 1·1628 |
| 0·88 | ·7744 | ·9381 | ·9583 | ·9748 | ·7265 | ·9502 | 1·1364 |
| 0·9 | ·81 | ·9487 | ·9555 | ·9791 | ·7684 | ·9587 | 1·1111 |
| 0·92 | ·8464 | ·9592 | ·9726 | ·9834 | ·8118 | ·9672 | 1·0870 |
| 0·94 | ·8836 | ·9695 | ·9796 | ·9877 | ·8567 | ·9756 | 1·0638 |
| 0·96 | ·9216 | ·9798 | ·9865 | ·9918 | ·9030 | ·9838 | 1·0427 |
| 0·98 | ·9604 | ·9899 | ·9933 | ·9960 | ·9507 | ·9920 | 1·0204 |
| 1· | 1· | 1· | 1· | 1· | 1· | 1· | 1· |
| 1·02 | 1·0404 | 1·0099 | 1·0065 | 1·0040 | 1·0508 | 1·0080 | 0·98039 |
| 1·04 | 1·0816 | 1·0198 | 1·0132 | 1·0079 | 1·1030 | 1·0158 | 0·96154 |
| 1·06 | 1·1236 | 1·0296 | 1·0196 | 1·0117 | 1·1569 | 1·0236 | 0·94340 |
| 1·08 | 1·1664 | 1·0392 | 1·0260 | 1·0155 | 1·2122 | 1·0313 | 0·92593 |
| 1·1 | 1·2100 | 1·0488 | 1·0323 | 1·0192 | 1·2691 | 1·0389 | 0·90909 |
| 1·12 | 1·2544 | 1·0583 | 1·0385 | 1·0229 | 1·3275 | 1·0464 | 0·89286 |
| 1·14 | 1·2996 | 1·0677 | 1·0446 | 1·0266 | 1·3876 | 1·0538 | 0·87719 |
| 1·16 | 1·3456 | 1·0770 | 1·0507 | 1·0301 | 1·4492 | 1·0612 | 0·86207 |
| 1·18 | 1·3924 | 1·0863 | 1·0567 | 1·0337 | 1·5126 | 1·0685 | 0·84746 |
| 1·2 | 1·4400 | 1·0954 | 1·0627 | 1·0371 | 1·5775 | 1·0757 | 0·83333 |
| 1·22 | 1·4884 | 1·1045 | 1·0685 | 1·0405 | 1·6440 | 1·0827 | 0·81967 |
| 1·24 | 1·5376 | 1·1136 | 1·0743 | 1·0440 | 1·7122 | 1·0899 | 0·80645 |
| 1·26 | 1·5876 | 1·1225 | 1·0801 | 1·0473 | 1·7821 | 1·0969 | 0·79365 |
| 1·28 | 1·6384 | 1·1314 | 1·0858 | 1·0506 | 1·8536 | 1·1038 | 0·78125 |
| 1·3 | 1·6900 | 1·1402 | 1·0914 | 1·0539 | 1·9269 | 1·1107 | 0·76923 |
| 1·32 | 1·7424 | 1·1489 | 1·0970 | 1·0571 | 2·0018 | 1·1175 | 0·75758 |
| 1·34 | 1·7956 | 1·1576 | 1·1025 | 1·0603 | 2·0786 | 1·1242 | 0·74627 |
| 1·36 | 1·8496 | 1·1619 | 1·1079 | 1·0634 | 2·1570 | 1·1309 | 0·73529 |
| 1·38 | 1·9044 | 1·1747 | 1·1133 | 1·0665 | 2·2372 | 1·1375 | 0·72464 |
| 1·4 | 1·96 | 1·1832 | 1·1187 | 1·0696 | 2·3196 | 1·1442 | 0·71429 |
| 1·42 | 2·0164 | 1·1916 | 1·1243 | 1·0734 | 2·4028 | 1·1522 | 0·70423 |
| 1·44 | 2·0736 | 1·2 | 1·1292 | 1·0757 | 2·4883 | 1·1570 | 0·69444 |
| 1·46 | 2·1316 | 1·2083 | 1·1344 | 1·0786 | 2·5756 | 1·1634 | 0·68493 |
| 1·48 | 2·1904 | 1·2166 | 1·1396 | 1·0816 | 2·6648 | 1·1698 | 0·67568 |
| 1·5 | 2·25 | 1·2247 | 1·1447 | 1·0845 | 2·7556 | 1·1761 | 0·66667 |
| 1·55 | 2·4025 | 1·2450 | 1·1573 | 1·0916 | 2·9911 | 1·1916 | 0·64516 |
| 1·6 | 2·56 | 1·2649 | 1·1696 | 1·0986 | 3·2382 | 1·2068 | 0·625 |
| 1·65 | 2·7225 | 1·2845 | 1·1817 | 1·1053 | 3·4971 | 2·2218 | 0·60606 |
| 1·7 | 2·89 | 1·3038 | 1·1935 | 1·1120 | 3·7681 | 1·2365 | 0·58824 |
| 1·75 | 3·0625 | 1·3229 | 1·2051 | 1·1184 | 4·0513 | 1·2509 | 0·57143 |
| 1·8 | 3·24 | 1·3416 | 1·2164 | 1·1247 | 4·3469 | 1·2651 | 0·55556 |
| 1·85 | 3·4225 | 1·3601 | 1·2276 | 1·1309 | 4·6551 | 1·2790 | 0·54054 |
| 1·9 | 3·61 | 1·3784 | 1·2386 | 1·1370 | 4·9760 | 1·2927 | 0·52632 |

Powers, Roots, and Reciprocals.

| Number | Square | Square Root | Cube Root | Fifth Root | Power of $\frac{1}{3}$ | Power of $\frac{2}{3}$ | Reciprocal |
|--------|---------|-------------|-----------|------------|------------------------|------------------------|------------|
| 1.95 | 3.8025 | 1.3964 | 1.2493 | 1.1429 | 5.3098 | 1.3062 | 0.51282 |
| 2 | 4 | 1.4142 | 1.2599 | 1.1487 | 5.6569 | 1.3195 | 0.5 |
| 2.1 | 4.41 | 1.4491 | 1.2806 | 1.1600 | 6.3834 | 1.3455 | 0.47619 |
| 2.2 | 4.84 | 1.4832 | 1.3006 | 1.1708 | 7.1790 | 1.3708 | 0.45455 |
| 2.3 | 5.29 | 1.5166 | 1.3200 | 1.1813 | 8.0227 | 1.3954 | 0.43478 |
| 2.4 | 5.76 | 1.5492 | 1.3389 | 1.1914 | 8.9214 | 1.4194 | 0.41667 |
| 2.5 | 6.25 | 1.5811 | 1.3572 | 1.2011 | 9.8823 | 1.4427 | 0.4 |
| 2.75 | 7.5625 | 1.6583 | 1.4010 | 1.2242 | 12.541 | 1.4988 | 0.36364 |
| 3 | 9 | 1.7321 | 1.4423 | 1.2457 | 15.589 | 1.5518 | 0.33333 |
| 3.25 | 10.5625 | 1.8028 | 1.4812 | 1.2658 | 19.041 | 1.6023 | 0.30769 |
| 3.5 | 12.25 | 1.8708 | 1.5183 | 1.2846 | 22.918 | 1.6505 | 0.28571 |
| 3.75 | 14.0625 | 1.9365 | 1.5536 | 1.3026 | 27.232 | 1.6967 | 0.26667 |
| 4 | 16 | 2 | 1.5874 | 1.3195 | 32 | 1.7411 | 0.25 |
| 4.25 | 18.0625 | 2.0616 | 1.6198 | 1.3356 | 37.2361 | 1.7838 | 0.23529 |
| 4.5 | 20.25 | 2.1213 | 1.6510 | 1.3510 | 42.9561 | 1.8251 | 0.22222 |
| 4.75 | 22.5625 | 2.1794 | 1.6810 | 1.3656 | 49.1731 | 1.8650 | 0.21053 |
| 5 | 25 | 2.2361 | 1.7099 | 1.3804 | 55.9010 | 1.9054 | 0.2 |
| 5.25 | 27.5625 | 2.2913 | 1.7380 | 1.3933 | 63.154 | 1.9414 | 0.19048 |
| 5.5 | 30.25 | 2.3452 | 1.7652 | 1.4063 | 70.943 | 1.9776 | 0.18182 |
| 5.75 | 33.0625 | 2.3979 | 1.7915 | 1.4189 | 79.283 | 2.0133 | 0.17391 |
| 6 | 36 | 2.4495 | 1.8171 | 1.4310 | 88.176 | 2.0477 | 0.16667 |
| 6.25 | 39.0625 | 2.5 | 1.8420 | 1.4427 | 97.657 | 2.0814 | 0.16 |
| 6.5 | 42.25 | 2.5495 | 1.8663 | 1.4541 | 107.71 | 2.1143 | 0.15385 |
| 6.75 | 45.5625 | 2.5981 | 1.8899 | 1.4651 | 118.38 | 2.1465 | 0.14815 |
| 7 | 49 | 2.6458 | 1.9129 | 1.4758 | 129.64 | 2.1779 | 0.14286 |
| 7.25 | 52.5625 | 2.6926 | 1.9354 | 1.4862 | 141.53 | 2.2087 | 0.13793 |
| 7.5 | 56.25 | 2.7386 | 1.9574 | 1.4963 | 154.04 | 2.2388 | 0.13333 |
| 7.75 | 60.0625 | 2.7839 | 1.9789 | 1.5061 | 167.21 | 2.2684 | 0.12903 |
| 8 | 64 | 2.8284 | 2 | 1.5157 | 181.01 | 2.2974 | 0.125 |
| 8.25 | 68.0625 | 2.8723 | 2.0206 | 1.5251 | 195.49 | 2.3258 | 0.12121 |
| 8.5 | 72.25 | 2.9155 | 2.0408 | 1.5342 | 214.64 | 2.3538 | 0.11765 |
| 8.75 | 76.5625 | 2.9580 | 2.0606 | 1.5431 | 226.48 | 2.3812 | 0.11429 |
| 9 | 81 | 3 | 2.0801 | 1.5518 | 243 | 2.4082 | 0.11111 |
| 9.5 | 90.25 | 3.0822 | 2.1179 | 1.5687 | 278.16 | 2.4609 | 0.10526 |
| 10 | 100 | 3.1623 | 2.1544 | 1.5849 | 316.23 | 2.5119 | 0.1 |
| 11 | 121 | 3.3166 | 2.2239 | 1.6154 | 401.31 | 2.6095 | 0.090909 |
| 12 | 144 | 3.4641 | 2.2894 | 1.6437 | 498.83 | 2.7019 | 0.083333 |
| 13 | 169 | 3.6056 | 2.3513 | 1.6702 | 609.34 | 2.7896 | 0.076923 |
| 14 | 196 | 3.7417 | 2.4101 | 1.6952 | 733.36 | 2.8738 | 0.071429 |
| 15 | 225 | 3.8729 | 2.4662 | 1.7188 | 871.42 | 2.9543 | 0.066667 |
| 16 | 256 | 4 | 2.5198 | 1.7411 | 1024 | 3.0314 | 0.0625 |
| 17 | 289 | 4.1231 | 2.5713 | 1.7623 | 1191.8 | 3.1058 | 0.058824 |
| 18 | 324 | 4.2426 | 2.6207 | 1.7826 | 1374.6 | 3.1777 | 0.055556 |

Powers, Roots, and Reciprocals.

| Number | Square | Square Root | Cube Root | Fifth Root | Power of $\frac{1}{2}$ | Power of $\frac{1}{3}$ | Reciprocal |
|--------|--------|-------------|-----------|------------|------------------------|------------------------|------------|
| 19 | 361 | 4'3589 | 2'6684 | 1'8020 | 1573'5 | 3'2472 | 0'052 632 |
| 20 | 400 | 4'4721 | 2'7144 | 1'8206 | 1788'8 | 3'3145 | 0'05 |
| 21 | 441 | 5'5826 | 2'7589 | 1'8384 | 2020'9 | 3'3798 | 0'047 619 |
| 22 | 484 | 4'6904 | 2'8020 | 1'8556 | 2270'11 | 3'4433 | 0'045 455 |
| 23 | 529 | 4'7958 | 2'8439 | 1'8722 | 2537'00 | 3'5050 | 0'043 478 |
| 24 | 576 | 4'8989 | 2'8845 | 1'8882 | 2821'8 | 3'5652 | 0'041 667 |
| 25 | 625 | 5' | 2'9240 | 1'9037 | 3125'0 | 3'6239 | 0'04 |
| 26 | 676 | 5'0990 | 2'9625 | 1'9186 | 3446'9 | 3'6812 | 0'038 462 |
| 27 | 729 | 5'1962 | 3' | 1'9332 | 3788'0 | 3'7372 | 0'037 037 |
| 28 | 784 | 5'2915 | 3'0366 | 1'9473 | 4148'5 | 3'7920 | 0'035 714 |
| 29 | 841 | 5'3852 | 3'0723 | 1'9610 | 4528'9 | 3'8455 | 0'034 483 |
| 30 | 900 | 5'4772 | 3'1072 | 1'9744 | 4929'5 | 3'8981 | 0'033 333 |
| 31 | 961 | 5'5678 | 3'1414 | 1'9873 | 5350'6 | 3'9493 | 0'032 258 |
| 32 | 1024 | 5'6569 | 3'1748 | 2' | 5792'6 | 4' | 0'031 250 |
| 33 | 1089 | 5'7746 | 3'2075 | 2'0124 | 6255'8 | 4'0495 | 0'030 303 |
| 34 | 1156 | 5'8309 | 3'2396 | 2'0244 | 6740'5 | 4'0982 | 0'029 412 |
| 35 | 1225 | 5'9161 | 3'2711 | 2'0362 | 7247'2 | 4'1460 | 0'028 571 |
| 36 | 1296 | 6' | 3'3019 | 2'0477 | 7776'0 | 4'1930 | 0'027 778 |
| 37 | 1369 | 6'0828 | 3'3322 | 2'0589 | 8327'3 | 4'2392 | 0'027 027 |
| 38 | 1444 | 6'1644 | 3'3620 | 2'0699 | 8901'4 | 4'2846 | 0'026 316 |
| 39 | 1521 | 6'2449 | 3'3912 | 2'0807 | 9498'6 | 4'3294 | 0'025 641 |
| 40 | 1600 | 6'3245 | 3'4200 | 2'0913 | 10120 | 4'3735 | 0'025 |
| 41 | 1681 | 6'4031 | 3'4482 | 2'1017 | 10763 | 4'4169 | 0'024 390 |
| 42 | 1764 | 6'4807 | 3'4760 | 2'1118 | 11432 | 4'4596 | 0'023 809 |
| 43 | 1849 | 6'5574 | 3'5034 | 2'1217 | 12124 | 4'5018 | 0'023 256 |
| 44 | 1936 | 6'6332 | 3'5303 | 2'1315 | 12842 | 4'5434 | 0'022 727 |
| 45 | 2025 | 6'7082 | 3'5569 | 2'1411 | 13584 | 4'5844 | 0'022 222 |
| 46 | 2116 | 6'7823 | 3'5830 | 2'1506 | 14351 | 4'6249 | 0'021 739 |
| 47 | 2209 | 6'8557 | 3'6088 | 2'1598 | 15144 | 4'6649 | 0'021 277 |
| 48 | 2304 | 6'9282 | 3'6342 | 2'1689 | 15962 | 4'7043 | 0'020 833 |
| 49 | 2401 | 7' | 3'6593 | 2'1779 | 16807 | 4'7433 | 0'020 408 |
| 50 | 2500 | 7'0711 | 3'6840 | 2'1867 | 17677 | 4'7818 | 0'02 |
| 51 | 2601 | 7'1414 | 3'7084 | 2'1954 | 18574 | 4'8198 | 0'019 608 |
| 52 | 2704 | 7'2111 | 3'7325 | 2'2039 | 19499 | 4'8574 | 0'019 231 |
| 53 | 2809 | 7'2801 | 3'7563 | 2'2124 | 20449 | 4'8945 | 0'018 868 |
| 54 | 2916 | 7'3484 | 3'7798 | 2'2206 | 21428 | 4'9313 | 0'018 519 |
| 55 | 3025 | 7'4162 | 3'8030 | 2'2288 | 22435 | 4'9676 | 0'018 182 |
| 56 | 3136 | 7'4833 | 3'8259 | 2'2369 | 23468 | 5'0035 | 0'017 857 |
| 57 | 3249 | 7'5498 | 3'8485 | 2'2448 | 24529 | 5'0391 | 0'017 544 |
| 58 | 3364 | 7'6158 | 3'8709 | 2'2526 | 25619 | 5'0742 | 0'017 241 |

Powers, Roots, and Reciprocals.

| Number | Square | Square Root | Cube Root | Fifth Root | Power of $\frac{1}{2}$ | Power of $\frac{1}{3}$ | Reciprocal |
|--------|--------|-------------|-----------|------------|------------------------|------------------------|------------|
| 59 | 3481 | 7·6811 | 3·8930 | 2·2603 | 26638 | 5·1091 | 0·016 949 |
| 60 | 3600 | 7·7460 | 3·9149 | 2·2679 | 27885 | 5·1435 | 0·016 667 |
| 61 | 3721 | 7·8102 | 3·9365 | 2·2754 | 29061 | 5·1776 | 0·016 393 |
| 62 | 3844 | 7·8740 | 3·9579 | 2·2829 | 30267 | 5·2114 | 0·016 129 |
| 63 | 3969 | 7·9372 | 3·9791 | 2·2902 | 31503 | 5·2449 | 0·015 873 |
| 64 | 4096 | 8· | 4· | 2·2974 | 32768 | 5·2780 | 0·015 625 |
| 65 | 4225 | 8·0623 | 4·0207 | 2·3045 | 34063 | 5·3109 | 0·015 385 |
| 66 | 4356 | 8·1240 | 4·0412 | 2·3116 | 35388 | 5·3434 | 0·015 152 |
| 67 | 4489 | 8·1854 | 4·0615 | 2·3185 | 36744 | 5·3756 | 0·014 925 |
| 68 | 4624 | 8·2462 | 4·0817 | 2·3254 | 38130 | 5·4076 | 0·014 706 |
| 69 | 4761 | 8·3066 | 4·1016 | 2·3322 | 39547 | 5·4393 | 0·014 493 |
| 70 | 4900 | 8·3666 | 4·1213 | 2·3389 | 40996 | 5·4707 | 0·014 286 |
| 71 | 5041 | 8·4261 | 4·1408 | 2·3456 | 42476 | 5·5018 | 0·014 085 |
| 72 | 5184 | 8·4853 | 4·1602 | 2·3522 | 43987 | 5·5326 | 0·013 889 |
| 73 | 5329 | 8·5440 | 4·1793 | 2·3588 | 45531 | 5·5639 | 0·013 699 |
| 74 | 5476 | 8·6023 | 4·1983 | 2·3651 | 47106 | 5·5936 | 0·013 514 |
| 75 | 5625 | 8·6603 | 4·2172 | 2·3714 | 48714 | 5·6237 | 0·013 333 |
| 76 | 5776 | 8·7178 | 4·2358 | 2·3777 | 50354 | 5·6536 | 0·013 158 |
| 77 | 5929 | 8·7750 | 4·2543 | 2·3840 | 52026 | 5·6832 | 0·012 987 |
| 78 | 6084 | 8·8318 | 4·2727 | 2·3901 | 53732 | 5·7127 | 0·012 821 |
| 79 | 6241 | 8·8882 | 4·2908 | 2·3962 | 55471 | 5·7418 | 0·012 658 |
| 80 | 6400 | 8·9443 | 4·3089 | 2·4022 | 57243 | 5·7708 | 0·012 5 |
| 81 | 6561 | 9· | 4·3267 | 2·4082 | 59049 | 5·7995 | 0·012 346 |
| 82 | 6724 | 9·0554 | 4·3445 | 2·4141 | 60888 | 5·8281 | 0·012 195 |
| 83 | 6889 | 9·1104 | 4·3621 | 2·4200 | 62762 | 5·8564 | 0·012 048 |
| 84 | 7056 | 9·1652 | 4·3795 | 2·4258 | 64669 | 5·8845 | 0·011 905 |
| 85 | 7225 | 9·2195 | 4·3968 | 2·4315 | 66611 | 5·9125 | 0·011 765 |
| 86 | 7396 | 9·2736 | 4·4140 | 2·4372 | 68589 | 5·9402 | 0·011 628 |
| 87 | 7569 | 9·3273 | 4·4310 | 2·4429 | 70559 | 5·9677 | 0·011 494 |
| 88 | 7744 | 9·3808 | 4·4480 | 2·4485 | 72645 | 5·9951 | 0·011 364 |
| 89 | 7921 | 9·4340 | 4·4647 | 2·4540 | 74726 | 6·0222 | 0·011 236 |
| 90 | 8100 | 9·4868 | 4·4814 | 2·4595 | 76843 | 6·0492 | 0·011 111 |
| 91 | 8281 | 9·5394 | 4·4979 | 2·4650 | 78995 | 6·0760 | 0·010 989 |
| 92 | 8464 | 9·5917 | 4·5144 | 2·4703 | 81183 | 6·1026 | 0·010 870 |
| 93 | 8649 | 9·6437 | 4·5307 | 2·4757 | 83408 | 6·1291 | 0·010 753 |
| 94 | 8836 | 9·6954 | 4·5468 | 2·4810 | 85668 | 6·1553 | 0·010 638 |
| 95 | 9025 | 9·7468 | 4·5629 | 2·4863 | 87964 | 6·1814 | 0·010 526 |
| 96 | 9216 | 9·7980 | 4·5789 | 2·4915 | 90298 | 6·2074 | 0·010 417 |
| 97 | 9409 | 9·8489 | 4·5947 | 2·4966 | 92668 | 6·2332 | 0·010 309 |
| 98 | 9604 | 9·8995 | 4·6·04 | 2·5018 | 95075 | 6·2588 | 0·010 204 |
| 99 | 9801 | 9·9499 | 4·6261 | 2·5068 | 97519 | 6·2843 | 0·010 101 |
| 100 | 10000 | 10· | 4·6416 | 2·5119 | 100000 | 6·3096 | 0·01 |

NOTE.—This table admits of finding the fourth and fifth powers of numbers.

Hydraulic Machines:—Return of Motive Power.

Deduced from Morin's experiments.

| | Proportion of Motive Power yielded | | Proportion of Motive Power yielded | Proportion of Water raised |
|----------------------|------------------------------------|---------------------------------|------------------------------------|----------------------------|
| Lift pump . . . | .316 | <i>Fire Engines.</i> | | |
| Force pump . . . | .516 | Merryweather . . . | .572 | .920 |
| Fire engine . . . | .233 | Tylor | .625 | .887 |
| Chinese wheel . . . | .36 | Letestu | .452 | .910 |
| Flash wheel . . . | .59 | Perry | .300 | .910 |
| Wirtz pump . . . | .181 | Flaud | .194 | .920 |
| | .640 | Perrin | .210 | .900 |
| <i>Rotary.</i> | | <i>Drainage Pumps.</i> | | |
| Stotz pump . . . | .43 | Denizot | .690 | .930 |
| Leclerc | .307 | Delpech | .600 | .926 |
| <i>Centrifugal.</i> | | Letestu | .513 | .940 |
| Piatti | .20 | Millus | .502 | |
| Appold | .70 | <i>Supply Pumps.</i> | | |
| Gwynne | .190 | At Ivry (feeder alone) . . . | .230 | |
| Girard | .300 | At Ivry (three pumps) . . . | .530 | |
| Vertical helix . . . | .19 | At St. Ouen | .696 | |
| <i>Water Rams.</i> | | At Lisbon (Farcot) | .652 | |
| Montgolfier . . . | .47 | Solid piston pumps | .900 | |
| | .80 | <i>Oscillating Pumps.</i> | | |
| Caligny | .43 | Vascile's fire-engine | .50 | |
| Foex | .55 | Gray's oscillating | .45 | |
| Dartige's balance | .72 | | | |
| Belidor | not used | | | |
| Huelgoat | .45 | | | |
| Pfetsch | .771 | | | |

Hydraulic Contrivances.

(By the Author.)

| | Coefficient of reduction for power | | Coefficient of reduction for power |
|-------------------------------|------------------------------------|--------------------------------|------------------------------------|
| Baling | 0.75 | Single chain of pots | 0.55 |
| Windlass | 0.50 | Double chain of pots | 0.60 |
| Dāl (Indian) | 0.70 | Single Môt (Indian) | 0.70 |
| Dāl (South India) | 0.70 | Double Môt (Indian) | 0.60 |
| Beam and bucket | 0.80 | Common pump | 0.50 |
| Picotah (S. Indian) | 0.80 | Lift and force-pump | 0.60 |

Memoranda for Conversion of Quantities.

Expressed in commercial measure.

MEASURES.

| | | | | |
|-------------|---|----------|--------------------|---|
| Feet | × | 0·015 | = Gunter's chains. | } See also pages 14 and 15 of the text for scientific system at 32° and 39° Fahr. |
| Feet | × | 0·00019 | = Miles. | |
| Square feet | × | 0·111 | = Square yards. | |
| Square feet | × | 0·000023 | = Acres. | |
| Cubic feet | × | 6·23 | = Gallons. | |
| Cubic feet | × | 0·779 | = Bushels. | |
| Cubic feet | × | 0·037 | = Cubic yards. | |

RAINFALL.

Feet of downpour × 193600 = cubic feet per square mile.

Feet of downpour × 302·5 = cubic feet per acre.

DRAINAGE AREAS.

The drainage from 1 square mile collecting 1 foot in depth yearly

} will irrigate 176 acres at a duty of 200 acres per cubic foot per second, will supply 47,580 inhabitants at a duty of 10 gallons daily, will yield ·8833 cubic feet per second throughout the year.

VELOCITIES.

| | | | |
|-----------------|---|------|------------------------|
| Feet per second | × | ·68 | give miles per hour. |
| Feet per second | × | 60 | give feet per minute. |
| Feet per second | × | 20 | give yards per minute. |
| Feet per second | × | 1200 | give yards per hour. |

DISCHARGES.

| | | | |
|--------------------|---|-----------------|-------------------------------------|
| Cub. feet per sec. | × | 2·2 | give cubic yards per minute. |
| Cub. feet per sec. | × | 133 | give cubic yards per hour. |
| Cub. feet per sec. | × | 3200 | give cubic yards per day. |
| Cub. feet per sec. | × | 6 $\frac{1}{4}$ | give gallons per second. |
| Cub. feet per sec. | × | 375 | give gallons per minute. |
| Cub. feet per sec. | × | 22 | give thousands of gallons per hour. |
| Cub. feet per sec. | × | 500 | give thousands of gallons per day. |
| Cub. feet per sec. | × | 2400 | give tons per day. |

WEIGHT.

| Cubic feet. | Gallons. | |
|------------------------------|----------------------------|-----------------------|
| 1 | = 6.232 | and weighs 62.32 lbs. |
| .1605 | = 1 | and weighs 10 lbs. |
| 1.8 | = 11.2 | and weighs 1 cwt. |
| 35.943 | = 224 | and weighs 1 ton. |
| 1 cubic inch | = .0036 | and weighs .0361 lbs. |
| 1 fluid ounce weighs | 437.5 grains. | |
| 1 Troy ounce measures | 8 fluid drams, 46 minims. | |
| 1 Avoirdupois ounce measures | 8 fluid drams. | |
| 1 lb. Troy = 5760 grains | = 6319.54 minims of water. | |
| 1 gallon = 76800 minims | = 70000 grs. of water. | |
| 1 lb. Avoir. = 7000 grains | = 7680 minims of water. | |

All comparisons between measures of capacity and those of weight are made with distilled water at a maximum density, at a specific gravity of 1; but, in commercial measure, the vessel is at a temperature of 62° Fahr.

PRESSURE OF WATER.

| | |
|--------------------------------------|---------------|
| H = head of water in feet | H = P × 0.016 |
| P = pressure in lbs. per square foot | P = H × 62.32 |

HORSE-POWER.

1 HP = 33000 lbs. of water raised 1 foot in 1 minute.
= 884 tons of water raised 1 foot in 1 hour.

Theoretical HP = .113 Q × fall in feet.

The drainage of 10 square miles collecting 12" yearly gives 1 HP for each foot of fall.

For pumping engines of the best class, allow HP = .142 Q H where Q = quantity raised in cubic feet per second, H = height in feet.

TOWAGE.

The general formula referred to in the text is

$$R = b T \cdot V^2,$$

where R = the pull on the rope in pounds,

T = the displacement of the barge in tons,

V = the velocity through the water,

b = a coefficient varying with the form of the barge, from .109 to .369.

Constants of Labour.

(Hurst.)

EARTHWORK.

Expressed in terms of a day's labour of 10 hours.

Days of a Labourer.

| | Soil | | |
|---|--------------|------------|--------------|
| | Soft | Moderate | Hard |
| Excavating only per cub. yard | '050 | '100 | '200 |
| „ in rock requiring blasting „ „ | | | '450 |
| Throwing 5 feet high, or filling trucks „ „ | Light '048 | Heavy '055 | Wet mud '065 |
| Filling barrows „ „ | '045 | '052 | '061 |
| Removing with wheelbarrow to 25 } yards' distance } „ „ | '026 | '030 | '030 |
| Filling at back of walls „ „ | '048 | '055 | '058 |
| Ramming earth in 6-inch layers . . „ „ | '040 | | |
| „ „ 12-inch „ „ | '025 | | |
| Levelling earth from barrow-heaps } without throwing } | '012 | '019 | |
| <hr/> | | | |
| Levelling and trimming slopes . . . per sq. yard | '020 to '030 | | |
| Turf 4 inches thick, cutting and } stacking only } | „ „ '045 | | |
| „ „ resodding only „ „ | '065 | | |

Days of driver, horse, and cart. (See also Cartage Table.)

| | |
|---|--------------|
| Removing 220 yards' distance, per c. yard . . . | '035 to '040 |
| Each additional 220 yards „ „ . . . | '020 to '025 |

N.B.—The vertical transport of earth is equal to 15 times the same horizontal distance when barrows are used, and 12 times when horses and carts are employed.

Days of an Indian Coolie.

| | Sand | Gravel | Stony soil |
|--|------|--------|------------|
| Excavating down to 9 feet, carrying to 25 yds. in a basket and depositing up to 6 ft., per cub. yard | 1'25 | 2'00 | 3'75 |
| Excavating down to 15 feet . . . „ „ | 2'00 | 2'75 | 4'50 |
| Add for each 3 feet more of depth or height of delivery, or for each 15 yards' additional distance per cub. yard | '25 | '25 | '25 |

Constants of Labour.

(Hurst.)

BRICKLAYERS' WORK.

Expressed in terms of a day's labour of 10 hours.

One Bricklayer's Labourer.

| | Days |
|--|------|
| Mixing concrete, wheeling and throwing from a stage, per cub. yard | '300 |
| Mixing mortar with a shovel " " | '720 |
| A two-horse pug-mill mixes 25 cubic yards of mortar in . . . | 1 |
| Picking up and stacking bricks without moving . . . per 1,000 | '150 |
| " " if handed to him " " | '100 |
| Selecting bricks for facings " " | '300 |
| Taking down old brickwork in mortar, cleaning and stacking per cub. yard | '410 |

One Bricklayer and Labourer.

| | Days |
|---|------|
| Brickwork in mortar to walls, exclusive of face work, per cub. yard | '320 |
| " in cement " " | '373 |
| " in mortar to covering arches " " | '410 |
| Pointing flat joint in mortar and raking out mortar joints per sq. yard | '110 |
| Pointing flat joint in cement and raking out cement joints . . . " | '170 |
| Pointing tuck in cement and raking out cement joints . . . " | '258 |
| Paving with stock bricks on edge in mortar " " | '086 |
| " " " " in cement " " | '100 |
| Laying and jointing in cement 3-inch drain pipes . . . per lin. yard | '024 |
| " " " 6 " " " | '048 |
| " " " 9 " " " | '069 |
| " " " 12 " " " | '093 |
| " " " 18 " " " | '150 |

One Bricklayer only.

| | Days |
|---|------|
| Working each fair face to brickwork and pointing . . . per s. yd. | '080 |
| Working each fair face in malms or facing of superior bricks | |
| per s. yd. | '117 |
| Working each fair face in malms, circular to template . . . " | '189 |
| Rough cutting to brickwork " " | '135 |
| Fair " " " " | '360 |

Constants of Labour—(continued).

(Hurst.)

MASONS' WORK.

Days of a Labourer.

| | | Days |
|---|-----------------|------|
| Rubble stone.—Filling barrows | per cubic yard | ·060 |
| „ Removing 25 yards and returning | „ „ | ·040 |
| „ Unloading barrows | „ „ | ·030 |
| „ Taking down old masonry in mortar, cleaning and stacking | „ „ | ·600 |
| Breaking stone to 1½" ordinary limestone | „ „ | ·700 |
| „ granite or very hard stone | „ „ | ·930 |
| Spreading the same for metalling 3" deep | per square yard | ·022 |

Days of a Mason and Labourer.

| | | Days |
|---|----------------|-------|
| Rubble masonry, dry in foundations | per cubic yard | ·240 |
| „ „ in mortar above foundations | „ „ | ·310 |
| „ „ all beds being horizontal | „ „ | ·480 |
| „ „ in cement „ | „ „ | ·570 |
| Ashlar masonry, 12" thick and in 12" courses, rubble with chisel-drafted margins | „ „ | 2·160 |
| Cubed stone hoisted and set in mortar | „ „ | ·756 |
| „ „ „ in cement | „ „ | ·945 |

Days of a Mason only.

| | | Days |
|--|-----------------|------|
| Add to rubble masonry for each fair face | per square yard | ·090 |
| „ „ if hammer dressed | „ „ | ·360 |
| „ „ if curved | „ „ | ·414 |
| Squaring 2" flags for paving | „ „ | ·072 |
| „ 4" „ | „ „ | ·135 |

Days of a Mason on stone of various sorts.

| | | Caen Days | Portland Days | Granite Days |
|---|-------|--------------|------------------|-----------------|
| Whole sawing, or axing, per square yard | | ·270 | ·540 | 1·270 |
| Plain work | } „ „ | ·540 | ·765 | 1·800 |
| „ circular | | ·900 | 1·395 | 2·160 |
| Sunk work | } „ „ | ·675 | 1·080 | 2·135 |
| „ circular | | 1·035 | 1·575 | 2·925 |
| Moulded work | } „ „ | 1·395 | 1·800 | 3·825 |
| „ circular | | 1·800 | 2·700 | 4·905 |

Constants of Labour—(continued).

(Hurst.)

PAVIORS', PLASTERERS', SLATERS', AND PAINTERS' WORK.

Days of a Pavior and Labourer.

| | | Days |
|--|-----------------|-------|
| Coursed pilcher paving, 6', in gravel, 6" deep | per square yard | 0·076 |
| " " " " " " " " " " " " | " " | 0·087 |
| Add for grouting and setting in mortar | " " | 0·035 |

Days of a Slate-mason.

| | | Days |
|---|-----------------|-------|
| Planing slate slabs | per square yard | 0·144 |
| Polishing slabs with very fine sand | " " | 0·270 |
| Plastering on under side of slating | " " | 0·050 |

Days of a Slater and Labourer.

| | | Days |
|--|------------|-------|
| Preparing and laying. Doubles | per square | 1·000 |
| " " " " " " " " " " " " | " " | 0·500 |

Days of a Labourer.

| | | Days |
|--|----------------|-------|
| Mixing lime and hair | per cubic yard | 0·032 |
| " " " " " " " " " " " " | " " | 1·080 |

Days of a Plasterer and Labourer.

| | | Days |
|--|-----------------|-------|
| Rendering and setting or floating | per square yard | 0·030 |
| Rendering two coats and setting | " " | 0·042 |
| Lathing with double fir laths | " " | 0·021 |
| Stucco trowelled | " " | 0·082 |
| Rendering with cement and sand | " " | 0·083 |
| Rough casting in lime and fine gravel | " " | 0·015 |
| Lime whitening | " " | 0·004 |
| Whiting and size, two coats, exc. scouring | " " | 0·009 |
| Colouring, stone or buff, two coats | " " | 0·012 |

Days of a Painter or Glazier.

| | | Days |
|--|-----------------|-------|
| Knotting, stopping, and painting, 1st coat | per square yard | 0·025 |
| Second or following coats, each | " " | 0·012 |
| Tarring with Stockholm tar, 1st coat | " " | 0·040 |
| Sash squares, each side, 2 coats | per square | 0·009 |
| Stopping, crown glass into new sashes | per square foot | 0·019 |
| " " " " " " " " " " " " | " " | 0·060 |

Constants of Labour—(continued).

(Hurst.)

CARPENTERS' WORK.

Days of a pair of Sawyers.

| | | | Days |
|---------|----------------------------------|-----------------|--------|
| Sawing. | Pine or Fir | per square foot | 0'0024 |
| „ | Ash, beech, elm, birch | „ „ | 0'0034 |
| „ | English oak, teak | „ „ | 0'0050 |

For arris-wise sawing add two-thirds.

Days of a Carpenter.

| | | Days |
|---|-----------------|-------|
| Working fir into rafters, purlins, joists, when under 16 sq. inches in section | per cubic foot | 0'080 |
| Under 36 sq. in. 0'069; under 81, 0'061, over 81 | „ „ | 0'054 |
| Working fir into rough frames as naked floors over 16" sq. in. | „ „ | 0'100 |
| Working fir into trusses, section 16" and over | „ „ | 0'135 |
| Priming and fixing fir, rough under 16" | „ „ | 0'160 |
| „ „ „ over 81" | „ „ | 0'108 |
| „ wrought two sides under 16" | „ „ | 0'232 |
| „ „ „ over 81" | „ „ | 0'138 |
| „ wrought all round under 16" | „ „ | 0'280 |
| „ „ „ over 81" | „ „ | 0'158 |
| Planing fir and squaring | per square foot | 0'017 |
| Sawing off end of sheeting piles | „ „ | 0'110 |
| Single tenon and mortice in fir under 16" | each | 0'040 |
| „ „ „ „ 81" | „ | 0'080 |
| „ „ „ „ 144" | „ | 0'10 |

For double tenon and mortice add one-third.

Cartage Table.
(By J. H. E. Hart.)

Constants of Labour per ton and per 100 cubic feet in terms of a day's work of a cart.

| Distance of 'lead' in miles | No. of trips | Cost of one trip | Constants for one ton | | | |
|-----------------------------|--------------|------------------|-------------------------|---------|--------|---------|
| | | | For a weight of load of | | | |
| | | | 8 cwt. | 8½ cwt. | 9 cwt. | 10 cwt. |
| 3/16 to 1/4 | 16 | ·0625 | ·156 | ·149 | ·139 | ·125 |
| 1/8 " 1/4 | 12 | ·083 | ·208 | ·196 | ·185 | ·167 |
| 1/4 " 1/2 | 10 | ·1 | ·25 | ·235 | ·222 | ·2 |
| 1/2 " 1/2 | 8 | ·125 | ·313 | ·294 | ·278 | ·25 |
| 3/4 " 1/2 | 6 | ·167 | ·417 | ·392 | ·370 | ·333 |
| 1 " 1/2 | 5 | ·2 | ·5 | ·471 | ·445 | ·4 |
| 1 1/4 " 1 1/4 | 4 | ·25 | ·625 | ·588 | ·556 | ·5 |
| 1 1/2 " 2 1/4 | 3 | ·333 | ·833 | ·784 | ·741 | ·667 |
| 2 1/2 " 3 1/4 | 2 | ·5 | 1·25 | 1·176 | 1·111 | 1·0 |
| 3 1/4 " 4 1/4 | 1 1/2 | ·667 | 1·67 | 1·57 | 1·48 | 1·33 |
| 4 1/4 " 5 1/4 | 1 1/4 | ·8 | 2·0 | 1·88 | 1·78 | 1·6 |
| 5 1/4 " 8 | 1 | 1·0 | 2·5 | 2·35 | 2·22 | 2·0 |

| Distance of 'lead' in miles | No. of trips | Cost of one trip | Constants for 100 cubic feet | | | | | | |
|-----------------------------|--------------|------------------|---|-------|--------|-------|-------|-------|-------|
| | | | For a capacity of load in cubic feet of | | | | | | |
| | | | 6 | 8 | 9 | 10 | 12 | 15 | 16 |
| 3/16 to 1/4 | 16 | ·0625 | 1·042 | ·781 | ·694 | ·625 | ·521 | ·417 | ·391 |
| 1/8 " 1/4 | 12 | ·083 | 1·389 | 1·042 | ·926 | ·833 | ·694 | ·556 | ·521 |
| 1/4 " 1/2 | 10 | ·1 | 1·667 | 1·25 | 1·111 | 1· | ·833 | ·667 | ·625 |
| 1/2 " 1/2 | 8 | ·125 | 2·083 | 1·563 | 1·389 | 1·25 | 1·042 | ·833 | ·781 |
| 3/4 " 1/2 | 6 | ·167 | 2·778 | 2·083 | 1·852 | 1·667 | 1·389 | 1·111 | 1·042 |
| 1 " 1/2 | 5 | ·2 | 3·333 | 2·5 | 2·222 | 2· | 1·667 | 1·333 | 1·25 |
| 1 1/4 " 1 1/4 | 4 | ·25 | 4·167 | 3·125 | 2·778 | 2·5 | 2·083 | 1·667 | 1·563 |
| 1 1/2 " 2 1/4 | 3 | ·333 | 5·556 | 4·167 | 3·704 | 3·333 | 2·778 | 2·222 | 2·083 |
| 2 1/2 " 3 1/4 | 2 | ·5 | 8·333 | 6·250 | 5·556 | 5· | 4·167 | 3·333 | 3·125 |
| 3 1/4 " 4 1/4 | 1 1/2 | ·667 | 11·111 | 8·333 | 7·407 | 6·667 | 5·556 | 4·444 | 4·167 |
| 4 1/4 " 5 1/4 | 1 1/4 | ·3 | 13·333 | 10· | 8·889 | 8· | 6·667 | 5·333 | 5· |
| 5 1/4 " 8 | 1 | 1· | 16·667 | 12·5 | 11·111 | 10· | 8·333 | 6·667 | 6·25 |



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