

HYDRAULIC
TABLES, COEFFICIENTS, AND FORMULÆ,
FOR
FINDING THE DISCHARGE OF WATER FROM ORIFICES,
NOTCHES, WEIRS, PIPES, AND RIVERS.

BY
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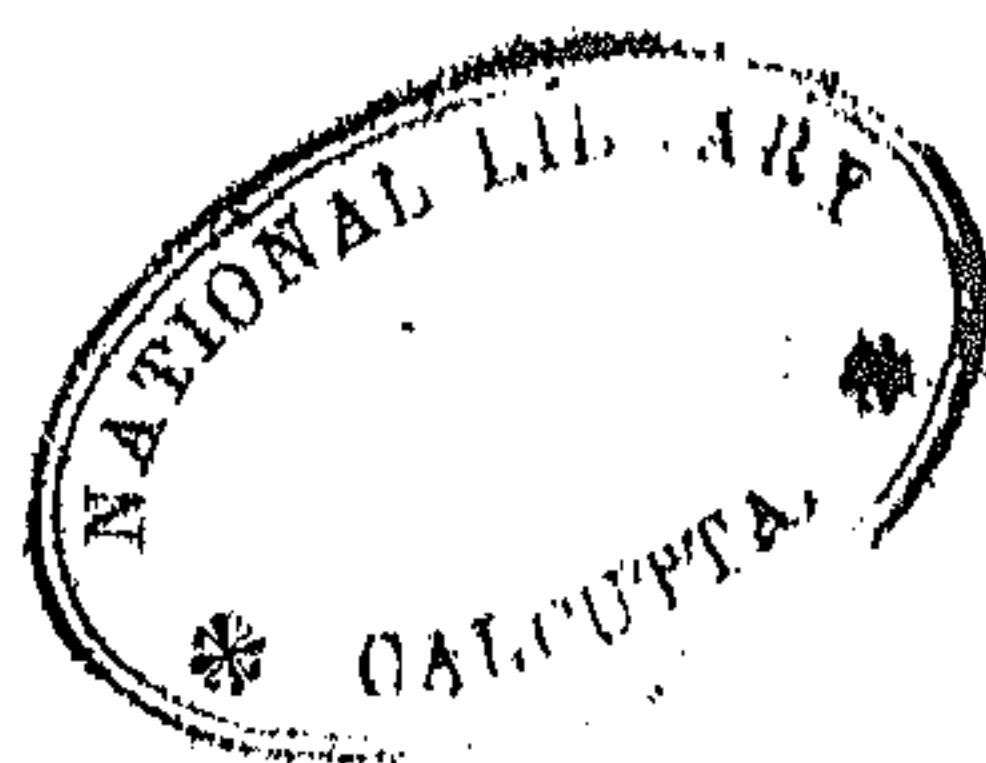
Second Edition :

WITH EXTENSIVE ADDITIONS, NEW FORMULÆ, TABLES, AND GENERAL
INFORMATION ON RAIN-FALL, CATCHMENT-BASINS, DRAINAGE, MINERALOGY,
WATER SUPPLY FOR TOWNS AND MILL POWER.

"It ought to be so generally known, that theory is a thing more than the conclusions of reason from numerous and accurately observed phenomena, and the deductions of the laws which connect causes with effects; that practice is the application of those general truths and principles to the common affairs and purposes of life; and that science is the recorded experience and discovery of mankind, or, as it has been well defined, 'the knowledge of many wisely and methodically digested, and arranged, so as to become attainable by one.'—*AMERICAN QUARTERLY REVIEW.*

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TO
MAJOR-GENERAL SIR THOMAS AISKEY LARCOM, K.C.B.,
M.D., F.R.S., M.B.I.A., &C.,
OF THE
ROYAL ENGINEERS,
UNDER SECRETARY OF STATE FOR IRELAND,
THIS WORK IS INSCRIBED
BY THE
AUTHOR.

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CORRECTION.

For $c = \frac{2}{3} c_d \sqrt{2gh}$, p. 120, line 18, read $c = \frac{2}{3} c_d \sqrt{2g}$.

INTRODUCTION

TO THE SECOND EDITION.

IN order to render this edition more valuable to the hydraulic engineer it has been considerably extended by the insertion of several new tables, formulae, experimental coefficients, examples, and general estimates of cost. It is hoped that the great extent and practical nature of the additions will render this work far more useful than the first in the ever-varying requirements of the profession. The six pages on catchment basins, rain-fall, and water power in the first edition have been extended into three new sections of one hundred and ten pages, embodying the subjects of water supply, sewerage discharge, drainage, and the method of determining the useful effect of water employed in turning the various kinds of water wheels. Considerable additions have also been made in Sections I, III, IV, VIII, and IX; the practical formula for gauging by weirs have been extended through twenty-one extra pages of new matter; the portion treating of the conveying powers of pipes and rivers has been also extended by the insertion of new tables at pages 42, 152, 191, 220, and 252, and of several new formulae, amongst which we believe that at page 216 (119A), will be found, in practice, the simplest and most accurate yet dis-

covered for ranges of velocity varying between one inch and twenty feet per second, and for all descriptions of channels, pipes, and rivers with which the engineer has to deal.

We have noticed at page 95 the erroneous notation of Morin and other engineers in giving only two-thirds of the coefficient of discharge for weirs. This notation assumes that the theoretical discharge from a notch is the same as if all the particles of water had the same velocity as those under test, which, being too large by one-third, the experimental coefficient has to be reduced in the same proportion. Mr. Blackwell and Mr. Hughes, in this country, lend themselves to this nomenclature; the latter gentleman says, page 328 of his useful treatise on *Water Works**, "Mr. Neville, in his tables of the discharge over weirs, termed the theoretical discharge $32\frac{1}{2}^3$ instead of $481\frac{1}{2}^3$ in cubic feet per minute." We were correct in doing so, for a weir one foot long; and those who adopt the latter formula are in error: this is a matter of demonstration, not of opinion. We can see no reason for sanctioning a different notation for notches, or orifices at the surface, and sunk orifices. The coefficients when in thin plates with large cisterns, have nearly the same general value, .615 to .628, and it tends to confusion to adopt in one place a coefficient for a correct formula, and in another a coefficient for an incorrect one; although the final result by an equality of contrary errors may be the same in both. We may here observe

* Weale's Series.

how very general the coefficient of two-thirds, and thereabouts, is for all orifices, notches, and also for the useful effect derived from the application of water power, as well as the relation of the velocity due to the fall and the velocity of water wheels to give a maximum result. The modifications of coefficients dependent on the position, thickness, form, and approaches of an orifice are seldom understood. The defects in the ordinary formula when the velocity of approach has to be considered are pointed out at pages 100, 115, and 116, and it is to be regretted that the authority of D'Aubuisson and others has misled many as to the correct form. Before the effective power of a water wheel, or water engine, can be determined we must know how to gauge the water supplied to it correctly. This can be done only by the application of formulae varied to suit the circumstances of the case under consideration. From causes, which it is not necessary to enter into here, this has seldom been done, and very little dependence can be placed on results obtained by the formula in common use when applied generally. It is pleasing to follow Francis and Thomson through the steps by which they get the effective power of their wheels, and we have accordingly made considerable use of their labors in Section XIV.

We have modified some of the old partial equations for the velocity in pipes, beginning at (86), from the form $v = \left\{ \frac{rs}{b} + \frac{a^3}{4b^3} \right\}^{\frac{1}{2}} - \frac{a}{2b}$, into $v = c \sqrt{rs} - \frac{a}{2b}$; principally for the purpose of giving the numerical values of c in the form $v = c \sqrt{rs}$. Of course these

modifications are only applicable when the value of $\frac{a}{2b}$ is small compared with that of the velocity.

Some remarks will be found at page 224 respecting M. Darcy's new formula for the velocity of water in iron pipes, as given by Morin in his *Hydraulique*. We have, only since those remarks were written, seen the original Mémoire, printed in Tome XV, *Mémoires présentés par divers savants à l'Académie des Sciences de l'Institut Impérial de France*, Paris, 1858. This Mémoire extends through 263 quarto pages, of which 34 contain tables calculated from the formula. We have not had time to do more than glance through it, but the deduced formula appears to be entirely derived from the author's experiments. This we consider, to some extent, objectionable; but, however this may be, there can be no doubt that the value of the coefficient c , in the formula $v = c \sqrt{rs}$, increases with the inclination, s , as well as with the diameter, $4r$, of the pipe; and as M. Darcy's formula makes the value of the multiplier, c , depend alone on the value of r , or $4r$, there appears an omission, in making the coefficient of friction entirely independent of the hydraulic inclination, and dependent only on the size of the channel. We shall give a few examples, taken at discretion, to show how limited this formula must be in its application.

1. Couplet's experiment, No. 43, p. 103, reduced to feet, gives $r = .3997$ feet, $s = .0035$, $rs = .001339$, and the observed velocity $v = 3.478$ feet $= 95 \sqrt{rs}$ nearly. Darcy's formula would give $v = 110.8 \sqrt{rs}$, our formula $106 \sqrt{rs}$ nearly, and Weisbach's

$105 \sqrt{rs}$ nearly. The pipe was probably an old one, and a deduction of about 10 per cent. might be made for the state of the bore. We have here, however, no means of judging the effect of a change of inclination on the multiplier, c .

2. From Du Buat's experiments with an inch pipe, nearly, Nos. 50 and 51, p. 103, we get, after reducing them to feet, in experiment 50, $r = .0222$, $s = .228$ and $v = 6.33$ feet $= 89.2 \sqrt{rs}$; or, after making the necessary deductions in the head for the velocity and the orifice of entry with the coefficient .815, $s = .147$ and $v = 6.33$ feet $= 111.4 \sqrt{rs}$. In experiment 51, we also get in feet $r = .0222$, $s = .3074$, and $v = 7.54 = 92 \sqrt{rs}$; or, by making allowance for the head due to the velocity and the orifice of entry, as before, $s = .179$, and $v = 7.54$ feet $= 119.7 \sqrt{rs}$. Here we see how the velocity or value of the inclination, s , affects the value of the multiplier, the diameter remaining constant. M. Darcy's formula, in each case, would only make $v = 80.3 \sqrt{rs}$.

3. In the excerpt proceedings of the Institution of Civil Engineers, p. 4, 6th February, 1855, James Simpson, president, in the chair, there is given for the "Colinton pipe" 16 inches diameter, eight or nine years in use, three observations. First, 29,580 feet long, a head of 420 feet and a discharge of 571 cubic feet per minute: these give $v = 6.816$ feet $= 99.2 \sqrt{rs}$ nearly. Secondly, a length of 25,765 feet a head of 184 feet, and a discharge of 440 cubic feet per minute: these give $v = 5.252$ feet $= 96.3 \sqrt{rs}$. And thirdly, a length of 3,815 feet a head of 184

feet, and a discharge of 1,215 cubic feet per minute; these give $v = 14.6$ feet $\approx 1.15 \sqrt{rs}$ nearly. In these three examples, the diameter, castings, and age of the pipes are the same. Yet we see, clearly, that the inclination affects the multiplier of \sqrt{rs} , which increases with the inclination, s , although M. Darcy's formula would make the multiplier the same in each case, and for all inclinations, viz. $v = 110 \sqrt{rs}$. Making those allowances inseparable from the state of the pipe, and all experimental observations, these results, as well as those from Du Buat's experiments, confirm the accuracy of our general formula (111A) page 215*, and those others we have given following it, as well also as that of Weisbach.

Dr. Young's formula, page 207, bears a resemblance to that of M. Darcy, in making the multiplier of \sqrt{rs} depend only on the diameter; but it works in a contrary manner: for the high velocities being derived from pipes, with small diameters in his experiments at his command, the value of c in $v = c \sqrt{rs}$, reduced from his formula, becomes larger in general for small than for larger diameters. No doubt an allowance should be made in small pipes for a thin film of water adjoining the pipe with little or no velocity; but within the limits which the engineer has to deal this may be neglected. Its effect,

* The form in which we first discovered this formula was $v = \left\{ 140 - \frac{10.6}{(rs)^{\frac{1}{2}}} \right\} \times \sqrt{rs}$. For measures in metres it becomes $v = 77.8 (rs)^{\frac{1}{2}} - 4.9 (rs)^{\frac{1}{2}}$; in which r is half the radius of a cylindrical pipe, or the hydraulic mean depth of any channel.

as well as that of all the other resistances, junctions, contractions, deposits, &c., is greater in small than in large pipes. We must refer to the body of the work for further remarks on this subject, but from lately appearing at such length in the *Mémoires* of the Imperial Academy of Sciences, M. Darcy's formula called for especial notice here.

The Statistics of rain-fall and catchment-basins have not yet received the attention which the subject deserves. The distribution of rain gauges with reference to elevation, contour, temperature, and isothermal lines has not been sufficiently attended to. The connexion of the rain-fall with the discharge generally, for the whole catchment, for the tributary catchments, and their sub-catchments, at the sea in the middle districts and at the sources, noting the geology, must be observed for several years before the questions of supply, discharge, absorption, and evaporation in any climate can be answered. The maximum and minimum discharges in each year and series of years must be observed, as well as the average mean discharges, and the maximums and minimums of these also, before the physical connexion of climate and catchment can be correctly ascertained, and the engineer furnished with reliable data. Heretofore observations, even when best, have been partial or limited, and a wide field is here yet open to competent physicists in connexion with our drainage works.

The general items of cost given in SECTION XIII. will be found of use; they are intended, however, more as guides than as standards for other

feet, and a discharge of 1,215 cubic feet per minute these give $v = 14.5$ feet $= 115 \sqrt{rs}$ nearly. In these three examples, the diameter, castings, and age of the pipes are the same. Yet we see, clearly, that the inclination affects the multiplier of \sqrt{rs} , which increases with the inclination, s , although M. Darcy's formula would make the multiplier the same in each case, and for all inclinations, viz. $v = 110 \sqrt{rs}$. Making those allowances inseparable from the state of the pipe, and all experimental observations, these results, as well as those from Du Buat's experiments, confirm the accuracy of our general formula (119A) page 215*, and those others we have given following it, as well also as that of Weisbach.

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works, the cost of which must depend on their own circumstances. Those who have practical experience of the differences between estimates, cost, and value, and how they are affected by time, locality, quality, and quantity, will estimate for each case in detail; but the discrepancies between estimates and cost, even under the same circumstances, are too well known to call for any remark here.

A few words about our publisher. Mr. Weale having purchased our interest in this edition, at once decided on adopting larger type and better paper, at a heavy extra outlay to himself. We had reason to be satisfied with the manner in which the first edition was brought out, in this, however, he has excelled, and we hope his enterprise will receive a fair return.

*Jocelyn Street, Dundalk,
October, 1860.*

INTRODUCTION

TO THE FIRST EDITION.

IN preparing the following work, we had three objects in view : first, a collection of useful hydraulic formulæ ; secondly, a collection of experimental results, and coefficients ; and, thirdly, a collection of useful practical tables, some calculated entirely from the formulæ and experiments, and others for the purpose of rendering the calculations more easy.

The TABLES at the end of the volume are all original, with the exception of TABLE I., which contains the well-known coefficients of PONCELET and LESBROS ; but those are newly arranged, the heads reduced to English inches, and the coefficients for heads measured over and back from the orifice, placed side by side, for more ready comparison. The coefficients in the small Tables throughout the work have been all calculated by us from the original experiments ; the formulæ have been carefully examined, and the continental ones reduced to English measures—some of them, as will be seen, for the first time. No labour has been spared in preparing the TABLES, and they are all purely hydraulic, though some of them are capable of being otherwise applied. We have filled no gap by the introduction of Tables applicable to other subjects, and in every-day use.

The correction of some of the experimental formulae, particularly the continental ones, as printed in some English books, cost us some labour. Even Du Buat's well-known formula is frequently misprinted; and in a late hydraulic work, $\sqrt{d-1}$, one of the factors, is printed $\sqrt{d-1}$ in every page where it is quoted. It is not always that such mistake can be avoided, but experimental formulae are so often copied from one work into another without sufficient examination, that an error of this kind frequently becomes fixed; and when applied to practical purposes erroneous formulae get the correct ones into disrepute. See note to formula (91).

The TABLES of velocities and discharges over weirs and notches have been calculated for a great number of coefficients to meet different circumstances of approach and overfall, and for various heads from $\frac{1}{4}$ th of an inch up to 6 feet. TABLE II. embodies the velocities acquired by falling bodies under the head of "theoretical velocity," and the velocities, suited to various coefficients, for heads up to 40 feet.

The formulae for calculating the effects of the velocity of approach to orifices and weirs, and the necessary corrections for the ratio of the channel to the orifice, as well as TABLE V., we believe to be original. They will be found of much value in determining the proper coefficients suited to various ratios. The remarks throughout SECTION IV. are particularly applicable to the proper use of this TABLE.

TABLE VII. of surface and mean velocities will be found to vary from those generally in use, and to be much more correct, and better suited for practical

purposes, particularly as applied to finding the mean velocities in rivers.

We have extended TABLE VIII. so as to make it directly available for hydraulic mean depths, from $\frac{1}{16}$ th of an inch to 12 feet, and for various hydraulic inclinations, even up to vertical, for pipes. The fall in rivers seldom exceeds 2 or 3 feet per mile, or the velocity 5 or 6 feet per second. The extension of the Table for great inclinations, and consequently great velocities, was made for purposes of calculation, and to include pipes. It must be understood throughout this TABLE that the velocities are those which continue unchanged for any length of channel, viz., when the resistance of friction is equal to the acceleration of gravity, the moving water and channel being then *in train*. Several of Du Buat's experiments were made with small vertical pipes. This TABLE is equally applicable to pipes and rivers, and gives directly either the hydraulic inclination, the hydraulic mean depth, or the velocity when any two of them are known.

Hydraulic formulæ have been frequently rendered unnecessarily complex, and unsuited for practical application, by combining them with those of mere mensuration in order to find the discharge. We have therefore given formulæ for finding the mean velocity principally,—unless in a few instances, as in orifices near the surface, where the discharge itself is first necessary to find the mean velocity; this once determined, the calculation of the discharge becomes one of simple mensuration.

We have preferred giving the mean velocity to the

discharge itself in TABLE VIII., because, while a infinite number of channels having the same hydraulic inclination (s) and the same hydraulic mean depth (m) must have the same velocity (v), yet the section areas, and consequently the discharges, may vary upwards from $6.2832r^2$, the area of a semicircular channel, to any extent; and the operation of multiplying the area by the mean velocity, to find the discharge, is so very simple that any tabulation for that purpose is unnecessary. Besides this, the banks of rivers, unless artificially protected, remain very seldom at a constant slope, and therefore any TABLES of discharge for particular side slopes are only of use so far as they apply to hypothetical cases. Indeed we have seen, in new river cuts, the banks, cut first to a given slope, alter very considerably in a few months; while the necessary regimen between the velocity of the water and the channel was in the course of being established. The velocity suited to the permanency of any proposed river channel, though too often entirely neglected, is the very first element to be considered.

For circular pipes, however, TABLE IX. gives the discharge in cubic feet per minute when the velocity in inches per second is known, or found from TABLE VIII., and is calculated for pipes from $\frac{1}{4}$ th of an inch up to 12 inches in diameter. TABLE XII. gives also the discharges in cubic feet per minute from the different equivalent river channels in TABLE XI.

TABLE X., for finding the heads on weirs of different lengths, TABLE XI., of equally discharging river channels, and TABLE XII., of the actual discharges

from the equivalents in TABLE XI., will be found of great practical value when new weirs and water-cuts have to be made. TABLES XI. and XII. are equally applicable to channels having side slopes, the widths being then the mean or central widths.

When the discharge and fall are known, and the hydraulic mean depth and the dimensions of *any* channel have to be determined, Problem III., section 8, as illustrated in EXAMPLE 17, section 1, gives a new and perhaps the most practically useful solution yet published. Tables XI., XII., and XIII. are particularly applicable to this problem.

A uniform notation is preserved throughout the work, so that the different experimental formulae can be compared without any further reduction. The letter h is used in every instance for the head, c for the coefficient, r for the mean radius or hydraulic mean depth, and s for the sine of the hydraulic inclination, unless it be otherwise stated. In order to designate particular values, the primary letters have *dependent or initial* letters below to explain them. Thus h_t is the head to the top of an orifice, h_b the head at the bottom, h_w the head on a weir, h_f the head due to friction, c_d the coefficient of discharge, c_v the coefficient of velocity, c_c the coefficient of contraction, &c. When the whole head is made up of different elements, such as the portions due to friction, velocity, contractions, bends, &c., it is expressed by the capital letter H .

Some writers and engineers appear to confound the inclination of a pipe, simply so called, or the head divided by the length, with the hydraulic inclination ;

and consequently have fallen into error in applying such of the known formulæ as take into consideration only the head due to the resistance of friction. When pipes are of considerable length, and the water is supplied from a reservoir at one end, the inclination, found as above, and the hydraulic inclination, may be taken equal to each other without sensible error; but for shorter pipes, of say up to 100 feet long, or even longer, the greater number of formulæ, as Du Buât's and others, do not directly apply; and it is necessary to take into consideration the head due to the orifice of entry, the velocity in the tube, and also to the impulse of supply when there are junctions. These separate elements, and their effects, will be considered in the following pages; but it will be of use to refer here to some late experiments, and the imperfect application of formulæ to them, first premising that *a pipe may be horizontal, or even turn upwards, and yet have a considerable hydraulic inclination.*

Mr. Provis's valuable experiments* with $1\frac{1}{2}$ -inch pipes, from 20 to 100 feet long, have been used in a recent work for the purpose of testing the accuracy of Du Buât's and some other formulæ; but the head divided by the length is assumed to be the hydraulic inclination throughout, and no allowance is made for the head due to the orifice of entry and velocity in the pipe. Of course the writer's conclusions are erroneous. We have shown, SECTION I., page 30, how very nearly the formulæ and experiments agree.

The formulæ appear to have been also misunder-

* Transactions of the Institution of Civil Engineers, Vol. II., pp. 201—210.

stood by the surveyor who experimented for the General Board of Health; for the inclination of the pipe in itself is assumed to be the hydraulic inclination, and no allowance is made for the head due to the impulse of supply. We quote from the CIVIL ENGINEER AND ARCHITECT'S JOURNAL, Vol. XV., page 366, in which it is stated that "the chief results as respect the house drains are thus described in the examination of the surveyor appointed to make the trials."*

"What quantity of water would be discharged through a 3 inch pipe on an inclination of 1 in 120?—Full at the head, it would discharge 100 gallons in three minutes, the pipe being 50 feet in length. This is with stone-ware pipe manufactured at Lambeth. This applies to a pipe receiving water only at the inlet, the water not being higher than the head of the pipe.

"What water was this?—Sewage-water of the full consistency, and it was discharged so completely that the pipe was perfectly clean.

"At the same inclination what would a 4-inch pipe discharge with the same distances?—Twice the amount (that I found from experiment); or, in other words, 100 gallons would be discharged in half the time. This likewise applies to a pipe receiving water only at the inlet, and of not greater height than the head. In those cases the section of the stream is diminished at the outlet to about half the area of the pipe.

"Before these experiments were made, were there not various hypothetical formulæ proposed for general use?—Yes.

* Minutes of Information with reference to Works for the removal of Soil, Water, or Drainage, &c., &c. Presented to both Houses of Parliament, 1852.

† It is a mistake to call these formulæ hypothetical, unless so far as the hypothesis is founded on facts. Every formulæ with which we are acquainted is founded on experiments, and has been deduced from them, but these formulæ are too often hypothetically applied to short tubes without the necessary corrections. It will be seen from SECTION VIII. that the experiments from

"What would those formulae have given with a 3-inch pipe, an at an inclination of 1 in 100? and what was the result of your experiments with the 3-inch pipe?—The formulae would give 7 cubic feet, the actual experiment gave $11\frac{1}{2}$ cubic feet; converting it into time, the discharge, according to the formulae, compared with the discharge found by actual practice, would be as 2 to 3.

"How would it be with a 4-inch pipe?—The formulae would give about 14.7 cubic feet per minute, whereas practice gave 23 cubic feet per minute.

"Take the case of a 6-inch pipe of the same inclination?—The results, according to Mr. Hawkesley's formula, would be $40\frac{1}{2}$ cubic feet per minute; from experiment it was found to be $63\frac{1}{2}$ cubic feet per minute.

"Then with respect to mains and drainage over a flat surface, the result of course becomes of much more value, as the difference proved by actual practice increases with the diminution of the inclination?—Certainly, to a very great extent. For example, the tables give only 14.2 cubic feet per minute as the discharge from a pipe 6 inches diameter, with a fall of 1 in 800; practice shows that, under the same conditions, 47.2 cubic feet will be discharged.

"Will you give an example of the practical value of this when it is required to carry out drainage works over a very flat surface?—An inclination of 1 in 800 gives only 14 cubic feet per minute, according to theory, while, according to actual experiment, and with the same inclination, 47 cubic feet are given.

"Then this difference may be converted either into a saving of water to effect the same object, or into power of water to remove feculent matter from beneath the site of any houses or town?—It may be so.

"And also the power of small inclinations properly managed?—Yes; for example, if it was required to construct a water course that should discharge, say 200 feet per minute, the formula would require an inclination of 1 in $60=2$ inches in 10 feet; whereas, experiment has shown that the same would be discharged at an inclination of 1 in $200=\frac{1}{2}$ inch in 10 feet, thus effecting a considerable saving in excavation, or a smaller drain would suffice at the greater inclination."

which the formulae there given were derived, were in every way greatly more extensive than those made by the directions of the Board of Health. The formula named is, substantially, Eytelwein's algebraically transformed.

We have extracted and tabulated the results given above, in the following Table, and also eight of the experiments made for the Metropolitan Commissioners of Sewers*; and assuming for the present, with the surveyor, examined by the Commissioners, that the inclinations of the pipes and hydraulic inclinations of the formulæ are the same, *which is incorrect*, we give the calculated discharges, found by means of TABLES VIII. and IX., in the last column of the Table.

Diameter of pipe in inches.	Inclination of pipe.	Discharge in cubic feet per minute by experiment.	Hypothetical discharge by Du Buât's formula.
3	1 in 120	5.3	6.6
4	1 in 120	10.7	14
3	1 in 100	11.2	7.6
4	1 in 100	23	15.3
6	1 in 100	60.6	43.8
6	1 in 800	47.2	13.3
6	1 in 60	75	50.3
6	1 in 100	60	40.8
6	1 in 160	54	33.4
6	1 in 200	52	29.2
6	1 in 320	49	21.8
6	1 in 400	48.5	19.6
6	1 in 800	47.2	13.3
6	Level	40	00

Du Buât's formula, therefore, gives larger results than the experiments in the two first cases, because the water received at one end only barely filled it, and the pipe was not full at the lower end; but less in the others. If in these the head due to the im-

* Adcock's Engineer's Pocket Book, 1852, pp. 261 and 262.

pulse of entry, at the upper end, and at the side junctions, were known, and the proper hydraulic inclination determined by the experiments, the formula would be found to give larger approximate results in every case, as might have been expected from the sewage-water used. In the last eight experiments it is stated*, that "the water was admitted at the head of the pipe, and at *five junctions or tributary pipes* on each side, so regulated as to keep the main pipe full," and that "without the addition of junctions the transverse sectional area of the stream of water near the discharging end was reduced to one-fifth of the corresponding area of the pipe, *and that it required a simple head of water of about 22 inches to give the same result as that accruing under the circumstances of the junctions.*" It is also stated, that "in the case of the 6-inch pipe, which discharged 75 cubic feet per minute, the lateral streams had a velocity of a few feet per minute."

Now, the head of "about 22 inches" is wholly neglected in the foregoing calculations, *though in a pipe 100 feet long it would be equal to an inclination of 1 in 55!* It however includes three elements at least, viz. the portion due to the orifice of entry, the portion due to the velocity in the pipe, and the portion due to friction. Let us assume the case of *the horizontal pipe, which discharged 46 cubic feet per minute†.* This is equal to a mean velocity of

* Adcock's Engineer's Pocket Book, 1852, pp. 261 and 262.

† The horizontal pipe would discharge equally at both ends, unless there was a head of water at either, or an equivalent in

46.9 inches per second; with this velocity, we find from TABLE VIII. the hydraulic inclination of a 6-inch pipe to be 1 in 94, and, therefore, the head due to friction in a pipe 100 feet long is 12.7 inches. Assuming the coefficient for the orifice of entry and velocity to be .815, we also find from TABLE II. a head of $4\frac{1}{4}$ inches due to these. We then have,

Head due to the velocity and orifice of	
entry	4.25 inches
Head due to the resistance of friction	12.70 „
Radius of pipe	3.00 „
	<hr/>
Total	19.95

which is about 2 inches less than the observed head: this, however, is not stated definitely. *It is therefore evident, that the formula gives, if anything, larger results than these experiments*, as might have been expected, instead of less in the ratio of 2 to 3, as is stated in the Report.*

Wherever junctions are applied, as in the examples above referred to, the formulae in general use require correction; for the quantity of water then flowing below each junction is increased. A certain amount of error is, perhaps, inseparable from every calculation of this kind; but before we condemn formulae deduced from experiment by men every way quali-

the velocity of approach. Of course, a smaller pipe with a fall, must be better than the larger one with none at all, in preventing deposits.

* This is also true of the other formulae, for finding the discharge from pipes, given in this work.

fied for the task, it would be well that we should learn to understand and properly apply them.

The diameter of a short pipe gives in itself the means of increasing very considerably the surface inclination of the fluid stream, by reducing the section at the lower end. If we assume a horizontal pipe 50 feet long and 6 inches in diameter, we perceive, that if the receiving end be full, and the discharging end one-third full, this inclination

will be $\frac{6 - 2}{50 \times 12} = \frac{1}{150}$; and that the discharging end cannot be kept full unless a head of several inches be maintained at the receiving end, or an equivalent from a lateral supply. When the pipe is about two diameters long it becomes a short tube; and when the length vanishes, the transverse section becomes, simply, a discharging orifice.

We have been led into the foregoing remarks, not from any desire to find fault with a Report containing so much valuable information as the one referred to, but for the purpose of defending from unmerited reproach, in a *Blue Book*, the researches in this department, of

“Those dead but sceptred sovereigns who still rule
Our sciences from their urns—”

Du Buat, Young, Eytelwein, Prony, and others.

We do not pretend to any particular accuracy in the sketches scattered throughout the work; they are only intended to illustrate the text, and were sketched while writing it, without further aim; neither do we pretend to have entered fully into the principles or

practice of hydraulics, our object being to select, construct, and arrange useful hydraulic formulæ, experiments, and Tables for the use of all classes of engineers. We make, however, no apology for preferring formulæ, in their simplicity, to any written rules which may be deduced from them, as being in every way more general, concise, and elegant. In conclusion, it is hoped that any errors of consequence in the work, will be found corrected in the errata.

Roden Place, Dundalk,
January, 1868.

ON THE
DISCHARGE OF WATER
FROM
ORIFICES, WEIRS, PIPES, AND RIVERS.

SECTION I.

APPLICATION AND USE OF THE TABLES, FORMULÆ, &c.

To find the velocity of a falling body from the height fallen, or the height fallen from the velocity.

RULE. — MULTIPLY THE SQUARE ROOT OF THE HEIGHT IN INCHES BY 27.8, AND THE PRODUCT WILL BE THE VELOCITY IN INCHES.* To FIND THE HEIGHT FROM THE VELOCITY, SQUARE THE VELOCITY IN INCHES AND DIVIDE THE SQUARE BY 772.84, THE QUOTIENT WILL BE THE HEIGHT IN INCHES. See equation (1). TABLE II., column 1, will give the velocity from the height, found in the column of "altitudes," or the height from the velocity, directly.

EXAMPLE 1.—*What is the velocity acquired by a heavy body falling $\frac{1}{8}$ th of an inch?* In the Table opposite to $\frac{1}{8}$ th of an inch, found in the column headed "altitudes h ," we find 9.829 in column 1, for the required velocity, in inches per second.

EXAMPLE 2.—*What is the velocity acquired by a*

* The square root of the height in feet multiplied by 8.025 gives the velocity per second in feet; and the square of the velocity in feet divided by 64.4 will give the height in feet.

fall of 11 feet 3 inches? Opposite to 11 feet 3 inches, as before, we find 323.007 inches, for the velocity required.

EXAMPLE 3.—*What height must a heavy body fall through to acquire a velocity of $40\frac{1}{2}$ feet per second?* Here $40\frac{1}{2}$ feet is equal 486 inches, opposite to the nearest number to which, found in column 1, we find 25 feet 6 inches for the required fall. In this example, the nearest number to 486 found in the Table is 486.301. The difference .301 corresponds, very nearly, to $\frac{3}{8}$ ths of an inch in altitude, and, therefore, the true head according to the rule would be 25' 6 $\frac{3}{8}$ "; but for all practical purposes the difference is immaterial.

By means of TABLE II. we can find, directly, or by simple interpolation, the velocity due to all heights from $\frac{1}{16}$ part of an inch up to 40 feet, and the heights from the velocities. For a greater height than 40 feet it may be divided by 4, 9, or some square number s^2 , and the velocity found for the quotient, from the Table, multiplied by 2, 3, or s , the square root of the divisor, will give the velocity required.

EXAMPLE 4.—*What is the velocity acquired by a fall of 45 feet?* $\frac{45}{4} = 11' 3"$, the velocity corresponding to which, found from the Table, is 323.007. Hence, $323.007 \times \sqrt{4} = 323.007 \times 2 = 646'' .014 = 53' 10'' .014$ is the velocity per second required. The reverse of this example is equally simple.

Columns 3, 4, 5, 6, 7, 8, 9, 10, 11, and 12 in the Table, give the values of $\sqrt{2gh}$ multiplied by the

coefficients therein stated. These columns will be found of great practical use in finding the mean velocities in the *vena-contracta*, in the orifice, and in short tubes; and consequently also in finding the mechanical force, as well as the discharge. An examination of the coefficients in the small Tables in SECTION III., and also of those in TABLES I. and V., at the end of the work, will show how much they vary; but those most generally useful, and their products by the theoretical velocity due to different heads, up to 40 feet, are given in the columns referred to.

EXAMPLE 5. — *What is the discharge from an orifice 4 inches by 8 inches, the centre sunk 20 feet below the surface of a reservoir?* From TABLE II., we find 430.676 inches equal 35.89 feet for the

theoretical velocity of discharge: hence, $\frac{8 \times 4}{144} \times$

$35.89 = \frac{2}{9} \times 35.89 = 7.976$ cubic feet per second is

the theoretical discharge. If the discharge takes place through a thin plate, or if the inner arrises next the water in the reservoir be *perfectly square*, and the water in flowing out does not fill the passage so as to convert the orifice into a short tube, the coefficient will be found from TABLE I. to be .603. The true discharge then is $7.976 \times .603 = 4.809$ cubic feet per second.

For the determination of the coefficient suited to any particular orifice, and the circumstances of its position, we must refer generally to the following pages. If in the example just given, the arrises

next the reservoir were rounded into the form of the contracted vein, see Fig. 4, the coefficient would increase from .603 to .974 or .956, for a passage not exceeding a couple of feet in length. With the former the discharge would be $7.976 \times .974 = 7.769$ cubic feet, and with the latter $7.976 \times .956 = 7.625$ cubic feet. We may find from Table II. the latter results otherwise. With a head of 20 feet and the coefficient .974, the velocity is 419.48 inches = 34.957 feet; hence, the discharge is $\frac{2}{9} \times 34.957 = 7.768$ cubic feet. With a coefficient of .956, the velocity is 411.73 inches = 34.31 feet, and $\frac{2}{9} \times 34.31 = 7.624$ cubic feet. These results are the same, practically, as those previously found.

If the inner arrises be square, and the passage out be from 18 inches to 2 feet long, the orifice will be converted into a short tube, the coefficient for which is .815. With this coefficient, and a head of 20 feet, we find as before, from TABLE II., the mean velocity of discharge equals 351 inches = 29.25 feet; hence, the discharge now is $\frac{2}{9} \times 29.25 = 6.5$ cubic feet per second.

The velocities in inches per second, given in TABLES II. and VIII., or elsewhere in the following pages, may be converted into velocities in feet per minute, by multiplying by 5, equal $\frac{60}{12}$.

EXAMPLE 6.—*The discharge from a small orifice having its centre placed 10 feet below the surface of a*

reservoir is 18 feet per minute, what will be the discharge from the same orifice at a depth of 17 feet? The discharges will be to each other as $\sqrt{10} : \sqrt{17}$, or as $1 : \sqrt{1.7}$; or, from TABLE III., as $1 : 1.3038$, whence we get the discharge sought equal $1.3038 \times 18 = 23.4684$ cubic feet.

EXAMPLE 7.—What is the value of the expression $c_d \left\{ 1 + \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{1}{2}}$ in equation (45), when $c_d = .617$, and $m = 2$? Here we have—

$$\frac{c_d^2}{m^2 - c_d^2} = \frac{.617^2}{4 - .617^2} = \frac{.3807}{3.6193} = .1052;$$

whence the first expression becomes equal to $.617 (1.1052)^{\frac{1}{2}}$ equal, from TABLE III., $.617 \times 1.0513 = .649$, the value sought. TABLE V. contains the values of this expression for various values of c_d and m , which latter, m , stands for the ratio of the channel to an orifice; and we can immediately find from it, opposite 2 in the first column, and under the coefficient $.617$ in the sixth column, $.649$ the value sought. When the head due to the pressure, and to the velocity of approach, are both known, we can determine the new coefficient of discharge by the above expression, and thence the discharge itself. The coefficient suited to the velocity of approach may however be found directly in TABLE V. The usual methods for finding the effects of the velocity of approach, given by d'Aubuisson and others, are incorrect in principle, see SECTION IV.

EXAMPLE 8.—What is the discharge from an orifice 17 inches long and 9 inches deep, having the upper edge placed 4 inches below the surface, and the

lower edge 13 inches? The expression for the discharge is $\frac{2}{3} \times A \sqrt{2gd} \times c_d \left\{ \left(1 + \frac{h_1}{d}\right)^{\frac{3}{2}} - \left(\frac{h_1}{d}\right)^{\frac{3}{2}} \right\}$ equation (43), in which we must take $d = 9$ inches; $h_1 = 0$ inches; $A = 17 \times 9 = 153$ square inches; and $\sqrt{2gd}$, found from TABLE II. = 83.4 inches. We have, also, $\frac{h_1}{d} = \frac{4}{9} = .444$, and hence the value of

$$(1.444)^{\frac{3}{2}} - (.444)^{\frac{3}{2}} = (\text{from TABLE IV.}) 1.44.$$

Assuming the coefficient of discharge to be .617, we then have the discharge in cubic inches per second equal to

$$\frac{2}{3} \times 153 \times 83.4 \times .617 \times 1.44 =$$

$$\frac{2}{3} \times 12760.2 \times .88848 = 7558.$$

Consequently, $\frac{7558}{1728} = 4.374$ is the discharge in cubic feet per second. From equation (6), we get the discharge equal to

$$\frac{2}{3} \times .617 \times 27.8 \times 17 \times \{13^{\frac{3}{2}} - 4^{\frac{3}{2}}\}.$$

But $13^{\frac{3}{2}} - 4^{\frac{3}{2}} = 46.872 - 8$, from TABLE IV., equal to 38.872, whence the discharge is

$$\frac{2}{3} \times .617 \times 27.8 \times 17 \times 38.872 = 11.4351 \times 17 \times 38.872 = 194.3967 \times 38.872 = 7557 \text{ cubic inches} \\ = 4.374 \text{ cubic feet, the same as before.}$$

It is shown in equation (31), that by using the mean depth for orifices near the surface, the discharge will approximate very closely to the true discharge, and that even for weirs the error will not exceed 6 per cent. The discharge is then expressed by

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$\cdot 617 \sqrt{2g} \times 8\frac{1}{2} \times 9 \times 17 =$ (from TABLE II.) $50\cdot 01 \times 153 = 7651\cdot 53$ cubic inches $= 4\cdot 427$ cubic feet per second. The head to the centre of the orifice is here $8\frac{1}{2}$ inches, and the depth of the orifice 9 inches, therefore, in equation (31), $h = d$ very nearly; and, therefore, this result must be multiplied by $\cdot 989$, as shown in that equation; then $\cdot 989 \times 4\cdot 427 = 4\cdot 378$ cubic feet, which gives a result differing from those otherwise found, by a very small quantity, which, practically, is of no value. By means of TABLE VI. the discharge from rectangular orifices near the surface can be found with very great facility.

We may always find the discharge from an orifice near the surface with sufficient accuracy, for practical purposes, by taking the head to the centre, in the same manner as if the orifice were sunk to a considerable depth; then by applying the corrections given in equation (31); or if the orifice be circular, those given in equation (28); extreme accuracy, according to the correct formula, is obtainable.

EXAMPLE 9.—*What is the discharge from a circular orifice 4 inches in diameter, having its centre placed 4 inches below the surface, when the coefficient of discharge is $\cdot 617$?* The area of the orifice is $4 \times 4 \times \cdot 7854 = 12\cdot 566$ square inches. The velocity in the orifice at the mean depth of 4 inches, with a coefficient of $\cdot 617$, is 34·31 inches, whence the discharge is $12\cdot 566 \times 34\cdot 31 = 431\cdot 139$ cubic inches $= \cdot 2496$ cubic feet per second, or 14·97 cubic feet per minute. By means of TABLE IX. the discharge in cubic feet per minute can be found very readily when the velocity, 34·31 inches per second, is known. Thus,

	Inches.		Cubic feet.
For a velocity of	30.00	the discharge is	18.089
"	4.00	"	1.746
"	.80	"	0.181
"	.01	"	0.004
"	<u>34.81</u>	"	<u>19.980</u>

By applying the coefficient found from equation (28), which is .992, when the depth at the centre is twice the radius, as it is in this example, we get $.992 \times 14.97 = 14.85$ for the correct discharge in cubic feet per minute. Here the difference in the results is only 1 in 125.

The application of TABLE VI. will enable us to find the discharge from rectangular orifices near the surface very quickly. Resuming "EXAMPLE 8," the discharge may be found from this Table for each foot in length of the orifice, as follows. The discharge in cubic feet per minute, when the coefficient is .617 for a notch 1 foot long and 13 inches deep, is 223.323; and for a notch of 4 inches deep, 38.116; therefore, the discharge from an orifice 9 inches deep, with the upper edge 4 inches below the surface, is $223.323 - 38.116 = 185.207$ cubic feet per minute. But as the length of the orifice is 17 inches, this must be multiplied by $\frac{17}{12}$, and the product 262.377 is the discharge in cubic feet per minute; this is equal to a discharge of 4.373 cubic feet per second, and agrees with that before found. This is the simplest way of finding the discharge from rectangular orifices near the surface.

EXAMPLE 10.—*What is the discharge in cubic feet*

per minute, from an orifice 2 feet 6 inches long and 7 inches deep, the upper edge being 3 inches below the surface, and the coefficient of discharge .628? From TABLE VI. we find the discharge from a notch 1 foot long and 10 inches deep to be 153.353, and for a notch 3 inches deep, 25.199. The difference, or 128.154, multiplied by $2\frac{1}{2}$, will be the discharge required; viz. $2\frac{1}{2} \times 128.154 = 320.385$ cubic feet per minute.

EXAMPLE 11.—*The size of a channel is 2.75 times the size of an orifice, what is the coefficient of discharge when that for a very large channel in proportion to the orifice is .628?* We find from TABLE V. the coefficient to be .645, when the approaching water suffers full contraction. By attending to the auxiliary Tables in the text, we find, for this case, $\frac{\text{orifice}}{\text{channel}} = \frac{1}{2.75} = .36$. We must, therefore, multiply 2.75 by .857, which gives 2.36 for the ratio of the mean velocities in the orifice and in the channel approaching it. With this new value of the ratio of the channel to the orifice, we find, as before, the value of the coefficient from TABLE V. to be .651. The remarks throughout the work, with the auxiliary tables, will be found of much use in determining the coefficients for different ratios of the channel to the orifice, notch, or weir, and the corrections suited to each. If in this example we were considering,—other things being the same,—the alteration in the coefficient for a notch, or weir, it would be found from the Table, column 4, to be .672 instead of .645 found in column 3, for an orifice sunk some depth

below the surface. For the corrections suited to mean and central velocity, and to the nature of the approaches, we must refer to the body of this work and to the auxiliary tables therein at the end of SECTION IV.

EXAMPLE 12.—*What is the discharge over a weir 50 feet long; the circumstances of the overfall, crest, and approaches, being such that the coefficient of discharge is .617, when the head measured from the water in the weir basin, 6 feet above the crest, is $17\frac{1}{2}$ inches?* TABLE VI. will give the discharge in cubic feet per minute, over each foot in length of weir, for various depths up to 6 feet. It is divided into two parts; the first for "greater coefficients," viz. .667 to .617; and the second for "lesser coefficients," viz. .606 to .518. The coefficient assumed being .617, we find the discharge over 1 foot in length, with a head of $17\frac{1}{2}$ inches, to be 348.799 cubic feet per minute; hence the required discharge is $50 \times 348.799 = 17439.95$ cubic feet.

The determination of the coefficient suited to the circumstances of the overfall, crest, approaches, and approaching section, will be found discussed elsewhere through this work. The valuable Table derived from Mr. Blackwell's experiments will also be of use; but the heads being taken at a much greater distance back from the crest than is generally usual, the coefficients taken from it for heads greater than 5 or 6 inches, will be found under the true ones for heads measured immediately at or about 6 feet, above the crest. For heads measured *on* the crest, the small Table of coefficients in SECTION III., applicable to the purpose, will be of use.

EXAMPLE 13.—*What is the mean velocity in a large channel, when the maximum velocity along the central line of the surface is 31 inches per second?* TABLE VII. gives 25.89 inches for the required velocity, and for smaller channels 24.86 inches. In order to find the mean velocity at the surface from the maximum central velocity, the latter must be multiplied by .914.

The velocity at the surface is best found by means of a floating hollow ball, which just rises out of the water. The velocity at a given depth is best found by means of two hollow balls connected with a link, the lower being made heavier than the upper, and both so weighted by the admission of a certain quantity of water that they shall float along the current, the upper one being in advance but nearly vertical over the other. The velocity of both will then be the velocity at half the depth between them. The velocity at the surface, found by means of a single ball, being also found, the velocity lost at the half depth is had by subtracting the common velocity due to the linked balls from that of the single ball at the surface. The velocity at any given depth is then easily found by a simple proportion; but the result will be most accurate when the given depth is nearly half the distance between the balls, which distance can never exceed the depth of the channel. *Pitot's tube*, *Woltmann's tachometer*, the *hydrometric pendulum*, the *rheometer*, and several other hydrometers, have been used for finding the velocity; but these instruments require certain corrections suited to each separate instrument, as well as kind of instrument,

and are not so correct or simple, for measuring the velocity in open channels, as a ball and linked balls.

EXAMPLE 14.—*What is the discharge from a river having a surface inclination of 18 inches per mile, or 1 in 3520, 40 feet wide, with nearly vertical banks, and 3 feet deep?* The area is $40 \times 3 = 120$ feet, and the border $40 + 2 \times 3 = 46$ feet; therefore the hydraulic mean depth is $\frac{120}{46} = 2.61$ feet = 2 feet 7.3 inches*. With this and the inclination we find from

TABLE VIII. $28.27 + 2.75 \times \frac{1.3}{6} = 28.87$ inches per second = $28.87 \times 5 = 144.35$ feet per minute for the mean velocity; hence we got $144.35 \times 120 = 17,322$ cubic feet per minute for the required discharge. For channels with sloping banks we have only to divide the border, which is always known, into the area for the hydraulic mean depth, with which, and the surface inclination, we can always find the velocity by TABLE VIII, and thence the discharge. Unless the banks of rivers be protected by stone pavement or otherwise, the slopes will not continue permanent; it is therefore almost useless to give the discharges for channels of particular widths and side slopes. When the mean velocity is once known, the remaining calculations are those of mere mensuration, and they should be made separately. This example may also be solved, practically, by means of TABLES XI. and XII. A channel 40×3

* For greater hydraulic depths than 144 inches, the extent of the TABLE, divide by 9, and find the corresponding velocity. This multiplied by 9 will be the velocity sought.

has the same conveying power as one 70×2 , TABLE XI., which latter, TABLE XII. discharges with a fall of 1.8 inches in the mile, 17,157 feet; or about one per cent. less than that previously found.

EXAMPLE 15.—*The diameter of a very long pipe is $1\frac{1}{2}$ inch, and the rate of inclination, or whole length of the pipe divided by the whole fall, is 1 in $71\frac{1}{2}$; what is the discharge in cubic feet per minute?* The

hydraulic mean depth, or mean radius, is $\frac{1.5}{4} = .375$

inches $= \frac{3}{8}$ inch. Consequently we find from TABLE

VIII. the velocity in inches per second equal to $25.09 - 1.92 \times \frac{1.5}{10} = 25.09 - .29 = 24.80$. The

discharge in cubic feet per minute for a $1\frac{1}{2}$ -inch pipe is now found most readily by means of TABLE IX., as follows:—

Inches.			Cubic feet.		
For a velocity of	20.0	the discharge is	1.237		
"	"	4.0	"	"	.246
"	"	.8	"	"	.049
<hr/>			<hr/>		
"	"	24.8	"	"	1.521

Whence the discharge in cubic feet per minute is 1.521.

For short pipes, of 100 or 200 feet in length, and under, the height due to the velocity and orifice of entry must be deducted from the whole height to find the proper hydraulic inclination, and also the height due to bends, curves, cocks, slides, and erogation. The neglect of these corrections has led some writers into mistakes in applying certain formulæ, and in test-

ing them by experimental results obtained with short pipes. We shall now apply the TABLES to the determination of the discharge from short pipes, and compare the results with experiment, referring generally to equation (153) and the remarks preceding it for a correct and direct solution.

EXAMPLE 16.—*What is the discharge in cubic feet per minute from a pipe 100 feet long, with a fall or head of 35 inches to the lower end, when the diameter is $1\frac{1}{2}$ inch? Find also the discharge from pipes 80 feet, 60 feet, 40 feet, and 20 feet, of the same diameter and having the same head.* If the water be admitted by a stop-cock at the upper end, the coefficient due to the orifice of entry will probably be about .75 or less, .815 being that for a clear entry to a short cylindrical tube. The approximate inclination is $\frac{100 \times 12}{35} = 1$ in 34.3; but as a portion of the fall must be absorbed by the velocity and orifice of entry, we may assume for the present that the inclination is 1 in 35. With this inclination and the mean radius $\frac{1\frac{1}{2}}{4} = \frac{3}{8}$ inches, we find the mean velocity from TABLE VIII. to be 38.06 inches. Now when the coefficient due to the orifice of entry and velocity is .75, we find from TABLE II. the head due to this velocity to be $3\frac{3}{8}$ inches nearly, whence $35 - 3\frac{3}{8} = 31\frac{1}{8} = 31.625$ inches is the height due to friction, and $\frac{100 \times 12}{31.625}$ equals 1 in 37.9, the inclination, very nearly. With this new inclination we find, as before, from TABLE VIII. the mean velocity of discharge to be now 36.35

inches ; and by repeating the operation we shall find the velocity to any degree of accuracy in accordance with the table, and the shorter the pipe is, the oftener must it be repeated. The height due to 36.35 inches taken from TABLE II. as before, with a coefficient of .750, is $3\frac{1}{8} = 3.125$ inches. The corrected fall due to the friction is now $35 - 3.125 = 31.875$, and $\frac{1200}{31.875}$ equal 1 in 37.6, the corrected inclination. With this inclination we find the corrected velocity to be now 36.53 inches per second. It is not necessary to repeat the operation again. The discharge determined from TABLE IX. is as follows :—

	Inches.		Cubic feet.
For a velocity of	30.00	the discharge is	1.844
"	6.00	"	.308
"	.50	"	.031
"	.03	"	.002
"	36.53	"	2.249

The experimental discharge found by Mr. Provis was 2.264 cubic feet per minute in one experiment, and 2.285 in another. The discharge from the shorter pipes may be found in a similar manner, and we place the results alongside the experimental ones given in the work referred to below* in the following short table :—

* "Transactions of the Institution of Civil Engineers," vol. ii, p. 203. "Experiments on the Flow of Water through small Pipes." By W. A. Provis. The small Tables in Sections VI. and VIII. of this edition give at once the coefficient to be multiplied by $\sqrt{2gH}$, or $8\sqrt{H}$, to find the velocity when the ratio of the diameter to the length of the pipe is known. They will be found of great advantage in calculating directly the velocity from short pipes. For long pipes, see the Tables pp. 42 and 43.

EXPERIMENTAL AND CALCULATED DISCHARGES FROM SHORT PIPES.

Length of pipe, in feet.	Head, in inches.	Observed discharge, in cubic feet.	Velocity per second.	Head due to the orifice and velocity.	Head due to friction.	Hydraulic inches.	Calculated velocity.	Calculated discharge.
100	35	2.275	37.089	3½	31½	37.0	30.53	2.242
80	35	2.500	40.750	3½	31½	30.8	41.18	2.521
60	35	2.874	46.840	5	30	24.0	48.02	2.946
40	35	3.504	57.115	7½	27½	17.5	58.50	3.500
20	35	4.528	73.801	12½	22½	10.7	78.61	4.824

The velocities in the fourth column have been calculated by the writer from the observed quantities discharged, from which the height due to the orifice of entry and velocity in column 5 is determined, and thence the quantities in the other columns as above shown. The differences between the experimental and calculated results are not large, and had we used a lesser coefficient than .750 for calculating the reduction of head due to the velocity, stop-cock, and orifice of entry, say .715, the calculated results, and those in all of Mr. Provis's experiments in the work referred to, would be nearly identical.*

EXAMPLE 17.—*It is proposed to supply a reservoir near the town of Drogheda with water by a long pipe, having an inclination of 1 in 480, the daily supply to be 80,000 cubic feet; what must the diameter of*

* In a late work, "Researches in Hydraulics," the author is led into a series of mistakes as to the accuracy of Du Buat's and several other formulæ, from neglecting to take into consideration the head due to the velocity and orifice of entry when testing them by the experiments above referred to.

the pipe be? The discharge per minute must be $\frac{80,000}{1440} = 56^*$ cubic feet, nearly. Assume a pipe whose "mean radius" is 1 inch, or diameter 4 inches, and the velocity per second found from TABLE VIII. will be 14.41 inches. We then have from TABLE IX.,

	Inches.		Cubic feet.
For a velocity of	10.00	a discharge of	4.3031
" "	4.00	" "	1.745
" "	.40	" "	.175
" "	.01	" "	.004
<hr/>			
" "	14.41	" "	6.287

The discharge from a pipe 4 inches in diameter would be therefore 6.287 cubic feet per minute. We then have

$4^{\frac{5}{2}} : d^{\frac{5}{2}} :: 6.287 : 56$, or $1 : d^{\frac{5}{2}} :: .196 : 56 :: 1 : 286$; therefore $d^{\frac{5}{2}} = 286$, and $d = 9.61$ inches, nearly, as may be found from TABLE XIII., &c. This is nearly the required diameter. It is to be observed that the diameters thus found will not always agree exactly with those found from Du Buat's or other formulae, nor with each other, because the discharges are not strictly as $d^{\frac{5}{2}}$; but in practice the difference is immaterial, and the approximative value thus found can be easily corrected. If we assumed a pipe whose diameter is 1, the operation would have been more simple; for the velocity would then be, TABLE VIII., at the given inclination, 6.4 inches; and the discharge .175 cubic feet, TABLE IX. Hence we get $d^{\frac{5}{2}} =$

* Hydraulic Tables, Weale, 1854, give at once this discharge for a pipe between 0 and 10 inches diameter, also the TABLE, p. 49.

$\frac{56}{.175} = 320$, and, therefore, TABLE XIII., $d = 10$ inches nearly, which differs about half-an-inch from the former value, 9.6 inches, found by assuming a pipe of 4 inches to calculate from. It is necessary to understand that different results must be expected, in working from practical formulae, for different operations. When once an approximative value is obtained, it can be easily corrected to any required degree of accuracy.

Again the velocity in inches per second, from a cylindrical pipe 6 inches in diameter, is nearly equal to the discharge in cubic feet per minute; and as $6^{\frac{5}{2}} = 88.2$, we have $88.2 : d^{\frac{5}{2}} ::$ the velocity in inches per second from a 6-inch pipe : the discharge per minute from a pipe whose diameter is d . Hence this proportion would enable us to find, very nearly, the discharge from the diameter and fall; or the diameter from the discharge and fall by finding the velocity only, due to a 6-inch pipe. See TABLE pp. 42 and 43.

EXAMPLE 18.—*The area of a channel is 50 square feet, and the border 20.6 feet; the surface has an inclination of 4 inches in a mile; what is the mean*

velocity of discharge? $\frac{50}{20.6} = 2.427$ feet $= 29.124$ inches is the hydraulic mean depth; and we get from Table VIII. $12.03 - \frac{1.30 \times .876}{6} = 12.03 - .19 = 11.84$ inches per second for the required velocity. Though this velocity will be found under the true value for straight clear channels, it will yet be more correct for ordinary river courses, with bends

and turns, of the dimensions given, than the velocity found from equation (114). For a straight clear channel of these dimensions, Watt found the mean velocity to be 13·5 to 14 inches; that is to say, 17 at top, 10 at bottom, and 14 in the middle. Our formula $v = 140 (r s)^{\frac{1}{2}} - 11 (r s)^{\frac{1}{2}}$ gives $v = 1·143$ feet, or nearly a mean of these two.

EXAMPLE 19.—*A pipe 5 inches in diameter, 14,637 feet in length, has a fall of 44 feet; what is the discharge in cubic feet per minute?* The inclination is $\frac{14,637}{44} = 332·7$, and mean radius $\frac{5}{4} = 1\frac{1}{4}$. We then find from TABLE VIII. the velocity equal to $19·81 + \frac{·41 \times 4·8}{12·5} = 19·81 + ·16 = 19·97$, or 20 inches per second very nearly; and by TABLE IX. the discharge in cubic feet per minute is, as before found to be, 13·635. The TABLE, p. 42, gives, by inspection, 13·6 feet.

EXAMPLE 20.—*What is the velocity of discharge from a pipe or culvert 4 feet in diameter, having a fall of 1 foot to a mile?* Here $s = \frac{1}{5280}$, and $r = 1$ foot. We then find the velocity of discharge from TABLE VIII. to be 14·09 inches, equal to 1·174 feet per second. By calculating from the different formulas referred to below, we shall find the velocities, when $r s = ·0001894$, and $\sqrt{r s} = ·01376$, as follows.

	Velocity in feet.
Reduction of Du Buat's formula equation (81)	1·174
„ Girard's do. (Canals with aquatic plants and very slow velocities) „ (88)	·624
„ Prony's do. (Canals) „ (88)	1·201

		Velocity in feet.
Reduction of Prony's formula (Pipes) . . .	equation (90)	1.257
" Prony's do. (Pipes and Canals) . . .	" (92)	1.229
" Eytelwein's do. (Rivers) . . .	" (94)	1.200
" Eytelwein's do. (Rivers) . . .	" (96)	1.285
" Eytelwein's do. (Pipes) . . .	" (98)	1.364
" Eytelwein's do. (Pipes) . . .	" (100)	1.350
" Dr. Young's do.	" (104)	1.120
" *D'Aubuisson's do. (Pipes) . . .	" (109)	1.259
" *D'Aubuisson's do. (Rivers) . . .	" (111)	1.199
" The writer's do. (Clear straight Channels with small velocities)	" (114)	1.208
" Weisbach's do. (Pipes) . . .	" (119)	1.285
" The author's, for Pipes and Rivers	"	1.205

We have calculated this example from the several formulæ above referred to, whether for pipes or rivers, in order that the results may be more readily compared. The formula from which the velocities and tables for the discharges of rivers are usually calculated is, for measures in feet, $v = 94.17 \sqrt{r} \text{ s.}$ This gives the mean velocity, for the foregoing example, equal to 1.205 feet per second. This is the same as is found from my general formula for all velocities; but the particular expression, $v = 99.17 \sqrt{r} \text{ s.}$ is only suited for velocities of about 15 inches per second; the results found from it for lesser velocities are too much, and for higher velocities too little, if bends and curves be allowed for separately. For ordinary practical purposes the result of Du Buat's general formula, equation (81), may be safely adopted; and we have, accordingly, preferred retaining the results in TABLE VIII. calculated for our first edition from it, notwithstanding the greater accuracy and simplicity of our own general equation (119 A)

* These two formulæ of D'Aubuisson's are, simply, adoptions of Eytelwein's and Prony's.

for the velocity in pipes and rivers, viz., $v = 140 (rs)^{\frac{1}{4}}$
 $- 11 (rs)^{\frac{1}{3}}$.

Dr. Young's formula gives lesser results for rivers and large pipes than Du Buat's, but they are too small unless when the curves and bends are numerous and sudden. Girard's formula (86) is only suited for small velocities in canals containing aquatic plants, and it is entirely inapplicable to rivers or regular channels for conveyance of water. A knowledge of various formulæ, and their comparative results, applied to any particular case, will be found of great value to the hydraulic engineer, and the differences in the results show only an amount of error that may be expected in all practical operations, and which becomes of less importance when we consider that by increasing the dimensions of a channel every way, by only one-third, we shall more than double its discharging power. See TABLE XIII.

EXAMPLE 21.—*Water flowing down a river rises to a height of $10\frac{1}{2}$ inches on a weir 62 feet long; to what height will the same quantity of water rise, on a weir similarly circumstanced, 120 feet long?* $\frac{62}{120} = .517$,

nearly. In TABLE X. we find, by inspection, opposite to .517, the ratio of the lengths, the coefficient .644, rejecting the fourth place of decimals; whence $10\frac{1}{2} \times .644 = 6.76$ inches, the height required. When the height is given in inches it is not necessary to take out the coefficient to further than two places of decimals.

EXAMPLE 22.—*The head on a weir 220 feet long is 6 inches; what will the head be on a weir 60 feet long, similarly circumstanced, the same quantity of water*

flowing over each? $\frac{60}{220} = .273$. As this lies between .27 and .28, we find from TABLE X. the coefficient .4208; hence $\frac{6}{.4208} = 14.26$ inches, the head required.

TABLE X. will be found equally applicable in finding the head above the pass into weir basins, and above contracted water channels. See SECTION X.

EXAMPLE 23.—*A river channel 40 feet wide and 4.5 feet deep is to be altered and widened to 70 feet; what must the depth of the new channel be so that the surface inclination and discharge shall remain unaltered?* In "TABLE XI., OF EQUALLY DISCHARGING RECTANGULAR CHANNELS," we find opposite to 4.54, in the column of 40 feet widths, 3 in the column of 70 feet widths, which is the depth required in feet.

EXAMPLE 24.—*It is necessary to unwater a river channel 70 feet wide and 1 foot deep, by a rectangular side cut 10 feet wide; what must the depth of the side cut be, the surface inclination remaining the same as in the old channel?* In TABLE XI. we find 4.5 feet for the required depth. When the width of a channel remains constant, the discharge varies as $\sqrt{rs} \times d$, in which d is the depth; and when the width is very large compared with the depth, the hydraulic mean depth r approximates very closely to the depth d , and therefore $d = r$; consequently the discharge then varies as $d^{\frac{3}{2}} \times s^{\frac{1}{2}}$, and when the discharge is given $d^{\frac{3}{2}}$ must vary inversely as $s^{\frac{1}{2}}$; or more generally $dr^{\frac{3}{2}}$ must vary inversely, as $s^{\frac{1}{2}}$, when the width and discharge remain constant.

In narrow cuts for unwatering, it is prudent to make the depth of the water half the width of the

cut very nearly, when local circumstances will admit of these proportions; for then a maximum effect will be obtained with the least possible quantity of excavation; but for rivers and permanent channels the proper relation of the depth to the width must be regulated by the principles referred to in SECTION IX.

TABLE XI. is equally applicable, whether the measures be taken in feet, yards, or any other standards whatever.

EXAMPLE 25.—*A new river channel is to have a fall of eighteen inches in a mile, and must discharge 18,700 cubic feet per minute, what shall the dimensions be?* In TABLE XII., in the column of 18 inches per mile, we shall find opposite to 18,766, that a primary channel 70×2.125 will be sufficient; and opposite to 2.125 in TABLE XI. we shall find the equivalent rectangular channels 60×2.37 ; 50×2.70 ; 40×3.19 ; 35×3.52 ; 30×3.96 ; 25×4.61 ; 20×5.58 ; 15×7.29 ; and 10×11.37 , to select from. If the sides shall have any given slopes, the discharge will not be practically affected as long as the depth and area of the rectangular channel and the one with sloping banks remain the same. See SECTION IX.

EXAMPLE 26.—*A pipe 100 feet long and 1 inch in diameter has a head of 150 feet over the lower end, what will be the discharging velocity?* Here $r = .020833$ in feet, and $s = 1.5$, therefore $rs = .03125$. Hence by formulæ (119A) $v = 140 \times (.03125)^{\frac{1}{2}} - 11 \times (.03125)^{\frac{1}{2}} = 140 \times .1766 - 11 \times .1766 = 24.724 - 3.465 = 21.259$ feet per second. If allowance is required for the orifice of entry, the velocity is corrected as follows. A square orifice of entry has a coefficient of .815. The head due to this coefficient for a

velocity of about $20\frac{1}{2}$ feet, or 246 inches, is about 10 feet, TABLE II.—The head due to friction is therefore $150 - 10 = 140$ feet, and $s = \frac{100}{140} = 1.4$; $r s$ now becomes $1.4 \times .020833 = .02917$. Hence $v = 140 \sqrt{r s} - 11 \sqrt[3]{r s}$ now becomes $140 \times .171 - 11 \times .308$ nearly, equal to $23.940 - 3.388 = 20.552$ feet, the velocity for a square junction.

EXAMPLE 27.—*A sewer 9 feet in diameter has a fall of 2 feet per mile, what will be the velocity and discharge of water flowing through it when full?*

Here $r = 2.25$ and $s = \frac{1}{2640}$, therefore $r s = .0008523$, $(r s)^{\frac{1}{2}} = .02919$ and $(r s)^{\frac{1}{3}} = .0948$; and by formula (119A) we have $v = 140 (r s)^{\frac{1}{2}} - 11 (r s)^{\frac{1}{3}} = 140 \times .02919 - 11 \times .0948 = 4.0866 - 1.0428 = 3.0438$ feet per second. Hence the discharge per minute is $9^2 \times .7854 \times 3.0438 \times 60 = 63.62 \times 182.6 = 11,617$ cubic feet nearly. The velocity from a circular pipe or sewer is however greatest when the circumference is open for about $78\frac{1}{2}$ degrees at the top, but the velocity of sewage matter would not be equal to that of water. It would vary according to the dilution in the sewer, and 50 per cent. should be allowed, at least, in deduction, unless the dilution be very considerable.

The TABLE for the values of $r s$ and v , calculated from the formula (119 A) SEC. VIII., will give the velocity at once when $r s$ is known, and $r s$ when the velocity is known, from the latter of which a definite value of r or s can be fixed upon, when the other may be found, by an operation of simple division.

EXAMPLE 28.—*Water is to be pumped through a*

pipe 3000 feet long and 2 feet in diameter, with a velocity not exceeding 4 feet per second, what head must be allowed extra for friction in the pipe in calculating horse power? We shall find from our TABLE of the values of the velocity and product of the hydraulic mean depth and hydraulic inclination, given near the conclusion of SECTION VIII., that for a velocity of 4 feet per second $r s = .00142$. The diameter of the pipe is 2 feet, therefore $r = .5$, whence $s = \frac{.00142}{.5} = .00284$, and as the length of the pipe is 3000 feet we get $3000 \times .00284 = 8.52$ feet, the head required. The TABLE, p. 43, would give 9.6 feet nearly, which corresponds with Du Buat's formula. If the velocity in the pipe was 10 feet instead of 4 feet per second, then, from our table, $r s = .007576$, and $\frac{r s}{r} = s = \frac{.007576}{.5} = .015152$, and, therefore, $h = l s = 3000 \times .015152 = 45.456$ feet, or about six times as much as when the velocity was only 4 feet per second. The great loss of head arising from pumping at high velocities, from friction alone, is therefore apparent. Were the velocity double, or 8 feet per second, the head would be 30 feet nearly, or from the TABLE, p. 43, 31.6 feet.

For velocities of about 2.1 feet per second, v is equal to $100 \sqrt{r s}$, and for velocities of about $5\frac{1}{2}$ feet per second, $v = 110 \sqrt{r s}$. If l be the length of a pipe, we would find in the former case the head h in feet due to friction from the formula $h = \frac{l v^2}{10,000 r} = l s$; and in the latter $h = \frac{l v^2}{12,100 r} = l s$.

In questions of this kind, however, the diameter of

a pipe, d should be used in preference to the hydraulic mean depth, and as $d = 4r$ we shall find in the first case $h = \frac{lv^3}{2500d} = ls$; and in the second case, $h = \frac{lv^3}{3025d} = ls$.

If we wish to substitute the fall per mile for the hydraulic inclination, the first of these will again become $h = \frac{2.11v^3}{d} = ls$ for the loss per mile; and in the second case, $h = \frac{1.72v^3}{d} = ls$ for the loss per mile in feet.

If the velocity were so low as about 1 foot per second, then $v = 90 \sqrt{rs}$, and we should find $h = \frac{lv^3}{2025d} = ls$.

If for the inclination we substitute the fall per mile, this will become $h = \frac{2.61v^3}{d} = ls$ for the loss per mile in feet.

The loss of head varies in the same pipe with the velocity, and must be calculated differently, for small and for high velocities, when using the common formulæ. The TABLE near the end of SECTION VIII. will always give the correct value of rs , and thence $s = \frac{rs}{r}$.

In addition to the loss of head arising from friction, losses also occur from straight or curved bends, from diaphragms, from junctions, and from the orifices of entry and discharge; these must be determined separately for each case, as is shown hereafter, and added together and to the loss arising from friction, and the sum to the height the water is to be raised, before the full or total head for determining the power of an engine can be accurately known.

The TABLE on the next two pages will be found of great practical utility in solving all questions connected with water-pipes and sewers discharging fully-diluted sewage. In using it we can interpolate, by inspection, for intermediate diameters or inclinations. For greater diameters, divide those given by 4, and multiply the corresponding velocity found in the table by 2, and the corresponding discharge in the table by 32. If the object be to find the size of the channel, divide greater given velocities by 2, and multiply the diameters or inclinations found from the table by 4; also divide greater discharges by 32, and multiply the diameters found from the table by 4. The small auxiliary table, p. 43, embodied in the larger one, is of great use in making allowance for the velocity and orifice of entry in short pipes, before finding the head due to friction. The table also gives the different diameters and inclinations which, taken together, give the same velocity or discharge; and it enables us, from inspection, to select that relation of diameter to declivity which is best suited for other engineering aspects of the question. Taken in connexion with TABLES VIII., XI., XII., and XIII., this table completes the means of finding, by inspection, the dimensions, inclinations, velocities, and discharges of every class of water-channel or sewage-conduit required in engineering practice.

TABLE XIV. gives the comparative values of English and French measures; and TABLE XV. gives the weight, specific gravity, and ultimate strength and elasticity of various materials with which the engineer has to operate.

TABLE for finding, very nearly, the velocity and discharge from cylindrical water pipes, diameter and fall are given. Any two of the four quantities, the velocity, discharge, diameter, inclination, being given, the others can be found in the table from inspection.

Height in ft. due to friction per mile of length.	Mean hydraulic inclination of pipe or sewer.	THE VELOCITY IN INCHES PER SECOND is given in the first horizontal line of or fall; and the DISCHARGE IN CUBIC FEET PER MINUTE in the next								
		1 in. diameter	2 in. diameter	3 in. diameter	4 in. diameter	5 in. diameter	6 in. diameter	7 in. diameter	8 in. diameter	9 in. diameter
1	5280	1.7	2.5	3.2	3.8	4.2	4.7	5.1	5.5	5.9
		.05	.27	.70	1.0	2.0	4.6	8.8	14.2	20.8
2	2640	2.5	3.8	4.7	5.5	6.3	6.9	7.5	8.1	8.7
		.07	.41	1.2	2.4	4.3	6.8	10.1	14.2	19.2
3	1760	3.1	4.7	5.9	7.0	7.9	8.7	9.4	10.2	11.0
		.08	.51	1.4	3.0	5.3	8.6	12.6	17.7	23.8
4	1320	3.6	5.5	6.9	8.2	9.2	10.2	11.1	12.0	12.9
		.10	.60	1.7	3.6	6.3	10.0	14.8	20.8	27.9
5	1056	4.1	6.2	7.8	9.3	10.4	11.6	12.5	13.5	14.5
		.11	.68	1.9	4.0	7.1	11.3	16.8	23.6	31.6
6	880	4.6	6.9	8.7	10.3	11.5	12.8	13.9	15.0	16.1
		.12	.73	2.2	4.6	7.9	12.6	18.6	26.1	34.1
7	754	5.0	7.5	9.5	11.2	12.4	13.7	14.9	16.1	17.3
		.14	.82	2.3	4.9	8.6	13.7	20.2	28.5	37.8
8	660	5.4	8.1	10.2	12.0	13.3	14.6	15.9	17.2	18.5
		.15	.89	2.5	5.3	9.2	14.8	21.8	30.6	40.7
9	597	5.7	8.7	11.0	12.9	14.3	15.6	16.9	18.2	19.5
		.16	.95	2.7	5.6	9.9	15.8	23.3	32.8	43.8
10	528	6.1	9.2	11.6	13.7	15.1	16.4	17.7	19.0	20.3
		.17	1.00	2.9	6.0	10.2	16.7	24.7	34.8	46.8
11	480	6.4	9.7	12.3	14.4	15.8	17.1	18.4	19.7	21.0
		.17	1.1	3.0	6.3	11.1	17.7	26.1	36.7	49.0
12	440	6.7	10.2	12.9	15.2	16.6	17.9	19.2	20.5	21.8
		.18	1.1	3.2	6.6	11.6	18.6	27.4	38.6	51.6
13.2	400	7.1	10.8	13.6	16.0	17.4	18.7	20.0	21.3	22.6
		.19	1.2	3.3	6.9	12.3	19.6	28.9	40.7	54.7
15.1	350	7.7	11.6	14.7	17.2	18.6	19.9	21.2	22.5	23.8
		.21	1.3	3.6	7.5	13.2	21.2	31.2	43.9	58.0
17.6	300	8.4	12.7	16.0	18.8	20.2	21.5	22.8	24.1	25.4
		.23	1.4	3.9	8.2	14.4	23.1	34.1	47.8	63.7
21.1	250	9.4	14.1	17.8	20.9	22.3	23.6	24.9	26.2	27.5
		.28	1.5	4.4	9.2	16.0	25.7	37.8	53.3	71.6
26.4	200	10.6	16.0	20.2	23.8	25.2	26.5	27.8	29.1	30.4
		.29	1.7	5.0	10.4	18.3	29.2	43.1	60.6	81.7
35.2	150	12.5	19.0	23.9	28.1	30.5	31.8	33.1	34.4	35.7
		.34	2.1	5.9	12.3	21.6	34.6	50.9	71.6	96.1
52.8	100	15.9	24.1	30.4	35.7	40.4	44.8	49.2	53.6	58.0
		.43	2.6	7.4	15.6	27.3	43.8	64.6	90.0	120.0
58.7	90	16.9	25.6	32.3	38.0	42.7	47.1	51.5	55.9	60.3
		.46	2.8	7.9	16.6	29.1	46.0	68.7	96.7	131.7
66.0	80	18.1	27.6	34.6	40.7	45.8	50.2	54.6	59.0	63.4
		.49	3.0	8.5	17.8	31.2	49.9	73.7	103.7	139.7
75.4	70	19.6	30.0	37.5	44.1	49.0	53.4	57.8	62.2	66.6
		.53	3.2	9.2	19.2	33.8	54.1	79.8	112.3	151.7
88.0	60	21.5	32.6	41.1	48.3	54.4	58.8	63.2	67.6	72.0
		.59	3.6	10.1	21.1	37.1	59.3	87.6	123.2	167.7
105.6	50	24.0	36.4	45.8	53.9	60.7	65.1	69.5	73.9	78.3
		.65	4.0	11.2	23.5	41.3	66.3	97.0	137.4	187.1
132.0	40	27.4	41.6	52.5	61.7	69.4	73.8	78.2	82.6	87.0
		.75	4.5	12.8	26.9	47.3	75.7	111.7	157.2	211.7
176.0	30	32.6	49.5	62.5	73.4	82.6	91.7	100.8	109.9	119.0
		.89	5.4	15.3	32.0	56.3	90.0	132.9	187.1	251.7
212.2	25	36.4	55.3	69.8	82.0	92.2	102.4	111.5	120.6	129.7
		.99	6.0	17.1	35.8	62.9	100.0	148.4	208.9	281.7
264.1	20	41.7	63.3	79.9	93.8	105.0	117.3	127.2	137.1	147.0
		1.14	6.9	19.6	40.9	72.0	116.1	169.0	239.2	321.7
352.0	15	49.6	75.3	95.0	111.7	125.0	139.0	151.3	163.6	175.9
		1.35	8.2	23.3	48.3	85.6	137.0	202.2	284.0	383.7
528.0	10	63.3	96.0	121.2	142.4	160.2	178.0	192.0	207.0	221.0
		1.73	10.5	29.7	62.1	109.2	174.7	257.8	362.9	488.0

TABLE for finding, very nearly, the velocity and discharge from Cylindrical Water Pipes or Sewers, when the diameter and fall are given. Any two of the four quantities, the velocity, discharge, diameter, or inclination being given, the others can be found in THE TABLE from inspection.

Height in ft. due to fric- tion per mile of length.	Mean hy- draulic incli- nation of pipe or sewer	The VELOCITY IN INCHES PER SECOND is given in the first horizontal line for each inclination or fall; and the DISCHARGE IN CUBIC FEET PER MINUTE in the next following one.									
		12 in. diameter	14 in. diameter	16 in. diameter	18 in. diameter	20 in. diameter	22 in. diameter	24 in. diameter	26 in. diameter	28 in. diameter	30 in. diameter
1	One in 5280	6.8	7.4	8.0	8.5	8.9	9.4	9.8	10.3	10.7	11.1
2	2640	26.9	40.9	56.7	75.5	98.2	124.4	155.5	189.9	228.7	271.1
3	1760	40	58	82	110	144	183	228	278	336	400
4	1320	50	73	103	138	180	229	285	349	421	500
5	1056	58	86	120	163	212	269	335	410	494	588
6	880	66	97	136	184	240	305	380	464	560	665
7	754	73	108	151	203	265	337	420	514	620	737
8	660	80	118	165	222	289	368	458	560	676	804
9	587	86	127	177	239	312	397	493	604	728	866
10	528	92	135	192	255	333	424	527	645	778	925
11	480	97	144	201	271	354	450	560	685	826	982
12	440	103	152	213	286	373	475	590	723	871	1036
13.2	400	108	159	223	300	392	498	620	759	916	1089
15.1	350	114	168	236	317	414	526	655	801	966	1149
17.6	300	123	181	254	342	446	568	707	865	1043	1240
21.1	250	134	198	278	374	487	620	772	944	1139	1354
26.4	200	149	219	308	415	541	688	857	1048	1264	1504
35.2	150	169	249	350	472	616	783	975	1192	1438	1710
52.8	100	200	294	414	558	728	925	1152	1409	1700	2021
58.7	90	254	374	526	708	923	1174	1462	1788	2157	2565
66	80	270	398	559	753	982	1250	1556			
75.4	70	290	427	600	808	1054	1340	1669			
88	60	314	463	650	875	1141	1451	1807			
105.6	50	344	506	712	959	1251	1591	1981			
132	40	384	565	795	1070	1396	1775	2211			
176	30	467	665	908	1224	1597	2031	2529			
212.2	25	523	770	1082	1456	1900	2417	3010			
264.1	20	584	860	1208	1627	2122	2699	3362			
352	15	668	984	1383	1862	2430	3090	3848			
528	10	795	1171	1646	2216	2891	3677	4579			
		1014	1496	2098	2826	3686	4688	5839			

Heads in inches due to the velo- city & junction.	Velocities in inches, per second, due to heads.			
	Best Curved Junction.	Square Junction.	Flow at the junction projecting into the cistern.	
1	27.8	22.7	20	
2	39	32	28	
3	48	39	34	
4	56	45	40	
6	68	56	49	
8	79	64	56	
10	88	72	63	
12	96	78	69	
24	136	111	97	
36	167	136	119	
48	193	157	138	
60	215	176	154	
72	236	192	169	
84	255	208	182	
96	272	222	194	
108	289	235	207	
120	305	248	218	

SECTION II.

FORMULÆ FOR THE VELOCITY, AND DISCHARGE, FROM ORIFICES, WEIRS, AND NOTCHES.—COEFFICIENTS OF VELOCITY, CONTRACTION, AND DISCHARGE.—PRACTICAL REMARKS ON THE USE OF THE FORMULÆ.

The quantity of water discharged in a given time through an aperture of a given area in the side or bottom of a vessel, is modified by different circumstances, and varies more or less with the form, position, and depth of the orifice; but the discharge may be easily found, when we have determined the velocity and the contraction of the fluid vein.

VELOCITY.

If g be the velocity acquired by a heavy body falling from a state of rest for one second, *in vacuo*, then it has been shown by writers on mechanics, that the velocity v per second acquired by falling from a height h , will be

$$(1.) \quad v = \sqrt{2gh}.$$

The numerical value of g varies with the latitude; we shall assume $2g = 772.84$ inches $= 64.403$ feet. These will give for measures in inches,

$$v = 27.8 \sqrt{h},^* \text{ and } h = \frac{v^2}{772.84} = .001293v^2,$$

and for measures in feet,

* The velocities for different heights are given in the column number 1, TABLE II.

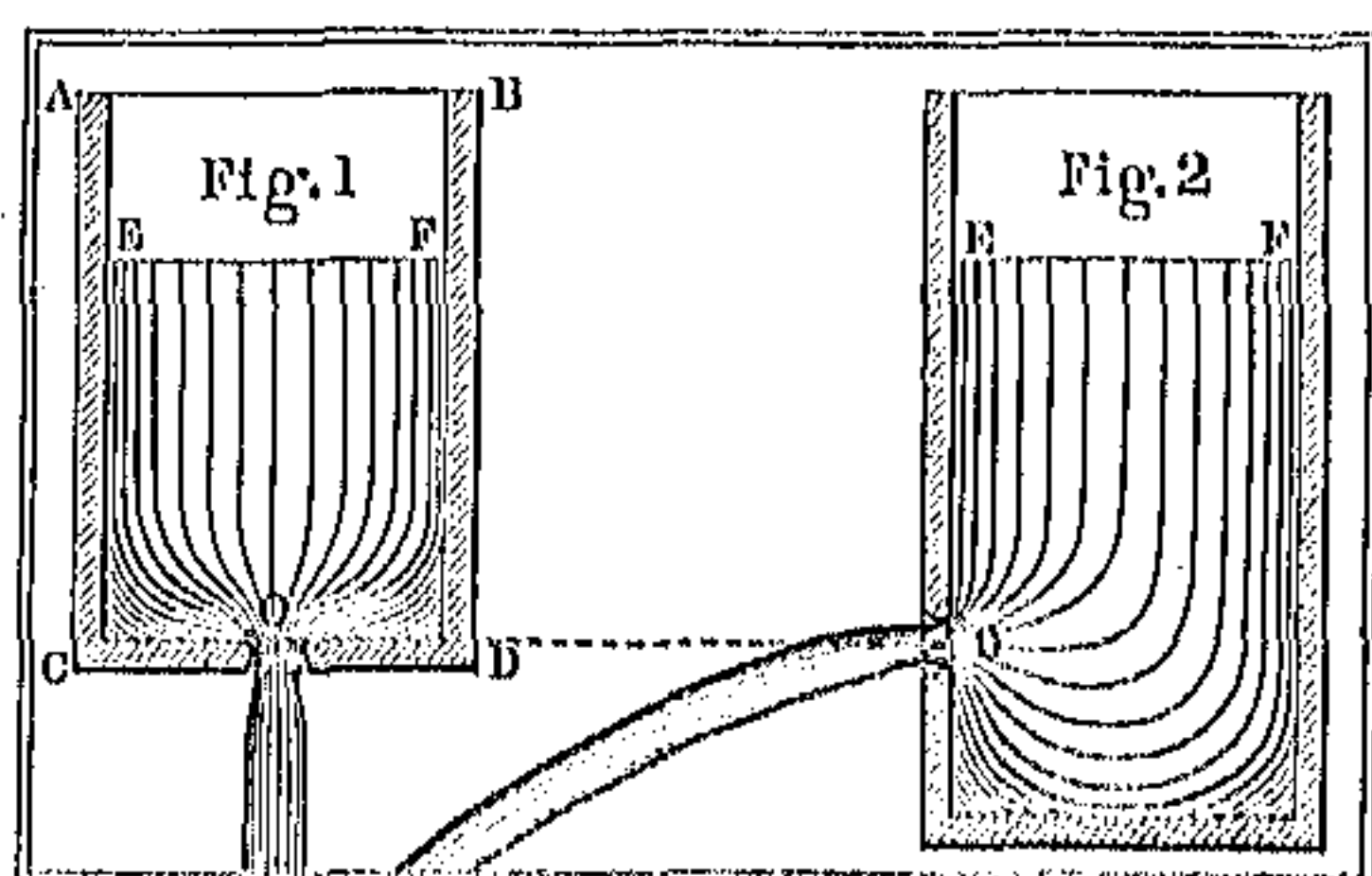
$$v = 8.025 \sqrt{h}, \text{ and } h = \frac{v^2}{64.403} = .01553 v^2.$$

If v be in feet, and h in inches, then

$$v = 2.317 \sqrt{h}, \text{ and } h = \frac{v^2}{5.367} = .1864 v^2.*$$

COEFFICIENT OF VELOCITY.

Let the vessel $A B C D$, Fig. 1, be filled with water to the level $E F$: then it has been found, by experi-



ment, that the velocity of discharge through a small orifice o , in a thin plate, at the distance of half the diameter outside it, in the *vena-contracta*, will be very nearly that due to a heavy body falling freely from the height h , of the surface of the water $E F$, above the centre of the orifice. The velocity of discharge

* The force of gravity increases with the latitude, and decreases with the altitude above the level of the sea, but not to any considerable extent. If λ be the latitude, and h the altitude, in feet, above the mean sea level, then we may, generally, take

$$g = 32.17 (1 - .0020 \cos 2\lambda) \times \left(1 - \frac{2h}{R}\right),$$

in which R , the radius of the earth at the given latitude is equal to

$$20887000 (1 + .0010 \cos 2\lambda).$$

determined by the equation $v = \sqrt{2gh}$, for falling bodies, is, therefore, called the "*theoretical velocity*." If we now put v_a for the actual mean velocity of discharge in the *vena-contracta*, and c_v for its ratio to the theoretical velocity v , we shall get $v_a = c_v v$; and by substituting for v , its value $\sqrt{2gh}$,

$$(2.) \quad v_a = c_v \sqrt{2gh},$$

c_v is termed "*the coefficient of velocity*;" its numerical value, at about half the diameter from the orifice, is about .974; and, consequently,

$$v_a = .974 \sqrt{2gh}.$$

This for measures in inches becomes

$$v_a = 27.077 \sqrt{h},*$$

and for measures in feet

$$v_a = 7.816 \sqrt{h}.$$

The orifice *o*, is termed an *horizontal orifice* in Fig 1, and in Fig 2 a *vertical* or *lateral orifice*. When

* The velocities for different heights calculated from this formula, are given in the column numbered 2, TABLE II. It has been latterly asserted in a *Blue Book* that theoretically $v_a = \frac{2}{3} \sqrt{2gh}$. It is not necessary here to combat this error, which confounds the discharge with its velocity, and a single practical fact, applicable only to a thin plate, with a theoretical principle. The experimental discharge approximates to $\frac{2}{3} \sqrt{2gh}$ multiplied by the area of the orifice; but the theoretical velocity $\sqrt{2gh}$ always approximates to the experimental velocity, or $.974 \sqrt{2gh}$, obtained immediately outside the orifice in the *vena-contracta*. It would be unnecessary to allude to this theory here if it were not supported and put forward by three eminent engineers whose authority may mislead others. *Vide* p. 4. Brief observations of Messrs. *Bidder*, *Hawksley*, and *Bazalgette* on the answers of the *Government Referees* on the METROPOLITAN MAIN DRAINAGE, ordered by the House of Commons to be printed 13th July, 1858.

VENA-CONTRACTA AND CONTRACTION.

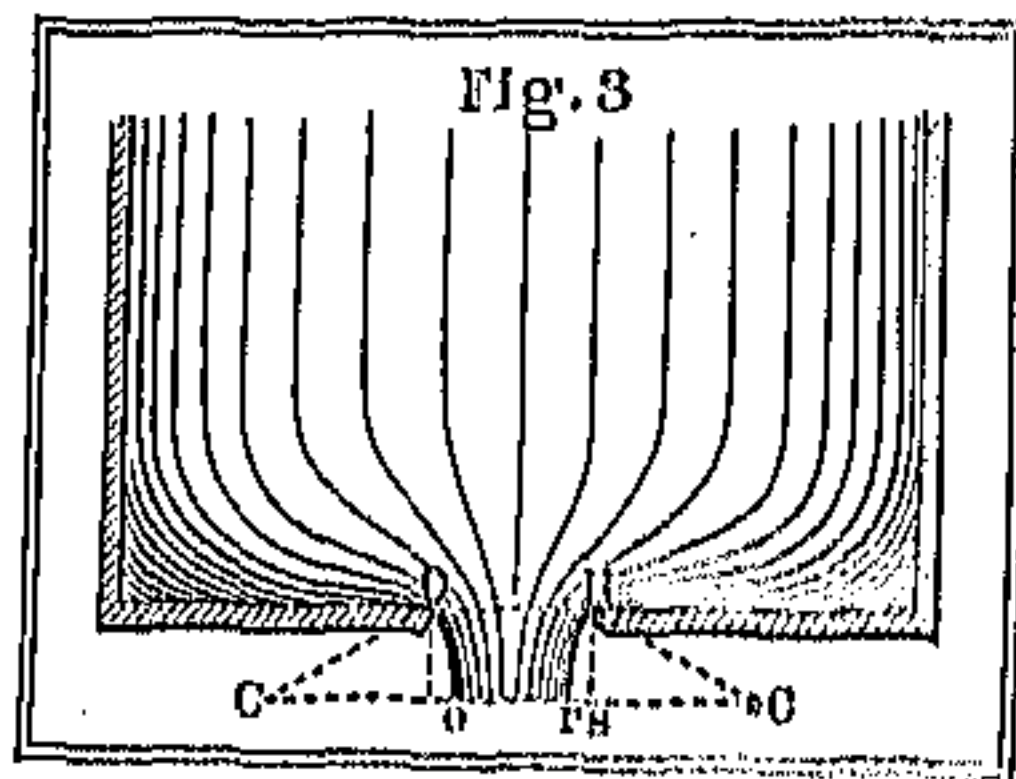
Newton makes it	1 :	·841,	(and, therefore, that of the areas as 1 :	·707
Poloni	1 :	·846 ·788	" "	1 : { ·7150 ·622
Borda	1 :	·809	" "	1 : ·6432
Michellotti	1 :	·8	" "	1 : ·64
Bossut	1 :	{ ·81 ·818	" "	1 : { ·676 ·669
Du Buat	1 :	·816	" "	1 : ·667
Venturi	1 :	·708	" "	1 : ·637
Eytelwein	1 :	·8	" "	1 : ·64
Bayer	1 :	·7854	" "	1 : ·617

Bayer's value for the contraction has been determined on the hypothesis, that the velocities of the particles of water as they approach the orifice from all sides, are inversely as the squares of their

distances from its centre; and the calculations made of the discharge from circular, square, and rectangular orifices, on this hypothesis, coincide pretty closely with experiments.

FORM OF THE CONTRACTED VEIN.

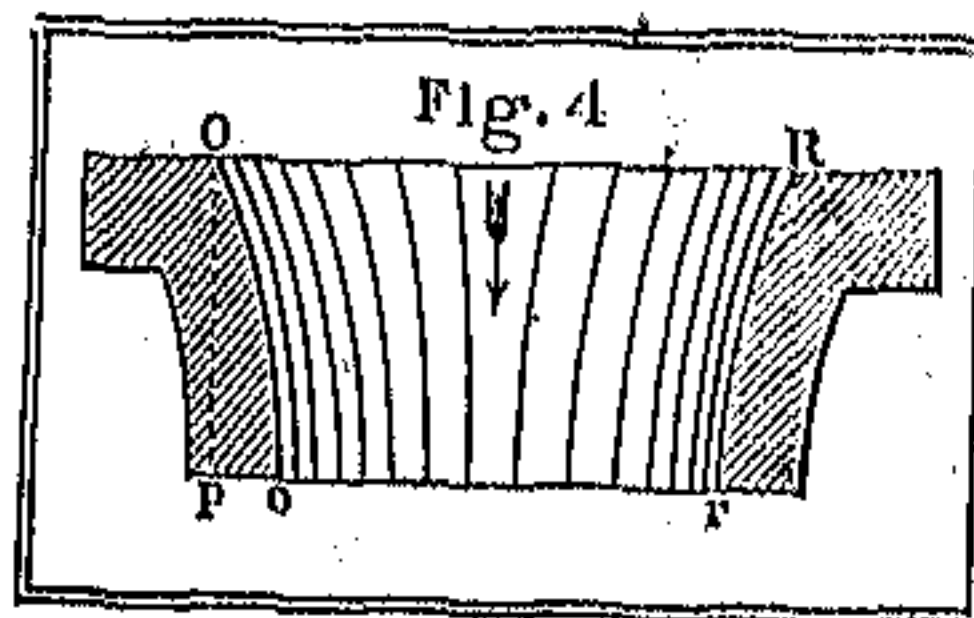
Let $OR = d$, Fig. 3, be the diameter of an orifice then at the distance $Rs = \frac{d}{2}$ the contraction is found to be greatest; we shall assume the contracted diameter $or = .7854 d$. If we suppose the fluid column between OR and or to be so reduced, that the curve lines Rr and oo shall become arcs of circles,



then it is easy to show from the properties of the circle, that the radius or must be equal to $1.22 d$. The mean velocity in the orifice, OR , is to that in the

vena-contracta, or , as $.617 : 1$; and the mouth piece, $Rroo$, Fig. 4, in which $op = \frac{1}{2} OR$, and $or = .7854 \times OR$, will give for the velocity of discharge at or , the *vena-contracta*,

$$v_a = .974 \sqrt{2gh} = 7.816 \sqrt{h},$$



the *vena-contracta*, and not that in the orifice itself,

in feet very nearly. In speaking of the velocity of discharge from orifices in thin plates, we always assume it to be the velocity in

which varies with the coefficient of discharge, unless in TABLE II., where the mean velocity in the latter, as representing $c_d \sqrt{2 g h}$, is also given.

COEFFICIENTS OF CONTRACTION AND DISCHARGE.

If we put A for the area of the orifice $o r$, Fig. 3, and $c_v \times A$ for that of the contracted section at $o r$, then c_v is called the "*coefficient of contraction*." The velocity of discharge v_d is equal to $c_v \sqrt{2 g h}$, equation (2). If we multiply this by the area of the contracted section $c_v \times A$, we shall get for the discharge

$$D = c_v \times c_c \times A \sqrt{2 g h}.*$$

It is evident $A \sqrt{2 g h}$ would be the discharge if there were no contraction and no change of velocity due to the height h ; $c_v \times c_c$ is therefore equal to the coefficient of discharge. If we call the latter c_d , we shall have the equation

$$(3.) \quad c_d = c_v \times c_c,$$

and hence we perceive that the "*coefficient of discharge*" is equal to the product of the coefficients of velocity and contraction. In the foregoing expression for the discharge D , h must be so taken, that the velocity at that depth shall be the mean velocity in the orifice A . *In full prismatic tubes the coefficients of velocity and discharge are equal to each other.*

* The expression $c_v c_c \sqrt{2 g h} = c_d \sqrt{2 g h}$ is the coefficient of the area A , and, consequently, represents the mean velocity in the orifice; the coefficient of which is, therefore, equal to c_d . The values of the velocity $c_d \sqrt{2 g h}$, for different heights and coefficients, are given in TABLE II.

MEAN AND CENTRAL VELOCITY.

In order to find the mean velocity of discharge from an orifice, it is, in the first instance, necessary to determine the velocity due to each point in its surface, and the discharge itself; after which, the mean velocity is found by simply dividing the area of the orifice into the discharge. The velocity due to the height of water at the centre of a circular, square, or rectangular orifice, is not strictly the mean velocity, nor is the latter in these, or other figures, that at the centre of gravity. When, however, an orifice is small in proportion to its depth in the water, the velocity of efflux determined for the centre approaches very closely to the mean velocity and, indeed, at depths exceeding four times the depth of the orifice, the error in assuming the mean velocity to be that at the centre of the orifice is so small as to be of little or no practical consequence, and for lesser depths it never exceeds 6 per cent. It is, therefore, for greater simplicity, the practice to determine the velocity from the depth $\frac{1}{2}$ of the centre of the orifice, unless in weirs or notches; and the coefficients of discharge and velocity in the following pages have been calculated from experiments on this assumption, unless it shall be otherwise stated.

DISCHARGES THROUGH ORIFICES OF DIFFERENT FORMS IN THIN PLATES.

The orifices which we have to deal with in practice are square, rectangular, or circular; and sometimes, perhaps, triangular or quadrangular in form. It will

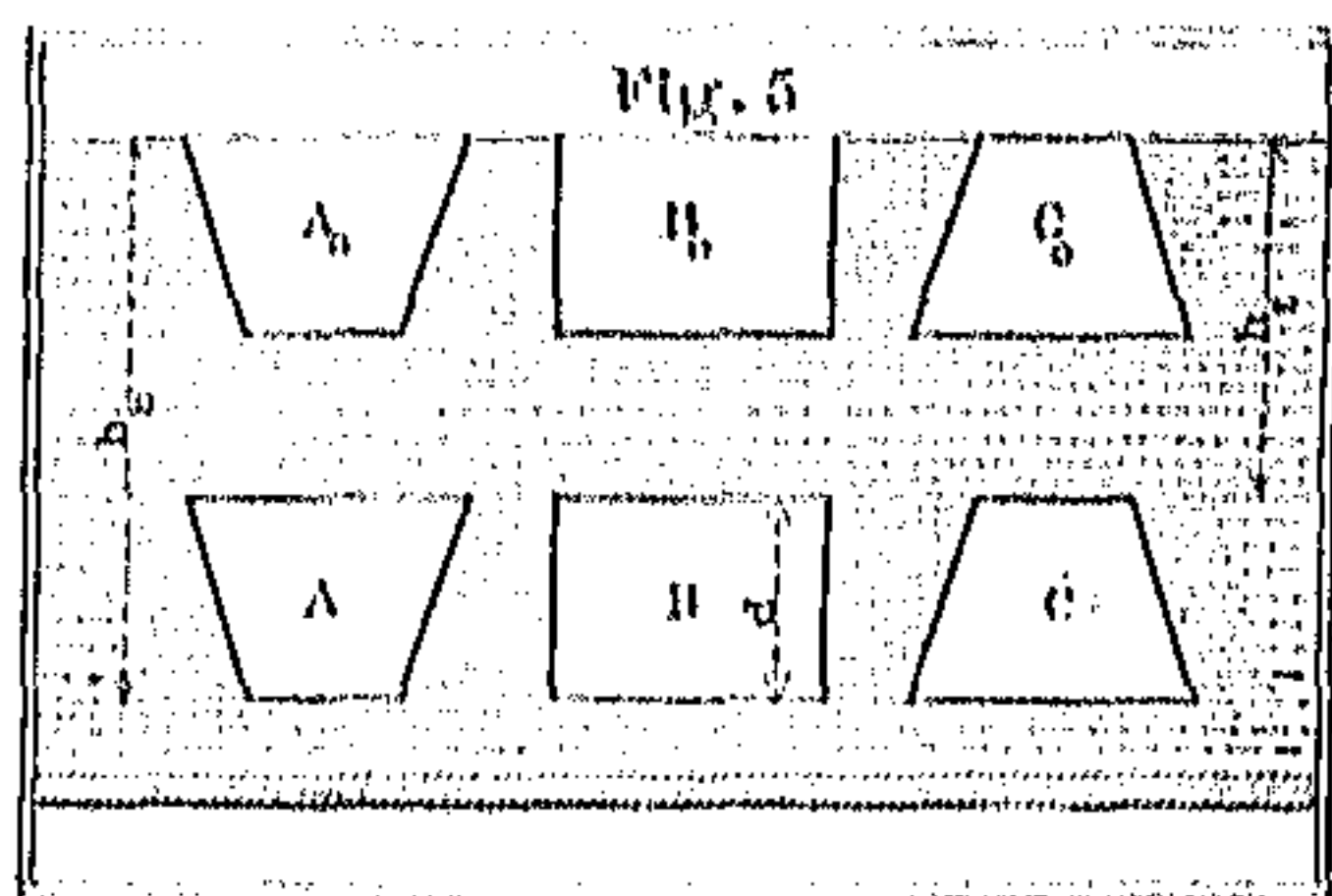
be necessary to give here only the theoretical expressions for the discharge and velocity for each kind of form, but as the demonstrations are unsuited to our present purposes we shall omit them.

TRAPEZOIDAL ORIFICES WITH TWO HORIZONTAL SIDES.

Put d for the vertical depth of an orifice, h_t for the altitude of pressure at top, above the upper side, and h_b for the altitude at bottom, above the lower side, we then get

$$h_b - h_t = d.$$

Let us also represent the top or upper side of the orifice Λ or α , Fig. 5, by l_t and the lower or bottom side by l_b , and put $\frac{l_t + l_b}{2} = l$.



Now, when $l_t = l_b$, the trapezoid becomes a parallelogram whose length is l and depth d ; and putting h for the depth to the centre of gravity, we get the equation

$$h_t + \frac{d}{2} = h_b - \frac{d}{2} = h.$$

The general expression for the discharge, ν , through a trapezoidal orifice, Λ , is then

$$(4.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} \left\{ l_b h_b^{\frac{3}{2}} - l_t h_t^{\frac{3}{2}} + \frac{2}{5} (l_t - l_b) \frac{h_b^{\frac{5}{2}} - h_t^{\frac{5}{2}}}{d} \right\},$$

in which c_d is the coefficient of discharge; and when the smaller side is uppermost as at c ,

$$(5.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} \left\{ l_b h_b^{\frac{3}{2}} - l_t h_t^{\frac{3}{2}} - \frac{2}{5} (l_b - l_t) \frac{h_b^{\frac{5}{2}} - h_t^{\frac{5}{2}}}{d} \right\}.$$

PARALLELOGRAMIC AND RECTANGULAR ORIFICES.

When $l_t = l_b = l$, the orifice becomes a parallelogram, or a rectangle, B , and we have for the discharge

$$(6.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} l \{ h_b^{\frac{3}{2}} - h_t^{\frac{3}{2}} \}.$$

NOTCHES.

When the upper sides of the orifices A , B , and C , rise to the surface as at A_o , B_o , and C_o , h_t becomes nothing, and we get, as $h_b = d$, for the trapezoidal notch A_o with the larger side up,

$$(7.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} d^{\frac{3}{2}} \left\{ l_b + \frac{2}{5} (l_t - l_b) \right\} \\ = \frac{2}{15} c_d \sqrt{2g} d^{\frac{3}{2}} (2 l_t + 3 l_b);$$

for the trapezoidal notch, C_o , with the smaller side up,

$$(8.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} d^{\frac{3}{2}} \left\{ l_b - \frac{2}{5} (l_b - l_t) \right\} \\ = \frac{2}{15} c_d \sqrt{2g} d^{\frac{3}{2}} (2 l_t + 3 l_b),$$

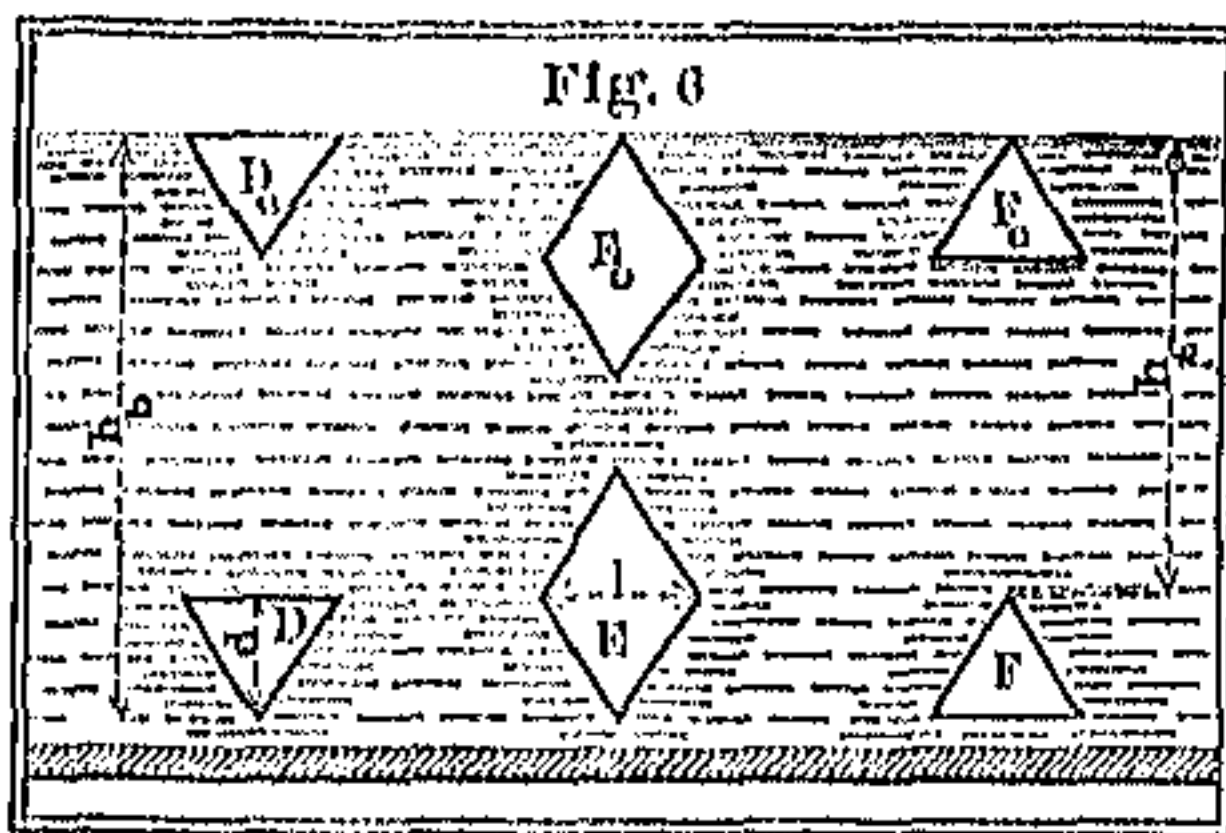
the same in form, but not in value, as the preceding equation; and for a parallelogramic or rectangular notch B_o ,

$$(9.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} l d^{\frac{3}{2}} = \frac{2}{3} c_d l d^{\frac{3}{2}} \sqrt{2g}.$$

It is easy to perceive that the forms of equations (4) and (5), and also of equations (7) and (8), are identical. The values for the discharge in equations (6) and (9) are equally applicable, whether the form of the orifice be a parallelogram or a rectangle, the only difference being in the value of the coefficient of discharge, c_d , which becomes slightly modified for each form of orifice.

TRIANGULAR ORIFICES WITH HORIZONTAL BASES, AND
RECTILINEAL ORIFICES IN GENERAL.

When the length of the lower side, $l_b = 0$, the orifice becomes a triangle, D , Fig. 6, with the base upwards.



In this case, equation (4) becomes

$$(10.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} l \left\{ \frac{2}{5} \times \frac{h_b^{\frac{5}{2}} - h_t^{\frac{5}{2}}}{d} - h_t^{\frac{3}{2}} \right\};$$

which gives the discharge through the triangular orifice, D .

When $l_t = 0$, in equation (5), the orifice becomes a triangle, F , with the base downwards; in this case, we find for the value of the discharge,

$$(11.) \quad D = c_d \sqrt{2g} \times \frac{2}{3} l_b \left\{ h_b^{\frac{3}{2}} - \frac{2}{5} \times \frac{h_b^{\frac{5}{2}} - h_t^{\frac{5}{2}}}{d} \right\}.$$

As any triangular orifice whatever can be divided into two others by a line of division through one of the angles parallel to the horizon; and as the discharge from the triangular orifice D or F is the same as for any other on the same base and between the same parallels, we can easily find, by such a division, the discharge from any triangle not having one side parallel to the horizon, and thence the discharge from any rectilineal figure whatever by dividing it into triangles.

If the triangle F be raised so that the base shall be on the same level with the upper side of the triangular orifice D ; if, also, the bases be equal, and also the depths, we shall find, by adding equations (10) and (11), and making the necessary changes indicated by the diagram,

$$(12.) \quad D = c_d \sqrt{2g} \times \frac{4}{15} \frac{l}{d} \{ h_b^{\frac{5}{2}} + h_t^{\frac{5}{2}} - 2 \times \overline{h_t + d}^{\frac{5}{2}} \}$$

for the discharge from a parallelogram E with one diagonal horizontal. Now this is the same as the discharge from any quadrilateral figure whatever, having the same horizontal diagonal, and also having the upper and lower angles on the same parallels, or at the same depths, as those of the parallelogram. If the orifices D , F , and E rise to the surface of the water, as at D_o , E_o , F_o , we shall then have for the discharge from the notch D_o ,

$$(13.) \quad D = c_d \sqrt{2g} \times \frac{4}{15} l_t d^{\frac{3}{2}};$$

which for a right angled triangle becomes

$$D = c_d \sqrt{2g} \times \frac{8}{15} d^{\frac{5}{2}}.*$$

For the discharge from the notch E_0 ,

$$(14.) \quad D = c_d \sqrt{2g} \times \frac{6}{15} l_b d^{\frac{3}{2}};$$

and for the discharge through the notch E_0 ,

$$(15.) \quad D = c_d \sqrt{2g} \times \frac{4l}{15} \{h_b^{\frac{5}{2}} - 2d^{\frac{5}{2}}\} = c_d \sqrt{2g} \times .9752 l d^{\frac{5}{2}}.$$

When the parallelogram E_0 becomes a square $l = 2d$, and hence,

$$(16.) \quad D = c_d \sqrt{2g} \times .9752 l^{\frac{5}{2}} \times \sqrt{\frac{1}{8}} = c_d \sqrt{2gl} \times .34478 l^2.$$

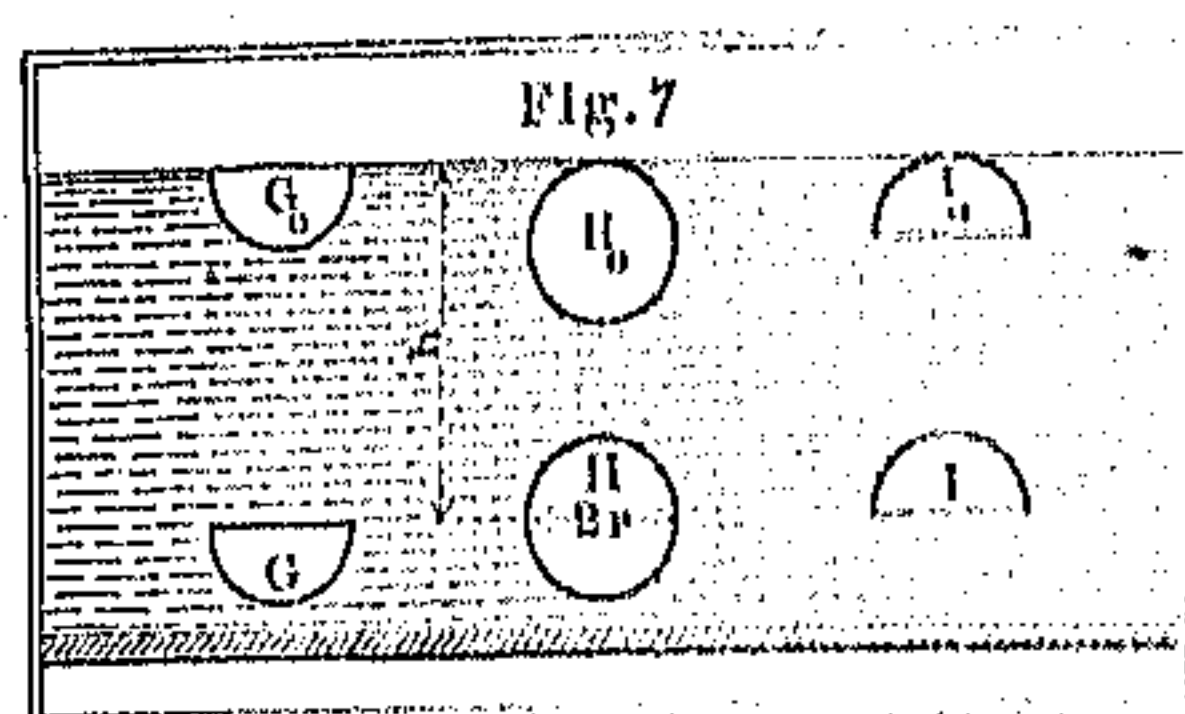
The foregoing equations will enable us to find an expression for the discharge from any rectilinear orifice whatever, as it can be divided into triangles, the discharge from each of which can be determined as already shown in the remark following equation (11.) The examples which we have given will be found to comprehend every form of rectilinear orifice which occurs in practice; but for the greater number of orifices, sunk to any depth below the surface, the

* In the Civil Engineer and Architect's Journal, 1858, p. 370, it is stated that Professor Thompson, Belfast College, gave at the British Association in Leeds for a right angled triangle, for discharges of from 2 to 10 cubic feet per minute, the expression $q = .317 \pi^{\frac{5}{2}}$, in which q is the quantity in cubic feet per minute, and π the head in inches. Now the above equation for a coefficient of .017 becomes, for inch measures, $D = 17.153 \times \frac{8}{15} d^{\frac{5}{2}} = 9.15 d^{\frac{5}{2}}$; or by multiplying by 60, and dividing by 1728, to reduce the discharge to feet per minute, we get $D = .317 d^{\frac{5}{2}}$, identically the same as Professor Thompson derived from his experiments. All sections of a triangular notch are similar triangles, and hence the advantage of a triangular-notch-gauge, where it can be used, as, probably, the coefficient remains constant. Professor Thompson, I believe, first drew attention to this.

discharge will be found with sufficient accuracy by multiplying the area by the velocity due to the centre.

CIRCULAR AND SEMICIRCULAR ORIFICES.

The discharge through circular and semicircular orifices in thin plates can only be represented by means of infinite series. Let us represent by s_1 the sum of the series



$$\left\{ \frac{1}{2} - \left(\frac{1}{2}, \frac{1}{4} \right) \frac{r^2}{h^2} - \left(\frac{1}{2}, \frac{1}{4}, \frac{3}{6}, \frac{5}{8} \right) \frac{r^4}{h^4} - \left(\frac{1}{2}, \frac{1}{4}, \frac{3}{6}, \frac{5}{8}, \frac{7}{10}, \frac{9}{12} \right) \frac{r^6}{h^6} - \&c. \right\} :$$

Let us also represent by s_2 the sum of the series

$$\frac{2}{3.1416} \left\{ \left(\frac{1}{2}, \frac{1}{8} \right) \frac{r^2}{h^2} + \left(\frac{1}{2}, \frac{1}{4}, \frac{3}{6} \right) \frac{r^4}{h^4} + \left(\frac{1}{2}, \frac{1}{4}, \frac{3}{6}, \frac{5}{8}, \frac{7}{10} \right) \frac{r^6}{h^6} + \&c. \right\} :$$

then the discharge from the semicircle a , Fig. 7, with the diameter upwards and horizontal, is

$$(17.) \quad D = c_d \sqrt{2gh} \times 3.1416 r^2 (s_1 + s_2).$$

And the discharge from the semicircle c , with the diameter downwards and horizontal, is

$$(18.) \quad D = c_d \sqrt{2gh} \times 3.1416 r^2 (s_1 - s_2).$$

If we put A for the area, we shall also have for the discharge from a circle b ,

$$(19.) \quad D = c_d \sqrt{2gh} \times 2As_1.$$

In each of these three equations (17), (18), and (19), h is the depth of the centre of the circumference below the surface, and r the radius.

When the orifices rise to the surface, we have for the discharge from a semicircular notch a_0 , with the diameter horizontal and at the surface,

$$(20.) \quad D = c_d \sqrt{2gr} \times .9586 r^2 = c_d \sqrt{2gr} \times .6103 A;$$

when the circumference of the semicircle is at the surface, and the diameter horizontal, as at 1_0 ,

$$(21.) \quad D = c_d \sqrt{2gr} \times \frac{4}{16} (\sqrt{128-7}) r^2 = c_d \sqrt{2gr} \times .7324 A;$$

when the horizontal diameter of the semicircle is uppermost, and at the depth r below the surface,

$$(22.) \quad D = c_d \sqrt{2gr} \times 1.8667 r^2 = c_d \sqrt{2gr} \times 1.1884 A;$$

and when the circumference of the entire circle is at the surface, as at u_0 ,

$$(23.) \quad D = c_d \sqrt{2gr} \times 3.0171 r^2 = c_d \sqrt{2gr} \times .9604 A.$$

If we desire to reduce equations (20), (21), and (22), to others in which the depth h of the centre of gravity from the surface is contained, we have only

to substitute $\frac{h}{.4244}$ for r in equation (20), and we shall get, for the discharge from a semicircle with the diameter at the surface,

$$(24.) \quad D = c_d \sqrt{2gh} \times .0367 A;$$

also, by substituting $\frac{h}{.5756}$ for r in equation (21), we get, for the discharge from a semicircle when the circumference is at the surface and the diameter horizontal,

$$(25.) \quad D = c_d \sqrt{2gh} \times .9653 A;$$

and when the horizontal diameter is uppermost, and at the depth r below the surface $r = \frac{h}{1.4244}$ and

$$(26.) \quad D = c_d \sqrt{2gh} \times .9957 A.$$

As A stands for the area of the particular orifice in each of the preceding expressions for the discharge, it must be taken of double the value, in equation (23) for instance where it stands for the area of a circle, that it is in equations (20), (21), or (23), where it represents only the area of a semicircle.

MEAN VELOCITY.

The mean velocity is easily found by dividing the area into the discharge per second given in the preceding equations. For instance, the mean velocity in the example represented in equation (9), is equal $\frac{2}{3} c_d \sqrt{2gd}$, which is had by dividing the area ld into the discharge; and in like manner the mean velocity in equation (23) is $.9604 c_d \sqrt{2gr}$.

PRACTICAL REMARKS ON THE DISCHARGE FROM CIRCULAR ORIFICES.

It has been shown, equation (19), that, for the discharge from a circle, we have

$$D = c_d \sqrt{2gh} \times 2 A s_1,$$

in which h is the depth of the centre, A the area, and s_1 the sum of the series

$$\left\{ \frac{1}{2} - \left(\frac{1}{2} \cdot \frac{1}{4} \right) \left(\frac{1}{2} \cdot \frac{1}{4} \right) \frac{r^2}{h^2} - \left(\frac{1}{2} \cdot \frac{1}{4} \cdot \frac{3}{6} \cdot \frac{5}{8} \right) \left(\frac{1}{2} \cdot \frac{1}{4} \cdot \frac{3}{6} \right) \frac{r^4}{h^4} - \&c. \right\} :$$

and it has also been shown, equation (23), that, when the circumference touches the surface, this value becomes

$$D = c_d \sqrt{2gr} \times .9604 \Lambda.$$

Now when h is very large compared with r , it is easy to perceive that $2s_1 = 1$, and hence

$$(27.) \quad D = c_d \sqrt{2gh} \times \Lambda.$$

As this is the formula commonly used for finding the discharge, it is clear, if the coefficient c_d remain constant, that the result obtained from it for D would be too large. The differences, however, for depths greater than three times the diameter, or $6r$, are practically of no importance; for, by calculating the values of the discharge at different depths, we shall find, when

$$(28.) \quad \left\{ \begin{array}{l} h = r, \text{ that } D = c_d \sqrt{2gh} \times .960 \Lambda; \\ h = \frac{5r}{4}, \text{ ,, } D = c_d \sqrt{2gh} \times .978 \Lambda; \\ h = \frac{8r}{3}, \text{ ,, } D = c_d \sqrt{2gh} \times .985 \Lambda; \\ h = \frac{7r}{4}, \text{ ,, } D = c_d \sqrt{2gh} \times .989 \Lambda; \\ h = 2r, \text{ ,, } D = c_d \sqrt{2gh} \times .992 \Lambda; \\ h = 3r, \text{ ,, } D = c_d \sqrt{2gh} \times .996 \Lambda; \\ h = 4r, \text{ ,, } D = c_d \sqrt{2gh} \times .998 \Lambda; \\ h = 5r, \text{ ,, } D = c_d \sqrt{2gh} \times .9987 \Lambda; \\ h = 6r, \text{ ,, } D = c_d \sqrt{2gh} \times .9991 \Lambda. \end{array} \right.$$

These results show very clearly that, for circular orifices, the common expression for the discharge $c_d \sqrt{2gh} \times \Lambda$ is abundantly correct for all depths

exceeding three times the diameter, and that for lesser depths the extreme error cannot exceed four per cent. in reduction of the quantity found by this formula. We shall show, hereafter, when discussing the value of c_d , that from the sinking of the surface, and perhaps other causes, the discharge at lesser depths is even larger than that exhibited by the expression $c_d \sqrt{2gh} \times A$, the value of the coefficient of discharge, c_d , being found to increase as the depths h decrease. In fact, the sides of the orifice, the rounding of the arrises, and the depth and position with reference to the sides of the vessel, and surface of the water, are of far greater practical importance than extreme accuracy in the mathematical formula.

PRACTICAL REMARKS ON THE DISCHARGE FROM RECTANGULAR ORIFICES.

It has been shown, equation (6), that the discharge from rectangular orifices, with two sides parallel to the horizon or surface of the water, is expressed by the equation

$$D = c_d \times \frac{2}{3} \sqrt{2g} \times l \{h_b^{\frac{3}{2}} - h_t^{\frac{3}{2}}\},$$

in which l is the horizontal length of the orifice, h_b the depth of water on the lower, and h_t the depth on the upper, side. As it is desirable in practice to change this form into a more simple one, in which the height h of the centre and depth d of the orifice only shall be included, we then have $h_b = h + \frac{d}{2}$ and

$h_v = h - \frac{d}{2}$. By substituting these values of h_v and h_s in the foregoing equations, and developing the result into a series, the terms of which, after the third, may be neglected, and putting A for the area $l d$, we shall find,

$$(29.) \quad D = c_d \sqrt{2 g h} \times A \left\{ 1 - \frac{d^2}{96 h^2} \right\} \text{very nearly.}$$

We have therefore for the accurate theoretical discharge

$$(30.) \quad D = c_d \sqrt{2 g h} \times \frac{2}{3} A \left\{ \frac{(h + \frac{1}{2} d)^{\frac{3}{2}} - (h - \frac{1}{2} d)^{\frac{3}{2}}}{d h^{\frac{1}{2}}} \right\};$$

for the approximate discharge

$$D = c_d \sqrt{2 g h} \times A \left\{ 1 - \frac{d^2}{96 h^2} \right\};$$

and for the discharge by the common formula

$$D = c_d \sqrt{2 g h} \times A.$$

When the head (h) is large compared with (d) the height of the orifice, each of the three last equations gives the same value for the discharge; but as the common expression $c_d \sqrt{2 g h} \times A$ is the most simple; and as the greatest possible error in using it for lesser depths does not exceed six per cent., viz. when the orifice rises to the surface and becomes a notch, it is evidently that formula best suited for practical purposes. The following table and equations will show more clearly the differences in the results as obtained from *the true, the approximate, and the common formulae*, applied to "lesser" heads; and they will also explain, to some extent, why "coefficients" determined from the common formula, and that used by Poncelet and Lesbros, should decrease as the orifice approaches the surface.

(31.)		
1	2	3
$h = \frac{d}{2},$	$D = c_d \sqrt{2gh} \times .9428 \Delta.$	$D = c_d \sqrt{2gh} \times .9583 \Delta.$
$h = \frac{5d}{8},$	„ „ „ $\times .9693 \Delta.$	„ „ „ $\times .9733 \Delta.$
$h = \frac{3d}{4},$	„ „ „ $\times .9796 \Delta.$	„ „ „ $\times .9815 \Delta.$
$h = \frac{7d}{8},$	„ „ „ $\times .9854 \Delta.$	„ „ „ $\times .9864 \Delta.$
$h = d$	„ „ „ $\times .9890 \Delta.$	„ „ „ $\times .9896 \Delta.$
$h = \frac{3d}{2},$	„ „ „ $\times .9953 \Delta.$	„ „ „ $\times .9954 \Delta.$
$h = 2d,$	„ „ „ $\times .9974 \Delta.$	„ „ „ $\times .9974 \Delta.$
$h = \frac{5d}{2},$	„ „ „ $\times .9983 \Delta.$	„ „ „ $\times .9983 \Delta.$
$h = 3d,$	„ „ „ $\times .9988 \Delta.$	„ „ „ $\times .9988 \Delta.$
$h = \frac{7d}{2},$	„ „ „ $\times .9991 \Delta.$	„ „ „ $\times .9991 \Delta.$
$h = 4d,$	„ „ „ $\times .9994 \Delta.$	„ „ „ $\times .9994 \Delta.$
$h = 10d,$	„ „ „ $\times .9999 \Delta.$	„ „ „ $\times .9999 \Delta.$

In the foregoing Table the first column contains the head at the centre of the orifice expressed in parts of its height d ; the second contains the values of the discharges according to equation (30); and the third column contains the approximate values determined from equation (29), the results in which are something larger than those in column 2, derived from the correct formula. The numerical coefficients of Δ , at every depth, are less in both than one, the constant coefficient according to the common formula. The latter, therefore (as in circular orifices), gives results exceeding the true ones, but the excess is in-

appreciable at greater depths than $h = 3d$, and for lesser depths than this the error cannot exceed six per cent. It may be useful to remark here, that when the orifice rises to the surface and becomes a notch, *the "centre of mean velocity" is at four-ninths of the depth, and the centre of gravity at two-thirds of the depth from the surface.* The former fraction is the square of the latter.

SECTION III.

EXPERIMENTAL RESULTS AND FORMULÆ.—COEFFICIENTS OF DISCHARGE.

We have heretofore dwelt but very partially on the numerical values of the general coefficient of discharge c_d . In order to determine its value under different circumstances more particularly, it will be now necessary to consider some of the experiments which have been made from time to time. These do not always give the same results, even when conducted under the same circumstances and by the same parties, and there appears to exist a certain amount of error, more or less, inseparable from the subject. The experiments with orifices in thin plates afford the most consistent results; but even here the differences are sometimes greater than might be expected. In many of the earlier experiments the value of the coefficient c_d comes out too large, which arises, very probably, from the orifices experimented with not being in thin plates, and partaking, more or less, of the nature of short tubes or mouth-pieces with rounded

arrises, which, we shall see, give larger coefficients than simple orifices. When an orifice is in the bottom of a vessel, it would appear more correct to measure the head from the surface to the *vena-contracta* than from the surface to the orifice itself; and as any error in measuring the head in any experiment must affect the value of the coefficient derived from such experiment, so as to increase it when the error is to make the head less, and *vice versa*, it appears that heads measured to an orifice in the bottom of a vessel, and not to the *vena-contracta*, must give larger coefficients from the experimental results than, perhaps, the true ones. The coefficients in the following pages have been almost all arranged and calculated, by the writer, from the original experiments.

In 1739 Dr. Bryan Robinson made some experiments on the discharge through small circular orifices, from one-tenth to eight-tenths of an inch in diameter, with heads of two and four feet*, which give the following coefficients.

COEFFICIENTS FROM DR. B. ROBINSON'S EXPERIMENTS.

Heads.	$\frac{1}{10}$ inch diameter.	$\frac{4}{10}$ inch diameter.	$\frac{5}{10}$ inch diameter.	$\frac{8}{10}$ inch diameter.
2 feet head	·708	·707	·701	·728
4 feet head	·708	·774	·705	·742

These results are pretty uniform, and the values from which they are derived are said to be "means taken from five or six experiments;" as values of c_d they are, however, too high. The apparatus made use of is not described; but it is probable, from the

* Helsham's Lectures, p. 390. Dublin, 1739.

results, that the plate containing the hole or orifice was of some thickness, and that the inner arris was slightly rounded. There is here, however, a very perceptible increase in the coefficients for the smaller orifices, but none for the smaller depth.

In a paper in the Transactions of the Royal Irish Academy, vol. ii. p. 81, read March 1st, 1788, Dr. Mathew Young determines the value of the coefficient for an orifice $\frac{2}{15}$ inch in diameter, with a mean head of 14 inches, to be .623. The manner in which this value is determined is very elegant, viz. by comparing the observed with the theoretical time of the water, in the vessel, sinking from 16 inches to 12 inches.

The following experiments by Michelotti, with circular orifices from 1 to about 3 inches diameter, and with from 6 to 23 feet heads, give for the mean value $c_d = .613$; and for square orifices of from 1 to 9 square inches in area, at like depths, the mean value of $c_d = .628$. The experiments are given in French feet and inches, according to which standard we have, in feet, $v = 7.77 \sqrt{h} \times t$, t being the time in seconds.* As the time of discharge in these ex-

* The value of $\sqrt{2gh}$, equation (1), for measures in French feet, is $7.77\sqrt{h}$, and for measures in French inches, $26.9\sqrt{h}$, g being equal to 30.2 feet, or 362.4 inches, French measure. One French foot is equal to 1.06578 English feet, and the inches preserve the same proportion. The resulting coefficients must be the same, whatever standards we make our calculations from. Many of the most valuable formulæ and experiments in hydraulics are given in French measures of the old style. As our object, however, in the present section, is simply to determine from experiment the relation of the experimental to the theoretical discharge, it is not necessary to reduce the experiments to other measures than those in the original; but the value of the force of

periments varies from ten minutes to an hour, and as the depths are considerable, the results must be looked upon as pretty accurate; and it is worthy of note that here the coefficients are larger for square than for circular orifices.

We may remark here in passing how universal the coefficients $\cdot 613$ to $\cdot 628$ are for all forms of orifices in thin plates, or with the outside arrises chamfered. Indeed, we may always use the coefficient $\cdot 62$ with certainty, for practical purposes, for every orifice of this kind, whether at the surface in the form of a notch, or at the sides or bottom of a vessel, if the section of the approaching water be large in proportion to the area of the discharging orifice or notch. By coefficient we of course here mean that decimal which, multiplied by the theoretical value, gives the practical result; and this is substantially the same for notches and orifices sunk below the surface, as will appear farther on. There appears to us, however, an utter want of accuracy in using the coefficient $\cdot 62$ or thereabouts in gauging for all orifices, weirs included, no matter what the thickness or form of the orifice or crest of a weir may be, or area of the approaching channel. These will cause the coefficient to vary from $\cdot 5$ to 1 or more, and hence the necessity for endeavouring to reduce this portion of our subject to rule.

gravity, g , must, of course, be taken in those measures with which the experiments were made. In the French decimal, or modern style, the metre is equal to $3\cdot 2809$ English feet, or $39\cdot 371$ inches. The tenth part of a metre is the decimetre, and the tenth part of the decimetre $\frac{1}{10}$ the centimetre, as the names imply. TABLE XIV. contains the weights and measures in general use in Great Britain and France, with their general ratios to each other.

COEFFICIENTS FROM MICHELETTI'S EXPERIMENTS.

Description and size of orifice, in French inches.	Depth of the centre of the orifice in French feet.	Quantity discharged in cubic feet.	Time of discharge in seconds.	Theoretical time, calculated from $\frac{D}{777.4\sqrt{h}}$	Resulting coefficient of discharge.
Square orifice, 3" x 3"	0.013	400.001	600	871.3	.010
	0.052	500.458	720	445.0	.010
	11.076	510.785	610	311.4	.010
	11.818	612.118	600	300.0	.011
	21.001	415.437	300	189.7	.012
	21.715	400.222	300	220.0	.013
Mean value of the coefficient; square orifice 3" x 3"011
Square orifice, 2" x 2"	0.025	320.800	300	504.	.000
	11.426	423.405	300	580.4	.015
	21.442	385.333	300	385.7	.013
Mean value of the coefficient; square orifice 2" x 2"010
Square orifice, 1" x 1"	0.757	158.540	1800	1585.	.028
	11.880	103.703	1440	880.0	.013
	21.507	502.041	3000	2240.0	.028
Mean value of the coefficient; square orifice 1" x 1"021
Circular orifice, 3" diameter	0.004	542.85	300	550.1	.011
	11.590	570.072	720	430.0	.010
	21.011	531.200	480	203.8	.012
Mean value of the coefficient; circular orifice 3" diameter ..					.011
Circular orifice, 2" diameter	0.785	488.087	1800	1108.1	.010
	11.722	580.535	1080	1010.4	.005
	21.003	575.480	1200	725.0	.005
Mean value of the coefficient; circular orifice 2" diameter ..					.009
Circular orifice, 1" diameter	0.875	247.354	3000	2227.	.019
	11.743	324.11	3000	2233.	.020
	22.014	444.535	3000	2237.2	.021
Mean value of the coefficient; circular orifice 1" diameter ..					.020

The experiments made by the Abbé Bossut, contained in the following table, give the mean value of c_d , for both circular and square orifices, equal to $\cdot616$ nearly; and it may be perceived that, for the small depth in the last experiment, the coefficient rises so high as $\cdot649$. These and other experiments led the

COEFFICIENTS FROM BOSSUT'S EXPERIMENTS.

Description, position, and size of orifice, in French inches.	Depth of the centre of the orifice in French inches.	Number of French cubical inches discharged per minute.	Theoretical discharge per minute, $D = 1014 A \sqrt{h}$.	Resulting coefficient.
Horizontal and circular, $\frac{1}{9}$ " diameter	140.832	2311	3700.8	$\cdot614$
Horizontal and circular, 1" diameter	140.832	9281	15043.3	$\cdot617$
Horizontal and circular, 2" diameter	140.832	37203	60173.1	$\cdot618$
Horizontal and rectangular, 1" \times $\frac{1}{4}$ "	140.832	2933	4788.4	$\cdot613$
Horizontal and square, 1" \times 1" ..	140.832	11817	19153.7	$\cdot617$
Horizontal and square, 2" \times 2" ..	140.832	47361	70614.6	$\cdot617$
Lateral and circular, $\frac{1}{4}$ " diameter ..	108.	2018	3293.3	$\cdot613$
Lateral and circular, 1" diameter ..	108.	8135	13173.3	$\cdot617$
Lateral and circular, $\frac{1}{2}$ " diameter ..	48.	1353	2105.5	$\cdot610$
Lateral and circular, 1" diameter ..	48.	5430	8782.2	$\cdot610$
Lateral and circular, 1" diameter ..	0.5833	928	968.	$\cdot649$

Abbé to construct a table of the discharges, at different depths, from a circular orifice 1 inch in diameter, from which we have determined the following table of coefficients. These increase, as the orifice

COEFFICIENTS DEDUCED FROM BOSSUT'S EXPERIMENTS.

Heads, in feet.	Coefficients.	Heads, in feet.	Coefficients.	Heads, in feet.	Coefficients.
1	$\cdot621$	6	$\cdot620$	11	$\cdot619$
2	$\cdot621$	7	$\cdot620$	12	$\cdot618$
3	$\cdot621$	8	$\cdot619$	13	$\cdot618$
4	$\cdot620$	9	$\cdot619$	14	$\cdot618$
5	$\cdot620$	10	$\cdot619$	15	$\cdot617$

approaches the surface, from $\cdot 617$ to $\cdot 621$; and at lesser depths than 1 foot other experiments show an increase in the coefficient up to $\cdot 650$. The experiments of Poncelet and Lesbros show, however, a reduction in the coefficients for square orifices $8'' \times 8''$ as they approach the surface from $\cdot 601$ to $\cdot 572$.

Brindley and Smeaton's experiments, with an orifice 1 inch square placed at different depths, give a mean

COEFFICIENTS CALCULATED FROM BRINDLEY AND SMEATON'S EXPERIMENTS.

1 foot head: orifice $1'' \times 1''$: coefficient $\cdot 680$	} mean $\cdot 637$.
2 feet head: orifice $1'' \times 1''$: coefficient $\cdot 685$	
3 feet head: orifice $1'' \times 1''$: coefficient $\cdot 648$	
4 feet head: orifice $1'' \times 1''$: coefficient $\cdot 632$	
5 feet head: orifice $1'' \times 1''$: coefficient $\cdot 632$	
6 feet head: orifice $\frac{1}{2}'' \times \frac{1}{2}''$: coefficient $\cdot 557$	

value for c_d of $\cdot 637$. The last experiment, with an orifice only $\frac{1}{2}$ inch by $\frac{1}{2}$ inch, gives so small a coefficient as $\cdot 557$ placed at a depth of 6 feet!

For notches 6 inches wide and from 1 to $6\frac{1}{2}$ inches deep, Brindley and Smeaton's experiments give the mean value of $c_d = \cdot 637$. The coefficients of discharge

COEFFICIENTS FOR NOTCHES, CALCULATED FROM BRINDLEY AND SMEATON'S EXPERIMENTS.

Ratio of the length to the depth.	Size of notches in inches.	Coefficients.	Ratio of the length to the depth.	Size of notches in inches.	Coefficients.
$\cdot 02$ to 1	$6 \times 0\frac{1}{2}$	$\cdot 633$	$3\cdot 7$ to 1	$6 \times 1\frac{1}{2}$	$\cdot 638$
$1\cdot 07$ to 1	$6 \times 5\frac{1}{2}$	$\cdot 571$	$4\cdot 4$ to 1	$6 \times 1\frac{3}{4}$	$\cdot 654$
$1\cdot 2$ to 1	6×6	$\cdot 600$	$4\cdot 8$ to 1	$6 \times 1\frac{1}{2}$	$\cdot 681$
$1\cdot 02$ to 1	$6 \times 3\frac{1}{2}$	$\cdot 602$	6 to 1	6×1	$\cdot 713$
$2\cdot 4$ to 1	$6 \times 2\frac{5}{16}$ *	$\cdot 630$	Mean value.		$\cdot 637$

* The depth is misprinted $2\frac{5}{16}$ inches in the Encyclopædias, the resulting coefficient for which would be $\cdot 508$ instead of $\cdot 630$ as above, for a depth of $2\frac{5}{16}$ inches.

for notches and orifices appear to differ as little from each other as those for either do in themselves. The results also show a general though not uniform increase in the coefficients for smaller depths.

Du Buât's experiments with notches 18.4 inches long, give the mean value of $c_d = .632$, which differs very little from the mean value determined from Brindley and Smeaton's experiments.

COEFFICIENTS FOR NOTCHES, CALCULATED FROM DU BUAT'S EXPERIMENTS.

Ratio of the length to the depth.	Size of notches in inches.	Coefficients.	Ratio of the length to the depth.	Size of notches in inches.	Coefficients.
2.72 to 1	18.4 × 6.753	.630	5.75 to 1	18.4 × 3.109	.624
3.94 to 1	18.4 × 4.605	.627	10.3 to 1	18.4 × 1.778	.648

Poncelet and Lesbros' experiments give the coefficients in the following table, for notches 8 inches

COEFFICIENTS FOR NOTCHES, BY PONCELET AND LESBROS.

Ratio of the length to the depth.	Size of notches in inches.	Coefficients.	Ratio of the length to the depth.	Size of notches in inches.	Coefficients.
.9 to 1	8 × 9	.577	3.93 to 1	8 × 2.4	.601
1 to 1	8 × 8	.585	5 to 1	8 × 1.6	.611
1.3 to 1	8 × 6	.590	6.7 to 1	8 × 1.2	.618
2 to 1	8 × 4	.592	10 to 1	8 × 0.8	.625
2.5 to 1	8 × 3.2	.595	20 to 1	8 × 0.4	.636

wide; the mean value of all the coefficients in these experiments is .603. Here the coefficients increase in every instance as the depths decrease, or as the ratio of the length of the notch to its depth increases. We shall have to refer to the valuable experiments made at Metz, on the discharge from differently-proportioned orifices immediately.

Rennie's experiments for circular orifices at depths

COEFFICIENTS FOR CIRCULAR ORIFICES, FROM RENNIE'S EXPERIMENTS.

Heads at the centre of each orifice in feet.	$\frac{1}{4}$ inch diameter.	$\frac{1}{2}$ inch diameter.	$\frac{3}{4}$ inch diameter.	1 inch diameter.	Mean values.
1	.671	.634	.644	.633	.645
2	.653	.621	.652	.610	.636
3	.600	.636	.632	.628	.630
4	.602	.620	.614	.584	.621
Means	.661	.629	.635	.616	.635

from 1 foot to 4 feet, and of diameters from $\frac{1}{4}$ inch to 1 inch, give the following coefficients. Here the increase in the coefficients for lesser orifices and at lesser depths exhibits itself very clearly, notwithstanding a few instances to the contrary. The mean value of the coefficient c_d derived from the whole, is .635. For small rectilinear orifices the coefficients were as follows:—

COEFFICIENTS FOR RECTANGULAR ORIFICES, FROM RENNIE'S EXPERIMENTS.

Heads at the centre of gravity, in feet.	Square orifice, 1 inch \times 1 inch.	Rectangular orifice, longer side horizontal, $2'' \times \frac{1}{2}''$.	Rectangular orifice, longer side horizontal, $1\frac{1}{2}'' \times \frac{3}{4}''$.	Equilateral triangle of 1 square inch, with base down.	Equilateral triangle of 1 square inch, with base up.
1	.617	.617	.603	"	.596
2	.635	.635	.608	"	.577
3	.606	.606	.606	"	.572
4	.593	.593	.593	.593	.593
Means	.613	.613	.632	.593	.585

The most valuable series of experiments of which we are possessed are those made at Metz, by Poncelet and Lesbros. They were made with orifices 8 inches wide, nearly, and of different vertical dimensions placed at various depths down to 10 feet. The discrepancies as to any general law in the relation of the different values of the coefficient of discharge c_d

to the size and depth of the orifice in the preceding experiments, have been remedied to a great extent by these. They give an increase of the coefficients for the smaller and very oblong orifices as they approach the surface, and a decrease under the same circumstances in those for the larger square and oblong orifices. There are a few depths where maximum and minimum values are obtained: we use the terms "maximum and minimum values" for those which are greater in the one case and less in the other than the coefficients immediately before and after them, and not as being numerically the greatest or least values in the column. We have marked with a *, in the arrangement of the coefficients, TABLE I., these maximum and minimum values. The heads given in this table were measured to the upper side of the orifices, and by adding half the depth (d) to any particular head, we shall obtain the head at the centre.

As a perceptible sinking of the surface takes place in heads less than from five to three times the depth of the orifice, the coefficients are arranged in pairs, the first column containing the coefficients for heads measured from the still water surface some distance back from the orifice, and the second obtained when the lesser heads, measured directly at the orifice, were used. A very considerable increase in the value of the coefficients for very oblong and shallow small orifices, may be perceived as they approach the surface, and the mean value for all rectilinear orifices at considerable depths, seems to approach to .605 or .606.

We have shown, equation (29), that the discharge is

$$D = c_d \times \left\{ 1 - \frac{d^2}{96 h^2} \right\} \times A \sqrt{2 g h},$$

approximately, in which expression d is the depth of the orifice, and h the head at its centre. Now it is to be observed, that it is not the value of c_d simply, which is given in TABLE I., but the value of $c_d \times \left\{ 1 - \frac{d^2}{96 h^2} \right\}$, the coefficient of $A \sqrt{2 g h}$, equation (29).

The coefficients in the table are, therefore, less than the coefficients of discharge, strictly so called, by a quantity equal to $\frac{c_d d^2}{96 h^2}$. The value of this expression

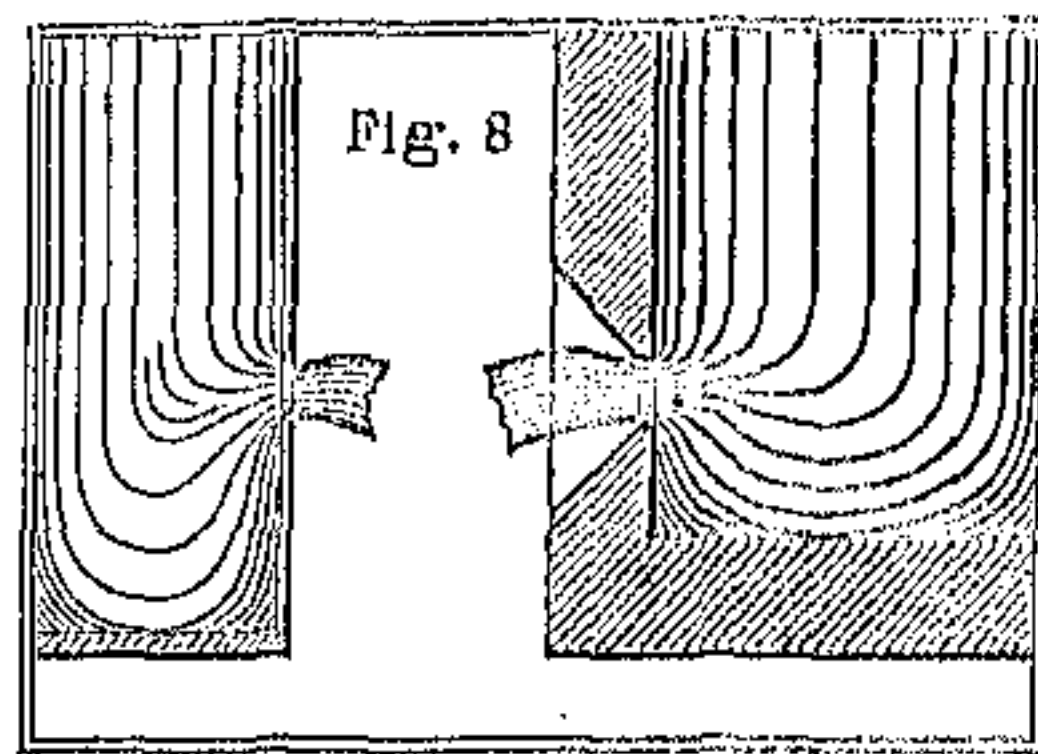
is in general very small, and it is easy to perceive from the first of the expressions in equation (31), p. 62, that it can never exceed 4.2 per cent., or more correctly .0417 in unity. If we wish to know the discharge from an orifice 4 inches square = $4'' \times 4''$, with its centre 4 feet below the surface, which is equivalent to a head of 3 feet 10 inches at the upper side, we find from the table the value of $c_d \left\{ 1 - \frac{d^2}{96 h^2} \right\} = .601$; hence we shall get

$$D = .601 \times A \sqrt{2 g h} = .601 \times \frac{1}{9} \times 8.025 \times 2 =$$

$$.601 \times \frac{1}{9} \times 16.05 = \frac{1}{9} \times 9.646 = 1.072$$

cubic feet per second. In the absence of any experiments with larger orifices, we must, when they occur, use the coefficients given in this table; and, in order to do so with judgment, it is only necessary to observe the relations of the sides and heads. For example, if the size of an orifice be $16'' \times 4''$, we must

seek for the coefficient in that column where the ratio of sides is as four to one, and if the head at the upper side be five times the length of the orifice, we shall find the coefficient .626, which in this case is the same for depths measured behind, or at the orifice. For lesser orifices, the results obtained from the experiments of *Michelotti* and *Bossut*, pages 67 and 68, are most applicable; and also the coefficients of *Rennie*, page 71. It is almost needless to observe, that all these coefficients are only applicable to orifices in thin plates, or those



having the outside arrises chamfered as in Fig. 8. Very little dependence can be placed on calculations of the quantities of water discharged from

other orifices, unless where the coefficients have been already obtained by experiment or correct inference for them. If the inner arris next the water be rounded, the coefficient will be increased.

NOTCHES AND WEIRS.

We have already given some coefficients, pages 69 and 70, derived from the experiments of *Du Buât*, *Brindley* and *Smeaton*, and *Poncelet* and *Lesbros*, for finding the discharge over notches in the sides of large vessels; and it does not appear that there is any difference of importance between these and those for orifices sunk some depth below the surface, when the proper formula for finding the discharge for each

is used. If we compare Poncelet and Lesbros' coefficients for notches, page 70, with those for an orifice at the surface, TABLE I., we perceive little practical difference in the results, the head being measured back from the orifice, unless in the very shallow depths, and where the ratio of the length to the depth exceeds five to one. The depths being in these examples less than an inch, it is probable that the larger coefficients found for the *orifice* at the surface, arise from the upper edge attracting the fluid to it and lessening the effects of vertical contraction, as well as from less lateral contraction. Indeed, the results obtained from experiments with very shallow weirs, or notches, have not been at all uniform, and at small depths the discharge must proportionably be more affected by movements of the air and external circumstances than when the depths are considerable. We shall see that in Mr. Blackwell's experiments the coefficient obtained for depths of 1 and 2 inches was .676 for a thin plate 3 feet long, while for a thin plate 10 feet long it increased up to .805.

The experiments of Castel, with weirs up to about 30 inches long, and with variable heads of from 1 to 8 inches, lead to the coefficient .497 for notches extending over one-fourth of the side of a reservoir; and to the coefficient .664 when they extend for the whole width. For lesser widths than one-fourth, the coefficients decrease down to .584; and for those extending between one-third of, and the whole width, they increase from .600 to .665 and .680. Bidone finds $c_d = .620$, and Eytelwein $c_d = .635$. It will be

perceived from these and the foregoing results, that the third place of decimals in the value of c_d , and even sometimes the second, is very uncertain; that the coefficient varies with the head and ratio of the notch to the side in which it is placed; and we shall soon show that the form and size of the weir, weir-basin, and approaches, still further modify its value.

When the sides and edge of a notch increase in thickness, or are extended into a shoot, the coefficients are found to reduce very considerably; and for small heads, to an extent beyond what the increase of resistance, from friction alone, indicates. Poncelet and Lesbros found, *for orifices*, that the addition of a horizontal shoot, 21 inches long, reduced the coefficient from $\cdot 604$ to $\cdot 601$, with a head of 4 feet; but for a head of only $4\frac{1}{2}$ inches, the coefficient fell from $\cdot 572$ to $\cdot 483$, the orifice being $8'' \times 8''$. For *notches* 8 inches wide, with a horizontal shoot 9 feet 10 inches long, the coefficient fell from $\cdot 582$ to $\cdot 479$, for a head of 8 inches; and from $\cdot 622$ to $\cdot 340$, for a head of only 1 inch. Castel found also, for a notch 8 inches wide with a shoot 8 inches long attached and inclined at an angle $4^\circ 18'$, that the mean coefficient for heads from 2 to $4\frac{1}{2}$ inches was only $\cdot 527$. Little dependence can be placed on experimental results obtained for shoots which partake of the nature of short pipes, and should be treated in like manner to find the discharge.*

We have obtained the following table of coefficients from some experiments made by Mr. Ballard, on the river Severn, near Worcester, "with a weir 2 feet

* *Traité Hydraulique*, par D'Aubuisson, pp. 46, 94 et 95.

COEFFICIENTS FOR RIGHT WEIRS OVER BOARDS.

Heads measured on the crest.

Depth in inches.	Coefficients.	Depth in inches.	Coefficients.	Depth in inches.	Coefficients.
1	·702	3	·801	5	·733
1 $\frac{1}{4}$	·693	3 $\frac{1}{4}$	·795	5 $\frac{1}{4}$	·713
1 $\frac{1}{2}$	·673	3 $\frac{1}{2}$	·748	5 $\frac{1}{2}$	·735
1 $\frac{3}{4}$	·693	3 $\frac{3}{4}$	·740	5 $\frac{3}{4}$	·729
2	·684	4	·759	6	·727
2 $\frac{1}{4}$	·702	4 $\frac{1}{4}$	·731	7	·710
2 $\frac{1}{2}$	·750	4 $\frac{1}{2}$	·744	8	·720
2 $\frac{3}{4}$	·780	4 $\frac{3}{4}$	·745	Mean	·732

long, formed by a board standing perpendicularly across a trough."* *The heads or depths were here measured on the weir, and hence the coefficients are larger than those found from heads measured back to the surface of still water.*

Experiments made at Chew-Magna, in Somersetshire, by Messrs. Blackwell and Simpson, in 1850†, give the following coefficients.

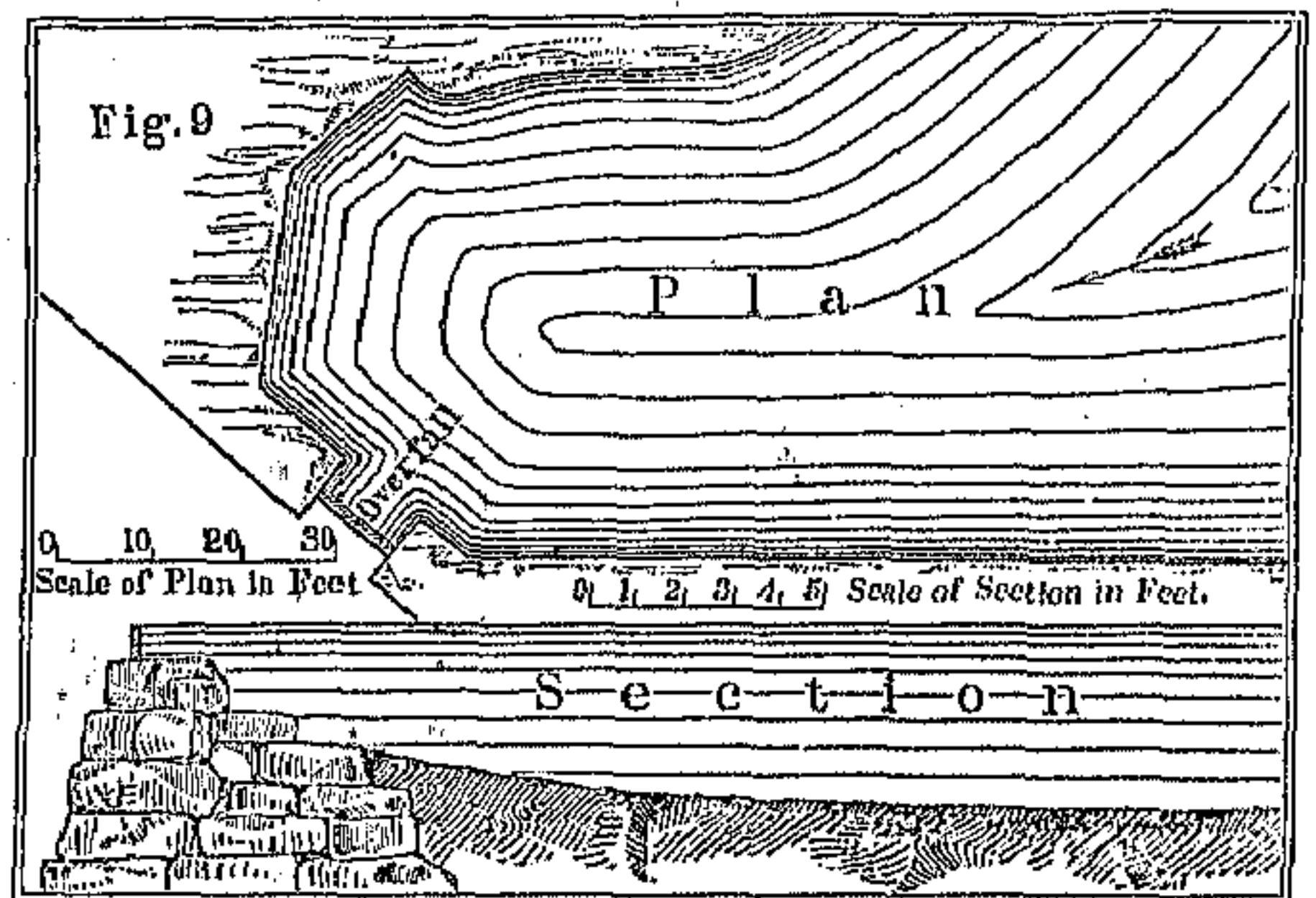
COEFFICIENTS DERIVED FROM THE EXPERIMENTS OF BLACKWELL AND SIMPSON.

Heads in inches.	Coefficients.	Heads in inches.	Coefficients.	Heads in inches.	Coefficients.
1 to 3	·501	4 $\frac{1}{4}$	·743	6	·740
1 to 1 $\frac{1}{10}$	·620	4 $\frac{5}{10}$	·700	6 $\frac{3}{10}$	·748
2 $\frac{3}{10}$ to 2 $\frac{1}{2}$	·682	4 $\frac{7}{10}$	·741	6 $\frac{7}{10}$ to 6 $\frac{1}{4}$	·747
2 $\frac{7}{10}$	·665	4 $\frac{9}{10}$	·750	6 $\frac{9}{10}$	·772
2 $\frac{9}{10}$	·670	5	·725	7 $\frac{1}{10}$	·717
2 $\frac{11}{10}$	·665	5	·780	8	·802
2 $\frac{13}{10}$	·653	5 $\frac{5}{10}$	·781	8 to 8 $\frac{3}{10}$	·737
2 $\frac{15}{10}$	·654	5 $\frac{7}{10}$	·740	8 $\frac{5}{10}$	·750
3 to 3 $\frac{1}{10}$	·725	5 $\frac{9}{10}$ to 5 $\frac{11}{10}$	·751	9	·781
4	·745	5 $\frac{11}{10}$	·728	Mean	·723

* Civil Engineer and Architect's Journal for 1851, p. 647.

† Civil Engineer and Architect's Journal for 1851, pp. 642 and 645.

"The overfall bar was a cast-iron plate 2 inches thick, with a square top." The length of the overfall was 10 feet. The heads were measured from still water at the side of the reservoir, and at some distance up in it. The area of the reservoir was 21 statute perches, of an irregular figure, and nearly 4 feet deep on an average. It was supplied from an upper reservoir, by a pipe 2 feet in diameter and of 19 feet fall; the distance between the supply and the weir was about 100 feet. The width of the



reservoir as it approached the overfall was about 50 feet, and the plan and section, Fig. 9, of the weir and overfall in connection with it, will give a fair idea of the circumstances attending the experiments. For heads over 5 inches the velocity of approach to the weir was "perceptible to the eye," though its amount was not determined. We perceive that the coefficient (derived from two experiments) for a depth of 8 inches is .802, while the coefficient (derived from three experiments) for a depth of $7\frac{1}{2}$ inches is

·717, and for depths from 8 to $8\frac{1}{4}$ inches the mean coefficient is ·743 : as all the attendant circumstances appear the same, these discrepancies and others must arise from the circumstances of the case : perhaps the supply, and, consequently, the velocity of approach, was increased while making one set of experiments, without affecting the still water near the side where the heads appear to have been taken. By comparing the results with those obtained by one of the same experimenters, Mr. Blackwell, on the Kennet and Avon Canal, we shall immediately perceive that the velocity of approach, and every circumstance which tends to alter and modify it, has a very important effect on the amount of the discharge, and, consequently, on the coefficient.

The experiments made by Mr. Blackwell, on the Kennet and Avon Canal, in 1850*, afford very valuable instruction, as the form and width of the crest were varied, and brought to agree more closely with actual weirs in rivers than the thin plates, or boards of earlier experimenters. We have calculated and arranged the coefficients in the following table from these experiments. The variations in the values for different widths of crest, other circumstances being the same, are very considerable ; and the differences in the coefficients, at depths of 5 inches and under, for thin plates and crests 2 inches wide, are greater than mere friction can account for ; and greater also than the differences at the same depths between the coefficients for crests 2 inches thick, and 3 feet long.

* Civil Engineer and Architect's Journal, 1851, p. 642.

COEFFICIENTS FOR THE DISCHARGE OVER WEIRS, ARRANGED AND DERIVED FROM THE EXPERIMENTS OF MR. BLACKWELL.

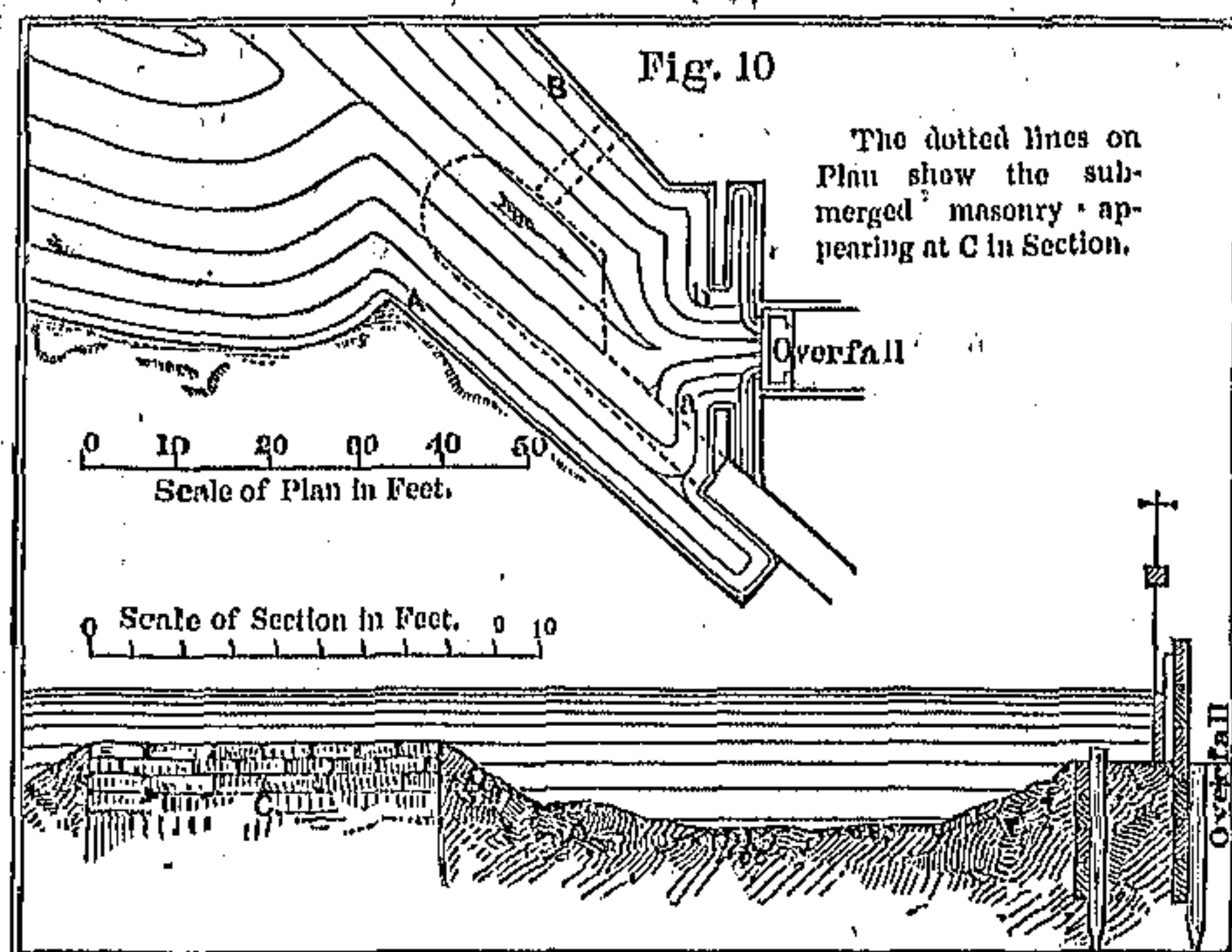
When more than one experiment was made with the same head, and the results were pretty uniform, the resulting coefficients are marked with a *. The effect of the converging wing-boards is very strongly marked.

NOTE.—Francis' experiments give a coefficient of .565 for a level crest 3 feet wide, and a head slope of $3\frac{1}{2}$ to 1, see p. 121.

Thin plates.		Planks 2 inches thick, square on crest.					Crests 3 feet wide.				
3 feet long.	10 feet long.	3 feet long.	6 feet long.	10 feet long.	10 feet long, wing-boards making an angle of 60°.	3 feet long, level.	3 feet long, fall 1 in 18.	3 feet long, fall 1 in 12.	6 feet long, level.	10 feet long, level.	10 feet long, fall 1 in 18.
1	.677	.809	.467	.459	.435†	.754	.452	.545	.467*	..	.467
2	.675	.803	.509*	.561	.585*	.675	.482	.546	.533	..	.479*
3	.630	.642*	.563*	.597*	.569*	..	.441	.537	.539	.492*	..
4	.617	.656	.549	.575	.602*	.656	.419	.431	.455	.497*	..
5	.602	.650*	.588	.601*	.609*	.671	.479	.516515
6	.593	..	.593*	.608*	.576*	..	.501*	..	.531	.507	..
7617*	.608*	.576*	..	.488	.513	.527	.497	..
8	..	.581*	.606*	.590*	.548*	..	.470	.491
9	..	.530	.600	.569*	.558*	..	.476	.492*	.498	.480*	.507
10614*	.539	.531*486	..
12525	.534*465*	..
14549*467*	..

† The discharge per second varied from .461 to .665 cubic feet in two experiments. The coefficient .435 is derived from the mean value.

The plan and section, Fig. 10, will give a fair idea of the approach to, and nature of the overfall made



use of in these experiments. The area of the reservoir was 2A. 1R. 30P., and the head was measured from the surface of the still water in it, which remained unchanged between the beginning and end of each experiment. The width of the approach A B from the reservoir was about 32 feet; the width at *ab* about 13 feet, below which the waterway widened suddenly, and again narrowed to the length of the overfall. The depth in front of the dam appears to have been about 3 feet; the depth on the dam, next the overfall, about 2 feet; and the depth on the sunk masonry in the channel of approach, about 18 inches. Altogether, the circumstances were such as to increase the amount of resistances between the reservoir, from which the head was measured, and the overfall, particularly for the larger heads, and we accordingly

see that the coefficients become less for heads over six inches, with a few exceptions. The measurements of the quantities discharged appear to have been made very accurately, yet the discharges per second, with the same head and same length of overfall, sometimes vary; for instance, with the plank 2 inches thick and 10 feet long, the discharge per second for 4 inches head varies from 6.098 cubic feet to 6.491 cubic feet, or by about one-sixteenth of the whole quantity. Most of the results, however, are means from several experiments. The quantities discharged varied from one-tenth of a cubic foot to 22 cubic feet per second, and the duration of the experiments from 24 to 420 seconds. If we compare the coefficients for a plank 10 feet long and 2 inches thick in the foregoing table with those for the same overfall at Chew-Magna, we shall immediately perceive how much the form of the approaches affects the discharge. Indeed, were the area of the reservoir at Chew-Magna even larger than that for the Kennet and Avon experiments, it would be found, notwithstanding, that the coefficients in the former would still continue the larger, though not fully as large as those found under the particular circumstances.*

* There is a very important omission in all the preceding experiments on weirs and notches. In Fig. 10, for instance, it would have been necessary to obtain the heads at A B and *ab* in each experiment, above the crest, and also the head on and a few feet above the crest itself. These are, perhaps, best calculated by means of the observed velocity of approach. They would indicate the resistances at the different passages of approach, and enable us to calculate the coefficients correctly, and thereby render them more generally applicable to practical purposes. The coefficients

The following table gives the mean results of 88 experiments made by Francis, at the Lower Lock, Lowell, Massachusetts, in 1852. The duration of each of these experiments varied from 180 to 822 seconds. The coefficients in column 10 have been

1	2	3	4	5	6	7	8	9	10
Average experiments.	Length of weir (l) in feet.	Observed mean depth over weir in feet (h).	Observed discharge in critical feet per second.	Observed velocity of approach in feet per second.	Depths (h') corrected for the velocity of approach when $h_2 = \frac{c^2}{2g}$	Values of h' in the formula in column 8.	Value of the formula $v = c \left\{ 1 + \frac{1}{2} \frac{h'}{h} \right\} \sqrt{h'^2}$ in cubic feet per second.	Values of the multiplier c in the formula in column 8.	Corresponding values of the coefficient of discharge c_d .
1	0.007	1.55	62.0	.78	1.50	1.50	62.0	0.32	.021
2	0.007	1.24	45.0	.50	1.25	1.25	45.4	0.33	.020
3	0.007	1.00	33.4	.44	1.00	1.00	33.5	0.32	.021
4	7.007	1.01	26.8	.30	1.02	1.02	26.9	0.30	.028
5	0.007	1.05	30.	.07	1.00	1.00	35.8	0.35	.020
6	0.005	0.08	33.0	.54	0.00	.08	32.4	0.31	.024
7	0.005	1.00	33.5	.55	1.01	1.00	33.3	0.30	.023
8	0.007	0.80	23.5	.30	.80	.80	23.4	0.32	.021
9	0.007	0.83	25.	.75	.80	.80	24.8	0.34	.024
10	0.005	0.80	20.0	.40	.80	.80	20.8	0.34	.024
11	0.007	0.02	10.2	.23	.02	.02	10.0	0.30	.020
12	0.007	0.05	17.5	.53	.05	.05	17.2	0.30	.020
13	7.007	0.08	14.0	.45	.08	.08	14.5	0.34	.023

calculated by ourselves, and the other results condensed from the large table given in Francis' Book.*

in the two previous tables are not as valuable as they otherwise would be from this omission. The level of still water near the banks is below that of the moving water in the current, therefore, heads measured from still water must give larger coefficients than if taken from the centre of the current. This may account, to some extent, for the larger coefficients in the first table, but apart from this, the short contracted channel immediately above the water-fall, Fig. 9, must increase the coefficients.

* Lowell Hydraulic Experiments. New York, 1855.

The heads given in the 6th column are those which would give the observed discharge from the formula

$$D = \frac{2}{3} c_d (2g)^{\frac{1}{2}} h'^{\frac{3}{2}}.$$

As we have also equation (39)

$$D = \frac{2}{3} c_d l (2g)^{\frac{1}{2}} \times \{(h + h_n)^{\frac{3}{2}} - h_n^{\frac{3}{2}}\}$$

we must, therefore, have

$$h' = \{(h + h_n)^{\frac{3}{2}} - h_n^{\frac{3}{2}}\}^{\frac{2}{3}},$$

the values of which are given in column 6. The values of h'' in column 8 are those which would be found by resolving the equation

$$D = c(l + n h'') h''^{\frac{3}{2}}$$

n being the number of end contractions, and c a multiplier varying from 3.32 to 3.36.

In this table the theoretical head $\frac{v_n^2}{2g} = .0155 v_n^2$ due to the velocity of approach has been used and does not exceed .02 of a foot. We are of opinion, however, that the head is much greater, and should be taken $= \frac{v_n^2}{c_d^2 \times 2g} = .04 v_n^2$ or thereabouts. This would reduce the values of the coefficient of discharge c_d in the 10th column. The differences between h , h' , and h'' in columns 3, 6, and 7 are here, practically, of little moment, and the value of c_d in column 10 would be nearly the same derived from either. The crest of the weir experimented upon was 1 inch thick. The weir measuring 10 feet \times 13 inches \times 1 inch, the top was rounded off at both arrises, leaving the central horizontal portion one quarter of an inch wide. The general result of these experiments verifies the

ordinary coefficient for notches in thin plates from .617 to .628 for the value of c_d .

Professor Thomson's experiments with right-angled triangular notches, in thin plates, give a mean coefficient of .617. *Vide* Note p. 55.

HEAD, AND FROM WHENCE MEASURED.

By referring to TABLE I., we shall see that there is a difference in the coefficients as obtained from heads measured on or above the orifice. This difference is greater in notches, or weirs, than in orifices sunk below the surface; and when the crest of a weir is of some width, the depths upon it vary. In the Kennet and Avon experiments, the heads measured from the surface of the water in the reservoir, and the depths at the "outer edge" (by which we understand the lower edge) of the crest were as follows:—

DIFFERENCE OF HEADS MEASURED ON AND ABOVE WEIRS.

Heads from reservoir to crest, in inches.	Heads on crests 3 inches thick.		Heads on crests 3 feet wide.					
	$\frac{15}{16}$ to $\frac{3}{8}$	$\frac{3}{8}$ to $\frac{3}{16}$	$\frac{7}{16}$ to $\frac{3}{8}$	$\frac{3}{8}$ to $\frac{1}{2}$	$\frac{1}{2}$ to $\frac{3}{4}$	$\frac{3}{4}$ to 1	1 to $1\frac{1}{4}$	$1\frac{1}{4}$ to $1\frac{1}{2}$
1	8	..	$\frac{7}{16}$..	4	4	$\frac{7}{16}$	$\frac{5}{16}$
2	8	8	$\frac{10}{16}$
3	..	$1\frac{15}{16}$	$1\frac{1}{4}$ to $1\frac{1}{4}$
4	3 to $2\frac{11}{16}$	$3\frac{1}{2}$	10	10	$1\frac{1}{2}$	$1\frac{1}{4}$
5	$3\frac{1}{2}$	$3\frac{5}{8}$	$3\frac{1}{4}$	12	12	..
6	40	$4\frac{3}{8}$	20	22	22	24
7	28	20
8	60	32	34
9	48	34	..
10	4	..

No intermediate heads are given, but those registered point out very clearly the great differences which often exist between the heads measured on a weir, or notch, and those measured from the still water above it; and how the form of the weir itself, as well as the nature of the approaches, alters the depth passing over. On a crest 2 feet wide, with $14\frac{1}{2}$ inches depth on the upper edge, we have found that the depth on the lower edge is reduced to $11\frac{1}{2}$ inches, or as 1.26 to 1. The head taken from 3 to 20 feet above the crest, where the plane of the approaching water surface becomes curved, is that in general which is best suited for finding the discharge by means of the common coefficients, but a correct section of the channel and water-line, showing the different depths upon and for some distance above the crest, is necessary in all experiments for determining accurately by calculation the value of the coefficient of discharge c_d .

Du Buat, finding the theoretical expression for the discharge through an orifice of half the depth h ,

$$D = \frac{2}{3} \sqrt{2g} \times l \left\{ h^{\frac{3}{2}} - \left(\frac{h}{2} \right)^{\frac{3}{2}} \right\},$$

equation (6)

$$= \frac{2}{3} l h \sqrt{2g h} \times \left\{ 1 - \left(\frac{1}{2} \right)^{\frac{3}{2}} \right\} = .646 \times \frac{2}{3} l h \sqrt{2g h},$$

to agree pretty closely with his experiments, seems to have assumed that the head h is reduced to $\frac{h}{2}$ in passing over. This is a reduction, however, which never takes place unless with a wide crest and at its lower edge, or where the head h is measured at a

considerable distance above the weir, and when a loss of head due to the distance and obstructions in channel takes place. When there is a clear weir basin immediately above the weir, we have found that, putting h for the head measured from the surface in the weir basin, and h_w for the depth on the upper edge of the weir, that

$$(32.) \quad h - h_w = .14 \sqrt{h},$$

for measures in feet, and

$$(33.) \quad h - h_w = .48 \sqrt{h},$$

for measures in inches. The comparative values of h and h_w depend, however, a good deal on the particular circumstances of the case. Dr. Robinson found* $h = 1.111 h_w$, when h was about 5 inches. The expressions we have given are founded on the hypothesis, that $h - h_w$ is as the velocity of discharge, or as the \sqrt{h} nearly. For small depths, there is a practical difficulty in measuring with sufficient accuracy the relative values of h and h_w . Unless for very small heads the sinking will be found in general to vary from $\frac{h}{10}$ to $\frac{h}{4}$, and in practice it will always be useful to observe the depths on the weir as well as the heads for some distances (and particularly where the widths contract) above it.

In order to convey to our readers a more definite idea of the differences between the coefficients for heads measured at the weir, or notch, and at some distance above it, we shall assume the difference of the heads $h - h_w = \frac{h_w}{r}$; then $\frac{h_w}{h - h_w} = r$, and $\frac{1}{r} = \frac{h - h_w}{h_w}$,

* Proceedings of the Royal Irish Academy, vol. iv. p. 212.

hence $h = \frac{r+1}{r} h_w$ and $h_w = \frac{r}{r+1} h$.

Now the discharge may be considered as that which would take place through an orifice whose depth is h_w with a head over the upper edge equal to $h - h_w = \frac{h_w}{r}$; hence from equation (6) the discharge is equal to

$$\frac{2}{3} l \sqrt{2g} \times c_d \left\{ h^{\frac{3}{2}} - \left(\frac{h_w}{r} \right)^{\frac{3}{2}} \right\},$$

and substituting for $h^{\frac{3}{2}}$ its value $\left(\frac{r+1}{r} h_w \right)^{\frac{3}{2}}$, we shall find the value of

$$(34.) \quad D = \frac{2}{3} l h_w \sqrt{2g} h_w \times c_d \left\{ \left(1 + \frac{1}{r} \right)^{\frac{3}{2}} - \left(\frac{1}{r} \right)^{\frac{3}{2}} \right\}.$$

As the value of the discharge would be expressed by

$$\frac{2}{3} l h_w \sqrt{2g} h_w \times c_d$$

if the head $h - h_w$ were neglected, it is evident the coefficient is increased, under the circumstances, from c_d to

$$c_d \times \left\{ \left(1 + \frac{1}{r} \right)^{\frac{3}{2}} - \left(\frac{1}{r} \right)^{\frac{3}{2}} \right\};$$

or, more correctly, the common formula has to be multiplied by $\left(1 + \frac{1}{r} \right)^{\frac{3}{2}} - \left(\frac{1}{r} \right)^{\frac{3}{2}}$, to find the true discharge, and the value of this expression for different values of $\frac{1}{r} = n$ will be found in TABLE IV. If we suppose that

$$h - h_w = \frac{h_w}{10}, \text{ then } \frac{1}{r} = \frac{1}{10} = n;$$

and we find from the table $\left(1 + \frac{1}{r} \right)^{\frac{3}{2}} - \left(\frac{1}{r} \right)^{\frac{3}{2}} = 1.1221$.

Now if we take the value of c_d for the full head h to be $\cdot 628$, we shall find $1\cdot 1221 \times \cdot 628 = \cdot 705$, rejecting the latter figures, for the coefficient when the head is measured at the orifice; and if $\frac{1}{r} = \frac{2}{10} = n$, we should find in the same manner the new coefficient to be $1\cdot 2251 \times \cdot 628 = \cdot 769$ nearly. The increase of the coefficients determined, page 77, from Mr. Ballard's experiments is, therefore, evident from principle, as the heads were taken at the notch; and it is also pretty clear that, *in order to determine the true discharge, the heads both on, at, and above a weir should be taken*. Most of the discrepancies in the coefficients determined from experiment have arisen from imperfect and limited observations of the facts. Amongst these the velocity of approach should never be neglected by observers, as its effect on the discharge is often considerable in increasing the quantity. The effect of the form of the weir and approaches is scarcely ever sufficiently considered by professional men. Most of the discussions which arose with reference to the gaugings on the *Metropolitan MAIN DRAINAGE QUESTION* would have been obviated if the calculators, or engineers, had taken into account the different circumstances attendant on it, instead of applying generally a formula suited to a particular case, namely, a thin crest, a small notch, and a large body of water immediately above it; and applied a correct formula for finding the effects of the velocity of approach.

The two following tables have been reduced to English measures of feet, from Boileau's experiments;

they show the relation of the head to the depth on the crest at the upper arris. The coefficient for the head h being known, we may, from our equation (34), calculate that due to h_w on the weir.

TABLE showing the ratio of the head, h , to the depth, h_w , on a Plank Weir of the full width of the Channel, immediately at the upper edge, or $\frac{h}{h_w}$, see equation (33), when the sheet of water is free after passing over, with air under it.

Head h in feet.	Values of the head h divided by the thickness of the sheet of water passing over the weir immediately at the upper edge; average $\frac{h}{h_w} = 1.2$ between heads of 3 and 14 inches.			
	Height of weir in feet, .80.	Height of weir in feet, 1.07.	Height of weir in feet, 1.33.	Height of weir in feet, 1.71.
.1	1.880	1.285
.13	1.382	..	1.320	1.250
.16	1.260	..	1.285	1.228
.20	1.234	1.243	1.240	1.214
.23	1.223	1.233	1.231	1.205
.26	1.216	1.232	1.223	1.200
.3	1.213	1.228	1.218	1.199
.33	1.210	1.225	1.217	1.199
.36	1.206	1.221	1.212	1.197
.40	1.202	1.216	1.206	..
.53	1.190	..	1.201	..
.60	1.190	..	1.195	..
.66	1.192	..	1.191	..
.82	1.186
.90	1.184
1.15	1.182

If we were to use the head h_w instead of h , to calculate the discharge, when $\frac{h}{h_w} = 1.2$, then a coefficient of .628 for the head h would become .769 for the head h_w in equation (34): for $\frac{1}{\frac{1}{2}} = .2$, and, therefore, TABLE IV., $.628 \times (1.2)^{\frac{3}{2}} - (.2)^{\frac{3}{2}} = .628 \times 1.2251 = .769$.

TABLE showing the ratio $\frac{h}{h_w}$, equation (33), when the sheet of water passing over is in contact with the crest and with the water immediately below a Plank Weir.

Head h in feet.	Values of $\frac{h}{h_w}$ for different heights of weirs and for different heads; mean value for heads between 3 and 14 inches, equal $\frac{h}{h_w} = 1.25$.		
	Height of weir in feet, 1.07.	Height of weir in feet, 1.9.	Height of weir in feet, 1.38.
.43	..	1.283	..
.46	..	1.276	1.201
.49	1.250	1.260	1.281
.53	1.250	1.253	1.271
.56	1.230	1.245	1.264
.60	1.225	1.232	1.241
.73	1.210	1.223	..
.79	1.203	1.210	..
.86	1.202	1.208	..
.92	1.198	1.203	..
.99	..	1.198	..

If we were to use the head h_w instead of h to calculate the discharge, when $\frac{h}{h_w} = 1.25$, then a coefficient of .628 for the head h would become .799 for the head h_w in equation (34); for $\frac{1}{r} = .25$; and, therefore, the value of $c_d \left\{ \left(1 + \frac{1}{r}\right)^{\frac{3}{2}} - \left(\frac{1}{r}\right)^{\frac{3}{2}} \right\}$, TABLE IV., is $.628 \times (1.25)^{\frac{3}{2}} - (.25)^{\frac{3}{2}} = .628 \times 1.2725 = .799$; and so on we may calculate the value of the coefficient to be applied to the depth h_w on the weir, for any other ratios between h and h_w by means of equation (34).

Boileau made some valuable experiments at Metz,

which were published in 1854. They give the following results for vertical plank weirs extending from side to side of the channel, when the water passed over without adhering to the crest :—

Height of weir over bottom of channel in feet.	Head above.	Mean coefficient.
3.	·2 to 1·6	·645
1·3	·16 to ·5	·622
·6	·15 to ·25	·625

When the water passing over was joined to the crest, and no air between the sheet passing over and the water below the weir, the experiments gave

Height of weir over bottom of channel in feet.	Head above.	Mean coefficient.
2.	1. to 1·6	·694
1·3	·6 to 1·8	·690
·6	·36 to 1·3	·675

When the plank weir leant up-stream 4 inches to a foot, the mean value of c_d was ·620, the height of weir being 1·5 foot, and with heads from ·23 to ·5 foot. When its crest was rounded to a semi-cylinder, the coefficient was, with a head of ·26 foot, ·696, and with a head of ·52 foot, ·843 ; the water adhering to the crest. With a head of ·6 foot the coefficient was ·867, and with a head of ·85 foot, ·840, when the water passed over without air between it and the water below the crest. The following tables give the experimental and reduced coefficients for vertical plank weirs of different heights, and with different heads, when the water passes over in a full sheet, and also when it is joined to the crest and lower water. Also for plank weirs suitable for sluices, leaning up-stream with a slope of one-third horizontal to one vertical.

when the water is free and not in contact with the slope, or water below; derived from Boileau's experiments.

Heights of weirs, in feet, over the bottom of the channel, and corresponding values of c_d in the formula $v = c_d \times \frac{1}{2} \sqrt{2gh}$.																			
Head h in feet.	66'	82'	99'	1-15'	1-32'	1-48'	1-65'	1-81'	1-98'	2-14'	2-31'	2-47'	2-64'	2-80'	3-07'	3-23'	3-40'	Head h in feet.	
1-13'	.631	.637	.639	.636	.627	.616	.612	.607	.603	.603	.606	.610	.619	.630	.637	.634	.627	.612	1-13
1-16	.628	.633	.634	.633	.628	.622	.613	.606	.598	.595	.597	.603	.613	.628	.634	.631	.624	.616	1-16
1-20	.624	.630	.633	.630	.621	.610	.606	.600	.597	.597	.600	.606	.615	.628	.633	.630	.624	.616	1-20
1-23	.627	.630	.633	.630	.622	.613	.606	.600	.597	.597	.600	.606	.615	.628	.633	.630	.624	.616	1-23
1-26	.627	.635	.636	.634	.633	.628	.624	.618	.612	.609	.610	.613	.619	.628	.633	.630	.624	.616	1-26
1-30637	.636	.633	.627	.621	.615	.613	.613	.615	.619	.625	.628	.628	.624	.619	1-30
1-33637	.636	.633	.627	.621	.616	.613	.613	.615	.619	.625	.628	.628	.624	.619	1-33
1-40642	.640	.637	.631	.625	.619	.615	.612	.612	.615	.624	.630	.630	.624	.616	1-40
1-46648	.645	.645	.637	.627	.618	.612	.612	.615	.624	.633	.637	.636	.630	1-46
1-53654	.652	.648	.642	.636	.628	.624	.622	.625	.627	.639	.640	.637	.630	1-53
1-59654	.651	.646	.642	.640	.639	.642	.633	.642	.640	.636	.630	1-59
1-66657	.652	.649	.648	.648	.649	.643	.645	.643	.639	.633	1-66
1-72657	.652	.651	.651	.654	.655	.655	.651	.648	.642	.636	1-72
1-79652	.651	.655	.657	.658	.658	.655	.651	.646	.639	1-79
1-86655	.654	.655	.657	.658	.658	.655	.651	.646	.639	1-86
1-92657	.658	.657	.658	.658	.655	.651	.646	.639	1-92
1-99658	.661	.663	.666	.667	.666	.663	.655	.648	1-99
1-05667	.670	.667	.663	.655	.648	1-05
1-12667	.669	.667	.663	.655	.648	1-12
1-18667	.669	.667	.663	.655	.648	1-18
1-25667	.669	.667	.663	.655	.648	1-25
1-32667	.669	.667	.663	.655	.648	1-32
1-39667	.669	.667	.663	.655	.648	1-39
1-45667	.669	.667	.663	.655	.648	1-45
1-52667	.669	.667	.663	.655	.648	1-52
1-58673	.670	.666	.660	.654	.648	1-58
1-65676	.670	.666	.660	.654	.648	1-65

COEFFICIENTS of *Vertical Plank Weirs at right angles to the Channel, when the edge is chamfered at the lower arris, and when the head passing over is in contact with the water at and below the Weir; or when the water immediately below the Weir rises to the crest. The maximum coefficient .733 appears to obtain when the height of the Weir is double the depth passing over the crest.*

Head h in feet.	Heights of weirs, in feet, over the bottom of the channel, and corresponding values of the coefficient of discharge c_d in the formula $v = c_d \times \frac{1}{2} \sqrt{2gh}$.									Head h in feet.
	.66'	.82'	.99'	1.16'	1.32'	1.48'	1.65'	1.81'	1.98'	
.30	.72730
.33	.72433
.36	.72136
.39	.71839
.43	.71443
.46	.70946
.40	.702	.708	.715	.72440
.53	.694	.699	.708	.71853
.56	.687	.693	.700	.712	.72956
.59	.679	.687	.694	.705	.72159
.63	.676	.682	.689	.700	.71763
.66	.672	.678	.684	.696	.71466
.73	.667	.672	.678	.690	.708	.73373
.79	.661	.666	.673	.685	.705	.72979
.86	.655	.660	.669	.681	.700	.72486
.92	.648	.655	.666	.678	.699	.72092
.99	.640	.652	.666	.678	.693	.703	.712	.720	.720	.99
1.05	.631	.645	.657	.669	.681	.691	.702	.711	.720	1.05
1.12	.627	.636	.646	.657	.667	.679	.690	.700	.711	1.12
1.19	.625	.636	.646	.657	.666	.675	.685	.694	.703	1.19
1.25	.625	.636	.646	.657	.666	.675	.682	.690	.696	1.25
1.32	.625	.636	.646	.657	.666	.673	.679	.685	.691	1.32
1.39666	.672	.678	.682	.684	1.39
1.45664	.670	.675	.679	.684	1.45
1.52661	.667	.672	.676	.681	1.52
1.58658	.663	.669	.672	.675	1.58
1.65655	.658	.663	.666	.667	1.65

The following table gives the result of experiments on chamfered plank weirs, for gauging, extending across a channel at right angles to it, when the back-water below was joined to the head-water at passing over, and when there was no air between:—

Height of weir over the bot- tom of the channel below...	feet	feet	feet	feet	feet	feet	feet	feet
	.66	.83	1.00	1.16	1.32	1.48	1.65	2.00
Heads passing over the weir in each case, when absorbed at the crest into the back-water	.23	.31	.38	.45	.51	.59	.66	.92

which shows that the head was drowned (*noyée*) when the depth of the lower channel below the crest of the weir was less than $2\frac{1}{2}$ times the head passing over, taking a general average.

TABLE of *Experimental Coefficients for Plank Weirs leaning up-stream, when the crest has the down-stream arris rounded to a quadrant; and when the crest is cylindrical and projecting up-stream in the form of a knob.*

Head H in feet.	Plank weir leaning up-stream one-third to one; the lower arris of crest rounded off to a quadrant of a circle with a radius the full thickness of the plank.		Plank weir leaning upwards one-third to one, the crest rounded and projecting in front beyond the plank, so as to be thicker than it.	
	Water free from curve of crest .13 foot thick.	Water in contact with curve of crest .17 foot thick.	Water in contact with curve of crest .3 foot thick.	Water in contact with curve of crest .33 foot thick.
.10	.589	.651
.20	.589	.672
.23	.594	.697
.26	.612	.697
.30	.633	.721	..	.670
.33	.642	.747	.604	.686
.36	.649	.766	.625	.700
.39	.655	.768	.648	.714
.43	.661	.795	.669	.727
.46	.667	.802	.687	.741
.49	.675	..	.702	.753
.53	.679	..	.715	.765
.56	.685	..	.720	.775
.59741	.786
.63753	.795
.66762	.802
.69808
.72813

The effect of the form of the crest in increasing the coefficients is distinctly observable in this Table, although the weirs experimented on overhung the water above, between the crest and the bottom of the channel.

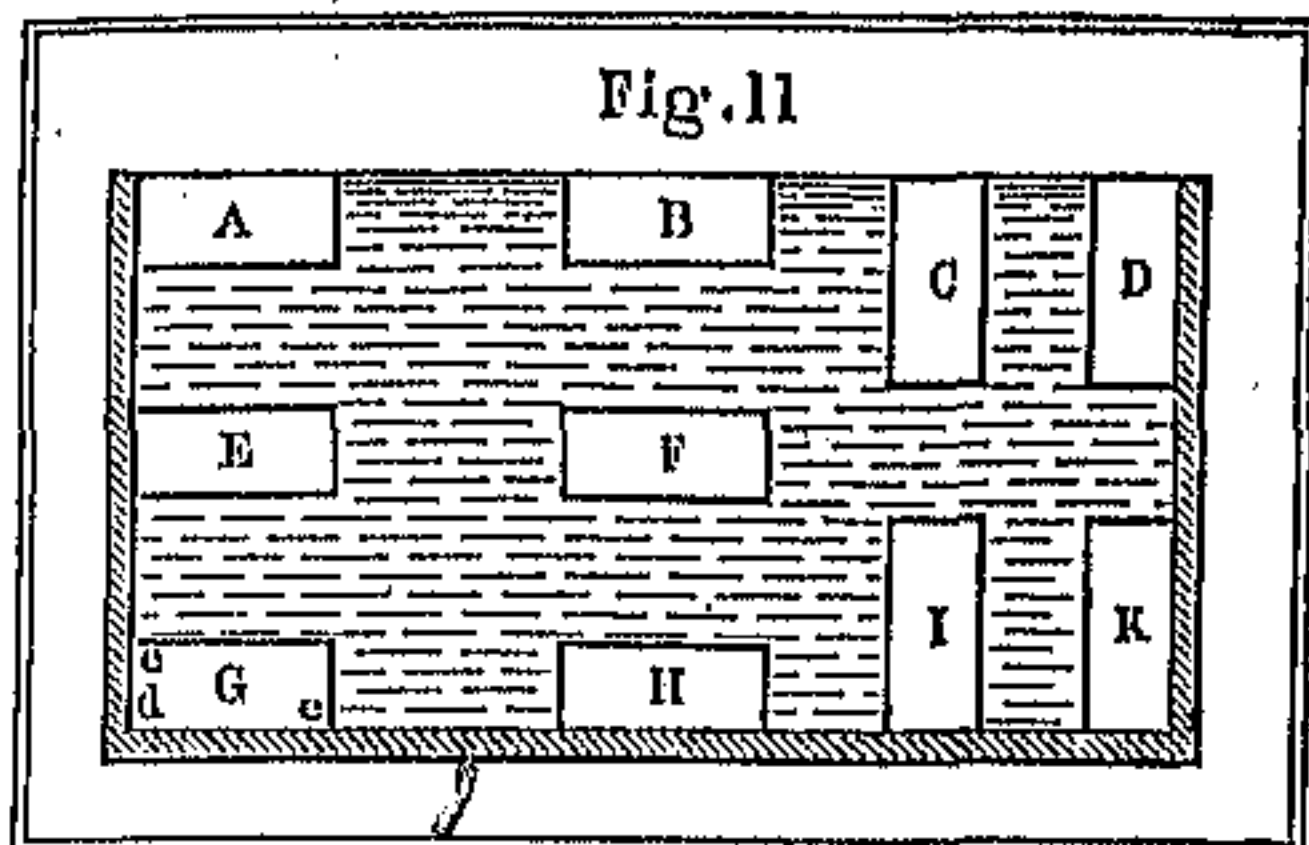
We must protest against the notation adopted by Boileau and Morin, of giving only two-thirds of the coefficient of discharge, c_d , for weirs, instead of the full and true value. The correct formula for the discharge from a weir is $D = \frac{2}{3} l h \sqrt{2 g h}$. Now they assume a coefficient due to an incorrect formula $D = l h \sqrt{2 g h}$, which reduces c_d to $\frac{2}{3} c_d$ to give the same final results. This leads also to an unnecessary distinction between the coefficients of orifices at the surface, or notches, and orifices sunk to some depth, which, practically, have the same, or nearly the same, general value.

SECTION IV.

VARIATIONS IN THE COEFFICIENTS FROM THE POSITION OF THE ORIFICE.—GENERAL AND PARTIAL CONTRACTION.—VELOCITY OF APPROACH.—PRACTICAL FORMULÆ FOR THE DISCHARGE OVER WEIRS AND NOTCHES.—CENTRAL AND MEAN VELOCITIES.

A glance at TABLE I. will show us that the coefficients increase as the orifices approach the surface, to a certain depth dependent on the ratio of the sides, and that this increase increases with the ratio of the length to the depth: some experimenters have found the increase to continue uninterrupted for all orifices up to the surface, but this seems to hold only for depths taken at or near the orifice when it is square or nearly so: it has also been found that the coefficient increases as the orifice approaches to the sides or bottom of a vessel: as the contraction becomes imperfect the coefficient increases. These facts probably arise from the velocity of approach being more direct and concentrated under the respective circumstances. The lateral orifices A, B, C, D, E, F, G, H, I, and K, Fig. 11, have coefficients differing more or less from each other. The coefficient for A is found to be larger than either of those for B, C, E, or D; that for G or K larger than that for H or I; that for H larger than that for I; and that for F, where the contraction is general, least of all. The contraction of the fluid on entering the orifice F removed from the bottom and sides is complete; it is termed, therefore, "*general contraction*;" that at the orifices A, E, G, H, I, K, and D, is interfered with by the sides; it is therefore incomplete,

and termed "*partial contraction*." The increase in the coefficients for the same-sized orifices at the same



mean depths may be assumed as proportionate to the length of the perimeter at which the contraction is partial, or from which the lateral flow is shut off; for example, the increase for the orifice G is to that for H as $cd + de : de$; and in the same manner the increase for G is to that for E as $cd + de : cd$. If we put n for the ratio of the contracted portion cde to the entire perimeter, and, as before, c_a for the coefficient of general contraction, we shall find the coefficient of partial contraction to be equal to

$$(35.) \quad c_a + .09 n = c_a + .1 n \text{ nearly,}$$

for rectangular orifices. The value of the second term $.09 n$ is derived from various experiments. If we assume $.617$ for the mean value of c_a , we may change the expression into the form $(1 + .146 n) c_a$. When $n = \frac{1}{4}$, this becomes $1.036 c_a$; when $n = \frac{1}{2}$, it becomes $1.073 c_a$; and when $n = \frac{3}{4}$, contraction is prevented for three-fourths of the perimeter, and the coefficient for partial contraction becomes $1.109 c_a$. The form which we have given equation (35) is, however, the simplest; but the value of n must not exceed $\frac{3}{4}$. If in this case $c_a = .617$, the coefficient

for partial contraction becomes $\cdot 617 + \cdot 09 \times \frac{2}{3} = \cdot 617 + \cdot 067 = \cdot 684$. Bidone's experiments give for the coefficient of partial contraction $(1 + \cdot 152 n) c_d$; and Weisbach's $(1 + \cdot 132 n) c_d$.

VARIATION IN THE COEFFICIENTS FROM THE EFFECTS OF
THE VELOCITY OF APPROACH.

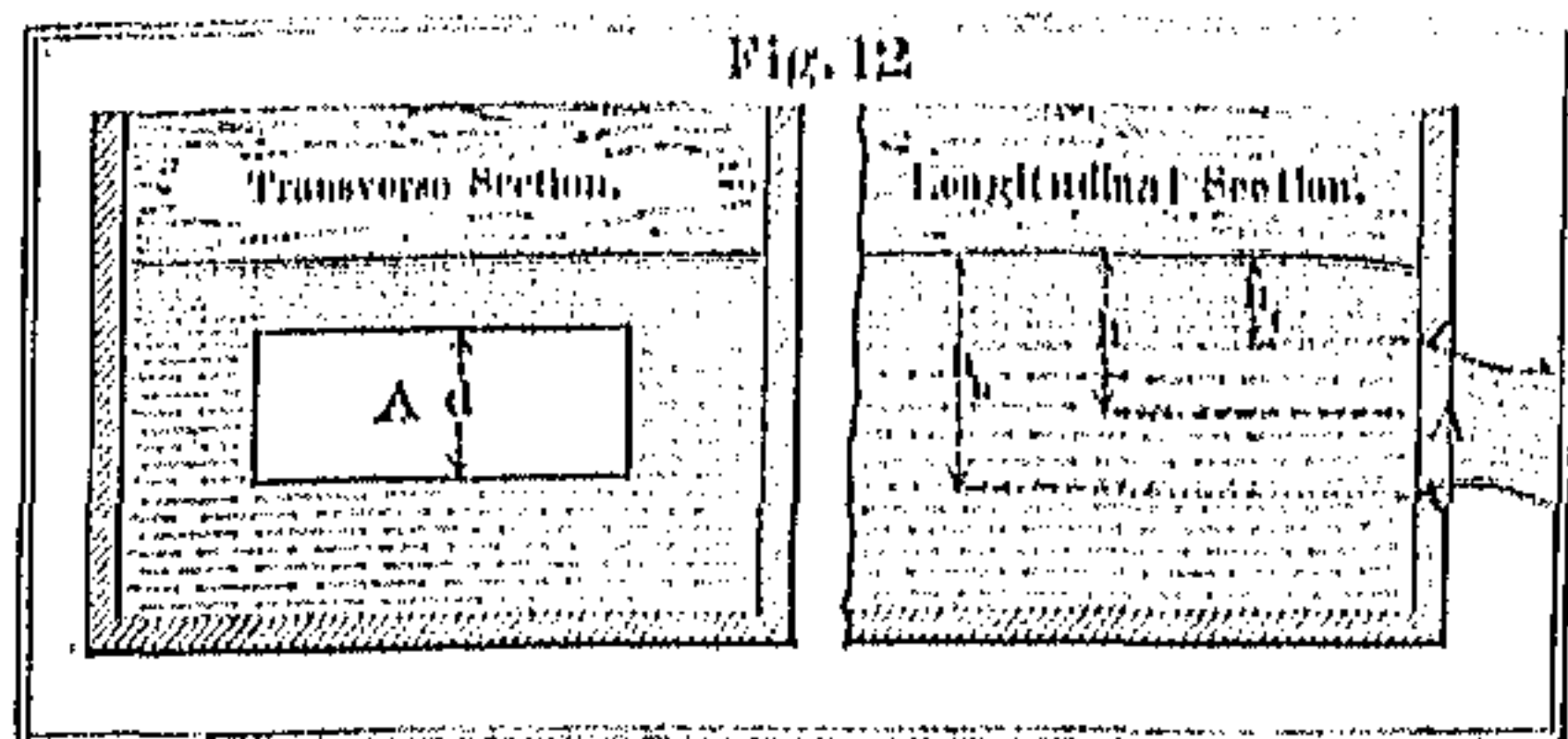
Heretofore we have generally supposed the water in the vessel to be almost still, its surface level unchanged, and the vessel consequently large compared with the area of the orifice. When the water flows to the orifice with a perceptible velocity, the contracted vein and the discharge are both found to be increased, other circumstances being the same. If the area of the vessel or channel in front exceed thirty times that of the orifice, the discharge will not be perceptibly increased by the induced velocity in the conduit; but for lesser areas of the approaching channel corrections due to the velocity of approach become necessary. It is clear that this velocity may arise from either a surface inclination in the channel, an increase of head, or a small channel of approach.

We get equation (6) for the discharge from a rectangular orifice A, Fig. 12, of the length l , with a head measured from still water

$$D = \frac{2}{3} c_d l \sqrt{2g} \times \{h_0^{\frac{3}{2}} - h_1^{\frac{3}{2}}\},$$

in which h_0 and h_1 are measured to the surface at some distance back from the orifice, as shown in the section. The water here, however, must move along the channel towards the orifice with considerable velocity. If A be the area of the orifice, and a the area of the

channel, we may suppose with tolerable accuracy that



this velocity is equal to $\frac{A}{O} v_o$, in which v_o represents the mean velocity in the orifice. If we also represent by v_a the velocity of approach, we get the equation

$$(36.) \quad v_a = \frac{A}{O} \times v_o,$$

and consequently the theoretical height (h_a) due to it is

$$(37.) \quad h_a = \frac{A^2}{O^2} \times \frac{v_o^2}{2g} = .0155 \frac{A^2 v_o^2}{O^2} \text{ in feet measures.}^*$$

The height h_a may be considered as an increase of head, converting h_b into $h_b + h_a$, and h_t into $h_t + h_a$. The discharge therefore now becomes

$$(38.) \quad D = \frac{2}{3} c_d l \sqrt{2g} \left\{ (h_b + h_a)^{\frac{3}{2}} - (h_t + h_a)^{\frac{3}{2}} \right\};$$

which, for notches or weirs, is reduced to

* When the approaching velocity passes through the orifice without contraction, it is evident that the head h_a required to produce that velocity, in the orifice with contraction, must be $h_a = \frac{A^2}{O^2} \times \frac{v_o^2}{2g \times c_d^2}$

instead of $h_a = \frac{A^2}{O^2} \times \frac{v_o^2}{2g}$, in which case equation (40) becomes

$h_a = \frac{D^2}{O^2} \times \frac{1}{2g c_d^2}$. In like manner we must have $h_a = \frac{v_a^2}{c_d^2 \times 2g} = .04 v_a^2$ in feet measures when v_a is the velocity of approach and $c_d = .617$.

$$(39.) \quad D = \frac{2}{3} c_d l \sqrt{2g} \left\{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\}, *$$

as h_t then vanishes. As D is also equal to $A \times v_o$, equation (37) may be changed into

$$(40.) \quad h_a = \frac{D^2}{g^2} \times \frac{1}{2g} = .0155 \frac{D^2}{g^2} \text{ in feet measures.}$$

If this value for h_a be substituted in equations (38) and (39), the resulting equations will be of a high order and do not admit of a direct solution; and in (38) and (39), as they stand, h_a involves implicitly the value of D , which we are seeking for. By finding at first an approximate value for the velocity of approach, the height h_a due to it can be easily found, equation (37); this height, substituted in equation (38) or (39), will give a closer value of D , from which again a more correct value of h_a can be determined; and by repeating the operation the values of D and h_a can be had to any degree of accuracy. In general the values found at the second operation will be sufficiently correct for all practical purposes.

It has been already observed that, for orifices, it is advisable to find the discharge from a formula in which only one head, that at the centre, is made use of; and though TABLE IV., as we shall show, enables us to calculate the discharge with facility from either formula, it will be of use to reduce equation (38) to

* The formula for the discharge over weirs, taking into account the velocity of approach, $D = 2.95 c_d l \sqrt{h + .115 v_a^2}$, given by D'Aubuisson, *Traité Hydraulique*, seconde édition, pp. 78 et 95, and adopted by some English writers and engineers, is incorrect in principle. In feet measures it becomes $D = 5.35 c_d l h \times \sqrt{h + .03494 v_a^2}$, which form,—with alterations in the numerals and measures,—was used for calculating discharges of sewers during the METROPOLITAN MAIN DRAINAGE discussion.

a form in which only the head (h) at the centre is used. The error in so doing can never exceed six per cent., even at small depths, equation (31), and this is more than balanced by the observed increase in the coefficients for smaller heads.

The formula for the discharge from an orifice, h , being the head at the centre, is

$$D = c_d \sqrt{2gh} \times A;$$

and when the additional head h_a due to the velocity of approach is considered,

$$D = c_d \sqrt{2g(h+h_a)} \times A,$$

which may be changed into

$$(41.) \quad D = A \sqrt{2gh} \times c_d \left\{ 1 + \frac{h_a}{h} \right\}^{\frac{1}{2}}.$$

Equation (39), for notches, may be also changed to the form

$$(42.) \quad D = \frac{2}{3} A \sqrt{2gh_b} \times c_d \left\{ \left(1 + \frac{h_a}{h_b} \right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b} \right)^{\frac{3}{2}} \right\};$$

this is similar in every way to the equation

$$(43.) \quad D = \frac{2}{3} A \sqrt{2gd} \times c_d \left\{ \left(1 + \frac{h_t}{d} \right)^{\frac{3}{2}} - \left(\frac{h_t}{d} \right)^{\frac{3}{2}} \right\},$$

for the discharge from a rectangular orifice whose depth is d , with the head h_t at the upper edge.

TABLE III. contains the values of $\left\{ 1 + \frac{h_a}{h} \right\}^{\frac{1}{2}}$ in equation

(41), and TABLE IV. the values of $\left(1 + \frac{h_a}{h_b} \right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b} \right)^{\frac{3}{2}}$

in equation (42), or the similar expression in (43),

$\frac{h_a}{h_b}$ or $\frac{h_t}{d}$ being put equal to n ; and we perceive that

the effect of the velocity of approach is such as to in-

crease the coefficient from c_d to $c_d \left\{ 1 + \frac{h_a}{h} \right\}^{\frac{1}{2}}$ for orifices

sunk some distance below the surface, and into

$$c_d \left\{ \left(1 + \frac{h_a}{h_b} \right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b} \right)^{\frac{3}{2}} \right\}$$

for weirs when h_a is the height due to the velocity of approach, h the depth of the centre of the orifice, and h_b the head on the weir. A few examples, showing the application of the formulæ (41), (42), and (43), and the application of TABLES I., II., III., and IV. to them, will be of use. We shall suppose, for the present, the velocity of approach v_a to be given, and no extra head be required to maintain it through the orifice: in other words when $h_a = \frac{v_a^2}{2g \times .956^2} = .017 v_a^2$ in feet measures nearly.

EXAMPLE I. *A rectangular orifice, 12 inches wide by 4 inches deep, has its centre placed 4 feet below the surface, and the water approaches the head with a velocity of 28 inches per second; what is the discharge?* For an orifice of the given proportions, and sunk to a depth nearly four times its length, we shall find from TABLE I.

$$c_d = \frac{.616 + .637}{2} = .621 \text{ nearly.}$$

As the coefficient of velocity, equation (2), for water flowing in a channel is about .956, we shall find, column No. 3, TABLE II. the height $h_a = 1\frac{1}{8} = 1.125$ inch nearly, corresponding to the velocity 28 inches. Equation (41),

$$D = A \sqrt{2gh} \times c_d \left\{ 1 + \frac{h_a}{h} \right\}^{\frac{1}{2}},$$

now becomes

$$D = 12 \times 4 \sqrt{2gh} \times .621 \left\{ 1 + \frac{1.125}{48} \right\}^{\frac{1}{2}}.$$

We also find $\sqrt{2gh} = 192.6$ inches when $h = 48$ inches, TABLE II.; therefore

$$D = 12 \times 4 \times 192.6 \times .621 \left\{ 1 + \frac{1.125}{48} \right\}^{\frac{1}{2}}$$

$$= 9244.8 \times .621 \{ 1 + .0234 \}^{\frac{1}{2}} = 9244.8 \times .621 \times 1.0116,$$

(as $\{ 1.0234 \}^{\frac{1}{2}} = 1.0116$ from TABLE III.) $= 9244.8 \times$

$$.628 \text{ nearly} = 5805.7 \text{ cubic inches} = 3.36 \text{ cubic feet}$$

per second. Or thus: The value of $.621 \times (1.0234)$

being found equal $.628$, $D = A \times .628 \sqrt{2gh} \times 48$. Now

for the coefficient $.628$, and $h = 48$ inches, TABLE II.

gives us $.628 \sqrt{2gh} \times 48 = 120.96$ inches; hence we

get $D = 12 \times 4 \times 120.96 = 5806.08$ cubic inches $= 3.36$

cubic feet, the same as before, the difference $.38$ in

the cubic inches being of no practical value. If we

find h_a from the formula $h_a = \frac{v_a^2}{2g} = 2.6$ inches, then

we shall get $D = 3.41$ cubic feet nearly.

If the centre of the orifice were within 1 foot of the surface, the effect of the velocity of approach would be much greater; for then

$$C_d \times \left\{ 1 + \frac{h_a}{h} \right\}^{\frac{1}{2}} = (\text{from TABLE I.}) .623 \left\{ 1 + \frac{1.125}{12} \right\}^{\frac{1}{2}}$$

$$= (\text{from TABLE III.}) .623 \times 1.047 = .652 \text{ instead of}$$

$$.628. \text{ In this case the discharge is } D = 12 \times 4 \times .652$$

$$\sqrt{2gh} \times 12 = 12 \times 4 \times .652 \times 96.3 (\text{from TABLE II.})$$

$$= 12 \times 4 \times 62.8 = 3014.4 \text{ cubic inches} = 1.744$$

$$\text{cubic feet per second. Or we may find the value of}$$

$$.652 \sqrt{2gh} \text{ directly from TABLE II. thus:}$$

$$\text{The value of } .628 \sqrt{2gh} \times 12 = 60.48 \quad .628$$

$$\text{The value of } .666 \sqrt{2gh} \times 12 = 64.14 \quad .652$$

$$\frac{38}{3.66} :: 24 : 2.31.$$

Hence $.652 \sqrt{2gh} = 60.48 + 2.31 = 62.79$, and the

discharge $= 12 \times 4 \times 62.79 \times 3013.92$ cubic inches
 $= 1.744$ cubic feet per second, the same as before.

If we take $h_a = \frac{v_a^2}{2gc_a^2} = 2.6$ inches, we shall find $D = 1.833$ cubic feet nearly.

EXAMPLE II. A rectangular notch, 7 feet long, has a head of 8 inches measured at about 4 feet above the orifice, and the water approaches the head with a velocity of $16\frac{1}{4}$ inches per second; what is the discharge? For a still head we shall assume $c_a = .628$ in this case, and we have from equation (42)

$$D = \frac{2}{3}A \sqrt{2gh_b} \times c_a \left\{ \left(1 + \frac{h_a}{h_b}\right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b}\right)^{\frac{3}{2}} \right\}.$$

As in the last example, we shall find from TABLE II. (h_a) the height due to the velocity of approach ($16\frac{1}{4}$ inches) to be $\frac{3}{8} = 0.375$ inch, assuming the coefficient of velocity to be .956. We have, therefore, $h_a = .375$, $h_b = 8$, $c_a = .628$, and $A = 7 \times 12 \times 8$, or for measures in feet $\frac{h_a}{h_b} = .047$, $h_b = \frac{2}{3}$, and $A = 7 \times \frac{2}{3}$; hence

$$D = \frac{2}{3} \times 7 \times \frac{2}{3} \sqrt{2g \times \frac{2}{3}} \times .628 \left\{ (1.047)^{\frac{3}{2}} - (.047)^{\frac{3}{2}} \right\}.$$

The value of $(1.047)^{\frac{3}{2}} - (.047)^{\frac{3}{2}}$ will be found from TABLE IV. equal to 1.0612; the value of $\sqrt{2g \times \frac{2}{3}}$ will be found from TABLE II. equal to 6.552, viz. by dividing the velocity 78.630, to be found opposite 8 inches, by 12; hence

$$\begin{aligned} D &= \frac{2}{3} \times 7 \times \frac{2}{3} \times 6.552 \times .628 \times 1.0612 \\ &= \frac{2}{3} \times 7 \times 4.368 \times .628 \times 1.0612 \end{aligned}$$

$$= \frac{2}{3} \times 7 \times 4.368 \times .666 \text{ nearly}$$

$$= \frac{2}{3} \times 7 \times 2.909 = 7 \times 1.939$$

$= 13.573$ cubic feet per second $= 814.38$ cubic feet per minute. *Or thus:* From TABLE VI. we find, when the coefficient is .628, the discharge from a weir 1 foot long, with a head of 8 inches, to be 109.731 cubic feet per minute. The discharge for a weir 7 feet long, when $\frac{h_a}{h} = .047$ is therefore $109.731 \times 7 \times 1.0612 = 815.12$ cubic feet per minute. The difference between this value and that before found, 814.38 cubic feet is immaterial, and has arisen from not continuing all the products to a sufficient number of places of decimals. If $h_a = \frac{v_a^2}{2g c_d^2} = .87$ inch, then $D = 14.51$ cubic feet per second nearly.

We have, in equations (36) and (37), pointed out the relations between the channel, orifice, velocity of approach, and velocity in the orifice, viz.

$$v_a = \frac{A}{C} \times v_o, \text{ and } h_a = \frac{A^2}{C^2} \times \frac{v_o^2}{2g} = \frac{D^2}{2g C^2}, \text{ in which } h_a = \frac{v_a^2}{2g}$$

(neglecting, for the present, the coefficient of velocity in passing through the orifice). As v_o is the actual velocity in the orifice, $\frac{v_o}{c_d}$ must be the theoretical velocity due to the head $h + h_a$, and therefore

$$h + h_a = \frac{v_o^2}{c_d^2 \times 2g}, \text{ and } h = \frac{v_o^2}{c_d^2 \times 2g} - \frac{v_a^2}{2g}; \text{ hence}$$

$$\frac{h_a}{h} = \frac{1}{\frac{v_o^2}{v_a^2 \times c_d^2} - 1} = \frac{c_d^2 v_a^2}{v_o^2 + c_d^2 v_a^2} = \frac{c_d^2 A^2}{C^2 - c_d^2 A^2}, \text{ for } \frac{v_o^2}{v_a^2} = \frac{C^2}{A^2}.$$

We have hence

$$(44.) \quad \frac{h_a}{h} = \frac{c_d^2 A^2}{C^2 - c_d^2 A^2};$$

substituting this value in equations (41) and (42), there results

$$(45.) \quad \begin{cases} D = A \sqrt{2gh} \times c_d \left\{ 1 + \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{1}{2}} \text{ or,} \\ D = A \sqrt{2gh} \times \frac{c_d}{\left(1 - \frac{c_d^2}{m^2} \right)^{\frac{1}{2}}} \end{cases}$$

in which $m = \frac{C}{A}$, for the discharge from an orifice at some depth, and for the discharge from a weir,

$$(46.) \quad D = \frac{2}{3} A \sqrt{2gh_b} \times c_d \left\{ \left(1 + \frac{c_d^2}{m^2 - c_d^2} \right)^{\frac{3}{2}} - \left(\frac{c_d^2}{m^2 - c_d^2} \right)^{\frac{3}{2}} \right\}.$$

The two last equations give the discharge when the

ratio of the channel to the orifice $\frac{C}{A} = m$ is known,

and also when the whole quantity of water passing through the orifice, *that due to the velocity of approach as well as that due to the pressure, suffers a contrac-*

tion whose coefficient is c_d . When $h_a = \frac{v_a^2}{2g} \times \frac{v_a^2}{c_d^2}$, that is when the velocity of approach v_a passes through the orifice without contraction, we shall get

$$\frac{h_a}{h} = \frac{v_a^2}{v_0^2 - v_a^2} = \frac{A^2}{C^2 - A^2} = \frac{1}{m^2 - 1},$$

consequently, in this case, equation (45) becomes

$$(45a.) \quad D = A \sqrt{2gh} \times c_d \times \left\{ 1 + \frac{1}{m^2 - 1} \right\}^{\frac{1}{2}};$$

and equation (46) in like manner changes into

$$(46a.) \quad D = \frac{2}{3} A \sqrt{2gh_b} \times c_d \times \left\{ \left(1 + \frac{1}{m^2 - 1} \right)^{\frac{3}{2}} - \left(\frac{1}{m^2 - 1} \right)^{\frac{3}{2}} \right\}.$$

The last members of these two equations are the same as the like members in (45) and (46), when h_a within the brackets $= 1$; consequently we shall easily find their values for the coefficient 1 in the last page of TABLE V., for the respective values of $n = \frac{C}{A}$; and also for those of $\frac{h_a}{h_b} = \frac{1}{m^2 - 1}$. When $h_a = 1$, equation (45) may be changed into

$$D = A \left\{ \frac{2g h}{1 - \left(\frac{1}{m}\right)^2} \right\}^{\frac{1}{2}}.$$

This is the equation of Daniel Bernoulli, and only a particular case of the one we have given.

If we put $n = \frac{c_d^2}{m^2 - c_d^2}$, the values of $\left\{ 1 + \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{1}{2}}$, and of $\left\{ 1 + \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{3}{2}} - \left\{ \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{3}{2}}$, respectively, can be easily had from TABLES III. and IV. We have, however, calculated TABLE V. for different ratios of the channel to the orifice, and for different values of the coefficient of discharge. This table gives at once the values of

$$\left\{ 1 + \frac{c_d^2}{m^2 - c_d^2} \right\}^{\frac{1}{2}} \text{ and } c_d \left\{ \left(1 + \frac{c_d^2}{m^2 - c_d^2} \right)^{\frac{3}{2}} - \left(\frac{c_d^2}{m^2 - c_d^2} \right)^{\frac{3}{2}} \right\},$$

the new coefficients, and the corresponding value of

$$\frac{h_a}{h_b}, \text{ or } \frac{h_a}{h_b} = \frac{c_d^2}{m^2 - c_d^2}.*$$

* When $\frac{h_a}{h_b} = \frac{1}{m^2 - 1} = \frac{A^2}{C^2 - A^2}$ we shall have in EXAMPLE II. $C = 1.1$ and $\left(1 + \frac{h_a}{h_b} \right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b} \right)^{\frac{3}{2}} = 1.133$, TABLE IV., (or

It is equally applicable, therefore, to equations (41) and (42) as to equations (45) and (46). For instance, we find here at once the value of 628 $\{(1.047)^{\frac{3}{2}} - (.047)^{\frac{3}{2}}\}$ in EXAMPLE II., p. 104, equal to .666, as $\frac{h_a}{h_b} = .047$, and the next value to it for the coefficient .628, in the table, is .046, opposite to which we find .666, the new coefficient sought. The sectional area of the channel in this case, as appears from the first column, must be about three times that of the weir or notch.

TABLE V. is calculated from coefficients, c_a , in still water, which vary from .550 to 1. Those from .606 to .650, and the mean value .628 are most suited for application in practice. When the channel is equal to the orifice, the supply must equal the discharge, and for open channels, with the mean coefficient .628, we find, accordingly, from the table, the new coefficient 1.002 for weirs; or 1 very nearly as it should be. We also find, in the same case, viz. when $A = c$, and $c_a = .628$, that for short tubes, Fig. 13, the resulting new coefficient becomes .807. This, as we shall afterwards see, agrees very closely with the experimental results. When the coefficients in still water are less than .628, or more correctly .62725, the orifice, according to our

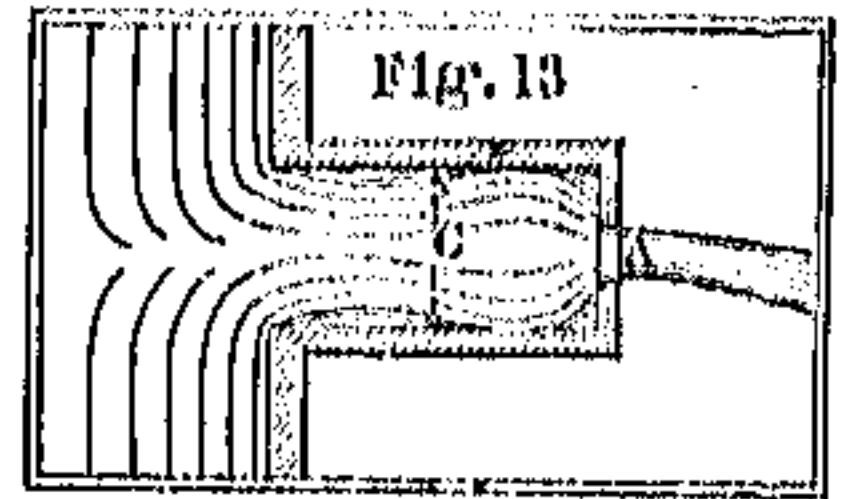
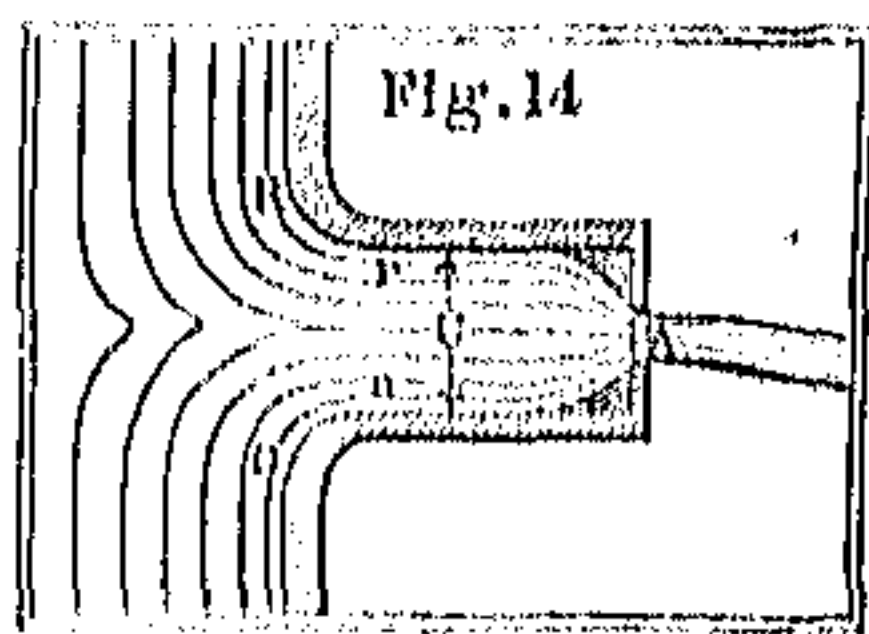


TABLE V. for the coefficient 1.) Hence in this case $.628 \times 1.133 = .712$ the new coefficient suited to the velocity of approach. Here of course $h_a = \frac{v_a^2}{2g c_a^2}$ (see Note p. 99).

formula, cannot equal the channel unless other resistances take place—as from friction in tubes longer than one and a half or two diameters, or in wide crested weirs; and for greater coefficients the junction of the short tube with the vessel must be rounded, Fig. 14, on one or more sides; and in weirs or notches the approaches must slope from the crest and ends to the bottom and sides, and the overfall be sudden. The converging form of the approaches must, however, increase the velocity of approach; and therefore



is greater than $\frac{A}{C} \times v_0$ when C is measured between o and ro , Fig. 14, to find the discharge, or new coefficient of an orifice placed at ro .

As the coefficients in TABLE V. are suited for orifices at the end of short cylindrical or prismatic tubes at right angles to the sides or bottom of a vessel, a correction is required when the junction is rounded off as at ro or ro , Fig. 14. When the channel is equal to the orifice, the new coefficient in equation (45) becomes

$$c_a \left\{ 1 + \frac{c_a^2}{1 - c_a^2} \right\}^{\frac{1}{2}} = c_a \times \left\{ \frac{1}{1 - c_a^2} \right\}^{\frac{1}{2}}.$$

The velocity in the short tube Fig. 14 is to that in the short tube Fig. 13 as 1 to $c_a \left\{ \frac{1}{1 - c_a^2} \right\}^{\frac{1}{2}}$ nearly, or the mean value $c_a = .628$, as 1 to .807. Now, as v_0 is assumed equal to $\frac{v_0}{v_a}$ in the cylindrical or pris-

matic tube, Fig. 13, $\frac{.807 \, c}{A} = \frac{v_o}{v_n}$ in the tube Fig. 14 with the rounded junction, for v_n becomes $\frac{v_n}{.807}$; hence, in order to find the discharge from orifices at the end of the short tube, Fig. 14, we have only to multiply the numbers representing the ratio $\frac{c}{A}$ in the first column, TABLE V., by .807, or more generally by $c_d \left\{ \frac{1}{1 - c_d^2} \right\}^{\frac{1}{2}}$, and find the coefficient opposite to the product. Thus if $c_d = .628$, we find, when $\frac{c}{A} = 1$, $c_d \left\{ \frac{1}{1 - c_d^2} \right\}^{\frac{1}{2}} = .807$ in the table. If, again, we suppose $\frac{c}{A} = 3$, then $3 \times .807 = 2.421$, the value of $\frac{v_o}{v_n}$ for the tube Fig. 14, and opposite this value of $\frac{c}{A}$, taken in column 1, we shall find .651 for the new coefficient. For the cylindrical or prismatic tube, Fig. 13, the new coefficient would be only .642. When the head h_n is however equal to $\frac{v_n^2}{2g c_d^2}$ the results must be modified accordingly (see Note p. 99).*

* Professor Rankine gives the value of the coefficient of discharge, or contraction, for varying values of A and c at a diaphragm in a pipe by the formula

$$c_d = \frac{.618}{\left(1 - .618 \times \frac{A^2}{c^2}\right)^{\frac{1}{2}}}.$$

When $\frac{A}{c} = 0$, $c_d = 1$; and when $\frac{A}{c} = 1$, $c_d = .618$; as it should be very nearly for an orifice in a *thin* plate, to which only, and to A , in the short tube, Fig. 14, the formula is suited (see SECTION X).

PRACTICAL FORMULÆ FOR THE DISCHARGE OVER WEIRS.

In order to reduce the preceding formulæ for weirs and notches to some of the forms in common use, with definite combined numerical coefficients, by substituting 8.025 for $\sqrt{2g}$, equation (39) becomes for feet measures

$$(A.) \quad p_a = 5.35 c_d l \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \},$$

and for inch measures, as $\sqrt{2g} = 27.8$, the discharge, taken also in cubic feet, becomes

$$(B.) \quad p_a = .01072 c_d l \{ (h_b - h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}.$$

When the length l is taken in feet and the depth in inches, we shall have

$$(C.) \quad p_a = .1287 c_d l \{ (h_b - h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}.$$

The three last equations being for seconds of time, we shall get, when the time is taken in minutes for feet measures, the discharge in cubic feet

$$(D.) \quad p_a = 321 c_d l \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \};$$

for inch measures

$$(E.) \quad p_a = .6433 c_d l \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \};$$

and for lengths (l) in feet and depths in inches

$$(F.) \quad p_a = 7.72 c_d l \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}.$$

The latter equation, when the coefficient of discharge, c_d , is taken at .614 becomes

$$(G.) \quad \begin{cases} p_a = 4.74 l \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ p = 4.74 l h^{\frac{3}{2}}, \text{ when the velocity of ap-} \\ \quad \text{proach vanishes.} \end{cases}$$

for a coefficient of .617

$$(H.) \begin{cases} D_a = 4.76 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 4.76 \, l \, h^{\frac{3}{2}} \text{ when the velocity of ap-} \\ \text{proach vanishes.} \end{cases}$$

For a coefficient of .623

$$(I.) \begin{cases} D_a = 4.81 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 4.81 \, l \, h^{\frac{3}{2}} \text{ with no perceptible approach.} \end{cases}$$

For a coefficient of .628

$$(K.) \begin{cases} D_a = 4.85 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 4.85 \, l \, h^{\frac{3}{2}} \text{ with no perceptible approach.} \end{cases}$$

For a coefficient of .648

$$(L.) \begin{cases} D_a = 5 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 5 \, l \, h^{\frac{3}{2}} \text{ with no perceptible approach.} \end{cases}$$

For a coefficient of $\frac{2}{3}$ or .667

$$(M.) \begin{cases} D_a = 5.14 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 5.14 \, l \, h^{\frac{3}{2}} \text{ with no perceptible approach.} \end{cases}$$

For a coefficient of .712

$$(N.) \begin{cases} D_a = 5.5 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 5.5 \, l \, h^{\frac{3}{2}} \text{ with no perceptible approach.} \end{cases}$$

And finally for a coefficient of .81

$$(O.) \begin{cases} D_a = 6.3 \, l \, \{ (h_b + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}, \text{ and} \\ D = 6.3 \, l \, h^{\frac{3}{2}} \text{ when the velocity of approach} \\ \text{vanishes.} \end{cases}$$

The theoretical value of h_a in each of the foregoing equations is in terms of the velocity of approach v_a

$$h_a = \frac{v_a^2}{2g}$$

in which $2g$ must be taken equal to 64.403 for heads in feet, and equal to 772.84 for heads in inches. But it is evident that in order to produce the velocity per second v_a passing through the notch with a

nearly still-water basin above it, that h_a must be increased from its theoretical value $\frac{v_a^2}{2g}$ to $\frac{v_a^2}{c_d^2 2g}$, in which expression c_d is the coefficient of discharge due to the particular notch, or weir, and its attendant circumstances; whence we must take

$$(P.) \quad h_a = \frac{v_a^2}{c_d^2 2g} = \frac{\text{Theoretical head}}{c_d^2}.$$

Now, unquestionably, the most general coefficient both for notches and submerged orifices, in thin plates, for gauging, whether triangular, rectangular, or circular, is .617, when the orifice or notch is small compared with the approaching channel; whence for measures in feet

$$h_a = .0408 v_a^2, \text{ and } v_a = 4.95 \sqrt{h_a}.$$

For measures in inches,

$$h_a = .0034 v_a^2, \text{ and } v_a = 17.2 \sqrt{h_a}.$$

And for measures in which v_a is expressed in feet per second, and h_a in inches

$$h_a = .49 v_a^2, \text{ and } v_a = 1.43 \sqrt{h_a}.$$

By substituting these values of h_a , found in terms of the approaching velocity, according to the standards used in the equations from (A) to (F) inclusive, and also in equation (H), we shall be enabled to find the proper discharge from a notch in a *thin plate*. The values of h_a , equation (P), can be found at once in inches from the observed values of v_a , to be also taken in inches, for coefficients varying from .584 to .974, by means of TABLE II. Thus, with a coefficient of .617, we shall find, for an approaching velocity of 36 inches per second, that h_a becomes $4\frac{3}{8} = 4.4$ inches nearly, while for a coefficient of .666, it is only $3\frac{3}{4} =$

3.8 inches; and for a coefficient of 1, the theoretical head is but $1\frac{3}{4} = 1.7$ inches nearly.

From the very nature of the case the approaching velocity must continue nearly unimpaired through the notch with but a very slight reduction arising from the viscosity of the water when it enters the aperture, and separates from the lateral fluid. But in order to give this unimpaired velocity by means of an extra head h_a , it is evident that h_a must be increased above the theoretical value by the amount due to the coefficient of discharge; or, as before stated, h_a must be increased from $\frac{v_a^2}{2g}$ to $\frac{v_a^2}{c_d^2 2g}$. This value of h_a is, perhaps, something too large, owing to the reduction of v_a at the moment it enters the notch and is acted upon by the overfall, drawing it away, as it were, from the lateral water above the crest.

The numerical results of the respective formulae from (A) to (o), inclusive, can be obtained by modifying the form as in equation (42) into

$$(Q.) \begin{cases} D_a = D \times \left\{ \left(1 + \frac{h_a}{h_b}\right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b}\right)^{\frac{3}{2}} \right\} \text{ or,} \\ D_a = c_d \times \frac{2}{3} l h_b \sqrt{2g h_b} \times \left\{ \left(1 + \frac{h_a}{h_b}\right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b}\right)^{\frac{3}{2}} \right\} \end{cases}$$

in which D is the discharge found, when there is no velocity of approach, by the common form $D = 5.35 \times c_d l h^{\frac{3}{2}}$, for which separate values are given in equations from (H) to (o) inclusive; and numerical values in TABLE VI.; and $\left\{ \left(1 + \frac{h_a}{h_b}\right)^{\frac{3}{2}} - \left(\frac{h_a}{h_b}\right)^{\frac{3}{2}} \right\}$ a multiplier suited to the velocity of approach, the values of which can be found from TABLE IV. Suppose, for

example, $D = 158.1$ cubic feet per minute, $h_b = 10$ inches, and $h_a = 4$ inches, which is that due to an approaching velocity of 3 feet per second with a coefficient of .648; then the multiplier becomes $(1 + .4)^{\frac{3}{2}} = 1.4035$, TABLE IV. Hence the discharge due to an approaching velocity of 3 feet is $158.1 \times 1.4035 = 221.9$ cubic feet, or an increase of about 40 per cent. Also, if the common formula were used, it is plain that the coefficient .648 should be increased to $.648 \times 1.4035$, or to .909 nearly, which approximates within 10 per cent. of the theoretical value. Nothing can show more clearly the necessity for varying the coefficients when the ordinary formulæ are used, even for a notch in a thin plate: for other notches the coefficients, even for still water above the crest, vary considerably.

The form of the equation used by D'Aubuisson and several other writers is

$$(R.) \quad D_a = c l \sqrt{h_b^3 + c v_a^2 h_b^2}$$

in which c and c are numerical coefficients, and v_a the velocity of approach. This form is incorrect in principle, although the values of c and c can be so taken as to give resulting values for D_a approximately correct. For feet measures, and time in seconds, Professor Downing makes, after D'Aubuisson, p. 37 of his translation,

$$D_a = c_d \times 5.35 l \sqrt{h_b^3 + .03494 v_a^2 h_b^2}.$$

Doctor Robinson* gives for like measures and time, values varying from

$$D_a = 3.55 l \sqrt{h_b^3 + .1395 v_a^2 h_b^2}, \text{ to}$$

$$D_a = 3.2 l \sqrt{h_b^3 + .1395 v_a^2 h_b^2}.$$

* Proceedings Royal Irish Academy, vol. iv. p. 212. $.1395 v_a^2$ is 10 times the theoretical head, and too much.

Mr. Taylor finds (for the Government Referees, see Report on the Main Drainage of the Metropolis, 13th July, 1858, p. 32) the discharge in cubic feet per minute, when the depth is taken in inches, and the length in feet to be,

$$D_a = 5.5 l \sqrt{h_b^3 + .8 v_a^2 h_b^2};$$

and the Messrs. Hawksley, Bidder, and Bazalgette assume, (p. 33 *ibid.*) for like measures,

$$D_a = 5 l \sqrt{h_b^3 + .1875 v_a^2 h_b^2},$$

which they consider is in "*excess.*" The following table, copied and extended from the report just referred to, shows the results of the last two formulæ, and of our equations (L) and (N), in which the depth, h_b , must be taken equal to 10 inches, and the length, l , equal to 1 foot.

Formula.	Mean velocities approaching the notch in feet per second, and discharges in cubic feet per minute.						
	0	.5	1	1.5	2	2.5	3
$D_a = 5\sqrt{h_b^3 + .1875 v_a^2 h_b^2}$	158.1	158.6	159.5	161.4	164	167	171
Equation (L) when the head, h_a , due to the velocity of approach is taken at only its theoretical value	158.1	159.2	162.1	166.8	173	180	189
Equation (L) when the head, h_a , due to the velocity of approach is increased for the coefficient of velocity .648	158.1	160	167	177	190	205	222
Equation (N) when the head, h_a , due to the velocity of approach is taken at only its theoretical value	173.0	175.1	178.3	183.6	190.1	198.3	207.5
Equation (N) when the head due to the velocity of approach is increased for the coefficient of velocity .712	173.0	176	182	192	204	218	234
$D_a = 5.5\sqrt{h_b^3 + .8 v_a^2 h_b^2}$	173.0	175.7	180.8	188.0	199.8	213	228

In equations (L) and (N) we can get, TABLE II., the values of the head, h_a , due to velocity of approach v_a , as follows :

$v_a = .5, .1, 1.5, .2, 2.5, 3.0$; in feet per second.
 $h_a = .047, .180, .410, .745, 1.10, 1.08$; theoretical head in inches.

Then

$h_a = .111, .447, .007, 1.77, 2.70, 4$; for a coefficient of .648.

and

$h_a = .093, .300, .893, 1.47, 2.20, 3.81$; for a coefficient of .712.

Whence as $h_b = 10$ inches, we shall have in equation (Q),

$\frac{h_a}{h_b} = .011, .045, .1, .18, .28, .4$; for a coefficient of .648,

and

$\frac{h_a}{h_b} = .000, .037, .089, .15, .23, .33$; for a coefficient of .712;

and hence, by means of TABLE IV. $(1 + \frac{h_a}{h_b})^{\frac{3}{2}} - (\frac{h_a}{h_b})^{\frac{3}{2}}$

becomes of the following respective values suited to the above velocities,

$1.015, 1.060, 1.122, 1.205, 1.3, 1.403$; for a coefficient of .648,

and

$1.019, 1.040, 1.104, 1.176, 1.254, 1.344$; for a coefficient of .712.

These latter values multiplied in order by the initial values of the discharges, 158.1 and 173.9, in the above table, give the discharges in the third and fifth lines corresponding, due to the respective velocities of approach.

The accordance between the results in the last two lines of the table is remarkable. TABLE V. shows that if the coefficient be .617 when the water above the crest is still, it will be increased to .712 when the approaching channel is about 1.83 times the section of the water in the notch. If the arrises of the two-inch thick waste board be rounded, the coefficient must also be considerable, although

TABLE showing the actual discharges per foot in weir gauge four feet weir; and the discharges by different formulae brought forward during the Metropolitan Sewage Discussion. In the approach in feet per second. See p. 22, Letter to the Right Hon. Lord John Manners, M.P., from the Government Referees, dated 16th August, 1858. The manner of obtaining the experimental values is not stated in the Letter. The experimental discharges give values for the coefficient of discharge, c_d , from .695 to .737 nearly.

Descriptions and Formulæ.	Heads, initial velocities, and corresponding discharges in cubic feet per minute for each foot in length of weir.															
HEADS OF WATER ON WEIR.	1 inch, ·083 feet.	2 inches, ·167 feet.	3 inches, ·25 feet.	4 inches ·333 feet.	5 inches, ·417 feet.	6 inches, ·5 feet.	7 inches, ·583 feet.	8 inches, ·667 feet.								
Initial velocities of stream in feet per second	·033	·71	·092	·98	·16	·73	·18	1.02	·32	1.27	·43	2.8	·53	1.81	·44	2.14
Quantities calculated by the formula $D = 5.5 \sqrt{h^3 + .8 h^2 v_a^2}$	5.5	6.5	15.5	18.3	28.6	30.5	44.1	48.3	61.9	68.8	81.4	115.5	103.5	119.4	125.6	150.2
Quantities calculated by the formula $D = 1.66 \{ (3 h + v_a^2)^2 - v_a^3 \}$	5.5	6.5	15.6	18.5	28.7	30.7	44.2	48.8	62.2	69.6	82.1	114.0	104.0	120.5	125.0	151.6
Quantities calculated by the formula $D = 4.8 \sqrt{h^3 + .1875 h^2 v_a^2}$	4.8	5.0	13.5	14.1	24.9	25.3	38.4	39.3	53.7	54.9	70.9	78.7	89.1	92.72	108.9	110.6
Actual quantities measured	5.7	6.7	15.8	18.3	28.0	29.9	43.8	48.2	61.1	67.6	82.7	113.0	105.0	118.0	125.0	152.8
Values of the coefficients, x , in the formula $D = x \sqrt{h^3 + .8 h^2 v_a^2}$ as obtained from the actual quantities discharged	5.7	5.66	5.58	5.50	5.37	5.38	5.46	5.48	5.43	5.41	5.56	5.38	5.59	5.45	5.47	5.61

certain; but as the equation $D_a = 5.5 \sqrt{h_b^3 + .8 v_a^2 h_b^3}$ appears to have been framed by Mr. Taylor, to press special experiments made for Mr. Simpson, which the quantities varied from 5 to 152 cubic ft per minute, and for heads on a four-foot weir varying from 1 inch to 8 inches,* we must conclude the coefficient for heads measured from still water above the crest in those experiments suited to the form of the weir used, and its attendant circumstances, is .712.

The equations (39) and those from (A) to (O) may easily be changed into forms in which only the depth h_b , the velocity of approach, and the coefficient velocity (in this case equal to that of discharge) c_d are introduced. It is, however, only necessary here to reduce the general form (A) p. 111, for feet measures, which becomes, after substituting for h_a its value $\frac{v_a^2}{2g}$, and making some reductions,

$$\begin{cases} D_a = 5.35 c_d \times \left(\frac{1}{c_d^2} \times 64.4 \right)^{\frac{3}{2}} \{ (c_d^2 \times 64.4 h_b + v_a^2)^{\frac{3}{2}} - v_a^3 \} \\ D_a = \frac{.01034}{c_d^3} \{ (64.4 c_d^2 h_b + v_a^2)^{\frac{3}{2}} - v_a^3 \}; \end{cases}$$

for time in minutes the discharge is

$$D_a = \frac{.621}{c_d^3} \{ (64.4 c_d^2 h_b + v_a^2)^{\frac{3}{2}} - v_a^3 \};$$

which v_a still continues the velocity in feet per second, as determined from observation. These for-

*Vide p. 22, Letter dated 10th August, 1858, from the eminent Referees to the Right Hon. Lord John Manners, on the subject of the Metropolitan Main Drainage.

mulæ may be again reduced to many others. If we take h_b in inches (T) becomes

$$(U.) \quad D_a = \frac{.621}{c_a^3} l \{ (5.37 c_a^3 h_b + v_a^3)^{\frac{3}{2}} - v_a^3 \}.$$

Mr. Pole, in a letter to Mr. Simpson and Captain Galton, already referred to, gives the special value,

$$D_a = 1.06 l \{ (3 h_b + v_a^3)^{\frac{3}{2}} - v_a^3 \},$$

which corresponds very closely with the experiments made for Mr. Simpson. If we assume $c_a = .712$, which also closely corresponds with those experiments, our equation (U) becomes for them

$$D_a = 1.225 l \{ (2.72 h_b + v_a^3)^{\frac{3}{2}} - v_a^3 \};$$

but the amount of the discharge must always depend on the coefficient c_a , equation (U) suited to the special circumstances of the case under consideration.

The form of equation for the discharge proposed by Mr. Boyden* includes the effects of the end contractions: it is

$$D = C \{ l - b n h_b \} h_b^{\frac{3}{2}}$$

in which $C = \frac{2}{3} c_a \sqrt{2g h_b}$, n the number of end contractions, l the length of the weir, h_b the head measured from the surface of the water above the curvature of approach, and b a coefficient due to the nature of the end contractions. The mean numerical exponent of this formula, derived by Francis from his experiments, is for feet measures, per second,

$$D = 3.33 (l - .1 n h_b) h_b^{\frac{3}{2}} \dagger,$$

* Francis's Lowell Hydraulic Experiments, p. 74.

† Ibid, p. 119.

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the value of c varied from 3.303 to 3.3617. These results give corresponding values of $c_d = .617$ to $.628$, and when $c = 3.33$, $c_d = .623$. The experimental results compared with this formula have been referred to at p. 83.

Francis's Lowell experiments on a wooden dam 10 feet long, level and 3 feet wide at the crest, with each slope of $3\frac{1}{4}$ to 1 in a channel 10 feet wide, and for heads between 6 and 20 inches, a mean coefficient of .563 or .565. This for feet measures would give for the discharge per second

$$D = 3.02 h^{\frac{3}{2}}.$$

At greater depths, on this width of crest, the discharge would probably rise as high as $3.1 h^{\frac{3}{2}}$ or $3.3 h^{\frac{3}{2}}$. The section of the dam was the same as that erected by the Essex Company across the Merrimack River, at Lawrence, Massachusetts. See, also, table of coefficients, p. 80.

In equation (13), pp. 54 and 55, we have given a general expression for the value of D through a triangular notch. Professor Thomson, of the Queen's College, Belfast, in a paper read at the British Association at Leeds in 1858, says:—

The ordinary rectangular notches, accurately experimented on as they have been, at great cost and with high scientific skill, in various countries, with a view of determining the necessary formulas and coefficients for their application in practice, are for many purposes suitable and convenient. They are, however, but ill adapted for the measurement of very variable quantities of water, such as commonly occur to the engineer to be gauged in rivers and streams.

If the rectangular notch is to be made wide enough to allow the water to pass in flood times, it must be so wide that for long periods, in moderately dry weather, the water flows so shallow over its crest, that its indications cannot be relied on. To remove, in some degree, this objection, gauges for rivers or streams are sometimes formed, in the best engineering practice, with a small rectangular notch cut down below the general level of the crest of a large rectangular notch. If now, instead of one depression being made for dry weather, we use a crest wide enough for use in floods, we conceive of a large number of depressions extending so as to give the crest the appearance of a set of steps of stairs, and if we conceive the number of such steps to become infinitely great, we are led at once to the conception of the triangular instead of the rectangular notch. The principle of the triangular notch being thus arrived at, it becomes evident there is no necessity for having one side of the notch vertical, and the other slanting; but that, as may in many cases prove more convenient, both sides may be made slanting, and their slopes may be alike. It is then to be observed, that by the use of the triangular notch, with proper formulas and coefficients derivable by due union of theory and experiments, quantities of running water from the smallest to the largest may be accurately gauged by their flow through the same notch. The reason of this is obvious, from considering that in the triangular notch, when the quantity flowing is very small, the flow is confined to a small space admitting of accurate measurement; and that

the space for the flow of water increases as the quantity to be measured increases, but still continues so as to admit of accurate measurement.

Further, the ordinary rectangular notch, when applied for the gauging of rivers, is subject to a serious objection from the difficulty or impossibility of properly taking into account the influence of the bottom of the river on the flow of the water to the notch. If it were practicable to dam up the river so deep that the water would flow through the notch as if coming from a reservoir of still water, the difficulty would not arise. This, however, can seldom be done in practice, and although the bottom of the river may be so far below the crest as to produce but a little effect on the flow of the water when the quantity flowing is small, yet when the quantity becomes great, the velocity of approach comes to have a very material influence on the flow of the water, but an influence which is usually difficult, if not impracticable to ascertain with satisfactory accuracy. In the notches now proposed of a triangular form, the influence of the bottom may be rendered definite, and such as to affect alike (or at least by some law that may be readily determined by experiment) the flow of the water when very small or when very great, in the same notch. The method by which I propose that this may be effected consists in carrying out a floor, starting exactly from the vertex of the notch, and extending both up-stream and laterally, so as to form a bottom to the channel of approach, which will both be smooth and will serve as the lower bounding surface

of a passage of approach unchanging in form while increasing in magnitude, at the places at least which are adjacent to the vertex of the notch. The floor may be either perfectly level, or may consist of two planes, whose intersection would start from the vertex of the notch, and would pass up-stream perpendicularly to the direction of the weir board; the two planes slanting upwards from their intersection more gently than the sides of the notch. The level floor, although theoretically not quite so perfect as the floor of two planes, would probably for most practical purposes prove the more convenient arrangement.

“With reference to the use of the floor it may be said, in short, that by a due arrangement of the notch and the floor a discharge orifice and channel of approach may be produced, of which (the upper surface of the water being considered as the top of the channel and orifice) the form will be unchanged or but little changed, with variations of the quantity flowing; very much less certainly than is the case with rectangular notches.

“Whatever may be the result in this respect, the main object must be to obtain, for a moderate number of triangular notches of different forms, and both with and without floors at the passage of approach, the necessary coefficients for the various forms of notches and approaches selected, and for various depths in any one of them, so as to allow of water being gauged for practical purposes, when in future convenient, by means of similarly formed notches and approaches. The util-

ity the proposed system of gauging it is to be particularly observed, will not depend upon a perfect close agreement of the theory described with the experiments, because a table of experimental coefficients for various depths, or an empirical formula slightly modified from the theoretical one, will serve all purposes.

“One evident simplification in the proposed system of gauging, as compared with that by rectangular notches, I would here advert, namely, that in the proposed system the quantity flowing comes to be a function of only one variable—namely, the measured head of water—while in the rectangular notch it is a function of at least two variables, namely the head of water, and the horizontal width of the notch; and is commonly also a function of a third variable very difficult to be taken into account, namely the depth from the crest of the notch down to the bottom of the channel of approach, which must vary in its influence with all the varying ratios between it and the other two quantities of which the flow is a function.

“The proposed system of gauging also gives facilities in taking another element into account which often arises in practice—namely, the influence of back water on the flow of the water in the gauge, when, as frequently occurs in rivers, it is found impracticable to dam the river up sufficiently to give it a clear overfall free from the back or tail water. For a given ratio of the height of the tail water above the vertex of the notch to the height of head water above the vertex of the notch, I would an-

ticipate that the quantities flowing would still be approximately at least, proportional to the $\frac{5}{2}$ power of the head, as before; and a set of coefficients would have to be determined experimentally for different ratios of the height of the head water to the height of the tail water above the vertex of the notch.

"I have got some preliminary experiments made on a right-angled notch in a vertical plane surface, the sides of the notch making angles of 45° with the horizon, and the flow being from a deep and wide pool of quiet water, and the water thus approaching the notch uninfluenced by any floor or bottom. The principal set of experiments as yet made were on quantities of water varying from about 2 to 10 cubic feet per minute; and the depths or heads of the water varied from 2 inches to 4 inches in the right-angled notch. From these experiments I derive the formula

$$Q = 0.317 H^{\frac{5}{2}},$$

where Q is the quantity of water in cubic feet per minute, and H the head as measured vertically in inches from the still water level of the pool down to the vertex of the notch. This formula is submitted at present temporarily as being accurate enough for use for ordinary practical purposes for the measurement of water by notches similar to the one experimented on, and for quantities of water limited to nearly the same range as those in the experiments; but as being, of course, subject to amendment by more perfect experiments extending through a wider range of quantities of water."

In the first edition of this book we gave the general form of the equation for the discharge through

angular notches, and also showed the general application of the coefficients $\cdot 617$ to $\cdot 628$ for all forms of orifices and notches in thin plates. $\cdot 617$, as shown on p. 55, gives a result identical with the practical results of Professor Thomson's experiments.

The great advantage of the triangular notch for gauging is, that the sections for all depths flowing over are similar triangles, and therefore the coefficient probably remains constant, or nearly so, not only for one but for all species of triangles, when the depth at the open is not very little indeed in proportion to the width flowing over at the surface.

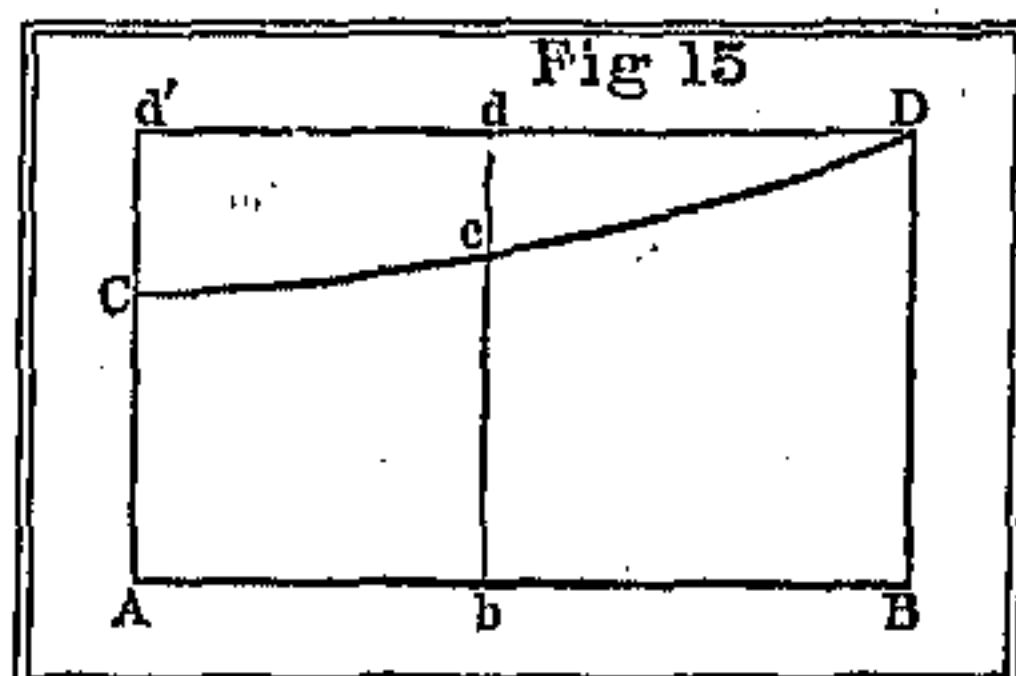
The disadvantage of the proposed triangular form of depression, if permanent in a dam, would be that the angular point should be at a lower level than the crest of a horizontal crest to maintain the same level, or the level, of the water during floods; and therefore the pressure of the water and head would be reduced at the period when most required for mill-power or navigation purposes; that is, during dry weather. For drainage purposes the winter level or that during floods, must evidently be kept down, unless the banks are steep, and along rapids; but these remarks do not apply to dams erected across mountain creeks or streams where the banks are, generally, considerably above floods. These remarks refer to occasions for permanent gauging to find the relations of evaporation, absorption, and discharge in given command areas. In notch gauging to determine the useful effect of water engines, rectangular forms in thin plates have the coefficients already well determined, and the calculations are easy.

DIFFERENT EFFECTS OF CENTRAL AND MEAN VELOCITIES.

There is, however, another element to be taken into consideration, and which we shall have to refer to more particularly hereafter; it is this, that the central velocity, directly facing the orifice, is also the maximum velocity in the tube, and not the mean velocity. The ratio of these velocities is $1 : .835$ nearly; hence, in the example, p. 110, where $\frac{C}{A} = 3$, we get $3 \times .835 = 2.505$ for the value of $\frac{C}{A}$ in column 1, TABLE V., opposite to which we shall find .649, the coefficient for an orifice of one-third of the section of the tube when cylindrical or prismatic, Fig. 13; and $3 \times .835 \times .807 = 2.02$ nearly, opposite to which we shall get .661 for the coefficient when the orifice is at the end of the short tube, Fig. 14, with a rounded junction. We have, therefore, $\frac{C}{A} \times .835$ equal to the new value of $\frac{C}{A}$ for finding the discharge from orifices at the end of cylindrical or prismatic tubes, and $\frac{C}{A} \times .835 \times .807 = \frac{C}{A} \times .67$ nearly for the new value of $\frac{C}{A}$ when finding the discharge from orifices at the end of a short tube with a rounded junction.

The ratio of a mean velocity in the tube to that facing the orifice cannot be less than .835 to 1, and varies up to 1 to 1; the first ratio obtaining when the orifice is pretty small compared with the sec-

tion of the tube, and the other when they are equal. If we suppose the curve DC , whose abscissæ (Ab) represent the ratio of the orifice to the section of the tube, and whose ordinates (bC) represent the ratio of the mean velocity in the tube to that issuing from the orifice, to be a parabola, we shall find the following values:—

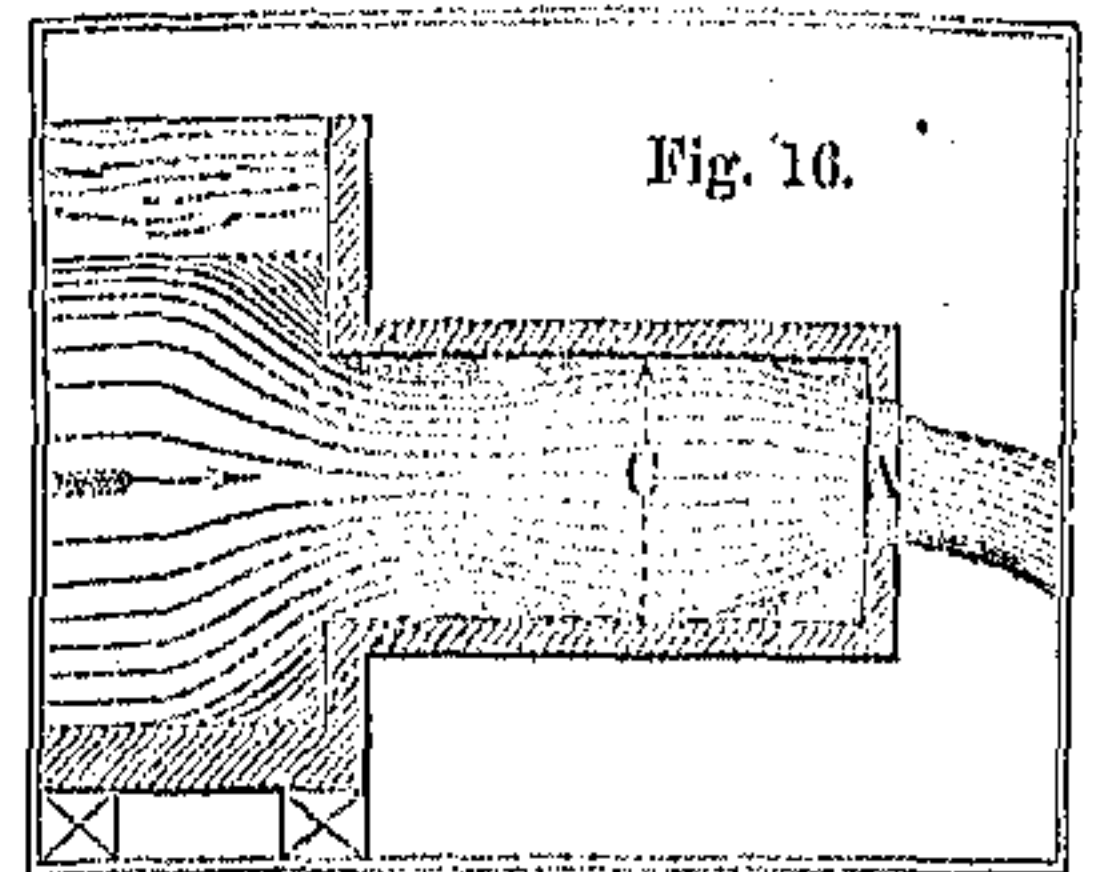


Ratio of the orifice to the channel, or values of $\frac{A}{C} = \frac{Ab}{AB}$	Values of dC	Ratio of the mean velocity of approach in a tube or channel to that directly opposite the orifice, or values of bC
·0	·165	·835
·1	·163	·837
·2	·158	·842
·3	·150	·850
·4	·139	·861
·5	·124	·876
·6	·106	·894
·7	·084	·916
·8	·059	·941
·9	·031	·969
1·0	·000	1·000

These values of bC are to be multiplied by the corresponding ratio $\frac{C}{A}$ in order to find a new value, to which will be found, in the table, the coefficient for orifices at the ends of short prismatic

or cylindrical tubes; and this new value again multiplied by $\cdot 807$, or more generally by $c_d \left\{ \frac{1}{1-c_d^2} \right\}^{\frac{1}{2}}$, will give another new value of $\frac{Q}{A}$, opposite to which, in the table, will be found the coefficient for orifices at the ends of short tubes with rounded junctions.

EXAMPLE III. *What shall be the discharge from an orifice A, Fig. 16, 2 feet long by 1 foot deep, when the value of $\frac{Q}{A}$ is 3, and the depth of the centre of A 1 foot 6 inches*

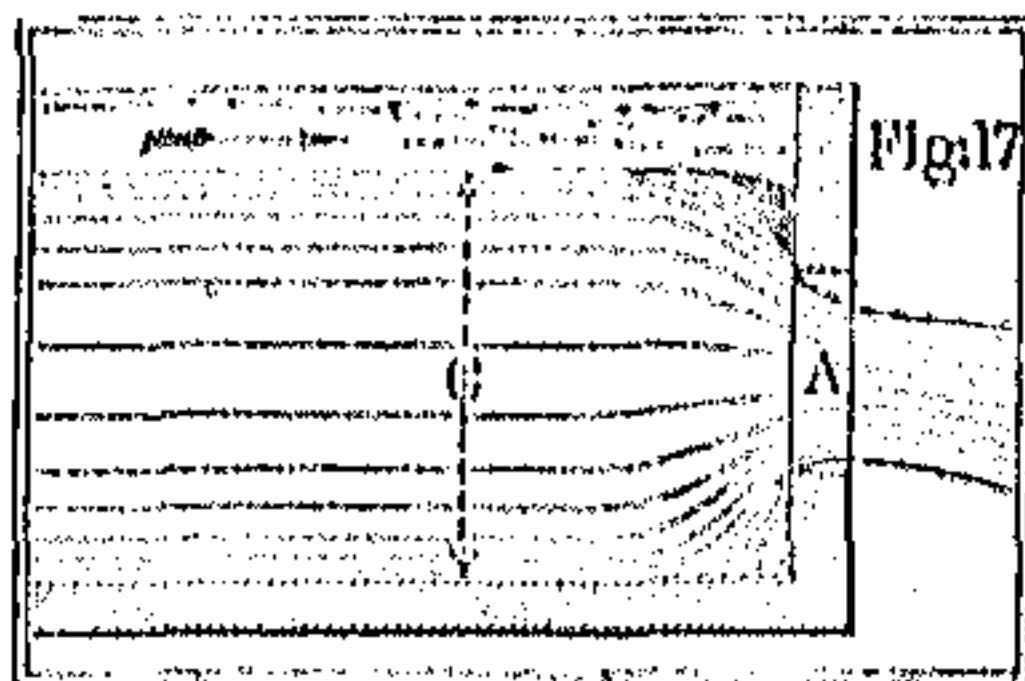


below the surface? We have $v_t = 2 \times 1 \times \frac{117.946}{12}$ (TABLE II.) $= 2 \times 9.829 \times 19.658$ cubic feet per second for the theoretical discharge. From the table on last page the coefficient for the mean velocity, facing the orifice, is about $\cdot 86$; hence $\frac{Q}{A} \times \cdot 86 = 3 \times \cdot 86 = 2.58$. If we take the coefficient from TABLE I., we shall find it (opposite to 2, the ratio of the length of the orifice to its depth) to be $\cdot 617$; and, for this coefficient, opposite to 2.58, in TABLE V., or the next number to it, we find the required coefficient $\cdot 636$; hence the discharge is $\cdot 636 \times 19.658 = 12.502$ cubic feet per second. If we assume the coefficient in still water to be $\cdot 628$, then we shall obtain the new co-

* See p. 106, with reference to the modifications of equations (45) and (46) into (45a) and (46a) suited to $h_n = \frac{v_n^2}{2g c_d^2}$.

eff nt $\cdot 647$, and the discharge would be $\cdot 647 \times$
 19 $8 = 12\cdot 719$ cubic feet. If the junction of the
 tu with the cistern be rounded, as shown by the
 do l lines, we have to multiply $2\cdot 58$ by $\cdot 807$, which
 giv $2\cdot 08$ for the new value of $\frac{Q}{A}$, opposite which we
 shu find, in TABLE V., when the first coefficient is
 $\cdot 62$ the new coefficient $\cdot 659$; and the discharge in
 thi ase would be $\cdot 659 \times 19\cdot 658 = 12\cdot 955$ cubic
 fee or second.

It is not necessary to
 tak out the coefficient
 of in velocity facing
 the lice to more than
 two aces of decimals.
 For range notches in
 thin plates placed in
 stre s and millraces, Fig. 17, the mean coefficient
 $\cdot 628$ for still water, may be assumed; thence the new
 coel ent suited to the ratio $\frac{Q}{A}$ may be found, as in
 the st portion of EXAMPLE III. We shall leave
 the picking out of the results when h_a is taken equal
 to $\frac{1}{2}h$ to the student.



EXAMPLE IV. *What shall be the discharge through the cistern A, equal 2 feet by 1 foot, when the channel is to the orifice as 3.375 to 1, and the depth of the centre of the orifice is 1.25 foot below the surface, taken at about 3 feet above the orifice? Here the coefficient of the approaching velocity is $\cdot 85$ nearly, whence the new value of $\frac{Q}{A}$ is $3\cdot 375 \times \cdot 85 = 2\cdot 87$; and as $c_d = \cdot 628$,*

we shall get from TABLE V, the new coefficient .644. Hence

$$D = 2 \times 1 \times \frac{107.669}{12} \times .644 \text{ (TABLE II.)} = 2 \times 8.972 \times .644 \\ = 17.944 \times .644 = 11.556 \text{ cubic feet per second.}$$

Weisbach finds the discharge, by an empirical formula, to be 11.31 cubic feet. If the coefficient be sought in TABLE I., we shall find it .617 nearly, from which, in TABLE V., we shall find the new coefficient to be .632: hence $17.944 \times .632 = 11.341$ cubic feet per second. If the coefficient .6225 were used, we should find the new coefficient equals .638, and the discharge 11.468 cubic feet. *Or thus:* The ratio of the head at the upper edge to the depth of the orifice is $\frac{9}{12} = .75$, and from TABLE IV. we find

$(1.75)^{\frac{3}{2}} - (.75)^{\frac{3}{2}} = 1.6655$. Assuming the coefficient to be .644, we find from TABLE VI. the discharge per minute over a weir 12 inches deep and 1 foot long to be

$$\frac{208.650 + 205.119}{2} = 206.884 \text{ cubic feet nearly; and}$$

as the length of the orifice is 2 feet, we have

$$\frac{2 \times 206.884 \times 1.6655}{60} = 11.482 \text{ cubic feet per second, which}$$

is the correct theoretical discharge for the coefficient .644, and less than the approximate result, 11.556 cubic feet above found, by only a very small difference. The velocity of approach in this example must be derived from the surface inclination of the stream. The working out of this example and the

increase of the discharge when $h_a = \frac{v_a^2}{2g c_d^2}$ will afford practice to the student.

For notches or Poncelet weirs the approaching velocity is a maximum at or near the surface. If the central velocity at the surface facing the notch be 1, the mean velocity from side to side will be .914. We therefore assume the variation of the central to the mean velocity to be from 1 to .914; and hence the ratio of the mean velocity at the surface of the channel to that facing the notch or weir cannot be less than .914 to 1, and varies up to 1 to 1; the first ratio obtaining when the notch or weir occupies a very small portion of the side or width of the channel, and the other when the weir extends for the whole width. Following the same mode of calculation as in Art. 129, Fig. 15, we shall find as follows:—

Ratio of the width of the notch to the width of the channel.	Values of $d e$, Fig. 15.	Values of $b c$, Fig. 15.
.0	.086	.914
.1	.085	.915
.2	.083	.917
.3	.078	.922
.4	.072	.928
.5	.064	.936
.6	.055	.945
.7	.044	.956
.8	.031	.969
.9	.016	.984
1.0	.000	1.000

These values of $b c$ are to be used as before in order to find the value of $\frac{Q}{A}$, opposite to which in the

tables, and under the heading for weirs, will be found the new coefficient.

EXAMPLE V. *The length of a weir is 10 feet; the width of the approaching channel is 20 feet; the head, measured about 6 feet above the weir, is 9 inches; and the depth of the channel 3 feet: what is the discharge?*

Assuming the circumstances of the overfall to be such that the coefficient of discharge for heads, measured from still water in a deep weir basin or reservoir, will be .617, we shall find from TABLE VI. the discharge to be $128.642 \times 10 = 1286.42$ cubic feet per minute; but from the smallness of the channel the water approaches the weir with some

velocity, and $\frac{C}{A} = \frac{20 \times 3}{10 \times \frac{3}{4}} = 8$. We have also the width of the channel equal to twice the width of the weir, and hence (small table, p. 133,) $8 \times .936 = 7.488$ for the new value of $\frac{C}{A}$. From TABLE V. we now find the

new coefficient $\frac{.622 + .624}{2} = .623$, and hence the dis-

charge is $\frac{1286.42 \times .623}{.617} = 1298.93$ cubic feet per minute.

Or thus: As the theoretical discharge, TABLE VI., is 2084.96 cubic feet, we get $2084.96 \times .623 = 1298.93$, the same as before. In this example, however, the mean velocity approaching the overfall bears to the mean velocity in the channel a greater ratio than $1 : .936$, as, though the head is pretty large in proportion to the depth of the channel, the ratio of the sections $\frac{A}{C} = \frac{1}{8}$ is small. We shall therefore be more correct by finding the multiplier from the small table,

9. By doing so the new value of $\frac{Q}{A}$ is $8 \times .838 = 6.704$. From this and the coefficient .617 we shall find as before from TABLE V., the new coefficient to be .627; hence we get $2084.96 \times .627 = 1307.27$ cubic feet per minute for the discharge.

The foregoing solution takes for granted that the velocity of approach is subject to contraction before arriving at the overfall or in passing through it; and as this reduces the mean velocity of approach from 1 to .784, TABLE V., when the coefficient for the velocity in still water is .617, we have to multiply the

value of $\frac{Q}{A} = 6.704$, last found, by .784, and we get

$6.704 \times .784 = 5.26$ for the value $\frac{Q}{A}$ due to this cor-

rection, from which we find the corresponding coefficient in TABLE V. to be .629, and hence the corrected discharge is $2084.96 \times .629 = 1311.44$ cubic

feet. *It is to be borne in mind that the value of $\frac{Q}{A}$ in*

TABLE V. is simply an approximate value for the ratio of the velocity in the channel facing the orifice to the velocity in the orifice itself; and the corrections applied in the foregoing examples were for the purpose of finding this ratio of velocity more correctly

the simple expression $\frac{Q}{A}$ gives it. The following

auxiliary table will enable us to find the correction, and hence the new coefficient, with facility. Thus,

if the channel be five times the size of the orifice, and the loss in the approaching velocity takes place equal to that in a short cylindrical tube, we get

AUXILIARY TABLE, TO BE USED WITH TABLE V. FOR MORE NEARLY FINDING THE COEFFICIENT OF DISCHARGE NEARLY SUITED TO EQUATIONS (45 a) AND (46 a).

Ratio of the orifice to the channel, or $\frac{a}{A}$	Multipliers due to the difference of the central and mean velocity only.	Multipliers for finding the new values of $\frac{C}{A}$ in TABLE V., when the water approaches and passes through the orifice, without contraction or loss of velocity.					
		Coefficient for orifices	Coefficient for weirs	Coefficient for orifices	Coefficient for weirs	Coefficient for orifices	Coefficient for weirs
0	.835	.60	.67	.65	.64	.63	.60
.1	.837	.70	.68	.66	.64	.62	.60
.2	.842	.70	.68	.66	.64	.62	.61
.3	.850	.71	.69	.67	.65	.63	.61
.4	.861	.72	.69	.68	.66	.64	.62
.5	.870	.73	.71	.69	.67	.65	.63
.6	.884	.74	.72	.70	.68	.66	.64
.7	.916	.76	.74	.72	.70	.68	.66
.8	.941	.78	.76	.74	.72	.70	.68
.9	.960	.81	.78	.76	.74	.72	.70
1.0	1.000	.831	.807	.784	.762	.740	.710

$5 \times .842 = 4.210$ for the new value of $\frac{C}{A}$, opposite to which, in TABLE V., will be found the coefficient sought. If the coefficient for still water be .606, we shall find it to be .612 for orifices and .623 for weirs. But when the water approaches without loss of velocity, we find from the auxiliary table .64 for the multiplier instead of .842, and consequently the new value of $\frac{C}{A}$ becomes $5 \times .64 = 3.2$, from which we shall find .617 to be the new coefficient for orifices and .636 for weirs. The auxiliary table is calculated by multiplying the numbers in the second column (see third column, table, p. 129) by the value of $c_a \times \left\{ \frac{1}{1 - c_a^2} \right\}^{\frac{1}{2}}$, which will be found from TABLE V., for the different values of c_a in the table, viz.

.61 .628, .617, .606, .595, .584, and .573, to be
 .81 .807, .784, .762, .740, .719, and .699 respec-
 tiv , as given in the top and bottom lines of figures.

en $\frac{c_d^2}{m^2 - c_d^2}$ in equations (45) and (46) is equal to

$\frac{1}{m^2}$ in equations (45a) and (46a), then $c_d = 1$,

and $\left\{1 + \frac{c_d^2}{m^2 - c_d^2}\right\}^{\frac{1}{2}}$ in equation (45) is equal to

$\left\{1 + \frac{1}{m^2 - 1}\right\}^{\frac{1}{2}}$ in equation (45a); and $c_d \left\{(1 + \frac{1}{m^2 - 1})^{\frac{3}{2}} - (\frac{1}{m^2 - 1})^{\frac{3}{2}}\right\}$ in equation (46) is equal to

$\left\{(1 + \frac{1}{m^2 - 1})^{\frac{3}{2}} - (\frac{1}{m^2 - 1})^{\frac{3}{2}}\right\}$ in equation (46a); and

the are the coefficient found from TABLE V. for

$c_d = 1$ will give the multiplier for c_d , outside the

bra ts, in (45a) and (46a), to find the new coeffi-

cier Thus in the last example $m = 5$, and hence

TAB V. for $c_d = 1$, we find $\left\{1 + \frac{1}{m^2 - 1}\right\}^{\frac{1}{2}} = 1.021$

and $\left\{(1 + \frac{1}{m^2 - 1})^{\frac{3}{2}} - (\frac{1}{m^2 - 1})^{\frac{3}{2}}\right\} = 1.055$. Hence

1.02 $\times .606 = .619$ nearly; and $1.055 \times .606 = .639$

near the new coefficients found from the other

metl being .617 and .636, the difference by both

metl s being of no great practical importance.

It necessary to observe, that in equations (45),

(46), 5a), and (46a), the head due to the velocity of

supp or approach, h_v , must be extra to the head h ,

and : part of it, and that—as is indicated by the

equal is— m can never be so small as unity. These

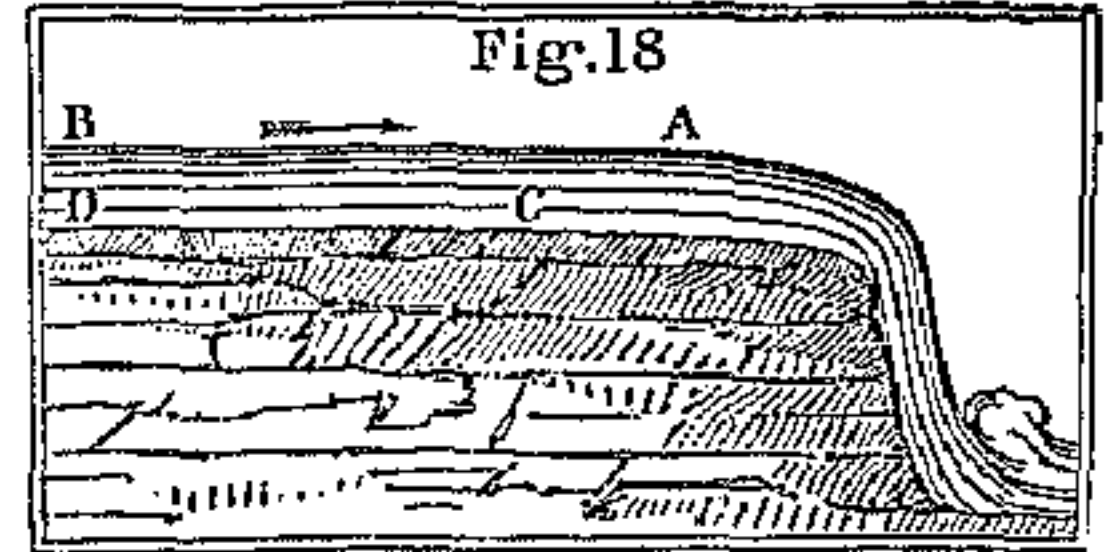
equal is are not, therefore, strictly applicable to

orific in the short tubes, Fig. 15 and Fig. 16, al-

though they can be made practically so within definite limits. The initial value of c_d itself varies considerably with the position and form of the orifice; for a mean value of .707 it changes according to the relation of c and A into $\frac{.707}{(1 - .5 \frac{A^2}{c^2})^{\frac{1}{2}}}$, and for a value of

.618 for an orifice, central in a thin plate, Professor Rankine's formula, p. 110, is applicable.

In weirs at right angles to channels with parallel sides, the sectional area can never equal that of the channel unless it be measured at or above the point A, where the sinking of the overfall commences; and unless also the bed $c d$ and surface $A B$ have the same inclination. In all open channels, as mill-races, streams, rivers, the supply is derived from the surface inclination of $A B$, and this inclination regulates itself to the discharging power of the overfall. When the overfall and channel have the same width, and it is considerable, we have, as shall appear hereafter, $91 \sqrt{h s}$ for the mean velocity in the channel, where h is the depth in feet and s the rate of inclination of the surface $A B$. We have also $\frac{2}{3} \sqrt{2 g h}$ for the theoretical velocity of discharge at the overfall, of equal depth with the channel, and, when both velocities are equal,

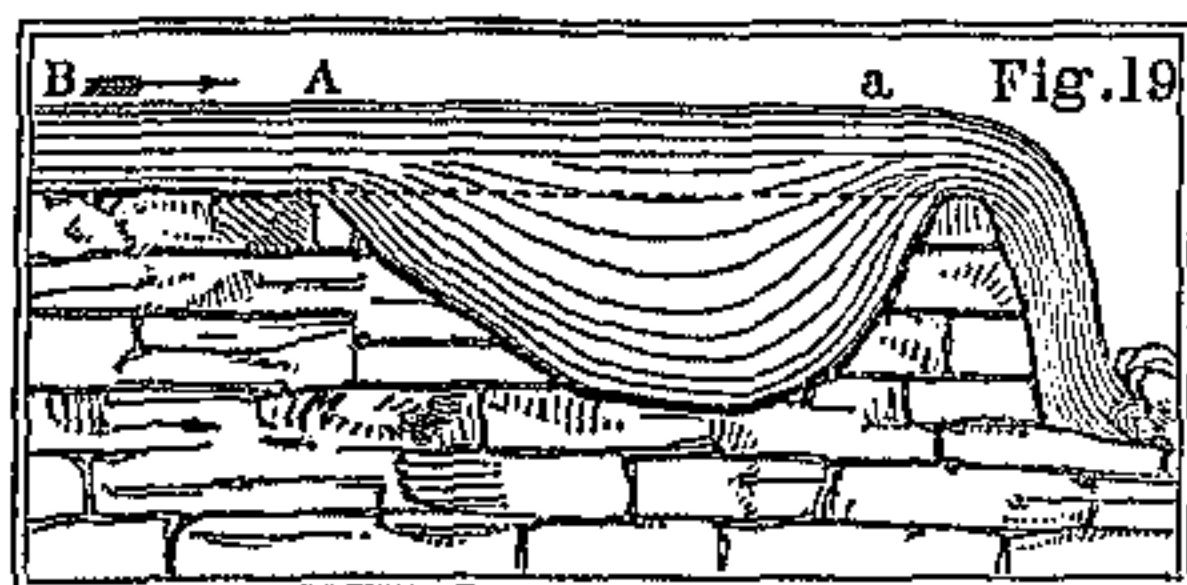


$$\frac{2}{3} \sqrt{2 g h} = 5.35 \sqrt{h} = 91 \sqrt{h s};$$

from which we find

$$s = \frac{1}{289} = .00346,$$

the inclination of B A when the supply is equal to the theoretical discharge at the overfall. If the coefficient at the overfall were .628, or, which is nearly the same thing, if a large and deep weir basin intervened between the weir and channel, Fig. 19, A a



would be level, the velocity of approach would be destroyed, and we should have

$$5.35 \times .628 \sqrt{h} = 3.36 \sqrt{h} = 91 \sqrt{hs};$$

and hence the inclination of A B

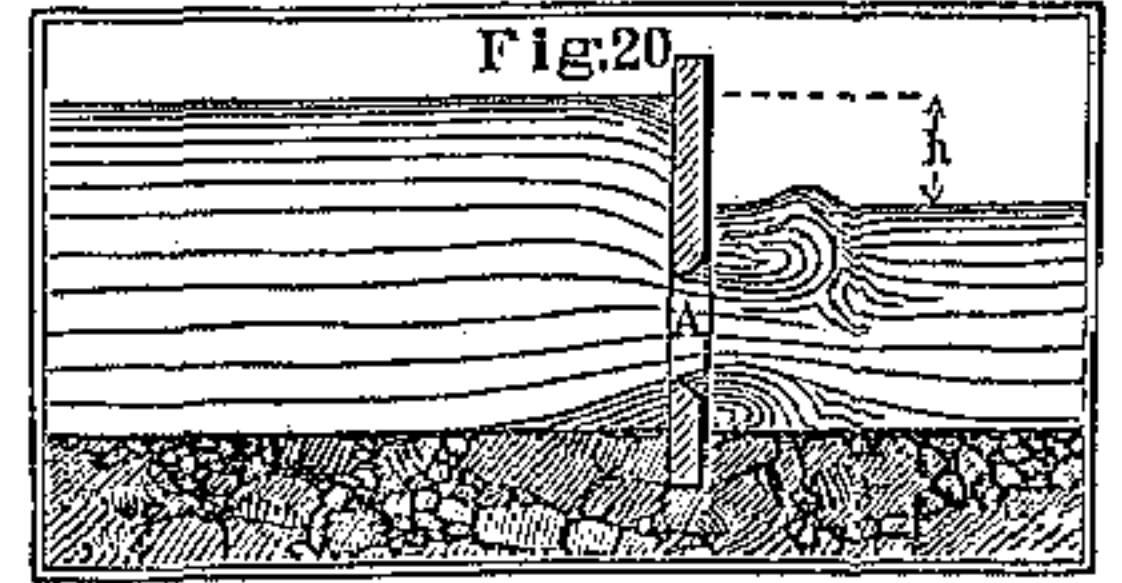
$$s = \frac{1}{734} = .00136$$

very nearly. When we come to discuss the surface inclination of rivers, we shall see that the conditions here assumed and the resulting surface inclinations would involve a considerable loss of head. If the quantity discharged under both circumstances be the same and h be the depth in the first case, Fig. 18, we shall then have the head in the latter case, Fig. 19, equal $\left(\frac{5.35}{3.36}\right)^{\frac{2}{3}} h = 1.36 h$ very nearly, from which and the surface inclination the extent of the backwater may be found with sufficient accuracy. When, in Fig. 19, the inclination of A B exceeds $\frac{1}{734}$, the head at a must exceed the depth of the river above A. We must refer to pages further on, SECTION X, for some remarks on the backwater curve.

SECTION V.

SUBMERGED ORIFICES AND WEIRS.—CONTRACTED
RIVER CHANNELS.

The available pressure at any point in the depth of the orifice A, Fig. 20, is equal to the difference of the pressures on each side.



This difference is equal to the pressure due to the height h , between the water surfaces on each side of the orifice; in this case, the velocity is

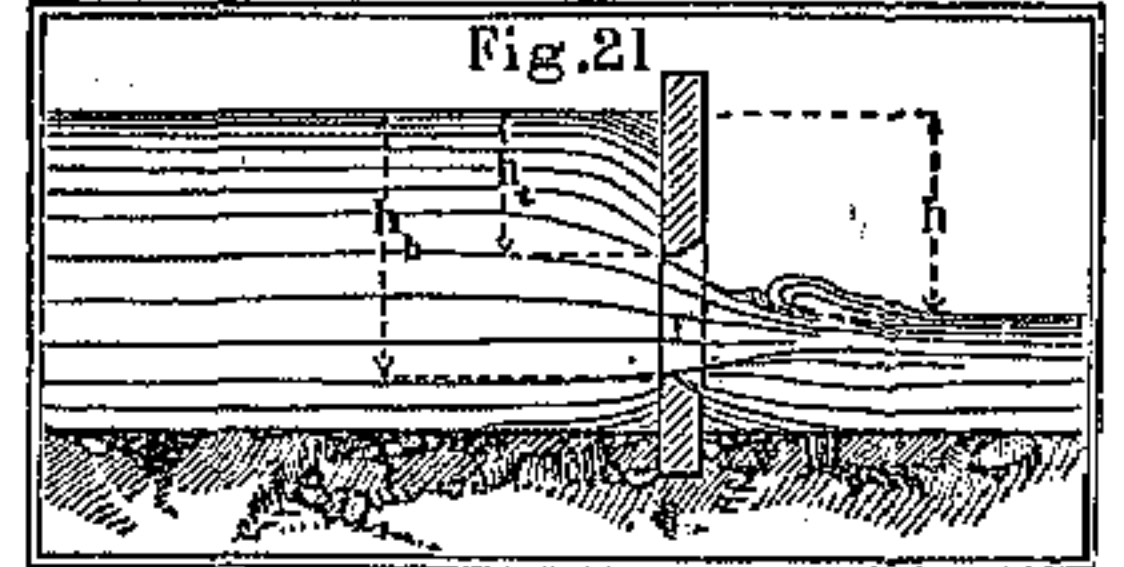
$$(47.) \quad v = c_d \sqrt{2gh};$$

and the discharge

$$(48.) \quad D = l d c_d \sqrt{2gh};$$

in which, as before, l is the length, and d the depth of the rectangular orifice A.

When the orifice is partly submerged, as in Fig. 21, we may put $h_o - h = d_2$ for the submerged depth,



and $h - h_i = d_1$, the remaining portion of the depth; whence $d_1 + d_2 = d$ is the entire depth. The discharge through the submerged depth d_2 is $c_d l d_2 \times \sqrt{2gh}$, and the discharge through the upper portion d_1 is

$$\frac{2}{3} c_d l \sqrt{2g} \{h^{\frac{3}{2}} - h_i^{\frac{3}{2}}\};$$

ence the whole discharge—assuming the coefficient of discharge c_d is the same for the upper and lower depths—is

$$D = c_d l \sqrt{2g} \left\{ d_2 \sqrt{h} + \frac{2}{3} (h^{\frac{3}{2}} - h_t^{\frac{3}{2}}) \right\}.$$

We may, however, equation (31), assume that

$$\frac{2}{3} c_d l \sqrt{2g} (h^{\frac{3}{2}} - h_t^{\frac{3}{2}}) = c_d d_1 l \sqrt{2g \left(h - \frac{d_1}{2} \right)}$$

nearly, and hence

$$D = c_d l d_2 \sqrt{2g h} + c_d l d_1 \sqrt{2g \left(h - \frac{d_1}{2} \right)}.$$

If $h + \frac{d_1}{2} = h - \frac{d_1}{2}$ this equation may be changed

$$D = c_d l d_2 \sqrt{2g h} + c_d l d_1 \sqrt{2g \left(h_t + \frac{d_1}{2} \right)}.$$

Whichever of these forms the values of

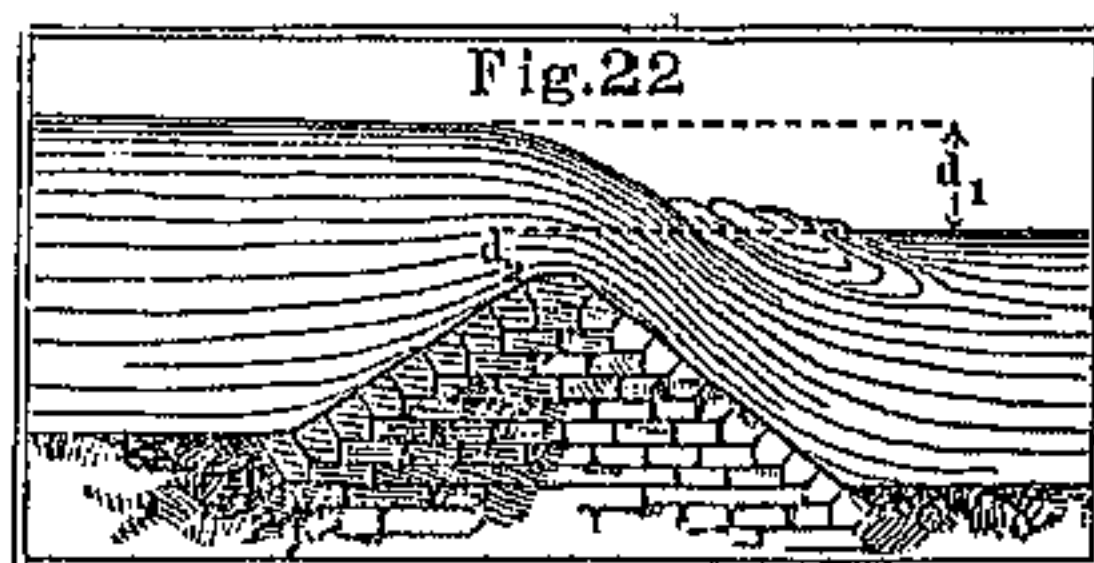
$$\sqrt{2g h}, c_d \sqrt{2g \left(h - \frac{d_1}{2} \right)}, \text{ and } c_d \sqrt{2g \left(h_t + \frac{d_1}{2} \right)}$$

be had from TABLE II., and the value of the discharge D thence easily found.

When the water approaches the orifice with a finite velocity, the height h_a due to that velocity can be found from TABLE II., and the discharge can be found by substituting $h + h_a$ and $h_t + h_a$ for h and h_t in the above equations.

In the submerged

Fig. 22, h becomes equal to d_1 , and D ; the discharge, from equation (49), then be-



$$(52.) \quad \begin{cases} D = c_d l d_2 \sqrt{2 g d_1} + \frac{2}{3} c_d l d_1 \sqrt{2 g d_1}, \text{ or} \\ D = c_d l \sqrt{2 g d_1} \left\{ d_2 + \frac{2}{3} d_1 \right\}. \end{cases}$$

When the water approaches with a velocity due to the height h_a , then h becomes $h + h_a$, $h_1 = h_a$, and equation (49) becomes

$$(53.) \quad D = c_d l \sqrt{2 g} \left\{ d_2 \sqrt{d_1 + h_a} + \frac{2}{3} (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\}.$$

In the improvement of the navigation of rivers, it is sometimes necessary to construct weirs so as to raise the upper waters by a given depth, d_1 . The discharge D is in such cases previously known, or easily determined, and from the values of d_1 and D we can easily determine, equation (52), the value of

$$(54.) \quad d_2 = \frac{D}{c_d l \sqrt{2 g d_1}} - \frac{2}{3} d_1;$$

or, by taking the velocity of approach into account,

$$(55.) \quad d_2 = \frac{D}{c_d l \sqrt{2 g (d_1 + h_a)}} - \frac{\frac{2}{3} (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}}{\sqrt{d_1 + h_a}}.$$

This value of d_2 must be the depth of the top of the weir below the original surface of the water, in order that this surface should be raised by a given depth, d_1 . When h_a is small compared with d_2 , we may take

$$\frac{2}{3} (d_1 + h_a)^{\frac{3}{2}} = \frac{2}{3} \times \frac{(d_1 + h_a)^{\frac{3}{2}} - (h_a)^{\frac{3}{2}}}{\sqrt{d_1 + h_a}} \text{ in equation (55).}$$

EXAMPLE VI. *A river whose width at the surface is 70 feet, whose hydraulic mean depth is 4.4 feet, and whose cross sectional area is 325 feet, has a surface inclination of 1 foot per mile; to what depth below, or height above the surface must a weir at right angles*

the channel be raised, so that the depth of water immediately above it shall be increased by $3\frac{1}{2}$ feet?

When the hydraulic mean depth is 4.4 feet, and a fall per mile 1 foot, we find from TABLE VIII. that the mean velocity of the river is 29.98 or 30 inches very nearly per second. The discharge is, therefore, $325 \times 2\frac{1}{2} = 812.5$ cubic feet per second,

48750 cubic feet per minute. Hence, $\frac{48750}{70} =$

696.4 cubic feet, must pass over each foot in length of the weir per minute. Assuming the coefficient $= .628$ in the first instance, we find from TABLE I. the head passing over a weir corresponding to a discharge to be 27.4 inches; but as the head is to be increased by $3\frac{1}{2}$ feet, or 42 inches, it is clear that the weir must be *perfect*; that is, have a clear fall, and rise $42 - 27.4 = 14.6$ inches over the original water surface. In order that the weir may not be submerged, or *imperfect*, the head could not be increased by more than 27.4 inches. Let us, therefore, assume in the example, that the increase shall be only 18 instead of 42 inches; the weir then becomes submerged, and we have, from equation (54),

$$d_2 = \frac{696.4}{.628 \sqrt{18'' \times 2g}} \times 18'' \text{ (as } l = 1 \text{ foot).}$$

The value of the first part of this expression is found from TABLE VI. or TABLE II. equal to

$$\frac{696.4}{\frac{18}{2} \times 370.341} = \frac{696.4}{370.341} = 1.88 \text{ feet} = 22.56 \text{ in.};$$

hence $22.56 - \frac{36}{3} = 10.56$ inches is the value of d_2 ;

that is, the submerged weir must be built within 10.56 inches of the surface to raise the head 18 inches above the former level. If, however, the velocity of approach be taken into account, we shall find this velocity equals $\frac{812.5}{430} = 2$ feet per second very nearly; and the height, or value of h_a , due to this velocity, taken from TABLE II., is $\frac{3}{4} = .75$ inches nearly; therefore, from equation (55),

$$d_2 = \frac{696.4}{.628 \sqrt{2g \times 18.75}} - \frac{2}{3} \times \frac{(18.75)^{\frac{3}{2}} - (.75)^{\frac{3}{2}}}{\sqrt{18.75}}.$$

The value of $\frac{696.4}{.628 \sqrt{2g \times 18.75}} =$ (from TABLE VI.)

$$\frac{696.4}{.628 \sqrt{2g \times 18.75}} = \frac{696.4}{378.8^*} = 1.84 \text{ feet} = 22.08 \text{ in.};$$

$$\frac{3}{2} \times \frac{12}{18.75} \times 393.75$$

$$\text{also, } \frac{2}{3} \times \frac{(18.75)^{\frac{3}{2}} - (.75)^{\frac{3}{2}}}{\sqrt{18.75}} = \frac{2}{3} \times 18.75 - \frac{2}{3} \times \frac{(.75)^{\frac{3}{2}}}{\sqrt{18.75}}$$

$$= 12.5 - \frac{2}{3} \times \frac{.65}{4.33} = 12.5 - .1 = 12.4.$$

Hence $d_2 = 22.08 - 12.4 = 9.68$ inches, or about 1 inch less than the value previously found from equation (54). The mean coefficient of discharge was here assumed to be .628. Experiments on submerged weirs show that the value of c_d varies from .5 up to .8, but as this coefficient would reduce the value of d_2 , or the depth of the top of the weir below the surface, it is safer (where a given depth above a weir

* This is found from TABLE II. more readily.

ist be obtained) to use the lesser and ordinary coefficients of perfect weirs, with a clear overfall, for finding the crest levels of submerged weirs, when it is necessary to construct them. If the coefficient .8 were used in the previous calculation, we should have found

$$= \frac{.628 \times 22.08}{.8} - 12.4 = 17.33 - 12.4 = 4.93 \text{ in.},$$

not much more than half the previous value; this would only increase the whole height of the weir by $9.68 - 4.93 = 4.75$ inches.

As $D = \frac{2}{3} c_d l \sqrt{2g} \{(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}\}$ for a perfect

weir with a free overfall, it is clear that when D is

greater than $\frac{2}{3} c_d l \sqrt{2g} \{(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}\}$, the weir

is imperfect or submerged. For backwater curve

SECTION X.

In the following table of coefficients from Lesbros*

h is measured from that point below the weir

where its value is a minimum. On examining equa-

tion (52), it will be seen that the equation $D = c_d l$

$(d_1 + d_2) \sqrt{2g d_1}$ adopted by Lesbros is incorrect,

and can only be safely used within the limits of his

experiments.

Vide p. 84, deuxième édition, Hydraulique, par Arthur
N. n. Paris, 1858.

that is, the submerged weir must be built within 10.56 inches of the surface to raise the head 18 inches above the former level. If, however, the velocity of approach be taken into account, we shall find this velocity equals $\frac{812.5}{430} = 2$ feet per second very nearly; and the height, or value of h_v , due to this velocity, taken from TABLE II., is $\frac{8}{4} = .75$ inches nearly; therefore, from equation (55),

$$d_2 = \frac{696.4}{.628 \sqrt{2g \times 18.75}} - \frac{2}{3} \times \frac{(18.75)^{\frac{3}{2}} - (.75)^{\frac{3}{2}}}{\sqrt{18.75}}.$$

The value of $\frac{696.4}{.628 \sqrt{2g \times 18.75}} =$ (from TABLE I.)

$$\frac{696.4}{\frac{8}{3} \times \frac{12}{18.75} \times 393.75} = \frac{696.4}{378.8*} = 1.84 \text{ feet} = 22.08 \text{ inches};$$

$$\begin{aligned} \text{Also, } \frac{2}{3} \times \frac{(18.75)^{\frac{3}{2}} - (.75)^{\frac{3}{2}}}{\sqrt{18.75}} &= \frac{2}{3} \times 18.75 - \frac{2}{3} \times \frac{(.75)}{\sqrt{18.75}} \\ &= 12.5 - \frac{2}{3} \times \frac{.65}{4.33} = 12.5 - .1 = 12.4. \end{aligned}$$

Hence $d_2 = 22.08 - 12.4 = 9.68$ inches, or about 1 inch less than the value previously found from equation (54). The mean coefficient of discharge was here assumed to be .628. Experiments on submerged weirs show that the value of c_d varies from .5 to .8, but as this coefficient would reduce the value of d_2 , or the depth of the top of the weir below the surface, it is safer (where a given depth above a weir

* This is found from TABLE II. more readily.

must be obtained) to use the lesser and ordinary coefficients of perfect weirs, with a clear overfall, for finding the crest levels of submerged weirs, when it is necessary to construct them. If the coefficient .8 were used in the previous calculation, we should have found

$$d_2 = \frac{.628 \times 22.08}{.8} - 12.4 = 17.33 - 12.4 = 4.93 \text{ in.},$$

or not much more than half the previous value; but this would only increase the whole height of the weir by $9.68 - 4.93 = 4.75$ inches.

As $D = \frac{2}{3} c_d l \sqrt{2g} \{ (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}$ for a perfect weir with a free overfall, it is clear that when D is greater than $\frac{2}{3} c_d l \sqrt{2g} \{ (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}$, the weir is imperfect or submerged. For backwater curve see SECTION X.

In the following table of coefficients from Lesbros* d_2 is measured from that point below the weir where its value is a minimum. On examining equation (52), it will be seen that the equation $D = c_d l (d_1 + d_2) \sqrt{2g d_1}$ adopted by Lesbros is incorrect, and can only be safely used within the limits of his experiments.

* *Vide* p. 84, deuxième édition, Hydraulique, par Arthur Morin. Paris, 1858.

Values of $\frac{d_1}{d_1 + d_2}$	Values of the coefficient c_d in the formula $D = c_d l (d_1 + d_2) \times \sqrt{2g d_1}$	Values of $\frac{d_1}{d_1 + d_2}$	Values of the coefficient c_d in formula $D = c_d l (d_1 + d_2) \times \sqrt{2g d_1}$
·001	·227	·000	·519
·003	·205	·000	·517
·009	·163	·100	·510
·001	·130	·150	·512
·003	·106	·200	·507
·006	·086	·250	·503
·007	·077	·300	·497
·008	·065	·350	·492
·009	·060	·400	·487
·010	·056	·450	·480
·015	·040	·500	·474
·020	·037	·550	·466
·025	·035	·600	·458
·030	·034	·700	·444
·035	·033	·800	·427
·040	·031	·900	·409
·045	·026	1·000	·390
·050	·023	"	"

The experimental values are those shown between the horizontal lines, the others are the upper ones, and below the lower ones, were deduced from calculations by Leslie.

The true value of the discharge is expressed by the equation $D = c_d l \left\{ \frac{2}{3} d_1 + d_2 \right\} \times \sqrt{2g d_1}$, and the values of c_d in the above table are, therefore, too small, applied to the correct formula. When $d_1 = d_2$ the table gives $c_d = .474$. Now for we are in which the sheet passing over is "drowned," the general value of the coefficient is about .67; this would give the coefficient for the lower portion in the true formula, equal to .503, and a mean coefficient c_d in the correct formula (52) equal to .49 nearly. When $d_2 = 200 d_1$, the apparent limits of the experiments on the other side, then the mean value of $c_d = .496$ nearly in equation (52). These results would show that the coefficient due to the submerged depth d_2 , in the first and last experiments,

is equal to about .5 nearly, (but varies to .6 nearly in some of the middle experiments,) or thereabouts, and, therefore, equation (52) for submerged weirs, as the coefficient for the upper part d_1 is .67, would become

$$(52A.) \quad D = l \times \{.445 d_1 + .5 d_2\} \times \sqrt{2g d_1};$$

which for feet measures would become again

$$(52B.) \quad D = l \times \sqrt{d_1} \times \{3.56 d_1 + 4 d_2\},$$

for the discharge in cubic feet per second over a submerged weir, Fig. 22.

CONTRACTED RIVER CHANNELS.

When the banks of a river, whose bed has a uniform inclination, approach each other, and contract the width of the channel in any way, as in



Fig. 23, the water will rise in the channel above the contracted portion A, until the increased velocity of discharge compensates for the reduced cross section. If we put, as before, d_1 for the increase of depth immediately above the contracted width, and d_2 for the previous depth of the channel, we shall find the quantity of water passing through the lower depth, d_2 , equal to $c_d l d_2 \sqrt{2g d_1}$, in which l is the width of the contracted channel at A, and the quantity of water overflowing through d_1 equal to $\frac{2}{3} c_d l d_1 \sqrt{2g d_1}$;

and hence the whole discharge through A is

$$(56.) \quad D = c_a l \sqrt{2g d_1} \left(d_2 + \frac{2}{3} d_1 \right).$$

When our object is to find the width l of the contracted channel, so that the depth of water in the upper stretch shall be increased by a given depth, we shall find

$$(57.) \quad l = \frac{D}{c_a \sqrt{2g d_1} \left(d_2 + \frac{2}{3} d_1 \right)}$$

When the velocity of approach is considerable, when the height h_a due to it becomes a large portion of d_1 , its effect must not be neglected. In this case, as before, we find the discharge through the depth d_2 equal to $c_a l d_2 \sqrt{2g(d_1 + h_a)}$; and the discharge through the depth d_1 equal to $\frac{2}{3} c_a l \sqrt{2g} \{ (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}$; and hence the whole discharge is

$$(58.) \quad D = c_a l \sqrt{2g} \left\{ d_2 (d_1 + h_a)^{\frac{1}{2}} + \frac{2}{3} [(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}] \right\}$$

from which we shall find

$$(59.) \quad l = \frac{D}{c_a \sqrt{2g} \left\{ d_2 (d_1 + h_a)^{\frac{1}{2}} + \frac{2}{3} [(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}] \right\}}$$

If the projecting spur or jetty at A be itself submerged, these formulæ must be extended; the manner of doing so, however, presents no difficulty, as it is only necessary to find the discharges of the different sections according to the preceding formulæ, and then add them together; but the resulting formula so found is too complicated to be of much practical value.

HEADS ARISING FROM PIERS AND BACKWATER ABOVE BRIDGES.

Equations (56), (57), (58), and (59), are applicable to cases of contraction of river channels caused by the construction of bridge-piers and abutments, when the width Z is put for the sum of the openings between them. The value of the coefficient c_a will depend on the peculiar circumstances of each case; we have seen that it rises from $\cdot 5$ to $\cdot 7$ in some cases of submerged weirs, and for cases of contracted channels it rises sometimes as high as $\cdot 8$, particularly when they are analogous to those for the discharge through mouth-pieces and short tubes. When the heads of the piers are square to the channel, the coefficient may be taken at about $\cdot 6$; when the angles of the cut-waters or sterlings are obtuse, it may be taken at about $\cdot 7$; and when curved and acute, at $\cdot 8$. With this coefficient, a head of $2\frac{3}{4}$ inches will give a velocity of very nearly 36 inches, or 3 feet per second; but as a certain amount of loss takes place from the velocity of the tail-water being in general less than that through the arch, also from obstructions in the passage, and from square-headed and very short piers, the coefficient may be so small in some cases as $\cdot 5$, which would require a head of $6\frac{3}{4}$ inches to obtain the same velocity. This head is to the former as 54 to 21. The selection of the proper coefficient suited to any particular case is, therefore, a matter of the first importance in determining the effect of obstructions in river channels: we shall have to recur to this subject again, but it is necessary to observe here, that the form of the approaches, the

and hence the whole discharge through A is

$$(56.) \quad D = c_a l \sqrt{2g d_1 \left(d_2 + \frac{2}{3} d_1 \right)}.$$

When our object is to find the width l of the contracted channel, so that the depth of water in the upper stretch shall be increased by a given depth d_2 , we shall find

$$(57.) \quad l = \frac{D}{c_a \sqrt{2g d_1 \left(d_2 + \frac{2}{3} d_1 \right)}}$$

When the velocity of approach is considerable, or when the height h_a due to it becomes a large portion of d_1 , its effect must not be neglected. In this case, as before, we find the discharge through the depth d_2 equal to $c_a l d_2 \sqrt{2g (d_1 + h_a)}$; and the discharge through the depth d_1 equal to $\frac{2}{3} c_a l \sqrt{2g \{ (d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \}}$; and hence the whole discharge is

$$(58.) \quad D = c_a l \sqrt{2g} \left\{ d_2 (d_1 + h_a)^{\frac{1}{2}} + \frac{2}{3} [(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}] \right\};$$

from which we shall find

$$(59.) \quad l = \frac{D}{c_a \sqrt{2g} \left\{ d_2 (d_1 + h_a)^{\frac{1}{2}} + \frac{2}{3} [(d_1 + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}] \right\}}$$

If the projecting spur or jetty at A be itself submerged, these formulæ must be extended; the manner of doing so, however, presents no difficulty, as it is only necessary to find the discharges of the different sections according to the preceding formulæ, and then add them together; but the resulting formula so found is too complicated to be of much practical value.

HEADS ARISING FROM PIERS AND BACKWATER ABOVE BRIDGES.

Equations (56), (57), (58), and (59), are applicable to cases of contraction of river channels caused by the construction of bridge-piers and abutments, when the width l is put for the sum of the openings between them. The value of the coefficient c_d will depend on the peculiar circumstances of each case; we have seen that it rises from $\cdot 5$ to $\cdot 7$ in some cases of submerged weirs, and for cases of contracted channels it rises sometimes as high as $\cdot 8$, particularly when they are analogous to those for the discharge through mouth-pieces and short tubes. When the heads of the piers are square to the channel, the coefficient may be taken at about $\cdot 6$; when the angles of the cut-waters or sterlings are obtuse, it may be taken at about $\cdot 7$; and when curved and acute, at $\cdot 8$. With this coefficient, a head of $2\frac{3}{4}$ inches will give a velocity of very nearly 36 inches, or 3 feet per second; but as a certain amount of loss takes place from the velocity of the tail-water being in general less than that through the arch, also from obstructions in the passage, and from square-headed and very short piers, the coefficient may be so small in some cases as $\cdot 5$, which would require a head of $6\frac{3}{4}$ inches to obtain the same velocity. This head is to the former as 5.4 to 2.1. The selection of the proper coefficient suited to any particular case is, therefore, a matter of the first importance in determining the effect of obstructions in river channels: we shall have to recur to this subject again, but it is necessary to observe here, that the form of the approaches, the

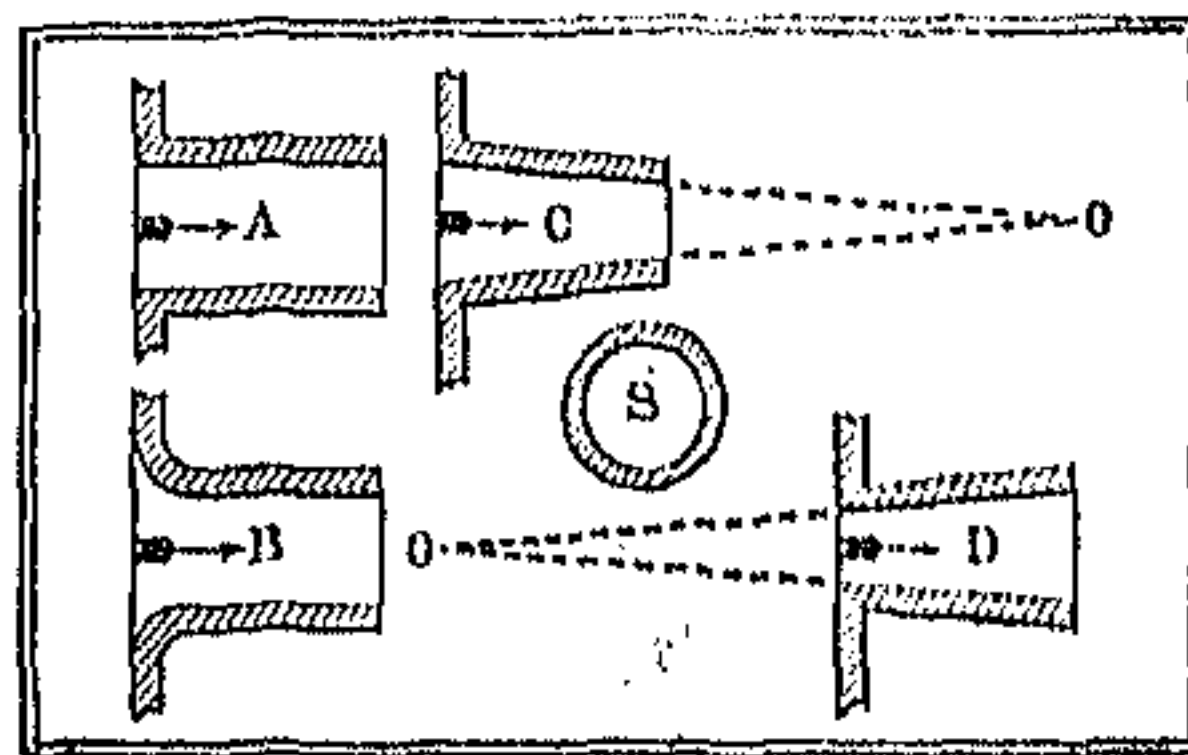
length of the piers compared with the distance between them, or span, and the length and form of the obstruction compared with the width of the channel, must be duly considered before the coefficient suited to the particular case can be fixed upon. Indeed, the coefficients will always approximate towards those, given in the next section, for mouth-pieces, shoots, and short tubes similarly circumstanced. For some further remarks on contracted channels, see SECTION X.

SECTION VI.

SHORT TUBES, MOUTH-PIECES, AND APPROACHES.—ALTERATION IN THE COEFFICIENTS FROM FRICTION BY INCREASING THE LENGTH.—COEFFICIENTS OF DISCHARGE FOR SIMPLE AND COMPOUND SHORT TUBES.—SHOOTS.

The only orifices we have heretofore referred to were those in thin plates or planks, with a few incidental exceptions. It has been shown, page 48, Fig. 4, that a rounding off, next the water, of the mouth-piece increases the coefficient; and when the curving

Fig. 24.



assumes the form of the *vena-contracta*, the coefficient increases to .986, or nearly unity. The discharge

from a short cylindrical tube A, Fig. 24, whose length is from one and a half to three times the diameter, is found to be very nearly an arithmetical mean between the theoretical discharge and the discharge through a circular orifice in a thin plate of the same diameter as the tube, or .814 nearly. If, however, the inner arris be rounded, or chamfered off in any way, the coefficient will increase until, in the tube B, Fig. 24, with a properly-rounded junction, it becomes unity very nearly. In the conical short tubes c and d the coefficients are found to vary according to some function of the converging or diverging angles ϕ , ϕ , and according as we take the lesser or greater diameter to calculate from. When the length of the tube exceeds twice the diameter, the friction of the water against the sides may be taken into account.

The following table, calculated by us, for a coefficient of friction .00699, due to a discharging velocity of about eighteen inches per second, see SECTION VIII, shows the resistance arising from friction in pipes of different lengths in relation to the diameter, and will be found of considerable practical value. It will be perceived that the calculations are made for three different orifices of entry. First, when the arrises are rounded, as in B, Fig. 24, with a coefficient of .986; secondly, when the arrises are square, as in A, with a coefficient of .815; and, thirdly, when the pipe projects into the vessel, when the coefficient of entry becomes reduced to .715. The velocity is

$$v = c_d \sqrt{2gh},$$

h being measured to the lower end of the tube.

COEFFICIENTS FOR SHORT AND LONG TUBES.

Number of diameters in the length of the pipe.	Corresponding coefficients of discharge, showing the effects of friction.			Number of diameters in the length of the pipe.	Corresponding coefficients of discharge, showing the effects of friction.		
2 diameters	.986	.814	.715	650 diameters	.228	.225	.223
5 "	.936	.779	.690	700 "	.220	.217	.215
10 "	.884	.747	.668	750 "	.213	.211	.209
15 "	.840	.720	.649	800 "	.206	.205	.203
20 "	.801	.695	.630	850 "	.201	.199	.197
25 "	.767	.673	.615	900 "	.195	.193	.192
30 "	.737	.653	.598	950 "	.190	.189	.187
35 "	.711	.634	.584	1000 "	.186	.184	.183
40 "	.688	.617	.570	1100 "	.177	.176	.175
45 "	.665	.601	.558	1200 "	.170	.169	.168
50 "	.646	.586	.546	1400 "	.158	.157	.156
100 "	.513	.480	.458	1600 "	.148	.147	.146
150 "	.439	.418	.403	1800 "	.139	.139	.138
200 "	.389	.375	.364	2000 "	.132	.132	.131
250 "	.354	.345	.334	2200 "	.126	.126	.125
300 "	.327	.318	.311	2400 "	.120	.120	.120
350 "	.304	.297	.292	2600 "	.116	.116	.116
400 "	.287	.280	.276	2800 "	.112	.112	.112
450 "	.271	.266	.262	3000 "	.108	.108	.108
500 "	.258	.254	.250	3200 "	.105	.105	.104
550 "	.247	.243	.240	3400 "	.102	.102	.101
600 "	.237	.234	.231	3600 "	.099	.099	.099

We see from this table, that the effect of adding to the length of the pipe is greatest next the orifice of entry. The effect of a few diameters added to the length in long pipes is, practically, immaterial; but in short pipes it is considerable.

As for orifices in thin plates, so also for short tubes, the coefficients are found to vary according to the depth of the centre below the surface of the water, and to increase as the depths and diameter of the tube decrease. Poleni first remarked that the discharge through a short tube was greater than that

through a simple orifice, of the same diameter, in the proportion of 133 to 100, or as $\cdot 617$ to $\cdot 821$.

CYLINDRICAL SHORT TUBES, A, FIG. 24.

The experiments of Bossût, as reduced by Prony, give the following coefficients, at the corresponding depths, for a cylindrical tube A, Fig. 24, 1 inch in diameter and 2 inches long. The depths are given in

COEFFICIENTS FOR SHORT TUBES, FROM BOSSÛT.

Heads in feet.	Coefficients.	Heads in feet.	Coefficients.	Heads in feet.	Coefficients.
1	$\cdot 818$	6	$\cdot 806$	11	$\cdot 805$
2	$\cdot 807$	7	$\cdot 806$	12	$\cdot 804$
3	$\cdot 807$	8	$\cdot 805$	13	$\cdot 804$
4	$\cdot 807$	9	$\cdot 805$	14	$\cdot 804$
5	$\cdot 806$	10	$\cdot 805$	15	$\cdot 803$

Paris feet in the original, but the coefficients remain the same, practically, for depths in English feet.

Venturi's experiments give a coefficient $\cdot 823$ for a short tube A, $1\frac{1}{2}$ inch in diameter and $4\frac{1}{2}$ inches long, at a depth of 2 feet $8\frac{1}{2}$ inches, the coefficient through an orifice in a thin plate of the same diameter and at the same depth being $\cdot 622$. We have calculated these coefficients from the original experiments. The measures were in Paris feet and inches, from which the calculations were directly made; and as the difference in the coefficient for small changes of depth or dimensions is immaterial or vanishes, as may be seen by the foregoing small table, and as 1 Paris inch or foot is equal to 1.0658 English inches or feet, the former measures exceed the latter by only

about $\frac{1}{15}$ th. We may therefore assume that *the coefficient* for any orifice, at any depth, is the same, whether the dimensions be in Paris or English feet or inches. This remark will be found generally useful in the consideration of the older continental experiments, and will prevent unnecessary reductions from one standard to another where the coefficients only have to be considered.

The mean value derived from the experiments of Michelotti, at depths from 3 to 20 feet, and with short tubes A from $\frac{1}{2}$ inch to 3 inches in width, is $c_d = .814$. Buff's experiments* give the following results for a tube $\frac{3}{16}$ of an inch wide and $\frac{5}{16}$ of an inch long, nearly.

BUFF'S COEFFICIENTS FOR SMALL SHORT TUBES.

Head in inches.	Coefficient.	Head in inches.	Coefficient.	Head in inches.	Coefficient.
$1\frac{1}{2}$.855	6	.840	23	.829
$2\frac{1}{2}$.861	14	.840	32	.826

The increase for smaller tubes and for lesser depths appears by comparing these results with the foregoing, and from the results in themselves, generally. Weisbach's experiments give a mean value for $c_d = .815$, and for depths of from 9 to 24 inches the coefficients .843, .832, .821, .810 respectively, for tubes $\frac{4}{10}$, $\frac{8}{10}$, $\frac{12}{10}$, and $\frac{16}{10}$ of an inch wide, the length of each tube being three times the diameter. D'Aubuisson and Castel's

* Annalen der Physik und Chemie von Poggendorff, 1830. vol. 46, p. 243.

experiments with a tube .61 inch diameter and 1.57 inch long, give .829 for the coefficient at a depth of 10 feet. When a pipe projects into a cistern and has a sharp edge, the coefficient falls so low as .715.

We have calculated the coefficients in the two following short tables, from Rennie's experiments with glass orifices and tubes, Table 7, p. 435, Philosophical Transactions for 1831. The form of the orifices, or length of the shorter tubes is not stated, but it is probable from the result, that the arrises of the ends were in some way rounded off; it is stated they were "enlarged." Indeed, the discharges from the short tube or orifice of $\frac{1}{4}$ inch diameter exceed the theoretical ones in the proportion of 1.261 to 1, and 1.346 to 1. These results could not have been derived from a simple cylindrical tube, but might have arisen from the arrises being more or less rounded at both ends, and the orifice partaking of the nature of a compound tube, which may be constructed, as we shall hereafter show, so as to increase the theoretical discharge from 1 up to 1.553. The resulting coefficients for the $\frac{1}{2}$

COEFFICIENTS FOR SHORT TUBES, THE ENDS ENLARGED.

Head in feet.	$\frac{1}{4}$ inch diameter.	$\frac{1}{2}$ inch diameter.	$\frac{3}{4}$ inch diameter.	1 inch diameter.
1	1.231	.831	.700	.012
2	1.201	.830	.820	.020
3	1.346	.838	.821	.860
4	1.201	.831	.820	.091

and $\frac{3}{4}$ inch tubes, approach very closely to those obtained by other experimenters, but those for the inch tube are too high, unless the arris at the ends was also rounded. The coefficients derived from the

experiments with a cylindrical glass tube 1 foot long, as here given, are very variable; like the others they

COEFFICIENTS DERIVED FROM EXPERIMENTS WITH A GLASS TUBE ONE FOOT LONG.

Heads in feet.	$\frac{1}{4}$ inch diameter.	$\frac{1}{2}$ inch diameter.	$\frac{3}{4}$ inch diameter.	1 inch diameter.
1	.802	.703	.601	.700
2	.914	.734	.718	.740
3	.931	.723	.700	.777
4	.914	.725	.677	.815

are, however, valuable, as exhibiting the uncertainty attending "experiments of this nature," and the necessity for minutely observing and recording every circumstance which tends to alter and modify them. Indeed, for small tubes, a very slight difference in the measurement of the diameter must alter the result a good deal, particularly when it is recollected that measurements are seldom taken more closely than the sixteenth of an inch, unless in special cases. As the author, however, states, p. 433 of the work referred to, that the "diameters of the tubes at their extremities were carefully enlarged to prevent wire edges from diminishing the sections;" this circumstance alone must have modified the discharges, and would account for most of the differences.

The coefficient for rectangular short tubes differs in no way materially from those given for cylindrical ones, and may be taken on an average at .814 or .815.

SHORT TUBES WITH A ROUNDED MOUTH-PIECE, B, FIG. 24.

When the junction of a short tube with a vessel is the form of the contracted vein, Figs. 3 and 4, 48, the mean value of the coefficient $c_d = .956$,

and the actual discharge is found to be from 93 to 99 per cent. of the theoretical discharge. Weisbach, for a tube $1\frac{1}{2}$ inch long and $\frac{1}{8}$ inch diameter, rounded at the junction, found at 1 foot deep $c_d = .958$, at 5 feet deep $c_d = .969$, and at 10 feet deep $c_d = .975$. These experiments show an increase in the coefficients, in this particular case, for an increase of depth. Any other form of junction than that of the contracted vein, will reduce the discharge, and the coefficients will vary from .715 to .814, and to .986, according to the change in the junction from the cylindrical, projecting into the vessel, to the square and properly curved forms. The coefficients derived from Venturi's experiments will be given hereafter.

SHORT CONICAL CONVERGENT TUBES, *c*, FIG. 24.

The experiments of D'Aubuisson and Castel lead to the following coefficients of discharge and velocity* from a conically convergent tube *c* at a depth of 10.

COEFFICIENTS FOR CONICAL CONVERGENT TUBES.

Converging angle α .	Coefficient of discharge.	Coefficient of velocity.	Converging angle α .	Coefficient of discharge.	Coefficient of velocity.
1°	.858	.868	14°	.948	.964
2°	.873	.873	16°	.957	.970
3°	.898	.898	18°	.961	.971
4°	.910	.900	20°	.962	.971
5°	.920	.910	22°	.917	.973
6°	.925	.923	24°	.904	.975
8°	.931	.933	36°	.895	.973
10°	.937	.950	40°	.800	.980
12°	.942	.955	60°	.844	.985

* *Traité d'Hydraulique*, Paris, p. 60.

feet. We have interpolated the original angles and coefficients so as to render the table more convenient to refer to, for practical purposes, than the original. The diameter of the tube at the smaller or discharging orifice in the experiments was .61 inches, and the length of the axis 1.57 inch; that is, the length was 2.6 times the smaller diameter of the tube. The coefficient became .829 for the cylindrical tube, *i. e.* when the angle at ϕ was nothing. The angle of convergence ϕ determines, from the proportions, the length of the inner and longer diameter of the tube. The coefficients of discharge increase up to .943 for an angle of $13\frac{1}{2}$ or 14 degrees, after which they again decrease; but the coefficients of velocity increase as the angle of convergence, ϕ , increases from .829, when the angle is zero up to .985 for an angle of 50 degrees.

When D is the discharge and A the area of the section, we have, as before shown, $D = c_d A \sqrt{2gh}$; but as, in conically convergent or divergent tubes, the inner and outer areas (or, as they may be called, the receiving and discharging sections) vary, it is clear that, the discharge being the same, and also the theoretical velocity $\sqrt{2gh}$, the coefficient c_d must vary inversely with the sectional area A , and that $c_d \times A$ must be constant. For the coefficients tabulated, the sectional area to be used is that at the smaller or outside end of a convergent tube ϕ , Fig. 24.

For a short tube ϕ , whose length is .92 inch, lesser diameter 1.21 inch, and greater diameter 1.5 inch, we have found, from Venturi's experiments, that $= .607$ if the larger diameter be used in the calcu-

lation, and $c_d = .934$ when the lesser diameter is made use of, the discharge taking place under a pressure of 2 feet $8\frac{1}{2}$ inches.

The earlier experiments of Poleni, when reduced, furnish us with the following coefficients: A tube 7.67 inches long, 2.167 inches diameter at each end, gave $c_d = .854$; the like tube with the inner or receiving orifice increased to $2\frac{3}{4}$ inches, $c_d = .903$; increased to 3.5 inches, $c_d = .898$; increased to 5 inches, $c_d = .888$; and increased to 9.83 inches, $c_d = .864$. The depth or head was 21.33 inches, the discharging orifice 2.167 inches diameter, and the length 7.67 inches, in each case.

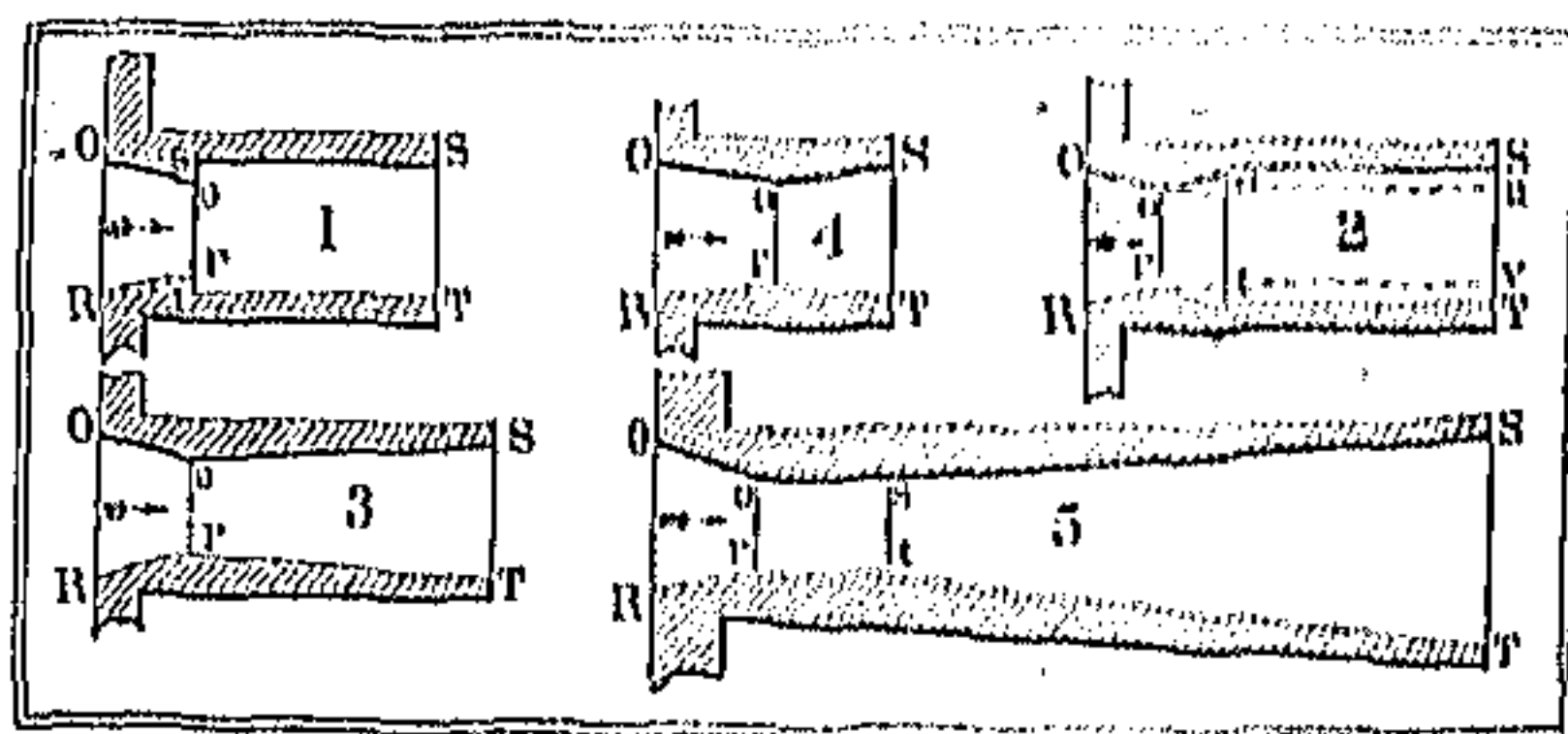
In the conically divergent tube *n*, Fig. 24, the coefficient of *discharge* is larger than for the same tube *c*, convergent, when the water fills both tubes, and the smaller sections, or those at the same distances from the centres *o o*, are made use of in the calculations. A tube whose angle of convergence, *o*, is 5° nearly, with a head of from 1 to 10 feet, whose axial length is $3\frac{1}{2}$ inches, smaller diameter 1 inch, and larger diameter 1.3 inch, gives, when placed as at *c*, .921 for the coefficient; but when placed as at *n*, the coefficient increases to .948. In the first case the smaller area, used in both calculations, being the receiving, and in the other the discharging, orifice. The coefficient of *velocity* is, however, larger for the tube *c* than for the tube *n*, and the discharging jet of water has a greater amplitude in falling. The effects of conically diverging tubes will, however, be better perceived from the experiments on compound short tubes.

EFFECTS OF COMPOUND ADJUTAGES AND ADMISSION OF AIR INTO SHORT TUBES.

If the tube A, Fig. 24, be pierced all round with small holes at the distance of about half its diameter from the reservoir, the discharge will be immediately reduced in the proportion of $\cdot 814$ to $\cdot 617$. Venturi found the reduction for a tube $1\frac{1}{2}$ inch diameter and $4\frac{1}{2}$ inches long, at a depth of 2 feet $10\frac{1}{2}$ inches, as 41 to 31, or as $\cdot 823$ to $\cdot 622$. As long as one hole remained open, the discharge continued at the same reduced rate; but when the last hole was stopped, the discharge again increased to the original quantity. If a small hole be pierced in a tube 4 diameters long, at the distance of $1\frac{1}{2}$ or 2 diameters at farthest from the junction, the discharge will remain unaffected. This shows that the contraction in the cylindrical tube extends only a short distance from the junction, probably $1\frac{1}{2}$ or $1\frac{1}{4}$ diameter, including the whole curvature of the contraction.

The contraction at the entrance into a tube from a reservoir accounts for the coefficients for a short tube A, Fig. 24, and the short tubes, diagrams 1 and 2, Fig. 25, being each the same decimal nearly, when

Fig. 25.



or :: $1 : \cdot 8$, or when or is not less than $OR \times \cdot 79$,

and is at the distance of nearly $\frac{or}{2}$ from or . The form of the junction $o o r r$ remaining as we have described it, the following coefficients will enable us to judge of the discharging powers of differently formed short mouth-pieces. They have been deduced and calculated by us, principally, from Venturi's experiments.*

These coefficients show very clearly that any calculations from the mere head of water and size of the orifice, without taking into consideration the form of the discharging tube and its connection with the reservoir, are very uncertain; and that the discharge can only be correctly obtained when all the circumstances of the case, including the form of the discharging orifice and its approaches, have been duly considered.

When a tube similar to diagram 5, Fig. 25, has the junction $o o r r$ rounded, as in Fig. 4, page 48, the outer extremity $s t s t$, such that $s t = or$, $ss = 9 st$, and the diameter $st = 1.8$ times the diameter st , with a *short* central cylindrical piece $o r s t$ between, the coefficient of discharge corresponding to the diameter $or = rs$ will increase to 1.493 or 1.555; that is, the discharge is $\frac{1.493}{.622} = 2.4$, or $\frac{1.555}{.622} = 2.5$ times as much as through an orifice (whose diameter is or) in a thin plate, and $\frac{1.555}{.822} = 1.9$ times as much as through a

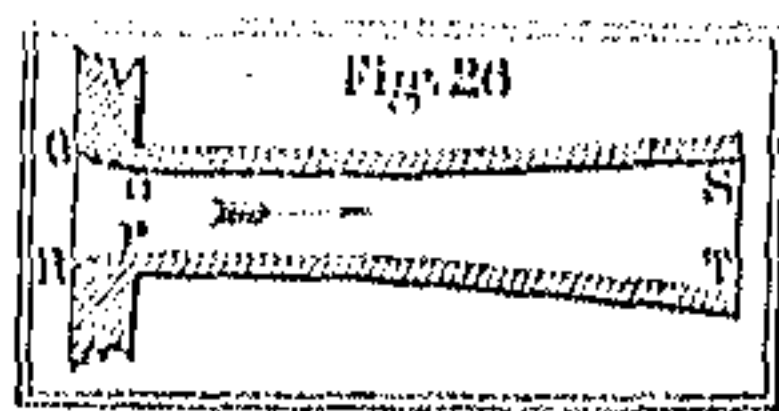
* See Nicholson's translation of Venturi's *Experimental Inquiries*, published in the *Tracts on Hydraulics*, London, 1836. The coefficients in the table, next page, have been all calculated for the first time by us.

TABLE OF COEFFICIENTS FOR SHORT TUBES AND MOUTH-PIECES.

Description of orifice, mouth-piece, or short tube.	Coefficients for the diameter $o r$.	Coefficients for the diameter $o r$.
1. An orifice $1\frac{1}{2}$ inch diameter in a thin plate...	.022	.074
2. A cylindrical tube $1\frac{1}{2}$ inch diameter and $4\frac{1}{2}$ inches long, A, Fig. 24.....	.823	.823
3. A short tube with a sharp end projecting into the cistern.....	.715	.715
4. A cylindrical tube, B, Fig. 24, having the junction rounded, as in Fig. 4, page 48.....	.611	.956
5. A short conical convergent mouth-piece, C, Fig. 24, of the proportions of $o o r r$, Fig. 25.....	.607	.934
6. The like tube divergent, with the smaller diameter at the junction with the reservoir; length $3\frac{1}{2}$ inches, lesser diameter 1 inch, and greater diameter $1\frac{3}{8}$ inch.....	.561	.948
7. The tube, $o o u v r r$, diagram 2, Fig. 25, when $o r = 1\frac{1}{2}$ inch, $o r = 1\cdot21$ inch, $u v = 1\cdot21$ inch, and $o u = r v = 2$ inches, the cylindrical portion being shown by dotted lines.....	.000	.023
8. The same tube when $o u = 11$ inches.....	.507	.873
„ The same tube when $o u = 23$ inches.....	.531	.817
9. The tube, $o o s s t r r$, diagram 2, Fig. 25, in which $o r = s t = s r = 1\frac{1}{2}$ inch, from o to s $1\frac{1}{2}$ inch, and $s s = 3$ inches, gives the same coefficient as the cylindrical tube, result No. 2 (see No. 10), viz.....	.823	1.206
10. The tube, diagram 1, Fig. 25, $o r = 1\frac{1}{2}$ inch...	.804	1.237
11. The same tube, having the spaces $o s o$ and $r t r$ between the mouth-piece $o o r r$ and the cylindrical tube $o s t r$ open to the influx of the water.....	.785	1.200
12. The double conical tube, $o o s t r r$, diagram 3, Fig. 25, when $o r = s t = 1\frac{1}{2}$ inch, $o r = 1\cdot21$ inch, $o o = \cdot92$ inches, and $o s = 4\cdot1$ inches...	.928	1.428
13. The like tube when, as in diagram 4, Fig. 25, $o o r r = o s t r$, and $o o s = 1\cdot84$ inch.....	.823	1.200
14. The like tube when, $s t = 1\cdot40$ inch, and $o s = 2\cdot17$ inches.....	.823	1.200
15. The like tube when $s t = 3$ inches, and $o s = 9\frac{3}{4}$ inches.....	.911	1.400
16. The like tube when $o s = 6\frac{1}{2}$ inches, and $s t$ enlarged to $1\cdot92$ inch.....	1.020	1.500
17. The like tube when $s t = 2\frac{1}{2}$ inches, and $o s = 12\frac{1}{4}$ inches.....	1.215	1.855
18. A tube, diagram 5, Fig. 25, when $o s = r t = 3$ inches, $o r = s t = 1\cdot21$ inch, and the tube $o s t r$ the same as described in No. 12, viz. $s t = 1\frac{1}{2}$ inch, and $s s = 4\cdot1$ inches.....	.805	1.377
19. The tube, diagram 2, Fig. 25, when $s t$ is enlarged to $1\cdot97$ inch, and $s s$ to 7 inches, the other dimensions remaining as in No. 9.....	.945	1.454
20. When the junction of $o s r t$ with $s s t r$, diagram 2, Fig. 25, is improved, the other parts remaining as described in No. 9.....	.850	1.309
„ Another experiment gives.....	.847	1.303

short cylindrical tube A, Fig. 24, whose diameter is also $o r$. Venturi was of opinion that this discharge continued even when the central cylindrical portion $o r s t$ was of considerable length; but this was a mistake, as the maximum discharge is obtained when it is reduced so that $o o r r$ and $s s t t$ shall join, as in diagram 3, Fig. 25. We see from No. 16 of the fore-

going coefficients that $\frac{1.569}{.622} = 2.52$ and $\frac{1.569}{.822} = 1.91$ are, perhaps, nearer to the maximum results obtainable by comparing the discharge from a compound tube $o o s t r r$, diagram 3, Fig. 25, with those through an orifice in a thin plate, and through a short cylindrical tube. When the form of the tube becomes curvilinear throughout, as in Fig. 26, $s t = 1.8 o r$ and $o s = 9 o r$, the coefficient suited to the diameter $o r$ will be 1.57



nearly, and the discharge will be $\frac{1.57}{.622} = 2.52$ times as much as through an orifice $o r$ in a thin plate.

The whole of the preceding coefficients have been determined from circumstances in which the coefficient for an orifice in a thin plate was .622, and for a short cylindrical tube .822 or .823. When the circumstances of head and approaches in the reservoir are such as to increase or decrease those primary coefficients, the other coefficients for compound advantages will have to be increased or decreased proportionately.

After examining the foregoing results, it appears sufficiently clear that the utmost effect produced by

the formation of the compound mouth-piece $o o s t r r$, with the exception of No. 17, is simply a restoration of the loss effected by contraction in passing through the orifice $o r$ in a thin plate, *and that the coefficient 2.5 applied to the contracted section at $o r$ is simply equal to the theoretical discharge, or the coefficient unity, applied to the primary orifice $o r$* ; for, as orifice $o r$: orificed $o r$:: 1 : .64, very nearly, when $o o r r$ takes the form of the *vena-contracta*, and the coefficient of discharge for an orifice $o r$ in a thin plate is .622, we get the theoretical discharge through the orifice $o r$, to the actual discharge through an orifice $o r$, so is 1 to $.622 \times .64$, so is 1 : .39808 :: 1 : .4 very nearly; and as $.4 \times 2.5 = 1$, it is clear that the form of the tube $o o s t r r$, when it produces the foregoing effect, simply restores the loss caused by contraction in the *vena-contracta*. Venturi's sixteenth experiment, from which we have derived the coefficients in No. 17, gives the coefficient 1.215 for the orifice $o r$. This indicates that a greater discharge than the theoretical, through the receiving orifice, may be obtained. It is, however, observable that Venturi, in his seventh proposition, does not rely on this result, and Eytelwein's experiments do not give a larger coefficient than 2.5 applied to the contracted orifice $o r$, which, we have above shown, is equal to the theoretical discharge through $o r$.

SHOOTS.

When the sides and under edge of an orifice or notch increase in thickness, so as to be converted into a shoot or small channel, open at the top, the

coefficients reduce very considerably, and to some extent beyond what the increased resistance from friction, particularly for small depths, indicates. Poncelet and Lesbros* found for orifices $8'' \times 8''$, that the addition of a horizontal shoot 21 inches long reduced the coefficient from .604 to .601, with a head of about 4 feet; but for a head of $4\frac{1}{2}$ inches the coefficient fell from .572 to .483. For notches $8''$ wide, with the addition of a horizontal shoot $9' 10''$ long, the coefficient fell from .582 to .479 for a head of $8''$; and from .622 to .340 for a head of $1''$. Castel also found for a notch $8''$ wide, with the addition of a shoot $8''$ long, inclined $4^\circ 18'$, the mean coefficient for heads from $2''$ to $4\frac{1}{2}''$, to be .527 nearly. The effects arising from friction alone will be perceived from the short table at the beginning of this section, p. 152.

The orifice of entry into a shoot and its position with reference to the sides and bottom modify the discharge, the head remaining constant. Lesbros† has given the coefficients suited to different positions of shoots both within and without a cistern, and from notches and submerged orifices; but, however valuable these are in some respects, they are of little practical use to the engineer. The general principles which are involved in the modification of these coefficients have, however, been already pointed out by us when discussing the effects of the position of the orifice, and the addition of short tubes, on the discharge. Equation (74B.), p. 189, is here applicable.

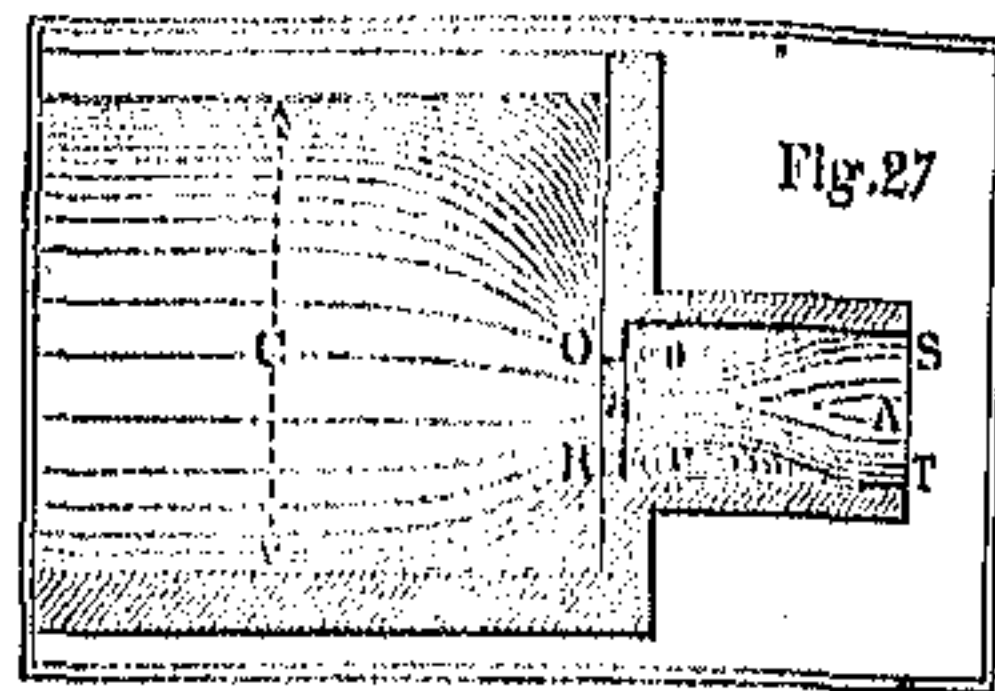
* *Traité d'Hydraulique*, pp. 46 et 94.

† *Vide Morin's Hydraulique*, deuxième édition, pp. 39 et 40.

SECTION VII.

LATERAL CONTACT OF THE WATER AND TUBE.—ATMOSPHERIC PRESSURE.—HEAD MEASURED TO THE DISCHARGING ORIFICE.—COEFFICIENT OF RESISTANCE.—FORMULA FOR THE DISCHARGE FROM A SHORT TUBE.—DIAPHRAGMS.—OBLIQUE JUNCTIONS.—FORMULA FOR THE TIME OF THE SURFACE SINKING A GIVEN DEPTH.—LOCK CHAMBERS.

The contracted vein $o r$ is about $\cdot 8$ times the diameter $o r$; but it is found, notwithstanding, that water, in passing through a short tube of not less



than $1\frac{1}{2}$ diameter in length, fills the whole of the discharging orifice $s r$. This is partly effected by the outflowing column of water carrying forward and exhausting the air between it and the tube, and by the external air then pressing on the column so as to enlarge its diameter and fill the whole tube. When once the water approaches closely to the tube, or is caused to approach, it is attracted and adheres with some force to it. The water between the tube and the *vena-contracta* is, however, rather in a state of eddy than of forward motion, as appears from the experiments, with the tube, diagram 2, Fig. 25, giving the same discharge as the simple cylindrical tube. If the entrance be contracted by a diaphragm, as at $o r$, Fig. 27, the water will also generally fill the tube, if it be only sufficiently long. Short cylindrical tubes do not fill when the discharge takes place in an exhausted receiver; but even diverging tubes,

D, Fig. 24, will be filled, under atmospheric pressure, when the angle of divergence, ϕ , does not exceed 7 or 8 degrees, and the length be not very great nor very short.

When a tube is fitted to the bottom or side of a vessel, it is found that the discharge is that due to the head measured from the surface of the water to the lower or discharging extremity of the tube. It must, however, be sufficiently long, and not too long, to get filled throughout. Guiglielmini first referred this effect to atmospheric pressure, but the first simple explanation is that given by Dr. Mathew Young, in the Transactions of the Royal Irish Academy, vol. vii., p. 56. Venturi, also, in his fourth proposition, gives a demonstration.

The values of the coefficients for short cylindrical tubes, which we have given p. 162, have been derived from experiment. Coefficients which agree pretty closely with them, and which are derived from the coefficients for the discharge through an orifice in a thin plate, may, however, be calculated as follows: Let σ be the area of the approaching section, Fig. 27, Λ the area of the discharging short tube, and α the area of the orifice $o r$ which admits the water from the vessel into the tube: also put, as before, h for the head measured from the surface of the water to the centre of the tube, and diaphragm $o r$; v for the actual velocity of discharge at $s t$; v_a for the velocity of approach in the section σ towards the diaphragm $o r$; and c_c for the coefficient of contraction in passing from $o r$ to $o r$; then we have $\sigma \times v_a = \Lambda \times v$, the contracted section $o r = c_c \times \alpha$, and consequently the velocity at the contracted section is equal to

$\frac{\Lambda v}{a c_o} = \frac{O v_n}{a c_o}$. Now a theoretical head equal to

$$\frac{v^2 - v_n^2}{2g} = \frac{v^2 \left(1 - \frac{\Lambda^2}{O^2}\right)}{2g}$$

is necessary to change the velocity v_n into v by the action of gravity; but as the water at the contracted section $o r$, moving with a velocity $\frac{\Lambda v}{a c_o}$, strikes against the water between it and $r s$, moving, from the nature of the case, with a slower velocity,* a certain loss of effect takes place from the impact. If this be, supposed, sudden, then writers on mechanics have shown *that a loss of head, equal to that due to the difference of the velocities, $\frac{\Lambda v}{a c_o} - v$, before and after the impact must take place.* This loss of head is therefore equal to

$$\frac{\left(\frac{\Lambda}{a c_o} - 1\right)^2 v^2}{2g},$$

whence we must have the whole head,

$$(60.) \quad h = \frac{\left(1 - \frac{\Lambda^2}{O^2}\right) v^2 + \left(\frac{\Lambda}{a c_o} - 1\right)^2 v^2}{2g},$$

from which we find for the velocity from a short tube,

$$(61.) \quad v = \sqrt{2gh} \left\{ \frac{1}{1 - \frac{\Lambda^2}{O^2} + \left(\frac{\Lambda}{a c_o} - 1\right)^2} \right\}^{\frac{1}{2}}.$$

Now, as $\sqrt{2gh}$ would be the velocity of discharge were there no resistances, or loss sustained, it is

evident that $\left\{ \frac{1}{1 - \frac{\Lambda^2}{O^2} + \left(\frac{\Lambda}{a c_o} - 1\right)^2} \right\}^{\frac{1}{2}}$ becomes as it

* *Vide* Sir Robert Kane's translation of Rühlman's book on Horizontal Water Wheels, p. 49.

were a coefficient of velocity. When the diameter of the diaphragm or becomes equal to the diameter st of the tube, $\Lambda = a$, and as the coefficient of velocity becomes equal to the coefficient of discharge when there is no contraction, we get in such case this coefficient, which we shall also call $c o f$, expressed by the formula

$$(62.) \quad c o f = \left\{ \frac{1}{1 - \frac{\Lambda^2}{a^2} + \left(\frac{1}{c_o} - 1 \right)^2} \right\}^{\frac{1}{2}}$$

and when the approaching section a is very large compared with the area Λ ,

$$(63.) \quad c o f = \left\{ \frac{1}{1 + \left(\frac{1}{c_o} - 1 \right)^2} \right\}^{\frac{1}{2}}.$$

If $c_o = .64$, we shall find from the last equation $c o f = .872$; if $c_o = .601$, $c o f = .833$; if $c_o = .617$, $c o f = .847$; and if $c_o = .621$, $c o f = .856$. These results are in excess of those derived from experiment with cylindrical short tubes, perfectly square at the ends and of uniform bore. As some loss, however, takes place in the eddy between or and the tube, and from the friction at the sides, not taken into account in the above calculation, they will account for the differences of not more than from 4 to

* When the diaphragm is placed in a tube of uniform bore, then $a = \Lambda$, and we shall get

$$c o f = \frac{1}{\frac{\Lambda}{a c_o} - 1} = \frac{c_o}{\frac{\Lambda}{a} - c_o},$$

and the loss of head, in passing the diaphragm, becomes

$$h = \left(\frac{\Lambda}{a c_o} - 1 \right)^2 \times \frac{v^2}{2g}.$$

It is evident from the equations that $\frac{\Lambda}{a}$ and c_o depend mutually on each other, and that they cannot be assumed arbitrarily. See equations (60), (67), (123), (124), and (125), with the corresponding remarks.

6 per cent. between the calculation and experiment. If c_o be assumed for calculation equal $\cdot 590$, then $c_o f = \cdot 821$; and as this result agrees very closely with the experimental one, c_o should be taken of this value in using the foregoing formulæ, from (60) to (63), for practical purposes. The thickness of the diaphragm itself and the relation of that thickness to the diameter, as well as the form of the orifice a , are necessary elements in the consideration of this question.

COEFFICIENT OF RESISTANCE.—LOSS OF MECHANICAL POWER
IN THE PASSAGE OF WATER THROUGH THIN PLATES AND
PRISMATIC TUBES.

The coefficients of contraction, velocity, and discharge have been already defined. *The coefficient of resistance is the ratio of the head due to the resistance, to the theoretical head due to the actual or final velocity.* If v be this latter velocity, the theoretical head due to it is $\frac{v^2}{2g}$; and if c_r be the coefficient of resistance, then the head due to the resistance itself is, from our definition, $c_r \times \frac{v^2}{2g}$. Now if c_v be the coefficient of velocity, the theoretical velocity of discharge must be $\frac{v}{c_v}$, and the head due to it is equal $\frac{v^2}{c_v^2 \times 2g}$; but as the theoretical head due to v is $\frac{v^2}{2g}$, we shall have

$$\frac{v^2}{c_v^2 \times 2g} - \frac{v^2}{2g} = \left(\frac{1}{c_v^2} - 1 \right) \frac{v^2}{2g}$$

: the head due to the resistance; and, therefore, from our definition, the coefficient of resistance

$$(64.) \quad c_r = \frac{1}{c_v^2} - 1;$$

from which we shall find the coefficient of velocity

$$(65.) \quad c_v = \left\{ \frac{1}{c_r + 1} \right\}^{\frac{1}{2}}.$$

These equations enable us to calculate the coefficient of resistance from the coefficient of velocity, and *vice versa*. If $c_v = 1$, $c_r = 0$, as it should be. The following short table, calculated from equation (65), will be of use. In short tubes, the coefficient of velocity c_v is equal to the coefficient of discharge c_d .

COEFFICIENTS OF VELOCITY AND RESISTANCE.

Coefficient of velocity.	Coefficient of resistance.	Coefficient of velocity.	Coefficient of resistance.	Coefficient of velocity.	Coefficient of resistance.
.990	.020	.910	.208	.890	.452
.970	.063	.890	.263	.820	.488
.950	.100	.870	.320	.814	.508
.930	.156	.850	.383	.810	.525

The coefficient of velocity for an orifice in a thin plate, or for a mouth-piece, Fig. 4, is .974; while that for a short prismatic tube, A, Fig. 24, is .814 nearly. The coefficient of resistance in the former case is .054, and in the latter .508; there is, therefore, 9.4 times as great a loss of mechanical power in the passage through short prismatic tubes, as through orifices in thin plates or tubes with a rounded junction, as in Fig. 4, the quantities of water discharged and the discharging velocities being the same.

If the quantities discharged and the heads be the same in both cases, then we shall have

$$\frac{v_i^2}{2g \times .814^2} = \frac{v_o^2}{2g \times .974^2} \text{ equal the head;}$$

* See the tables of resistances, discharge, and contraction, pp. 174 and 170.

that is, $\frac{v_t^2}{.663 \times 2g} = \frac{v_o^2}{.949 \times 2g}$, or $.949 v_t^2 = .663 v_o^2$; whence we get $v_t^2 = .698 v_o^2$ and $v_o^2 = 1.431 v_t^2$ for the relation of the discharging velocities, v_o , from an orifice, and, v_t , from a short tube. The height due to the resistance is therefore, $\left(\frac{1}{.814^2} - 1\right) \frac{v_t^2}{2g}$ for short prismatic tubes, and $\left(\frac{1}{.974^2} - 1\right) \frac{1.431 v_t^2}{2g}$ for orifices in thin plates. These are to each other as .508 to $.054 \times 1.431$, or as 5.08 to .773, that is to say, *the loss of mechanical power arising from the resistance in passing through short tubes is 6.57 times as great as when the water passes through thin plates or mouth-pieces, as in Fig. 4;* and the discharging mechanical power in plates, is to that in tubes as 1.431 to 1, or as 1 : .698, the heads and quantities discharged being the same.

The whole loss of mechanical power in the passage is 5.4 per cent. for the plates, and about 51 per cent. for short tubes. If the loss compared with the whole head be sought, we get, when v is the discharging velocity, $\frac{v}{.814}$ for the theoretical velocity due to the head in short tubes, and its square $\frac{v^2}{.814^2} = \frac{v^2}{.663}$ is as the whole head; therefore, the whole head is to the head due to the discharging velocity as $\frac{v^2}{.663}$ to v^2 , or as 1 to .663; and as .508 is the coefficient of resistance* for the discharging velocity, $.508 \times .663$.337 is the coefficient of resistance due to the

* Table, p. 171.

whole head; this is equal to a loss of 34 per cent. nearly, or about one-third. In like manner, we find $.974^2 \times .054 = .0512$ for the coefficient when the discharge takes place through thin plates, or $5\frac{1}{2}$ per cent. of the whole head.

DIAPHRAGMS.

When a diaphragm, *o r*, Fig. 27, is placed at the entrance of a short tube, we have shown, page 168,

that a loss of head equal $\frac{\left(\frac{\Lambda}{a c_o} - 1\right)^2 v^2}{2 g}$ takes place

when v is the discharging velocity, whence the coefficient of resistance is equal to $\left(\frac{\Lambda}{a c_o} - 1\right)^2$,* according

to our definition. The coefficient of contraction c_o , as we have before shown, page 170, should be taken equal to .590 in the application of formula (63); and, as it must also be taken equal to about .621 when the area of the tube Λ is very large compared with the area a of the orifice *o r* in the diaphragm, we may

assume that when $\frac{a}{\Lambda}$ is equal to

0, .1, .2, .3, .4, .5, .6, .7, .8, .9, and 1 successively, the coefficient c_o must be taken equal to .621, .618, .616, .613, .609, .606, .603, .600, .597, .593, and .590, in the same order. As the approaching section *o*

* For the sudden alteration in the velocity passing through a diaphragm, we must reject the hypothesis of D'Aubuisson, *Traité d'Hydraulique*, p. 238, and adopt that of Navier, taking the loss of head to correspond to the square of the difference and not to the difference of the squares of the velocities *in* and *after* passing the orifice. The coefficient of contraction must, however, be varied to suit the ratio of the channels, as it is in this and the following pages.

may be considered exceedingly large, the value of the coefficient of discharge or velocity, as the tube o r s t is supposed full, in equation (61), becomes

$$(66.)' \quad c_d = \left\{ \frac{1}{1 + \left(\frac{\Lambda}{a c_o} - 1 \right)^2} \right\}^{\frac{1}{2}},$$

and the coefficient of resistance

$$(67.) \quad c_r = \left(\frac{\Lambda}{a c_o} - 1 \right)^2;$$

from which equations and the above values of c_o , corresponding to $\frac{a}{\Lambda}$, we have calculated the following values of the coefficients of discharge and resistance through the tube o r s t, Fig. 27.

COEFFICIENTS OF CONTRACTION, DISCHARGE, AND RESISTANCE FOR DIAPHRAGM.

Ratio $\frac{a}{\Lambda}$	Coefficient c_c	Coefficient c_d	Coefficient c_r	Ratio $\frac{a}{\Lambda}$	Coefficient c_c	Coefficient c_d	Coefficient c_r
0.0	.021	.000	infinite.	0.0	.000	.403	3.115
0.1	.018	.000	231.	0.7	.000	.687	1.007
0.2	.015	.130	50.8	0.8	.507	.075	1.198
0.3	.012	.219	10.8	0.9	.503	.753	.702
0.4	.000	.307	0.6	1.0	.500	.831	.483
0.5	.000	.300	5.3

In this table c_o is the coefficient of contraction, c_d the coefficient of discharge, suited to the larger section of the pipe Λ , at s t; and c_r the coefficient of resistance. The discharge is found from equation (61), as c is here very large compared with Λ , to be

$$\begin{aligned} 7A.) \quad D &= \Lambda \sqrt{2 g h} \left\{ \frac{1}{1 + \left(\frac{\Lambda}{a c_o} - 1 \right)^2} \right\}^{\frac{1}{2}} \\ &= \Lambda \sqrt{2 g h} \left\{ \frac{1}{1 + c_r} \right\}^{\frac{1}{2}} = c_d \Lambda \sqrt{2 g h}. \end{aligned}$$

The coefficient of resistance c_r is here equal $\left(\frac{\Lambda}{a c_0} - 1\right)^2$,

and the coefficient of discharge $c_d = \frac{1}{(1 + c_r)^{\frac{1}{2}}}$.*

The tube must be so placed, that the water, after passing the diaphragm, shall fill it; for instance, between two cisterns, when the height h must be measured between the water surfaces, or when the tube is sufficiently long to be filled; in this case, however, *the height must be determined from the discharging velocity*, as a portion of the head is required to overcome the friction, which we shall have immediately to refer to more particularly.

The table shows that the head due to the resistance is 5.3 times that due to the discharging velocity, when the area of the diaphragm is half the area of the tube; that is, the whole head required is 6.3 times that due to the velocity, and that the coefficient of discharge is reduced to .399. In order to find the coefficients suited to the smaller area of the orifice in the diaphragm or a , when it is to be used in calculations of the discharge, we have only to divide

the numbers corresponding to $\frac{\Lambda}{a}$ into those of c_d , op-

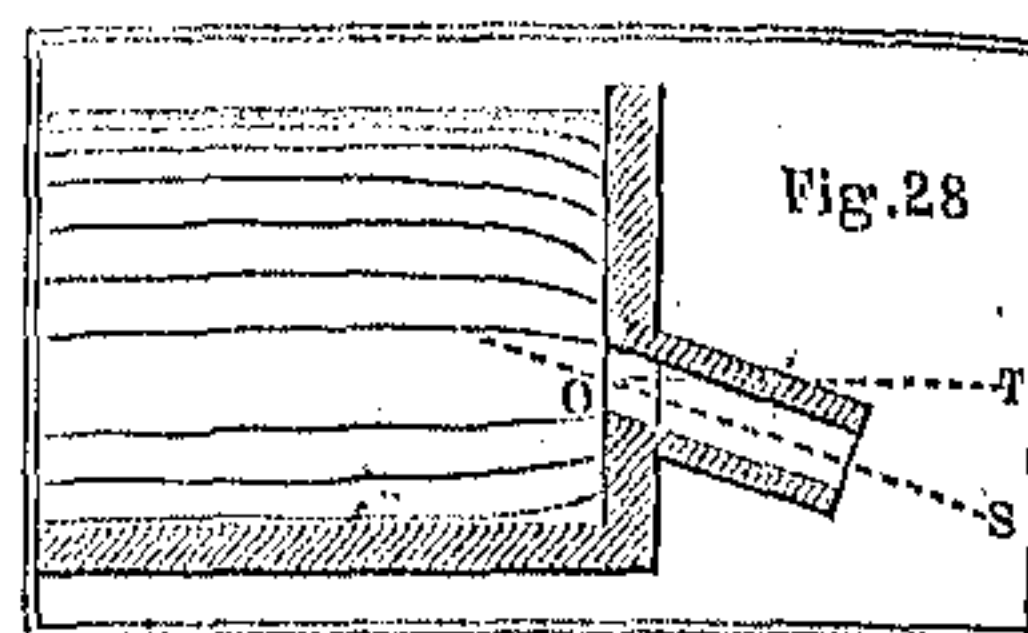
posite to them in the table. Thus, when $\frac{\Lambda}{a} = .8$, we have the coefficient of discharge suited to the area a ,

* For the loss sustained by contraction in the bore of a pipe by a diaphragm, see equations (123), (124), and (125). The actual value of c_0 in equation (67A) depends on the thickness of the diaphragm as well as on the relation of a and Λ . The form of the orifice a also affects the value of c_0 .

equal $\frac{.675}{.8} = .844$, and so of other values of the ratio $\frac{a}{A}$. The coefficients in the table, page 174, are for the larger orifice A in the formula $D = A c_a \sqrt{2 g h}$.

SHORT TUBES OBLIQUE AT THE JUNCTION.

When a tube is attached obliquely, as in Fig. 28, we have found that if the number of degrees in the angle



$\tau o s$, formed by the direction of the tube $o s$, with the perpendicular $o \tau$, be represented by ϕ , then $.814 - .0016 \phi$ will give the coefficient of discharge corresponding to the obliquely attached short tube in the Figure. This formula is, however, empirical, but it is simple, and agrees pretty closely with experimental results. As the coefficient of resistance is equal $\frac{1}{c_r^2} - 1$, equation (64), we have

here $c_r = \frac{1}{(.814 - .0016 \phi)^2} - 1$; from these equations we have calculated the following table for heads measured to the middle of the outside orifice:—

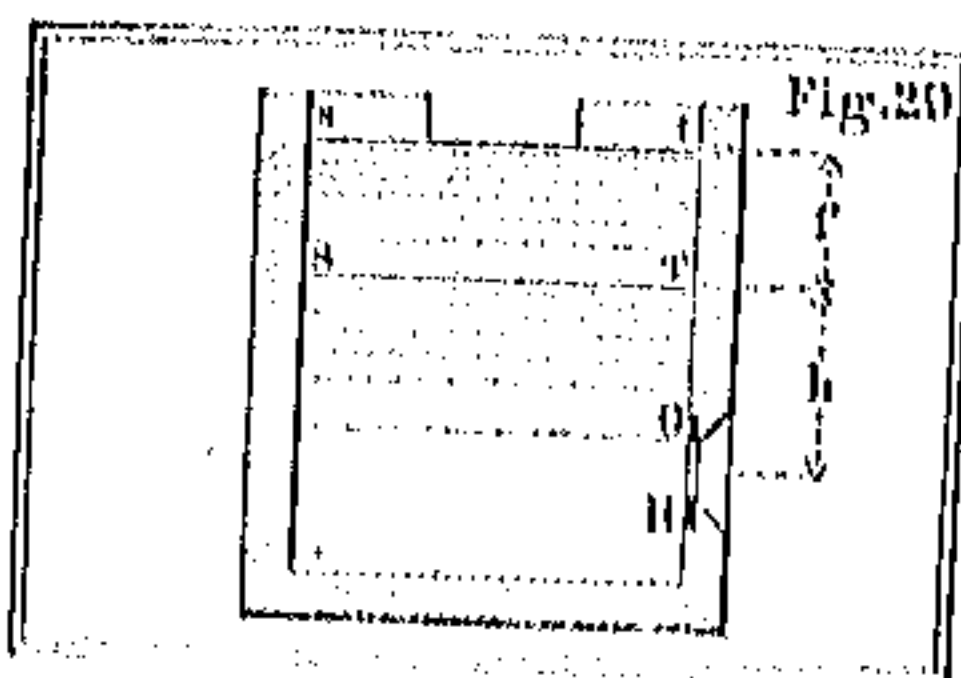
COEFFICIENTS OF DISCHARGE AND RESISTANCE FOR OBLIQUE JUNCTIONS.

ϕ in degrees.	Coefficient of discharge.	Coefficient of resistance.	ϕ in degrees.	Coefficient of discharge.	Coefficient of resistance.
0°	.814	.508	35	.758	.740
5	.806	.530	40	.750	.778
10	.798	.560	45	.742	.816
15	.790	.602	50	.734	.858
20	.782	.655	55	.726	.907
25	.774	.709	60	.718	.940
30	.766	.764	65	.710	.984

The coefficient of resistance for a tube at right angles to the side, is to the like coefficient when it makes an angle of 45 degrees as .508 to .816, or as 1 to 1.6 nearly; and the loss of head is greater in the same proportion. If the short tube be more than three or four diameters in length, friction will have to be taken into account. The head h is measured to the outside orifice.

FORMULA FOR FINDING THE TIME THE SURFACE OF WATER IN A CISTERN TAKES TO SINK A GIVEN DEPTH.—DISCHARGE FROM ONE VESSEL OR CHAMBER INTO ANOTHER.—LOOK CHAMBERS.

In experiments for finding the value of the coefficients of discharge, one of the best methods is to observe the time the water discharged from the orifice takes to sink the surface in a prismatic cistern a given depth; the ratio of the observed to the theoretical time will then give the coefficient sought. A formula for finding the theoretical time is, therefore, of much practical value. In Fig. 29, the time of falling from st to sr , in seconds, is



$$(68.) \quad t = \frac{\Lambda}{4.0125 a} \{ (h + f)^2 - h^2 \},$$

in which a is the area of the orifice $o n$, and Λ the area of the prismatic vessel at st or sr ; this formula is for measures in feet. For measures in inches, we have

$$(69.) \quad t = \frac{\Lambda}{13.9 a} \{ (h + f)^2 - h^2 \}.$$

EXAMPLE VII. A cylindrical vessel 5.74 inches in diameter has an orifice .2 inch in diameter at a depth

of 16 inches below the surface, measured to the centre; it is found that the water sinks 4 inches in 51 seconds; what is the coefficient of discharge?

The theoretical time t is found from equation (69), equal

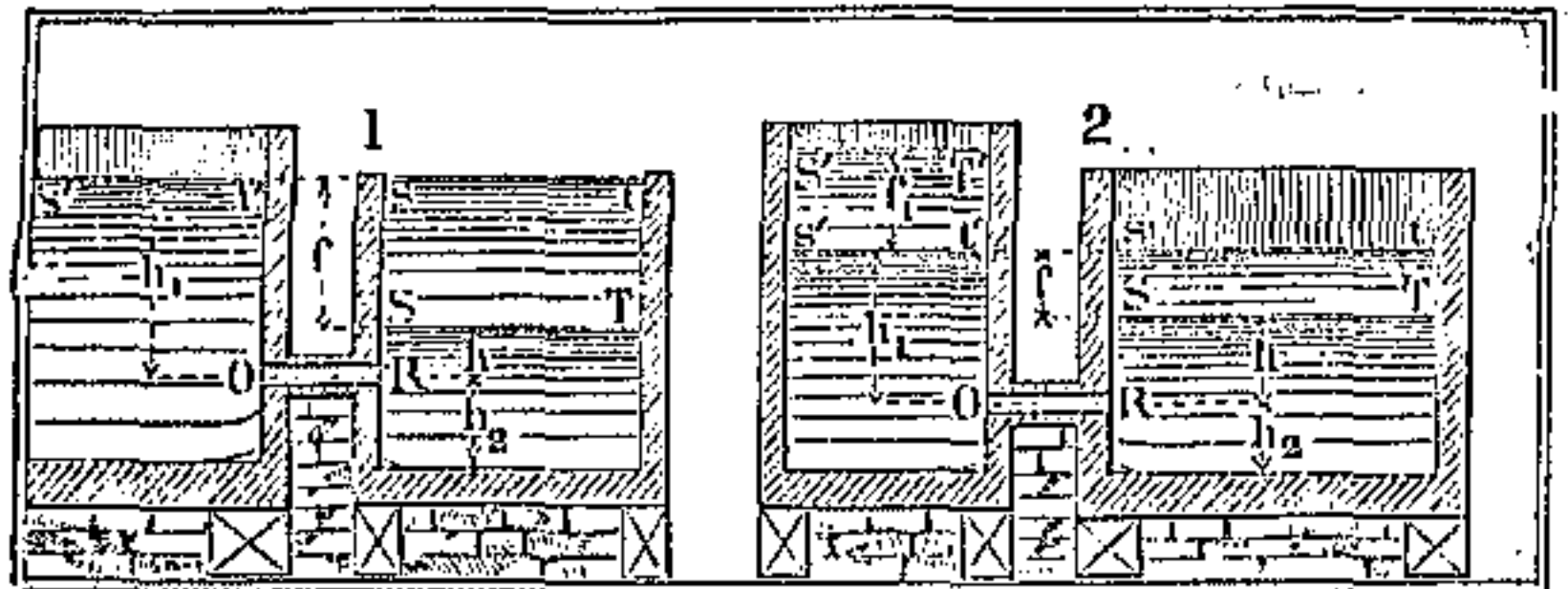
$$\frac{5.74^2 \times .7854}{13.9 \times .2^2 \times .7854} \{16^{\frac{1}{2}} - 12^{\frac{1}{2}}\} = \frac{32.9476}{.556} \{4 - 3.4641\}.$$

$$= \frac{17.6566}{.556} \times .5359 = 31.8 \text{ seconds; hence, } \frac{31.8}{51} = .624$$

is the coefficient sought. When the orifice $o r$ and the horizontal section of the vessel are similar figures, $\frac{A}{a}$ is equal $\frac{S T^2}{O R^2}$; and therefore, for circular cisterns and orifices, it is unnecessary to introduce the multiplier .7854.

We have given above, formulæ for the time water in a prismatic vessel takes to fall a given depth, when dis-

Fig. 29a.



charged from an orifice at the side or bottom. The time the surface $s t$, diagram 1, Fig. 29a, takes to rise to $s t$, when supplied through an orifice or tube $o r$, from an upper large chamber or canal, whose surface $s' t'$ remains always at the same level, is $\frac{2 A f^{\frac{1}{2}}}{c_a a \sqrt{2 g}}^*$

* The time of rising from s to s is exactly double the time it would take, if the pressure f remained uniform, to fill the same depth below r .

and we thence get the time of rising from r to s for measures in feet

$$(69A.) \quad t = \frac{A}{4.015 c_a a} \{h_1^{\frac{1}{2}} - f^{\frac{1}{2}}\},$$

and for measures in inches

$$(69B.) \quad t = \frac{A}{13.9 c_a a} \{h_1^{\frac{1}{2}} - f^{\frac{1}{2}}\},$$

in which A is the area of the horizontal section at s r ; a the sectional area of the communicating channel or orifice o r ; c_a the coefficient of discharge suited to it, and h_1 and f , as shown in the diagram.

In order to find the time of filling the lower vessel to the level s r , supposing it at first empty, we have the contents of the portion below o r equal to $A h_2$, and the time of filling it equal to

$$(69C.) \quad \frac{A h_2}{8.025 c_a a h_1^{\frac{1}{2}}};$$

then the time of filling up to any level s r , for measures in feet, is equal to the sum of (A) and (C); that is,

$$(69D.) \quad T = \frac{A h_1^{\frac{1}{2}}}{8.025 c_a a} \left\{ 2 + \frac{h_2}{h_1} - \frac{2 f^{\frac{1}{2}}}{h_1^{\frac{1}{2}}} \right\} \\ = \frac{A (2 h_1 + h_2 - 2 f^{\frac{1}{2}} h_1^{\frac{1}{2}})}{8.025 c_a a h_1^{\frac{1}{2}}},$$

and for measures in inches

$$(69E.) \quad T = \frac{A h_1^{\frac{1}{2}}}{27.8 c_a a} \left\{ 2 + \frac{h_2}{h_1} - \frac{2 f^{\frac{1}{2}}}{h_1^{\frac{1}{2}}} \right\} \\ = \frac{A (2 h_1 + h_2 - 2 f^{\frac{1}{2}} h_1^{\frac{1}{2}})}{27.8 c_a a h_1^{\frac{1}{2}}}.$$

When s r coincides with s t ,

$$(69F.) \quad T = \frac{A (2 h_1 + h_2)}{8.025 c_a a h_1^{\frac{1}{2}}},$$

for measures in feet, and

$$(69a.) \quad T = \frac{\Lambda (2h_1 + h_2)}{27.8 c_d a h_1^{\frac{3}{2}}},$$

for measures in inches. These equations are exactly suited to the case of a closed lock-chamber filled from an adjacent canal.

When the upper level $s' t'$ is also variable, as in Diagram 2, the time which the water in both vessels takes to come to the same uniform level $s' t' s t$, is

$$(69b.) \quad t = \frac{2\Lambda c(h_1 + f_1 - h)^{\frac{3}{2}}}{c_d a (\Lambda + c) \sqrt{2g}} = \frac{2\Lambda c(f + f_1)^{\frac{3}{2}}}{c_d a (\Lambda + c) \sqrt{2g}};$$

in which $h_1 + f_1 - h = f + f_1$ is the difference of levels at the beginning of the flow; c the horizontal section of the upper chamber; and the other quantities as in Diagram 1. As $cf_1 = \Lambda f$, we find

$$f + f_1 = \frac{c + \Lambda}{\Lambda} f_1 = \frac{c + \Lambda}{c} f.$$

Now, in order to find the time of falling a given depth d below the first level $s' t'$, we have the head above $s' t' s t$ equal to $f_1 - d$ in the upper vessel, and the depth below it in the lower vessel equal to $\frac{c(f_1 - d)}{\Lambda}$; whence the difference of levels in the two

vessels at the end of the fall d , is

$$f_1 - d + \frac{c(f_1 - d)}{\Lambda} = \frac{\Lambda + c}{\Lambda} (f_1 - d).$$

The time of falling through d is, therefore, from equation (69 b),

$$(69c.) \quad t = \frac{2\Lambda c(f + f_1)^{\frac{3}{2}}}{c_d a (\Lambda + c) \sqrt{2g}} - \frac{2\Lambda c \left\{ \left(\frac{c + \Lambda}{\Lambda} \right) (f_1 - d) \right\}^{\frac{3}{2}}}{c_d a (\Lambda + c) \sqrt{2g}} \\ = \frac{2\Lambda c}{c_d a (\Lambda + c) \sqrt{2g}} \left\{ (f + f_1)^{\frac{3}{2}} - \left(\frac{\Lambda + c}{\Lambda} (f_1 - d) \right)^{\frac{3}{2}} \right\},$$

in which $\sqrt{2g} = 8.025$ for measures in feet, and equal 27.8 for measures in inches. The whole time of filling to a level the lower empty vessel, is found by adding the time of filling the portion below r , determined in a manner similar to equations (68) and (69) to be

$$(69\kappa.) \quad \frac{2a}{c_a a \sqrt{2g}} \left\{ (h_1 + f_1)^{\frac{5}{2}} - \left(h_1 + f_1 - \frac{h_2 \Lambda}{c} \right)^{\frac{5}{2}} \right\},$$

to the time of filling above r , given in equation (11), when h is taken equal to zero. Equations (11), (1), and (κ) are applicable to the case of the upper and lower chambers of a double lock, after making the necessary change in the diagrams.

The above equations require further extensions when water flows into the upper vessel while also flowing from it into the lower; such extensions are, however, of little practical value, and we therefore omit them. For sluices in flood-gates with square arrises, c_a may be taken at about .545, but with rounded arrises, the coefficient will rise much higher. See SECTIONS III. and VI.

SECTION VIII.

FLOW OF WATER IN UNIFORM CHANNELS.—MEAN VELOCITY.—
 MEAN RADII AND HYDRAULIC MEAN DEPTHS.—BORDER.
 —TRAIN.—HYDRAULIC INCLINATION.—EFFECTS OF FRICTION.—
 FORMULÆ FOR CALCULATING THE MEAN VELOCITY.
 —APPLICATION OF THE FORMULÆ AND TABLES TO THE
 SOLUTIONS OF THREE USEFUL PROBLEMS.

In rivers the velocity is a maximum along the central line of the surface, or, more correctly, over the deepest part of the channel; and it decreases thence to the sides and bottom: but when backwater arises from any obstruction, either a submerged weir, Fig. 22, or a contracted channel, Fig. 23, the velocity in the channel approaching the obstruction is a maximum at the depth of the backwater below the surface, and it decreases thence to the surface, sides, and bottom. When water flows in a pipe of any length, the velocity at the centre is greatest, and it decreases thence to the sides or circumference of the pipe. If the pipe be supposed divided into two portions in the direction of its length, the lower portion or channel will be analogous to a small river or stream, in which the velocity is greatest at the central line of the surface, and the upper portion will be simply the lower reversed. A pipe flowing full may, therefore, be looked upon as a double stream, and we shall soon see that the formulæ for the discharge from each kind are all but identical, though a pipe may discharge full at all inclinations, while the velocity in rivers or streams, having uniform inclination, never exceed a few feet per mile.

MEAN VELOCITY.

It is found, by experiment, that the mean velocity is nearly independent of the depth or width of the channel, the central or maximum velocity being the same. From a number of experiments, Du Buât derived empirical formulæ equivalent to

$$v = \frac{v_b + v}{2} = v - v^{\frac{1}{2}} + \frac{1}{2}, \quad v_b = (v^{\frac{1}{2}} - 1)^2, \quad \text{and} \quad v = (v_b^{\frac{1}{2}} + 1)^2;$$

in these equations v is the mean velocity, v the maximum surface velocity, and v_b the velocity at the sides, or bottom, expressed in French inches. Tables calculated from these formulæ do not give correct results for measures in English inches, though they are those generally adopted. Disregarding the difference in the measures, which are as 1 to 1.0678, it will be found that, in the generality of channels, the mean velocity is not an arithmetical mean between the velocity at the central surface line and that at the bottom, though nearly so between the mean bottom and mean surface velocities. Dr. Young,* modifying Du Buât's formula, assumes for English inches that $v + v^{\frac{1}{2}} = v$, and hence $v = v + \frac{1}{2} - (v + \frac{1}{2})^{\frac{1}{2}}$. This gives results very nearly the same as the other formula for v , but something less, particularly for small surface velocities. For instance, Du Buât's formula gives .5 inch for the mean velocity when the central surface velocity is 1 inch, whereas Dr. Young's makes it .38 inch. For large velocities both formulæ agree very closely, disregarding the difference between the measures, which is only seven per

* Philosophical Transactions, 1808, p. 487.

cent. They are best suited to very small channels or pipes, but unless at mean velocities of about 3 feet per second, they are wholly inapplicable to rivers.

Prony found, from Du Buat's experiments, that for measures in metres $v = \left(\frac{2.37187 + v}{3.15312 + v} \right) v$, in which v is also the maximum surface velocity. This, reduced for measures in English feet, becomes

$$(70.) \quad v = \left(\frac{7.783 + v}{10.345 + v} \right) v;^*$$

and for measures in English inches,

$$(71.) \quad v = \left(\frac{93.39 + v}{124.14 + v} \right) v.$$

For medium velocities $v = .81 v$. The experiments from which these formulæ were derived were made with small channels. We have calculated the values of v from that of v , equation (71), and given the results, in columns 3, 6, and 9, in TABLE VII. Ximenes, Funk, and Brünnig's experiments in larger channels give the mean velocity at the centre of the depth equal $.914 v$, when the central or maximum surface velocity is v ; but as the velocity also decreases in nearly the same ratio at the surface from the centre to the sides of the channel, we shall get the mean velocity in the whole section equal

* Francis, "Lowell Experiments," p. 150, finds this formula to give 15 per cent. less than the result found by weir measurement from the formula $D = 3.33 (L - .1 n h) h^2$, the quantity discharged being about 250 cubic feet per second, and the velocity about 3.2 feet. It appears, however, that Francis uses the mean surface velocity, and not the maximum surface velocity required by the formula: if the latter were used, the difference would be reduced to 6 per cent., or thereabouts, in equation (72).

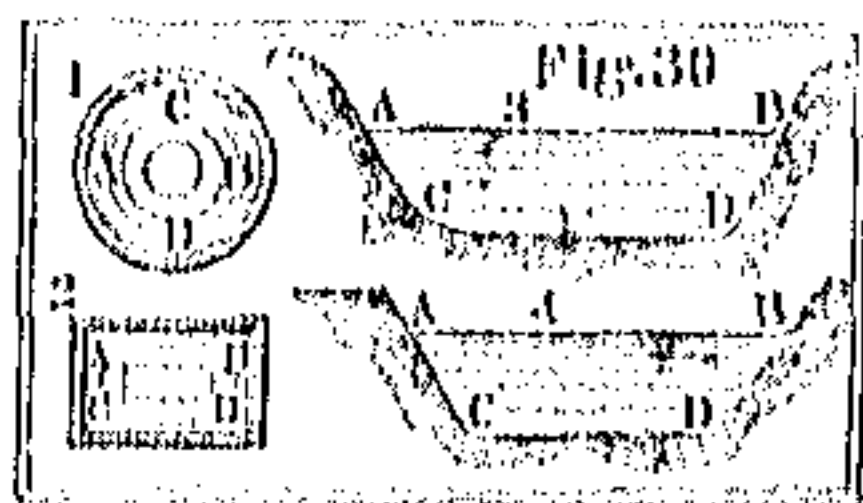
$\cdot 914 \times \cdot 914 v = \cdot 835 v$; and hence we have, for large channels,

$$(72.) \quad v = \cdot 835 v,$$

in which equation v is the *maximum velocity at the surface*. We have also calculated the values of v from this formula, and given the results in columns 2, 5, and 8 of TABLE VII. This table will be found to vary considerably from those calculated from Du Buât's formula in French inches, hitherto generally used in this country, and much more applicable for all practical purposes.

MEAN RADIUS.—HYDRAULIC MEAN DEPTH.—BORDER.—
COEFFICIENT OF FRICTION.

If, in the diagrams 1 and 2, Fig. 30, exhibiting the sections of cylindrical and rectangular tubes filled with flowing water, the areas be divided respectively by the perimeters $A O B D A$ and $A B D C A$, the quotients are termed "*the mean radii*" of the tubes, diagrams 1 and 2; and the perimeters in contact with the flowing water are termed "*the borders*." In the diagrams 3 and 4, the surface $A B$ is not in contact with the channel, and the width of the bed and sides, taken together, $A O D B$, becomes "*the border*." "*The mean radius*" is equal to the area $A B D C A$ divided by the length of the border $A O D B$. "*The hydraulic mean depth*" is the same as "*the mean radius*," this latter term being perhaps most applicable to pipes flowing full, as in diagrams 1 and 2; and the former to streams and rivers which



have a surface line A B, diagrams 3 and 4. We shall, throughout the following equations, designate the value of the "mean radius," "hydraulic mean depth," or quotient, $\frac{\text{area A B D C A}}{\text{border B D C A}}$,* by the letter r , remarking here that *for cylindrical pipes flowing full, or rivers with semicircular beds, it is always equal to half the radius, or one-fourth of the diameter.*

Du Buât was the first to observe that the head due to the resistance of friction for water flowing in a uniform channel increased directly as the length of the channel l , directly as the border, and inversely as the area of the cross-flowing section,† very nearly; that is, as $\frac{l}{r}$. It also increases as the square of the velocity, nearly; therefore the head due to the resistance must be proportionate to $\frac{v^2 l}{2 g r}$. If $c_r \times \frac{v^2 l}{2 g r} = h_r$, then c_r is the coefficient for the head due to the resistance of friction, as h_r is the head necessary to overcome the friction; c_r is therefore termed "*the coefficient of friction.*"

* M. Girard has conceived it necessary to introduce the coefficient of correction 1.7 as a multiplier to the border for finding r , to allow for the increased resistance from aquatic plants; so that, according to his reduction,

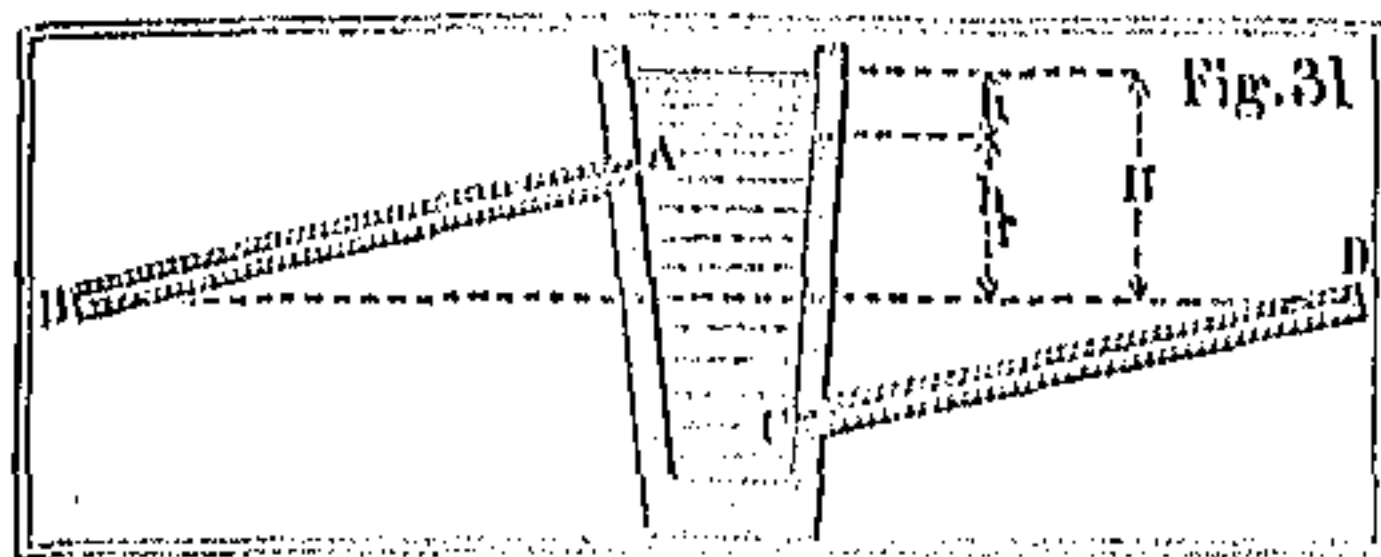
$$r = \frac{\text{area}}{1.7 \text{ border}}.$$

See Rennie's First Report on Hydraulics as a Branch of Engineering; Third Report of the British Association, p. 167; also, quotation (85), p. 201.

† Pitot had previously, in 1726, remarked that the diminution arising from friction in pipes is, *ceteris paribus*, inversely as the diameters.

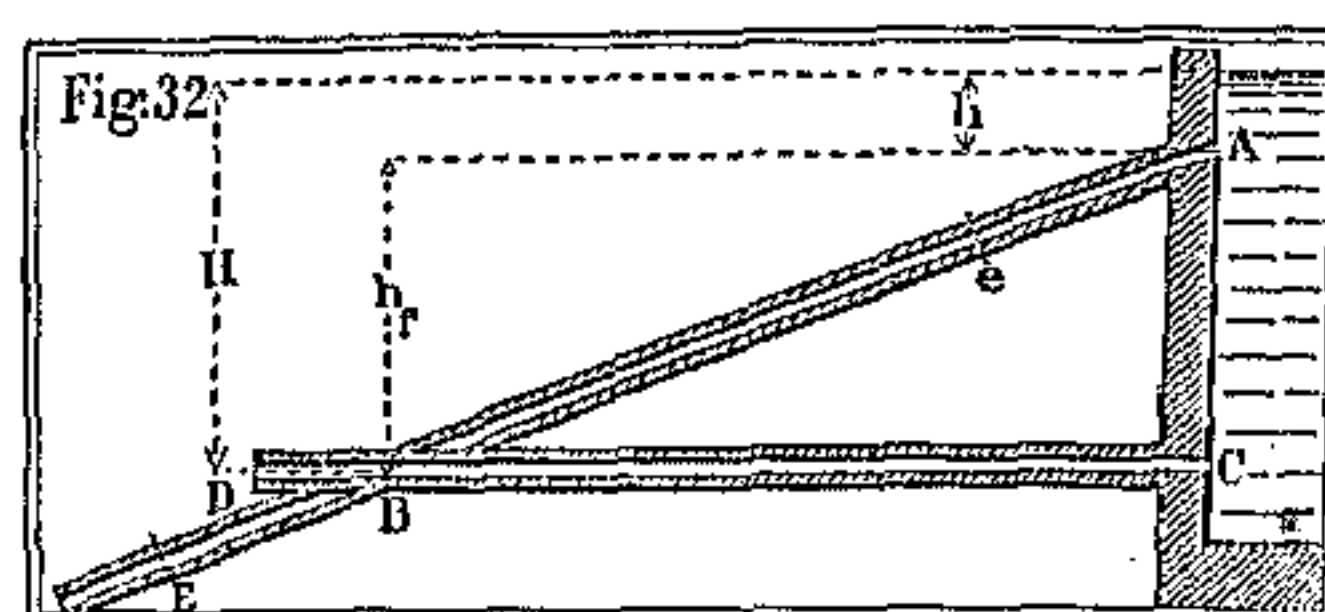
HYDRAULIC INCLINATION.—TRAIN.

If l be the length of a pipe or channel, and h_f the height due to the resistance of friction of water flowing in it, then $\frac{h_f}{l}$ is the *hydraulic inclination*. In Fig. 31 the tubes A B, C D, of the same length l , and



whose discharging extremities B and D are on the same horizontal plane B D, will have the same hydraulic inclination and the same discharge, no matter what the actual inclinations or the depth of the entrances at A and C may be, so they be of the same kind and bore; and as the velocities in A B and C D are the same, the height h due to them must be the same when the circumstances of the orifices of entry A and C are alike. We have the whole head $H = h + h_f$ (see pp. 166 and 167). The hydraulic inclination is not therefore the whole head H , divided by the length l of the pipe, as it is sometimes mistaken, but the height h_f (found by subtracting the height h , due to the entrance at A or C, and the velocity in the pipe, from the whole height) divided by the length l . When the height h is very small compared with the whole height H , as it is in very long tubes with moderate heads, $\frac{H}{l}$ may be substituted for $\frac{h_f}{l}$ without error; but for short pipes up to 100 feet in length

the latter only should be used in applying Du Buât's and some other formulæ; otherwise the results will be too large, and only fit to be used approximately in order to determine the height h from the velocity of discharge thus found. When the horizontal pipe $c d$, Fig. 32, is equal in every way to the inclined



pipe AB , and the head at A is that due to the velocity in CD , the discharge from the pipe AB will be equal to that from CD ; but a peculiar property belongs to the pipe AB in the position in which it is here placed; for *if we cut it short at any point e , or lengthen it to any extent, to E , the discharge will remain the same and equal to that through the horizontal pipe CD .* The velocity in AB at the angle of inclination ABC , when $AC = h_r$, and $AB = CD$, is therefore such that it remains unaffected by the length AE or Ae , to which it may be extended or cut short; and *at this inclination the water in the pipe AB is said to be "in train."* In like manner a river or stream is said to be "*in train*" when the inclination of its surface bears such a relation to the cross section that the mean velocity is neither decreased nor increased by the length of the channel; and we perceive from this *that the acceleration caused by the inclination is exactly counterbalanced*

by the resistances to the motion when the moving water in a pipe or river channel is in train.

As $h = (1 + c_r) \frac{v^2}{2g}$ where c_r is the coefficient of the height due to the resistance at the orifice of entry A or C , and $h_t = c_t \frac{v^2 l}{2gr}$, we therefore get

$$(73.) \quad H = (1 + c_r) \frac{v^2}{2g} + c_t \times \frac{lv^2}{2gr} = \left(1 + c_r + c_t \frac{l}{r}\right) \frac{v^2}{2g},$$

and hence we find the mean velocity of discharge

$$(74.) \quad v = \left\{ \frac{2gH}{1 + c_r + c_t \frac{l}{r}} \right\}^{\frac{1}{2}} = \left\{ \frac{\frac{r}{c_t} \times 2gH}{(1 + c_r) \frac{r}{c_t} + l} \right\}^{\frac{1}{2}};$$

or,

$$(74A.) \quad v = \left\{ \frac{2gH}{\frac{1}{c_d^2} + c_t \frac{l}{r}} \right\}^{\frac{1}{2}} = \left\{ \frac{2gHr}{\frac{r}{c_d^2} + c_t l} \right\}^{\frac{1}{2}};$$

as $c_d^2 = \frac{1}{c_r + 1}$, equation (65). We have also

$$(74B.) \quad v = \sqrt{2gH} \times \left\{ \frac{1}{\frac{1}{c_d^2} + c_t \times \frac{l}{r}} \right\}^{\frac{1}{2}};$$

the values of the second member on the right hand side of this equation, or of

$$\left\{ \frac{1}{\frac{1}{c_d^2} + c_t \times \frac{l}{r}} \right\}^{\frac{1}{2}}$$

are given, for different values of c_r , c_d , and $\frac{l}{r}$, in the small table at p. 152, and below at p. 191.

When h is small compared with h_t , or, which comes to the same thing, $1 + c_r$ small compared with $c_t \times \frac{l}{r}$,

$$(75.) \quad H = c_1 \times \frac{l v^2}{2 g r^2}$$

and

$$(76.) \quad v = \left\{ \frac{2 g r^2 H}{c_1 l} \right\}^{\frac{1}{2}}.$$

If, in the last equation, we substitute s for $\frac{H}{l}$, equal the sine of the angle of inclination $A B C$, we then have

$$(77.) \quad v = \left\{ \frac{2 g r^2 s}{c_1} \right\}^{\frac{1}{2}}.$$

The average value of c_1 for all pipes with straight channels, with velocities of about 1.5 foot per second, is .0069914, from which we find equation (77) becomes, for measures in feet,

$$(78.) \quad v = 96 \sqrt{r s}.$$

As the mean value of the coefficient of resistance c_1 for the entrance into a tube is .508, and as $2 g = 64.403$, and $c_1 = .0069914$, equation (74), for measures in feet, becomes

$$(79.) \quad \left\{ v = \left\{ \frac{64.403 H}{1.508 + .0069914 \frac{l}{r}} \right\}^{\frac{1}{2}}, \text{ or} \right. \\ \left. v = \left\{ \frac{H r}{.0234 r + .0001085 l} \right\}^{\frac{1}{2}} = 100 \left\{ \frac{H r}{234 r + 1.085 l} \right\}^{\frac{1}{2}} \right. \\ \left. = 50 \left\{ \frac{H d}{58 d + 1.085 l} \right\}^{\frac{1}{2}} \right.$$

This, multiplied by the section, gives the discharge.

For velocities between 2 and $2\frac{1}{2}$ feet per second, $c_1 = .0064403$, and therefore

$$v = \left\{ \frac{H r'}{0.0234 r + 0.0001 l} \right\}^{\frac{1}{2}} = 50 \left\{ \frac{H d}{58 d + l} \right\}^{\frac{1}{2}},$$

in which $d = 4r =$ diameter of a pipe.

The following table is calculated from equation

$$\text{VALUES OF } \left\{ \frac{1}{c_d^2 + c_f \times \frac{l}{r}} \right\}^{\frac{1}{2}}.$$

Number of diameters in the length of the pipe.	Corresponding coefficients of discharge.			Number of diameters in the length of the pipe.	Corresponding coefficients of discharge.		
2 diameters	.980	.813	.715	900 diameters	.239	.236	.233
5 "	.957	.791	.693	950 "	.234	.230	.227
10 "	.919	.760	.663	1000 "	.228	.225	.222
15 "	.880	.740	.640	1100 "	.218	.215	.213
20 "	.855	.731	.636	1200 "	.209	.207	.205
25 "	.828	.713	.613	1400 "	.194	.193	.191
30 "	.801	.698	.602	1600 "	.182	.180	.179
35 "	.781	.683	.590	1800 "	.172	.171	.170
40 "	.760	.668	.570	2000 "	.163	.162	.161
45 "	.741	.655	.560	2200 "	.155	.155	.154
50 "	.723	.643	.550	2400 "	.149	.149	.148
100 "	.695	.618	.514	2600 "	.144	.143	.142
150 "	.678	.605	.502	2800 "	.139	.138	.137
200 "	.661	.590	.493	3000 "	.134	.133	.133
250 "	.644	.575	.481	3200 "	.130	.129	.129
300 "	.628	.561	.468	3400 "	.126	.125	.125
350 "	.612	.546	.456	3600 "	.122	.121	.121
400 "	.596	.530	.442	3800 "	.119	.119	.118
450 "	.580	.514	.428	4000 "	.116	.116	.115
500 "	.564	.500	.416	4200 "	.113	.113	.113
550 "	.548	.485	.403	4400 "	.111	.111	.111
600 "	.532	.470	.390	4600 "	.108	.108	.108
650 "	.516	.455	.376	4800 "	.106	.106	.106
700 "	.500	.440	.364	5000 "	.104	.104	.104
750 "	.484	.425	.351	5200 "	.102	.102	.102
800 "	.468	.410	.338	5400 "	.100	.100	.100
850 "	.452	.395	.325	5600 "	.098	.098	.098

(74B) for a velocity of about 20 feet per second, when $c_f = .004556$, and for different orifices of entry, in which c_d varies from .986 for a rounded orifice, to .715 when the pipe projects into the vessel. It gives directly the coefficient, which, multiplied by $\sqrt{2gH}$, gives the velocity in the pipe, taking friction into account.

The small table, SECTION VI., p. 152, gives the like coefficients of $\sqrt{2gH}$ in equation (74B), when $c_f = .00699$ suited to a velocity of about 18 inches per second, and can be applied in like manner. The value of $\sqrt{2gH}$ is given, in inches, in column 2, TABLE II. For feet it is equal $8\sqrt{H}$ nearly.

DU BUAT'S FORMULA.

The coefficient of friction c_f is not, however, constant, as it varies with the velocity. That which we have just given answers for pipes when the velocity is 20 feet per second. For pipes and rivers it is found to increase as the velocity decreases; that is, the loss of head is proportionately greater for small than for large velocities. Du Buat found the loss of head to be also greater for small than large channels, and applied a correction accordingly in his formula. This, expressed in French inches, is

$$(80.) \quad v = \frac{297 (r^4 - .1)}{\left(\frac{1}{s}\right)^{\frac{1}{2}} - \text{hyp. log.} \left(\frac{1}{s} + 1.6\right)^{\frac{1}{2}}} - .3 (r^4 - .1),$$

maintaining the preceding notation, in which $s = \frac{h_1}{l}$.

In this formula .1, in the numerator of the first term, is deducted as a correction due to the hydraulic

mean depth, as it was found that $297 (r^{\frac{1}{2}} - 0.1)$ agreed more exactly with experiment than $297 r^{\frac{1}{2}}$ simply. The second term $\text{hyp. log.} \left(\frac{1}{s} + 1.6 \right)^{\frac{1}{2}}$, of the denominator is also deducted to compensate for the observed loss of head being greater for less velocities, and the last term $\cdot 3 (r^{\frac{1}{2}} - .1)$ is a deduction for a general loss of velocity sustained from the unequal motions of the particles of water in the cross section as they move along the channel. These corrections are empirical; they were, however, determined separately, and after being tested by experiment, applied, as above, to the radical formula $v = 297 \sqrt{r s}$.

Du Buât's formula was published in his *Principes d'Hydraulique*, in 1786. It is, as we have seen, partly empirical, but deduced by an ingenious train of reasoning and with considerable penetration from about 125 experiments, made with pipes from the 19th part of an inch to 18 inches in diameter, laid horizontally, inclined at various inclinations, and vertical; and also from experiments on open channels with sectional areas from 19 to 40,000 square inches, and inclinations of from 1 in 112 to 1 in 36,000. The lengths of the pipes experimented with varied from 1 to 3, and from 3 to 3600 feet.

In several experiments by which we have tested this formula, the resulting velocities found from it were from 1 to 5 per cent. too large for small pipes, and too small for straight rivers in nearly the same proportion. As the experiments from which it was derived were made with great care, those with pipes particularly so, this was to be expected. Expe-

riments with pipes of moderate or short lengths should have the circumstances of the orifice of entry from the reservoir duly noted; for the close agreement of this formula with them must depend a great deal, in such pipes, on the coefficient due to the height h , which must be deducted from the whole head H before the hydraulic inclination, $\frac{h_r}{l} = s$, can be obtained; but for very long pipes and uniform channels this is not necessary.

The experiments from which Du Buât's formula was constructed are given in full by the late Dr. Robinson in his able article on "rivers" in the *Encyclopædia Britannica*, pp. 268, 269, and 270, where the calculated and observed velocities are placed side by side in French inches per second. In all these experiments Du Buât carefully deducted the head due to the velocity and orifice of entry before finding the hydraulic inclination s , and those who attempt to calculate the velocity from the head and length of the channel only, without making this deduction, will find their calculated results very different from those there given. If there were bends, curves, or contractions, deductions would have to be made for these in like manner before finding s .

Under all the circumstances, and after comparing the results obtained from various other formulæ, we have preferred calculating tables for the values of v from this formula reduced for measures in English inches, which is

$$v = \frac{306.596 (r^{\frac{1}{2}} - .1032)}{\left(\frac{1}{s}\right)^{\frac{1}{2}} - \text{hyp. log.} \left(\frac{1}{s} + 1.6\right)} - .2906 (r^{\frac{1}{2}} - .1032),$$

or more simply,

$$(81.) \quad v = \frac{307 (r^4 - .1)}{\left(\frac{1}{s}\right)^4 - \text{hyp. log.} \left(\frac{1}{s} + 1.6\right)^4} - .3 (r^4 - .1).$$

This gives the value of v a little larger than the original formula, but the difference is immaterial. For measures in English feet it becomes

$$(82.) \quad v = \frac{88.51 (r^4 - .03)}{\left(\frac{1}{s}\right)^4 - \text{hyp. log.} \left(\frac{1}{s} + 1.6\right)^4} - .084 (r^4 - .03).$$

The results of equation (81) are calculated for different values of s and r , and tabulated in TABLE VIII., the first eight pages of which contain the velocities for values of r varying from $\frac{1}{16}$ th inch to 6 inches; or if pipes, diameters from $\frac{1}{4}$ inch to 2 feet, and of various inclinations from horizontal to vertical. The last five pages contain the velocities for values of r from 6 inches to 12 feet, and with falls from 6 inches to 12 feet per mile.

EXAMPLE VIII. *A pipe, $1\frac{1}{2}$ inch diameter and 100 feet long, has a constant head of 2 feet over the discharging extremity; what is the velocity of discharge per second?*

The mean radius $r = \frac{1\frac{1}{2}}{4} = \frac{3}{8}$ inches, and $\frac{100}{2} = 50 = \frac{1}{s}$, is the approximate hydraulic inclination. At page 2 of TABLE VIII., in the column under the mean radius $\frac{3}{8}$, and opposite to the inclination 1 in 50, we find 30.69 inches for the velocity sought. This, however, is but approximative, as the head due to the velocity should be subtracted from the whole head of 2 feet, before finding the true hydraulic inclination. This

head depends on the coefficient of resistance at the entrance orifice, or the coefficient of discharge for a short tube. In all Du Buât's experiments this latter was taken at .8125, but it will depend on the nature of the junction, as, if the tube runs into the cistern, it will become as small as .715; and, if the junction be rounded into the form of the contracted vein, it will rise to .974, or 1 nearly. In this case, the coefficient of discharge may be assumed .815,* from which, in TABLE II., we find the head due to a velocity of 30.69 inches to be $1\frac{7}{8} = 1.87$ inch nearly, which is the value of h ; and hence, $11 - h = h_1 = 24 - 1.87 = 22.13$ inches; and $\frac{l}{h_1} = \frac{100 \times 12}{22.13} = 54.2 = \frac{1}{s}$, the hydraulic inclination, more correctly. With this new inclination and the mean radius $\frac{3}{8}$, we find the velocity by interpolating between the inclinations 1 in 50 and 1 in 60, given in the table to be $30.69 - 1.34 = 29.35$ inches per second. This operation may be repeated until v is found to any degree of accuracy according to the formula; but it is, practically, unnecessary to do so. If we now wish to find the discharge per minute in cubic feet, we can easily do so from TABLE IX., in which, for an inch and a half pipe, we get

	Inches.		Cubic Feet.	
For a velocity of	20.00	per second,	1.22718	per minute.
"	9.00	"	.55223	"
"	.30	"	.01841	"
"	.04	"	.00245	"
"	29.34	"	1.80027	"

* See EXAMPLE 16, pp. 28, 30.

The discharge found experimentally by Mr. Provis, for a tube of the same length, bore, and head, was 1.745 cubic foot per minute.

If we suppose the coefficient of discharge due to the orifice of entry and stop-cock in Mr. Provis's 208 experiments* with $1\frac{1}{2}$ inch lead pipes of 20, 40, 60, 80, and 100 feet lengths, to be .715, the results calculated by the tables will agree with the experimental results with very great accuracy, and it is very probable, from the circumstances described, that the ordinary coefficient .815 due to the entry was reduced by the circumstances of the stop-cock and fixing to about .715; but even with .815 for the coefficient, the difference between calculation and experiment is not much, the calculation being then in excess in every experiment, the average being about 5 per cent., and not so much in the example we have given.

TABLE VIII. will give the velocity, and thence the discharge, immediately, for long pipes, and TABLE X. enables us to calculate the cubic feet discharged per minute, with great facility. For rivers, the mean velocity, and thence the discharge, is also found with quickness. See also TABLES XI., XII., and XIII., and the table at pp. 42 and 43.

EXAMPLE IX. *A watercourse is 7 feet wide at the bottom, the length of each sloping side is 6.8 feet, the width at the surface is 18 feet, the depth 4 feet, and the inclination of the surface 4 inches in a mile; what is the quantity flowing down per minute?*

* Transactions of the Institution of Civil Engineers, pp. 201, 210, vol. ii.

$$\text{Here } \frac{(18 + 7) \times \frac{4}{2}}{7 + 2 \times 6.8} = \frac{50}{20.6} = 2.4272 \text{ feet} = 29.126 \text{ inches}$$

$=r$, is the hydraulic mean depth; and as the fall is 4 inches per mile, we find at the 11th page of TABLE VIII., the velocity $v = 12.03 - .16 = 11.87$ inches per second; the discharge in cubic feet per minute is, therefore,

$$50 \times \frac{11.87}{12} \times 60 = 2967.5.$$

$$\begin{aligned} \text{If } 94.17 \sqrt{rs} = v, \text{ we have } v &= 94.17 \sqrt{2.427 + \frac{1}{15840}} \\ &= 94.17 \times \sqrt{\frac{1}{6526}} = \frac{94.17}{80.7} = 1.17 \text{ feet} = 14.04 \text{ inches.} \end{aligned}$$

Watt, in a canal of the fall and dimensions here given, found the mean velocity about $13\frac{1}{2}$ inches per second. This corresponds to a fall of 5 inches in the mile, according to the formula. Du Buat's formula is less by $12\frac{1}{2}$ per cent. or $\frac{1}{8}$ th; the common formula too much by 5 per cent.

In one of the original experiments with which the formula was tested on the canal of Jard, the measurements accorded very nearly with those in this example, viz. $\frac{1}{s} = 15360$, and $r = 29.1$ French inches; the observed velocity at the surface was 15.74, and the calculated mean velocity, from the formula, 11.61 French inches.* TABLE VII. will give 12.29 inches for the mean velocity, corre-

* These measures reduced to inches, give $r = 81.014$, $v = 12.374$; and the surface velocity 16.775 inches; reduced for mean velocity 13.101 inches.

sponding to a superficial velocity of 15.74 inches. This shows that the formula also gives too small a value for v in this case, by about $\frac{1}{17}$ th of the result, it being about $\frac{1}{8.8}$ part in the other. The probable error in the formula *applied to straight clear rivers* of about 2 feet 6 inches hydraulic mean depth is nearly $\frac{1}{17}$ th or 8 per cent. of the tabulated velocity, and this must be added for the more correct result; the watercourse being supposed nearly straight and free from aquatic plants.

Notwithstanding the differences above remarked on, we are of opinion that the results of this formula, which we have calculated and tabulated, may be more safely relied on as applied to *general practical purposes* than most of those others which we shall proceed to lay before our readers. Rivers or watercourses are seldom straight or clear from weeds, and even if the sections, during any improvements, be made uniform, they will seldom continue so, as "*the regimen*," or adaptation of the velocity to the tenacity of the banks, must vary with the soil and bonds of the channel, and can seldom continue permanent for any length of time unless protected. From these causes a loss of velocity takes place, difficult, if not impossible, to estimate accurately, but which may be taken at from 10 to 15 per cent. of that in the clear unobstructed direct channel; but be this as it may, *it is safer to calculate the drainage or mechanical results obtainable from a given fall and river channel, from formulæ which give lesser, than*

from those which give larger velocities. This is a principle engineers cannot too much observe.

We have before remarked, that for both pipes and rivers the coefficient of resistance increases as the velocity decreases. This is as much as to say, in the simple formula for the velocity, $v = m \sqrt{rs}$, that m must increase with v , and as some function of it. This is the case in TABLE VIII., throughout which the velocities increase faster than \sqrt{r} , the \sqrt{s} , or the \sqrt{rs} . In all formulæ with which we are acquainted but Du Buât's and Young's, *the velocity found is constant when \sqrt{rs} or $r \times s$ is constant*. In Du Buât's formula for $r \times s$ constant, v obtains maximum values between $r = \frac{3}{4}$ inch and $r = 1$ inch; the differences of the velocities for different values of r above 1 inch, $r \times s$ being constant, are not much. We may always find the maximum value, or nearly so, by assuming $r = \frac{3}{4}$ inch, and finding the corresponding inclination from the formula $\frac{4rs}{3}$, which is equal to it. For example, if $r = 12$ inches, and $s = \frac{1}{10560}$, the velocity is found equal 9.52 inches; but when rs is constant, the inclination s corresponding to $r = \frac{3}{4}$ inch is $\frac{4 \times 12''}{3 \times 10560} = \frac{1}{660}$, from which we find from the table $v = 10.25$ inches for the maximum velocity, making a difference of fully 7 per cent.

When $r = .01$ of an inch, or a pipe is $\frac{1}{100}$ th part of an inch in diameter, Du Buât's formula fails, but gives correct results for pipes $\frac{1}{4}$ th of an inch in diameter, and two of the experiments from which it

was derived were made with pipes 12 inches long and only $\frac{1}{16}$ th part of an inch in diameter.

COULOMB having shown that the resistance opposed to a disc revolving in water increases as the function $av + bv^2$ of the velocity v , we may assume that the height due to the resistance of friction in pipes and rivers is also of this form; and that

$$(83.) \quad h_f = (av + bv^2) \frac{l}{g},$$

and consequently,

$$(84.) \quad rs = av + bv^2, \text{ and } v = \left\{ \frac{rs}{b} + \frac{a^2}{4b^2} \right\}^{\frac{1}{2}} - \frac{a}{2b}.$$

GIARD first gave values to the coefficients a and b . He assumed them equal, and each equal to $\cdot 0003104$ for measures in metres, and thence the velocity in canals,

$$(85.) \quad v = (3221 \cdot 016 rs + \cdot 25)^{\frac{1}{2}} - \cdot 5; *$$

which reduced for measures in English feet becomes

$$(86.) \quad \begin{cases} v = (10567 \cdot 8 rs + 2 \cdot 67)^{\frac{1}{2}} - 1 \cdot 64, \text{ or} \\ v = 103 \sqrt{rs} - 1 \cdot 64, \text{ nearly.} \end{cases}$$

The value of $a = b = \cdot 0003104$ was obtained by means of twelve experiments by Du Buat and Chezy. Of course the value is four times this in the original, as we use the mean radius in all the formulæ instead of the diameter. This formula is only suited for very small velocities in canals, between locks, containing aquatic plants; it is inapplicable to rivers and channels in which the velocity exceeds an inch per second.

Prony found from thirty experiments on canals,

* See Browster's Encyclopedia, Article Hydrodynamics, p. 259.

that $a = .000044450$ and $b = 000309314$,* for measures in metres, from which we find

$$(87.) \quad v = (3232.96 \, r s + .00516)^{\frac{1}{2}} - .0719 ;$$

this reduced for measures in English feet is,

$$(88.) \quad \begin{cases} v = (10607.02 \, r s + .0556)^{\frac{1}{2}} - .236 ; \dagger \text{ or} \\ v = 103 \sqrt{r s} - .24 \text{ nearly :} \end{cases}$$

the velocities did not exceed 3 feet per second in the experiments from which this was derived.

For pipes, Prony found,‡ from fifty-one experiments made by Du Buât, Bossut, and Couplet, with pipes from 1 to 5 inches diameter, from 30 to 7,000 feet in length, and one pipe 19 inches diameter and nearly 4,000 feet long, that $a = .00001733$, and $b = .0003483$, from which values

$$(89.) \quad v = (2871.09 \, r s + .0006192)^{\frac{1}{2}} - .0249,$$

for measures in metres, and for measures in English feet,

$$(90.) \quad \begin{cases} v = (9419.75 \, r s + .00665)^{\frac{1}{2}} - .0816 ; \text{ or} \\ v = 97 \sqrt{r s} - .08 \text{ nearly.} \end{cases}$$

Prony also gives the following formula applicable to pipes and rivers. It is derived from fifty-one selected experiments with pipes, and thirty-one with open channels :

$$(91.) \quad v = (3041.47 \, r s + .0022065)^{\frac{1}{2}} - .0469734, \S$$

* Recherches Physico-Mathématiques sur la Théorie des Eaux Courantes.

† For *canals* containing aquatic plants, reeds, &c., we must substitute $\frac{r}{1.7}$ for r . See note, p. 186.

‡ Recherches Physico-Mathématiques sur la Théorie du Mouvement des Eaux Courantes, 1804.

§ Recherches Physico-Mathématiques sur la Théorie des Eaux courantes. A reduction of this formula into English feet is given page 6, Article Hydrodynamics, Encyclopedia Britannica; at

TABLE of the fifty-one Experiments referred to in Equation (80), the value of g in the sixth being taken at 9.8088 metres.

It will be perceived that Prony did not take into calculation, in framing his formula, the head due to the velocity in the pipe and to the orifice of entry.

Number of selected experiments.	Names of Experimenters.	Heads measured to the lower orifice in metres.	Diameters of pipes in metres.	Length of the pipes in metres.	Values of $\frac{g r^5}{v}$ in metres.	Experimental values of the velocity v in metres.	Calculated velocity from formula (89) in metres.
1	Du Buat	.0041	.0271	19.95	.000314	.0430	.0427
2	Couplet	.1511	.1353	2280.37	.000404	.0644	.0591
3	Couplet	.3008	.1353	2280.37	.000523	.0854	.0921
4	Du Buat	.0135	.02707	19.95	.000450	.0980	.0920
5	Couplet	.4534	.1333	2280.37	.000590	.1117	.1263
6	Couplet	.5105	.1333	2280.37	.000638	.1301	.1330
7	Couplet	.6407	.1333	2280.37	.000670	.1411	.1433
8	Couplet	.6767	.1333	2280.37	.000683	.1441	.1467
9	Du Buat	.0189	.0271	3.75	.001426	.2352	.2895
10	Du Buat	.1137	.0271	3.75	.001188	.2826	.3088
11	Du Buat	.1137	.0271	3.75	.001309	.2888	.3088
12	Bossut	.1083	.0271	16.24	.001337	.3308	.3350
13	Bossut	.3248	.0361	58.47	.001446	.3400	.3553
14	Du Buat	.1605	.0271	19.95	.001482	.3604	.3713
15	Bossut	.3248	.0361	48.75	.001549	.3807	.3915
16	Du Buat	.2106	.0271	19.95	.001713	.4091	.4287
17	Bossut	.3248	.0361	38.98	.001687	.4366	.4402
18	Du Buat	.2425	.0271	19.95	.001830	.4408	.4618
19	Bossut	.3248	.0544	58.47	.001672	.4433	.4416
20	Du Buat	.2425	.0271	19.95	.001793	.4500	.4618
21	Bossut	.3248	.0544	48.73	.001795	.4955	.4860
22	Bossut	.6407	.0361	58.47	.001922	.5115	.5122
23	Bossut	.3248	.0361	29.23	.001918	.5128	.5122
24	Du Buat	.3335	.0271	19.95	.002050	.5411	.5450
25	Bossut	.3248	.0544	38.98	.001981	.5605	.5458
26	Du Buat	.3700	.0271	19.95	.002174	.5676	.5700
27	Bossut	.6407	.0361	48.73	.002073	.5693	.5684
28	Du Buat	.3952	.0271	19.95	.002223	.5916	.5961
29	Bossut	.3248	.0271	20.24	.002201	.6032	.5990
30	Bossut	.3248	.0361	19.40	.002333	.6323	.6327
31	Bossut	.3248	.0544	29.23	.002300	.6444	.6344
32	Bossut	.6407	.0361	38.98	.002267	.6498	.6323
33	Bossut	.6407	.0544	58.47	.002214	.6695	.6344
34	Bossut	.6407	.0544	48.73	.002302	.7436	.6972
35	Bossut	.6407	.0361	29.23	.002588	.74	.7343
36	Du Buat	.6416	.0271	19.95	.002750	.7761	.7660
37	Bossut	.3248	.0544	19.40	.002812	.7908	.7823
38	Du Buat	.1624	.0271	3.75	.003620	.7943	.8030
39	Bossut	.6407	.0544	38.98	.002656	.8363	.7819
40	Bossut	.3248	.0361	9.74	.003287	.8976	.9048
41	Bossut	.65	.0361	19.40	.003161	.9332	.9048
42	Bossut	.65	.0544	29.23	.003062	.9681	.9071
43	Couplet	3.9274	.4873	1169.42	.003785	1.0600	1.0592
44	Bossut	.3248	.0544	9.74	.004073	1.0915	1.1164
45	Bossut	.6407	.0544	19.40	.003821	1.1640	1.1164
46	Bossut	.6407	.0361	9.74	.004491	1.3138	1.2896
47	Du Buat	.4873	.0271	3.17	.006470	1.5784	1.7043
48	Du Buat	.5671	.0271	3.75	.006307	1.5919	1.6898
49	Bossut	.6407	.0544	9.74	.005578	1.5945	1.5890
50	Du Buat	.7219	.0271	3.17	.007838	1.9301	2.0798
51	Du Buat	.9745	.0271	3.17	.008882	2.2094	2.4205

for measures in metres, which, reduced for measures in English feet, is

$$(92.) \quad \begin{cases} v = (9978.76 \, r \, s + .02375)^{\frac{1}{2}} - .15412; \text{ or} \\ v = 100 \sqrt{r \, s} - .15 \text{ nearly.} \end{cases}$$

EYTTELWEIN, following the method of investigation pursued previously by Prony, found from a large number of experiments, $\alpha = .0000242651$, and $b = .000365543$ in rivers, for measures in metres; and, therefore,

$$(93.) \quad v = (2735.66 \, r \, s + .001102)^{\frac{1}{2}} - .0332.*$$

This reduced for measures in English feet, is

$$(94.) \quad \begin{cases} v = (8975.43 \, r \, s + .0118858)^{\frac{1}{2}} - .1089; \text{ or} \\ v = 94.5 \sqrt{r \, s} - .11 \text{ nearly} = 1.3 \sqrt{f \, r} - .11 \\ \qquad \qquad \qquad = \sqrt{1.7 \, f \, r} - .11 \end{cases}$$

when f is the fall in feet per mile. He also shows,† that $\frac{1}{11}$ ths of a mean proportional between the fall in two English miles and the hydraulic mean depth, gives the mean velocity very nearly. This rule for measures in inches is equivalent to

$$(95.) \quad v = 324 \sqrt{r \, s};$$

and for measures in feet

$$(96.) \quad v = 93.4 \sqrt{r \, s}.$$

For the velocity of water in pipes, he found,‡ from the fifty-one experiments of Du Buat, Bossut, and

page 184, Third Report, British Association, by Rennie; and at pages 427 and 533, Article Hydrodynamics, Brewster's Encyclopedia. This reduction, $v = -0.1541 + (.02375 + 32800.0 \, r \, s)^{\frac{1}{2}}$ is entirely incorrect; and, being the same in each of those works, appears to have been copied one from the other.

* Mémoires de l'Académie de Berlin, 1814 et 1815. See notation (110).

Handbuch der Mechanik und der Hydraulik, Berlin, 1801.

Mémoires de l'Académie des Sciences de Berlin, 1814 et 1815.

Couplet, that $a = .0000223$, and $b = .0002803$, from which we get for measures in metres,

$$(97.) \quad v = (3567.29 \, r \, s + .00157)^{\frac{1}{2}} - .0397 ;$$

which reduced for measures in English feet becomes

$$(98.) \quad \begin{cases} v = (11703.95 \, r \, s + .01698)^{\frac{1}{2}} - .1303 ; \text{ or} \\ v = 1.08 \sqrt{r \, s} - .13 \text{ nearly.} \end{cases}$$

Another formula given by Eytelwein for pipes, which includes the head due to the velocity for the orifice of entry, is

$$(99.) \quad v = 50 \left(l + \frac{d \, \pi}{50 \, d} \right)^{\frac{1}{2}} ;$$

in which π is the head, l the length, and d the diameter of the pipe, all expressed in English feet. This is a particular value of equation (74) suited to velocities of about $2\frac{1}{2}$ feet per second. It must be here mentioned, that much of the valuable information presented by Eytelwein is but a modification of what Du Buât had previously given, to whom only for much that is attributed to the former we are primarily indebted.

In the foregoing as well as in the following equations for the velocity, we have, unless otherwise stated, maintained one class of standards. It is evident if we change these standards in part, or in whole, that apparently different forms of the equations will arise ; thus—if for s , the hydraulic incli-

nation, we substitute $\frac{m}{5280}$, we shall have the fall

m in feet per mile, in place of the inclination s ; so that equation (94), for instance, would become

$$v = (1.7 \, m \, r + .012)^{\frac{1}{2}} - .11 = (1.7 \, m \, r)^{\frac{1}{2}} - .11 \text{ nearly,}$$

in which v is the velocity in feet per second, m the

fall in feet per mile, and r the "hydraulic mean depth" in feet. In like manner equation (98) would become

$$v = (2.2 m r + .02)^{\frac{1}{2}} - .13 = (2.2 m r)^{\frac{1}{2}} - .13.$$

The first of these reductions, viz. :—

$$v = (1.7 m r + .0119)^{\frac{1}{2}} - .109,$$

is given in a book of tables calculated for river channels for the Commissioners of Public Works, Ireland, the original equation being Eytelwein's, and not D'Aubuisson's, who merely copied it, and is suited for velocities averaging about 1.3 feet per second.

Mr. Hawksley gives for pipes the formula

$$v_y = .77 \left\{ \frac{d_1 \pi_1}{l_y + 1.5 d_1} \right\}^{\frac{1}{2}},$$

in which l is the length in yards, π the head in inches, d the diameter in inches, and v the velocity in yards per second. For uniform feet measures, for, v , d , and π , this becomes

$$v = 48.045 \left\{ \frac{d \pi}{l + 54 d} \right\}^{\frac{1}{2}},$$

which is only Eytelwein's equation (99) slightly modified. Eytelwein's equation expressed in the measures used by Mr. Hawksley would be very nearly

$$v_y = .8 \left\{ \frac{d_1 \pi_1}{l_y + 1\frac{1}{2} d_1} \right\}^{\frac{1}{2}},$$

which is far the simplest of the two ; both, however, are but particular cases of the general equation 74, and only suited for velocities of about $2\frac{1}{2}$ feet per second.

DR. THOMAS YOUNG* also derives his formula from

* Philosophical Transactions for 1808.

the supposition, that the head due to the resistance of friction assumes the form of equation (83); calling the diameter of a pipe d , he takes

$$h_f = (av + bv^3) \frac{l}{d},$$

and the whole height $h = h_f + \frac{v^3}{586}$, expressed in inches. He found from some experiments of his own, those collected by Du Buat, and some of Gerstner's, that

$$(100.) \quad a = .0000002 \left\{ \frac{900 d^3}{d^3 + 1136} + \frac{1}{d^4} \left(1085 + \frac{13.21}{d} + \frac{1.0563}{d^2} \right) \right\};$$

and

$$(101.) \quad b = .0000001 \left\{ 413 + \frac{75}{d} - \frac{1440}{d + 12.8} - \frac{180}{d + .355} \right\};$$

then as $\frac{1}{586} = .00171$, we get

$$(102.) \quad v = \left\{ \frac{h d}{b l + .00171 d} + \left(\frac{a l}{2 b l + .00341 d} \right)^{\frac{3}{2}} - \frac{a l}{2 b l + .00341 d} \right\}^{\frac{2}{3}}.$$

When the length l of the pipe is very great compared with the head due to the orifice of entrance and velocity, $.00171 v^3$, we have

$$(103.) \quad v = \left\{ \frac{h d}{b l} + \frac{a^3}{4 b^3} \right\}^{\frac{2}{3}} - \frac{a}{2 b};$$

or by substituting for $\frac{h}{l}$ its value s , equal the sine of the inclination,

$$(104.) \quad v = \left\{ \frac{s d}{b} + \frac{a^3}{4 b^3} \right\}^{\frac{2}{3}} - \frac{a}{2 b}.$$

The values of a and b are for measures in inches. For most rivers, in which d must be taken equal $4r$, he finds for French inch measures, $v = \sqrt{20000 ds}$; this reduced for English inches is

$$(105.) \quad v = 292 \sqrt{rs};$$

which again reduced for feet measures, becomes

$$(106.) \quad v = 84.3 \sqrt{rs}.$$

These latter values, for rivers, are even smaller than those found from Du Buat's formula; less than the observed velocities, and less than those found from any other formula, with the exception of Girard's. The values of the coefficients a and b vary in this formula with the value of $d = 4r$; they are expressed generally in equations (101) and (102), from which we have calculated the following table for different values of d and r .

An examination of this table will show that a obtains a minimum value when d is between 10 and 11 inches; and b when the diameter is between $\frac{1}{2}$ and $\frac{3}{4}$ of an inch. Now, it appears from equation (102), that v increases with $\sqrt{\frac{Hd}{bl}}$ nearly, or, which is the same thing, as b decreases, there must, *cæteris paribus*, be a maximum value of v for a given value of $\frac{Hd}{l}$, or rs , when d is between $\frac{1}{2}$ and $\frac{3}{4}$

inch; but as $\frac{a}{2b}$ has a minimum value when d is nearly 12 inches, the maximum value of v referred to will be found between values of d from $\frac{3}{4}$ inch to 12 inches; in fact, when $d = 10$ inches nearly. We have already pointed out a similar peculiarity in Du

Buât's general theorem, at page 195. *It will not be necessary to take out the values of $\frac{a}{2b}$ and $\frac{a^3}{4b^3}$ to more than one place of decimals.*

The values of $\frac{a}{2b}$ are also given in the following table, and may be used in equation (104) for finding the discharge from long pipes. It is, however, necessary to remark, that this equation is sometimes misapplied in finding the velocity from short pipes, and those of moderate lengths. It is necessary to use equation (102), which takes into consideration the head due to the velocity and orifice of entry for such pipes.

For a pipe 11 inches in diameter, the expression for the velocity, equation (104), becomes for inch measures,

$$v = \left\{ \frac{sd}{.000034} + 1.49 \right\}^{\frac{1}{2}} - 1.22 :$$

and for feet measures, also substituting $4r$ for d ,

$$(106A.) \quad v = \left\{ \frac{sr}{.000102} \right\}^{\frac{1}{2}} - .1 = 100(rs)^{\frac{1}{2}} - .1$$

very nearly. For a pipe .7 inch in diameter we should find in a like manner for feet measures,

$$(106B.) \quad v = 118(rs)^{\frac{1}{2}} - .5,$$

which is only suitable for pipes with very high velocities.

SIR JOHN LESLIE states,* that the mean velocity of a river in miles per hour, is $\frac{1}{14}$ ths of the mean proportional between the hydraulic mean depth and

* Natural Philosophy, p. 428.

TABLE OF THE VALUES OF a , b , $\frac{a}{2b}$ AND $\frac{a^2}{4b^2}$ IN EQUATION (104).

d in inches.	r in inches.	a	b	$\frac{a}{2b}$	$\frac{a^2}{4b^2}$	d in inches.	r in inches.	a	b	$\frac{a}{2b}$	$\frac{a^2}{4b^2}$
1	0.25	0.00537	0.00066	6.341	40.207	6	1.5	0.00094	0.00032	1.468	2.155
2	0.5	0.00536	0.00035	7.657	58.629	7	1.75	0.00090	0.00033	1.363	1.857
3	0.75	0.00406	0.00028	7.250	52.562	8	2	0.00687	0.00033	1.316	1.737
4	1.00	0.00350	0.00025	7.120	50.694	9	2.25	0.00084	0.00034	1.235	1.526
5	1.25	0.00316	0.00025	6.320	39.942	10	2.50	0.00083	0.00034	1.220	1.489
6	1.50	0.00287	0.00024	5.973	35.750	11	2.75	0.00083	0.00034	1.220	1.489
7	1.75	0.00264	0.00024	5.500	30.250	12	3.00	0.00084	0.00035	1.200	1.440
8	2.00	0.00247	0.00025	4.940	24.403	15	3.75	0.00086	0.00035	1.223	1.509
9	2.25	0.00232	0.00025	4.640	21.529	20	5	0.00096	0.00036	1.233	1.777
1	2.50	0.00220	0.00025	4.400	19.360	40	10	0.00140	0.00038	1.242	3.392
1.5	3.75	0.00179	0.00027	3.315	10.983	50	12.5	0.00154	0.00039	1.274	3.893
2	5.00	0.00155	0.00028	2.757	7.651	60	15	0.00165	0.00039	2.115	4.474
2.5	6.25	0.00139	0.00028	2.482	6.161	80	20	0.00177	0.00040	2.212	4.895
3.0	7.50	0.00127	0.00029	2.189	4.794	100	25	0.00184	0.00040	2.300	5.290
3.5	8.75	0.00119	0.00030	1.983	3.933	300	75	0.00190	0.00041	2.317	5.369
4	1.000	0.00111	0.00031	1.790	3.205	500	125	0.00189	0.00041	2.305	5.312
5	1.250	0.00101	0.00031	1.629	2.653	Infinite	Infinite	0.00180	0.00041	2.195	4.818

the fall in two miles in feet. This rule is equivalent, for measures in feet, to

$$(107.) \quad v = 100 \sqrt{rs};$$

and is applicable to rivers with velocities of about 2½ feet per second.

D'AUBUISSON, from an examination of the results obtained by Prony and Eytelwein, assumes* for measures in metres that $a = .0000189$, and $b = .0003425$ for pipes, substituting these in equation (84) and resolving the quadratic

$$(108.) \quad v = (2919.71 rs + .00074)^{\frac{1}{2}} - .027;$$

which reduced for measures in English feet becomes

$$(109.) \quad \begin{cases} v = (9579 rs + .00813)^{\frac{1}{2}} - .0902, \text{ or} \\ v = 98 \sqrt{rs} - .1 \text{ nearly.} \end{cases}$$

For rivers he assumes with Eytelwein,† $a = .000024123$ and $b = .0003655$, for measures in metres, and hence

$$(110.) \quad v = (2735.98 rs + .0011)^{\frac{1}{2}} - .033;$$

which for measures in English feet is

$$(111.) \quad \begin{cases} v = (8976.5 rs + .012)^{\frac{1}{2}} - .109, \text{ or} \\ v = 94.5 \sqrt{rs} - .11 \text{ nearly.} \end{cases}$$

When the velocity exceeds two feet per second, he assumes, from the experiments of Couplet, $a = 0$, and $b = .00035875$; these values give

$$(112.) \quad v = \sqrt{2787.46 rs},$$

for measures in metres, and

$$(113.) \quad v = 95.6 \sqrt{rs} = \sqrt{9145 rs}$$

for measures in English feet. Equations (110) and (111) are the same as (93) and (94), found from Eytel-

* *Traité d'Hydraulique*, p. 224.

† *Traité d'Hydraulique*, p. 133. See Equation (93).

wein's values of a and b , and it may be remarked that D'Aubuisson's equations for the velocity generally, are simply those of Prony and Eytelwein.

The values which we have found to agree best with experiments on clear straight rivers are $a = .0000035$, and $b = .0001150$ for measures in English feet, from which we find

$$(114.) \quad \begin{cases} v = (8695.6 \, r \, s + .00023)^{\frac{1}{4}} - .0152, \text{ or} \\ v = 93 \sqrt{r \, s} - .02, \end{cases}$$

which for an average velocity of $1\frac{1}{2}$ foot per second will give $v = 92.3 \sqrt{r \, s}$ nearly, and for large velocities $v = 93.3 \sqrt{r \, s}$; for smaller velocities than $1\frac{1}{2}$ foot per second, the coefficients of $\sqrt{r \, s}$ decrease pretty rapidly. This formula will be found to agree more accurately with observation and experiment than any other we know of this form.

WEISBACH is perhaps the only writer who has modified the form of the equation $r \, s = a \, v + b \, v^2$. In Dr. Young's formula, a and b vary with r , but Weisbach assumes that $h_r = \left(a + \frac{b}{v^4}\right) \frac{l}{d} \times \frac{v^2}{2g}$, and finds from the fifty-one experiments of Couplet, Bossut, and Du Buât, before referred to, one experiment by Guemard, and eleven by himself, all with pipes varying from an inch to five and a half inches in diameter, and with velocities varying from $1\frac{1}{2}$ inch to 15 feet per second, that $a = .01439$, and $b = .0094711$ for measures in metres; hence we have for the metrical standard

$$(115.) \quad h_r = \left(.01439 + \frac{.0094711}{v^4}\right) \frac{l}{d} \times \frac{v^2}{2g}.$$

This reduced for the mean radius r is

$$(116.) \quad h_f = \left(.003597 + \frac{.0023678}{v^4} \right) \frac{l}{r} \times \frac{v^3}{2g};$$

from which we find for measures in English feet

$$(117.) \quad h_f = \left(.003597 + \frac{.0042887}{v^4} \right) \frac{l}{r} \times \frac{v^3}{2g},$$

and thence

$$(118.) \quad r s = \left(.003597 + \frac{.0042887}{v^4} \right) \frac{v^3}{2g};$$

and by substituting for $2g$, its value 64.403 ,

$$(119.) \quad r s = \left(.00005585 + \frac{.00006659}{v^4} \right) v^3.$$

In equation (117), $\left(.003597 + \frac{.0042887}{v^4} \right) = c_f$ is the coefficient of the head due to friction. The equation does not admit of a direct solution, but the coefficient should be first determined for different values of the velocity v and tabulated, after which the true value of v can be determined by finding an approximate value, and thence taking out the corresponding coefficient from the table, which does not vary to any considerable extent for small changes of velocity. In the following small table we have calculated the coefficients of friction, and also those of v^3 , in equation (119), for different values of the velocity v .

TABLE OF THE COEFFICIENTS OF FRICTION IN PIPES.

Velocity in feet.	c_f	$\frac{c_f}{64.4}$	$\frac{64.4}{c_f}$	$\sqrt{\frac{64.4}{c_f}}$	Velocity in feet.	c_f	$\frac{c_f}{64.4}$	$\frac{64.4}{c_f}$	$\sqrt{\frac{64.4}{c_f}}$
1	.017159	.0002664	3078.07	55.5	2.4	.006365	.0000988	10121.5	100.5
2	.018186	.0002047	4885.2	69.0	2.5	.006300	.0000970	10214.5	101.0
3	.011427	.0001774	5636.0	75.08	2.6	.006257	.0000972	10288.1	101.4
4	.010378	.0001611	6270.3	78.8	2.7	.006207	.0000964	10373.4	101.8
5	.009662	.0001500	6666.6	81.6	2.8	.006160	.0000956	10460.2	102.2
6	.009133	.0001418	7052.2	84.0	2.9	.006115	.0000949	10537.4	102.6
7	.008723	.0001354	7385.5	85.9	3	.006073	.0000943	10604.4	102.9
8	.008391	.0001308	7674.6	87.6	3.5	.005890	.0000914	10940.0	104.6
9	.008117	.0001260	7936.5	89.1	4	.005741	.0000891	11223.3	105.9
10	.007886	.0001224	8169.9	90.4	5	.005514	.0000856	11682.2	108.0
11	.007686	.0001193	8382.2	91.5	6	.005348	.0000830	12048.2	109.7
12	.007512	.0001166	8576.3	92.6	7	.005218	.0000810	12345.6	111.1
12.5	.007433	.0001154	8665.5	93.1	8	.005113	.0000794	12632.2	112.4
13	.007358	.0001142	8756.5	93.5	9	.005020	.0000780	12820.5	113.3
14	.007221	.0001121	8920.6	94.4	10	.004953	.0000769	13003.9	114.0
15	.007098	.0001102	9074.4	95.2	15	.004704	.0000730	13698.6	117.0
16	.006987	.0001085	9216.5	96.0	16	.004660	.0000725	13793.1	117.4
17	.006880	.0001069	9354.5	96.7	20	.004556	.0000707	14141.2	118.9
17.5	.006839	.0001062	9416.2	97.03	25	.004455	.0000691	14471.7	120.3
18	.006794	.0001054	9487.6	97.4	30	.004380	.0000680	14705.9	121.2
19	.006715	.0001042	9596.9	97.9	35	.004322	.0000671	14903.1	122.0
20	.006629	.0001029	9718.2	98.5	40	.004275	.0000664	15090.2	122.7
21	.006556	.0001018	9823.2	99.1	45	.004236	.0000658	15197.5	123.3
22	.006488	.0001007	9930.5	99.6	50	.004203	.0000653	15313.3	123.7
23	.006424	.0000997	10003	100	100	.004208	.0000625	16000.0	126.4

If the value of $\frac{64.4}{c_f}$ here found, be substituted in the equation $v = \sqrt{\frac{64.4}{c_f}} r s$, we shall have the value of v . According to this table the coefficient of friction for a velocity of six inches is more than twice that for a velocity of twenty feet, and the velocity is in the proportion of 81.6 to 118.9, or of 81.6 ($r s$)¹

to $118.9 (rs)^{\frac{1}{2}}$. On comparing these coefficients and those for pipes in the preceding formulæ, with those for rivers of the same hydraulic depth, we perceive that the loss from friction is greatest in the latter, as might have been anticipated; but this evidently arises from lesser velocities.

It has been already remarked that the coefficient of friction decreases as the velocity increases. The only general formula which properly meets this defect in the common formulæ is Weisbach's, but it does not give the velocity v directly, as this quantity is involved in both sides of his equation. As for several hydraulic works it is necessary to convey water through pipes to work machines under high heads, and for which the common formula would give results considerably under the true ones, it appeared to me desirable to obtain some simple expression for the velocity which might be easily remembered and applied, which would be equally correct with other formulæ for medium velocities of from one to two and a half feet, and which at the same time would give practically correct results for lesser and greater velocities within the limits of experiment. By reducing the velocity found from experiment to the form $v = m \sqrt{rs}$ for every case, and afterwards applying a correction of the form $n \sqrt{rs}$ to meet the increasing value of m as v increased, I discovered that the expression

$$(119A) \quad v = 140 (rs)^{\frac{1}{2}} - 11 (rs)^{\frac{1}{2}}$$

gave results not differing more from experiments than these frequently do from each other. The following table exhibits the velocities compared with those

obtained from the experiments made by Du Buât, Couplet, Watt, Mr. Provis, and Mr. Leslie, in the Minutes of the Institution of Civil Engineers for February 1855. The last experiment was furnished to me by Mr. Hodson of Lincoln. Numbers 34 and 35 were made by myself, and give the mean results of several experiments made with great care; the coefficient of the orifice of entry was found to be $\cdot 860$.* The measures have been all reduced to English feet. The results found by the same experimenters, at the same time, with the same apparatus, sometimes differ by three or four per cent., as may be seen by referring to Mr. Provis' experiments, (Transactions of the Institution of Civil Engineers, vol. II, p. 203,) and the difference in the experiments shown in the table are apparent. The difference in the velocities found from the experiments, do not exceed those inseparable from practical investigations, and they differ as much in themselves as from the formula, which for cylindrical pipes of diameter d may be thus expressed,

$$(119B.) \quad \begin{cases} v = 70 (d s)^{\frac{1}{2}} - 6.93 (d s)^{\frac{1}{2}}, \text{ or} \\ v = 70 (d s)^{\frac{1}{2}} - 7 (d s)^{\frac{1}{2}} \text{ nearly.} \end{cases}$$

The expression fails when $70 (d s)^{\frac{1}{2}}$ is equal to or less than $6.93 (d s)^{\frac{1}{2}}$, but as this only happens when $r s = \left(\frac{11}{140}\right)^2 = .00000235$, and for velocities below one inch per second, its practical value is not thereby affected. The expression of Du Buât fails with a

* The coefficient for the orifice of entry was found by cutting off the pipe at two diameters from the cistern at the conclusion of the experiments, and finding the time of emptying. *Vide* p. 177.

TABLE showing the Experimental Results of observed Velocities in Water Channels, with the Author's general formula for Pipes and Rivers, viz.

$$v = 140 (r s)^{\frac{1}{2}} - 11 (r s)^{\frac{1}{4}}.$$

No.	Heads in feet (H).	Lengths in feet (L).	Values of r.	Values of s.	Values of r s.	Velocities from experiment.	Velocities from the formula.	Velocity expressed in the form $v = \frac{140}{\sqrt{r s}} - 11$	Experimenters' Names.
1	0.8333	1080	0.052083	0.000076	0.00000896	1.00	1.05	52.6 $\sqrt{r s}$	Mr. Leslie
2	0.1332	66.37	0.022204	0.000196	0.00000434	1.10	1.13	54.0 "	Couplet
3	1.4583	1080	0.052083	0.00133	0.0000693	1.18	1.57	60.0 "	Mr. Leslie
4	4.0580	7482	1.11000	0.000066	0.0000131	1.78	1.67	61.5 "	Couplet
5	2.0833	1080	0.052083	0.00190	0.0000989	2.17	2.06	65.0 "	Mr. Leslie
6	4.5833	1080	0.052083	0.00417	0.0002170	3.61	3.16	74.1 "	"
7	1.48768	7482	1.11000	0.00198	0.0002220	3.66	3.48	74.1 "	Couplet
8	1.44800	1080	0.052083	0.01321	0.000688	7.15	7.11	85.7 "	Mr. Leslie
9	2.78125	1080	0.052083	0.02538	0.001322	1.085	1.050	91.3 "	"
10	"	"	2.427200	0.00063	0.001532	1.166	1.143	92.1 "	Watt
11	5.0000	100	0.01250	0.01741	0.0001482	1.023	1.122	92.2 "	Mr. Provis
12	2.78125	1080	0.052083	0.01348	0.002265	1.461	1.438	95.5 "	Mr. Leslie
13	4.76042	1080	0.052083	0.06110	0.003340	1.725	1.796	98.3 "	"
14	1.00580	127.0	0.01630	0.007748	0.0003458	1.339	1.810	98.9 "	Bossut
15	5.0000	40	0.01250	0.010810	0.0003378	1.711	1.816	98.7 "	Mr. Provis
16	1.00580	95.02	0.01630	0.010050	0.0004485	2.111	2.124	100.3 "	Bossut
17	1.5	100	0.01250	0.014156	0.0004422	2.005	2.103	100.5 "	Mr. Provis
18	0.0890	1080	0.052083	0.009174	0.0004779	2.095	2.185	100.6 "	Mr. Leslie
19	8.575	40	0.01250	0.018042	0.0005638	2.380	2.414	101.7 "	Mr. Provis
20	2.1310	191.0	0.01630	0.010548	0.0004708	2.463	2.183	100.6 "	Bossut
21	2.1310	159.0	0.01630	0.012524	0.0005589	2.440	2.404	101.7 "	"
22	"	"	0.052083	0.01286	0.0007440	2.800	2.823	103.5 "	Mr. Leslie
23	2.1310	127.0	0.01630	0.015350	0.0008651	2.744	2.696	103.0 "	Bossut
24	"	"	0.01630	0.027021	0.0012405	3.819	3.760	106.6 "	"
25	"	"	0.052083	0.025000	0.0013021	3.783	3.852	106.7 "	Mr. Leslie
26	3.27410	40	0.01250	0.018040	0.002093	5.054	5.000	109.3 "	Mr. Provis
27	2.3684	10.30155	0.022204	0.133680	0.0029679	6.322	6.048	111.0 "	Du Buât
28	3.27410	20	0.01250	0.111200	0.001750	6.723	6.572	111.5 "	Mr. Provis
29	3.4525	20	0.01250	0.113900	0.0015594	7.080	6.668	111.9 "	"
30	7.135	02.8822	0.020605	0.008861	0.0029268	6.157	5.999	110.9 "	Couplet
31	14.270	125.7644	0.029605	0.106151	0.0031426	6.151	6.239	111.3 "	"
32	21.405	188.0460	0.029605	0.108579	0.00321455	6.145	6.316	111.4 "	"
33	3.1974	10.30155	0.022204	0.176991	0.0039292	7.544	7.039	112.3 "	Du Buât
34	11.125	9.202	0.021250	0.713000	0.01515125	14.583	14.513	117.9 "	Mr. Neville
35	20.8	10.2	0.021250	0.814000	0.01729750	15.667	15.617	118.4 "	"
36	150	100	0.020833	1.000000	0.0201667	21.7	20.6	120.3 $\sqrt{r s}$	Mr. Hodson

tube of one twenty-fifth part of an inch in diameter, no matter what the head may be, as it then makes the velocity equal to nothing, although some of the experiments from which it was derived were made with tubes but the eighteenth part of an inch dia-

meter. The following expression is free from this defect:

$$(119c.) \quad v = 60 (rs)^{\frac{1}{2}} + 120 (rs)^{\frac{2}{3}},$$

and will give results approximating very closely to those found from Du Buât's formula, and, therefore, with those experiments with which it most nearly coincides, but agreeing much more closely with Watt's and other experiments, on rivers. It gives higher results than the previous formula for velocities below six inches, but the results found by different experimenters differ very much in those. For higher velocities it appears to differ occasionally only about one-twentieth from observation, being in general less, as far as twenty feet per second, where it coincides very closely with Mr. Hodson's experiment. As the errors appear to be of an opposite kind generally, in the two last expressions, we may get by combining them

$$(119d.) \quad v = 100 (rs)^{\frac{1}{2}} + 60 (rs)^{\frac{2}{3}} - 5.5 (rs)^{\frac{1}{3}},$$

an expression which, however, wants simplicity for ready practical application. When the length of the pipe does not exceed from 1000 to 2000 diameters, a correction is due to the velocity in it, and to the orifice of entry before finding the "hydraulic inclination" (s). The coefficient used in reducing the foregoing experiments for the orifice of entry was .815, which gives $1.508 \frac{v^2}{2g}$ for the height due to the joint effects of velocity and orifice. This must be deducted from the head (H) before dividing it by the length (l) to find the inclination (s) in our table.

The following table, calculated from the formula (119A), $v = 140 (rs)^{\frac{1}{2}} - 11 (rs)^{\frac{3}{2}}$, gives the corresponding values of rs and v , so that when one is known the other is immediately found from inspection. Thus, if $rs = .03125$, we find

$$v = 20.6 \quad \text{when} \quad rs = .029167$$

$$v = 24.7 \quad \text{when} \quad rs = .041666$$

Difference 4.1 corresponds to .012499

$$.03125$$

$$.02917$$

Difference .00208

Whence $.0125 : 4.1 :: .00208 : .7$ nearly, and $20.6 + .7 = 21.3$ is the velocity sought; the same practically as found in EXAMPLE 26, p. 37. If allowance is to be made for the head due to the orifice of entry and velocity, this head can be determined from the velocity due to the value of rs in the table next less than the given value with sufficient accuracy. In this case, this velocity is 20.6 feet per second = 247 inches nearly. If the orifice of entry be square, the coefficient is .815, and the head due to the velocity and this coefficient is, TABLE II., 10 feet nearly. If r be known separately, and also s , as well as the head H , and the length of the pipe l , we had at first

$$\frac{H}{l} = s, \text{ and, therefore, } \frac{H - 10}{l} = \frac{h}{l} = s.$$

In EXAMPLE 26, p. 37, $H = 150$, and $l = 100$ feet, therefore, the new value of $\frac{h}{l} = \frac{140}{100}$ is 1.4; and as r must be equal .020833, $rs = .02917$: the value

TABLE for finding the Velocity in feet per second, from the product of the hydraulic mean depths and hydraulic inclinations, and the reverse calculated from the Author's formula $v = 140 (rs)^{\frac{1}{2}} - 11 (rs)^{\frac{1}{2}}$, in which r , s , and v , are feet measures.

Values of rs .	Velocity v .	Values of rs .	Velocity v .	Values of rs .	Velocity v .	Values of rs .	Velocity v .
·00000296	·083	·0001302	1·04	·000689	2·70	·003559	6·67
·00000332	·091	·0001322	1·05	·000710	2·75	·003599	6·71
·00000395	·104	·0001420	1·09	·000744	2·83	·003630	6·74
·00000427	·111	·0001482	1·12	·000758	2·85	·003788	6·90
·00000543	·133	·0001532	1·14	·000789	2·91	·003929	7·04
·00000592	·142	·0001578	1·16	·000805	2·94	·003946	7·05
·00000690	·158	·0001610	1·17	·000833	3·00	·003977	7·08
·00000734	·167	·0001657	1·19	·000852	3·04	·004104	7·20
·00000947	·198	·0001736	1·21	·000900	3·13	·004167	7·27
·00000989	·206	·0001776	1·24	·000947	3·22	·004356	7·44
·00001184	·231	·0001815	1·26	·001042	3·40	·004546	7·62
·00001263	·241	·0001894	1·30	·001105	3·51	·004630	7·66
·00001420	·261	·0002052	1·35	·001136	3·57	·004735	7·78
·00001578	·280	·0002131	1·38	·001231	3·73	·005556	8·49
·00001677	·292	·0002265	1·43	·001246	3·76	·006944	9·61
·00001894	·316	·0002367	1·47	·001263	3·78	·007576	10·0
·00001973	·325	·0002552	1·50	·001302	3·85	·008333	10·5
·00002170	·345	·0002604	1·55	·001326	3·89	·009259	11·1
·00002367	·365	·0002652	1·57	·001420	4·04	·010417	11·8
·00002565	·385	·0002778	1·61	·001515	4·18	·011905	12·7
·00002841	·411	·0002841	1·63	·001576	4·28	·013889	13·8
·00003255	·448	·0003030	1·69	·001610	4·32	·015151	14·5
·00003354	·457	·0003157	1·73	·001667	4·41	·016067	15·3
·00003551	·473	·0003220	1·75	·001705	4·46	·017297	15·6
·00003748	·489	·0003314	1·79	·001735	4·51	·020833	17·1
·00003946	·505	·0003378	1·80	·001799	4·60	·027778	20·2
·00004143	·521	·0003409	1·81	·001894	4·73	·029167	20·6
·00004340	·536	·0003551	1·85	·001989	4·87	·041066	24·7
·00004632	·558	·0003630	1·89	·002052	4·94	·055556	28·8
·00005130	·594	·0003706	1·90	·002083	4·98	·062500	30·6
·00005327	·608	·0003788	1·92	·002093	5·00	·072916	33·2
·00005524	·622	·0003946	1·98	·002178	5·10	·083333	35·6
·00005919	·643	·0004022	1·10	·002210	5·14	·104107	40·0
·00006314	·674	·0004103	2·02	·002273	5·22	·125	43·0
·00006708	·699	·0004261	2·06	·002375	5·35	·145583	47·6
·0000688	·711	·0004419	2·10	·002402	5·46	·166667	51·1
·00007102	·724	·0004485	2·12	·002533	5·53	·208333	57·3
·00007694	·760	·0004546	2·14	·002652	5·68	·229167	60·2
·00008049	·781	·0004708	2·18	·002683	5·72	·260000	63·0
·00008523	·808	·0004735	2·18	·002841	5·90	·270833	65·7
·00008681	·828	·0004893	2·23	·002968	6·05	·312500	70·7
·00009270	·849	·0005051	2·27	·002999	6·08	·333333	73·2
·00009470	·861	·0005208	2·31	·003030	6·11	·354167	75·5
·00010259	·903	·0005303	2·33	·003143	6·23	·375000	77·7
·00010654	·923	·0005538	2·41	·003157	6·25	·395833	80·0
·00011048	·945	·0006061	2·52	·003214	6·31	·416667	82·1
·00011364	·960	·0006155	2·54	·003220	6·32	·437500	84·2
·00011837	·983	·0006313	2·57	·003314	6·42	·458333	86·2
·00012232	1·00	·0006440	2·60	·003409	6·51	·479166	88·3
·00012627	1·02	·0006629	2·64	·003475	6·58	·500000	90·2

corresponding to which, in our table, is 20.6, the velocity when allowance is made for the head due to the velocity and orifice of entry.

In general, by taking the value of v for the next less value of $r s$ in the table, we shall find the velocity with sufficient accuracy, and also the value of $r s$ from that of v by taking it as the next greater. If we had taken $r s = .0008523$, the table would give $v = 3.04$ feet, the same practically as already found in EXAMPLE 27, p. 38.

The value of $r s$, when known, determines the value of v . If r be assumed of any convenient dimensions, s is determined; and, in like manner, any suitable value of s determines r ; thus:

$$\frac{r s}{r} = s, \text{ and } \frac{r s}{s} = r.$$

It is well to remark, here again, that for pipes the value of r is the fourth part of the diameter d , and that

$$r = \frac{d}{4}, \text{ and } 4 r = d.$$

In 1857, M. Darcy, inspecteur des ponts et chaussées, published his *Recherches expérimentales relatives au Mouvement de l'Eau dans les Tuyaux*,* the result of 198 experiments, in which the velocities varied from .03 to 5 or 6 mètres per second, or from $1\frac{1}{2}$ inch to 16 or 19 feet, and with pipes varying from $\frac{1}{2}$ inch to 20 inches diameter. The formula by which he presents the results is in mètres,

$$(a.) \quad R J = 7.1 U^2,$$

in which R is the radius of the pipe, J the hydraulic

* Morin's *Hydraulique*, deuxième édition, Paris, p. 164.

inclination, b_1 a variable coefficient dependent on the circumstances, and v the velocity per second. For wrought and cast iron pipes of the same state of bore, the value of b_1 is expressed by M. Darcy, by the equation

$$(b.) \quad b_1 = .000507 + \frac{.00000647}{R},$$

the agreement between which and experiment is shown in the following table.

Diameters in English inches.	Diameters in mètres.	Values of b_1 from experiments.	Values of b_1 by the formula.	Remarks.
.5	.0122	.001673	.001568	Well polished bore. { Pipe already in use, but the bore cleaned.
1.	.0260	.000918	.000903	
1.5	.0395	.000785	.000835	
3.2	.0819	.000695	.000665	
5.4	.1370	.000553	.000601	
7.4	.1880	.000584	.000576	
11.7	.2970	.000612	.000561	
19.7	.5000	.000509	.000532	

For iron coated with bitumen, the value of b_1 in a pipe .196 mètres in diameter was .0004334; for a newly cast pipe of .188 mètres, b_1 was .000584; and for a pipe .2432 mètres in diameter, b_1 was .001168; the relative proportions of b_1 in these three instances, being as

1.1 to 1.5 and to 3;

and, therefore, the velocities, or discharges, would be inversely as the square roots of these, or as

.95 to .82 and to .58.

By substituting our notation for that of M. Darcy, shall have in mètres, from equations (a) and (b),

$$rs = \frac{b_1}{2} v^2 = \left\{ .0002535 + \frac{.0000016175}{r} \right\} v^2;$$

which for feet measures becomes (as 1 mètre = 3.281 feet)

$$rs = \left\{ .0002535 + \frac{3.281 \times .0000016175}{r} \right\} \times \frac{v^2}{3.281};$$

hence we get

$$v^2 = \left\{ \frac{rs}{.00007726 + \frac{.00000162}{r}} \right\},$$

and, therefore,

$$v = \left\{ \frac{rs}{.00007726 + \frac{.00000162}{r}} \right\}^{\frac{1}{2}}.$$

For all half-inch pipes this becomes

$$v = \left\{ \frac{rs}{.00023278} \right\}^{\frac{1}{2}} = 65.5 \sqrt{rs};$$

for all inch pipes,

$$v = \left\{ \frac{rs}{.00015502} \right\}^{\frac{1}{2}} = 80.3 \sqrt{rs};$$

for all two-inch pipes,

$$v = \left\{ \frac{rs}{.00011614} \right\}^{\frac{1}{2}} = 92.8 \sqrt{rs};$$

for all four-inch pipes,

$$v = \left\{ \frac{rs}{.0000967} \right\}^{\frac{1}{2}} = 101.7 \sqrt{rs};$$

for all six-inch pipes,

$$v = \left\{ \frac{rs}{.00009022} \right\}^{\frac{1}{2}} = 105.3 \sqrt{rs};$$

for all nine-inch pipes,

$$v = \left\{ \frac{rs}{.0000859} \right\}^{\frac{1}{2}} = 107.8 \sqrt{rs};$$

for all twelve-inch pipes,

$$v = \left\{ \frac{rs}{.00008374} \right\}^{\frac{1}{2}} = 109.3 \sqrt{rs};$$

for all eighteen-inch pipes,

$$v = \left\{ \frac{rs}{.00008158} \right\}^{\frac{1}{2}} = 110.7 \sqrt{rs};$$

for all twenty-four-inch pipes,

$$v = \left\{ \frac{rs}{.0000805} \right\}^{\frac{1}{2}} = 111.5 \sqrt{rs};$$

and when r is large, as for very large pipes and channels, we get the velocity

$$v = \left\{ \frac{rs}{.00007726} \right\}^{\frac{1}{2}} = 113.8 \sqrt{rs}.$$

There is evidently, on an examination of these results, a great error in the formula of M. Darcy. As long as the diameter of a long pipe continues constant, the velocity is always represented by a given fixed multiple of \sqrt{rs} , or of the square root of the product of the hydraulic inclination and hydraulic mean depth, no matter how small or great the velocity in the pipe may be. For an inch pipe this multiplier for feet measures is 80.3. Now with a lead pipe I have found, from several experiments, that for a velocity of about 15 feet per second, the multiplier to be 117 or 118; and for a velocity of about 22 feet per second, Mr. Hodson's experiment gives a multiplier of about 120. Taking the other

extreme for large pipes, the multiplier derived from M. Darcy's formula is 113.8, no matter how small the velocity may be. Now we have experiments in abundance to prove that for velocities of about 12 or 13 inches per second, the multiplier cannot exceed 95. We, therefore, look upon these researches of M. Darcy, as partial and defective, and his formula as a representation; at best, of a limited range of velocities, in which those at either side are omitted or not perceived.

For small pipes, any obstruction arising from defective bore, decomposition, encrustation, or from diminished bore, affects the discharge much more considerably than the same obstructions in a large pipe. In order to compare correctly the effects of the state of the bore on the discharge, we must use pipes of exactly the same diameter, and determine the value of b_1 from experiments in which the velocity is the same, otherwise the results, as deduced by M. Darcy and given by Morin, cannot be depended upon.

COEFFICIENTS DUE TO THE ORIFICE OF ENTRY.—PROBLEMS.

Unless where otherwise expressed, the head due to the velocity and orifice of entry is not considered in the preceding equations. In equation (74), where it is taken into calculation generally,

$$v = \left(\frac{2gH}{1 + c_r + c_r \times \frac{l}{r}} \right)^{\frac{1}{2}}$$

in which $1 + c_r$ is equal to $\left(\frac{1}{c_v}\right)^2$, c_r being the coefficient

of resistance due to the orifice of entry, and c_v the coefficient of velocity or discharge from a short tube. If the tube project into the reservoir, and be of small thickness, c_v will be equal $\cdot 715$ nearly, and therefore $c_r = \cdot 956$; if the tube be square at the junction, the mean value of c_v will be $\cdot 814$, and therefore $c_r = \cdot 508$; and if the junction be rounded in the form of the contracted vein, c_v is equal to unity very nearly, and $c_r = 0$. For other forms of junction the coefficients of discharge and resistance will vary between these limits, and particular attention must be paid to their values in finding the discharge from shorter tubes and those of moderate lengths; but in very long tubes $1 + c_r$ becomes very small compared with $c_r \times \frac{l}{r}$,

and may be neglected without practical error. These remarks are necessary to prevent the misapplication of the tables and formulæ, as the height due to the velocity and orifice of entry is an important element in all calculations for short tubes.

We have considered it unnecessary to give any formulæ for finding the discharge itself, because, the mean velocity once determined, the calculation of the discharge from the area of the section is one of simple mensuration; and the introduction of this element into the three problems to which this portion of hydraulic engineering applies itself, renders the equations of solution complex, though easily derived; and presents them with an appearance of difficulty and want of simplicity which excludes them, nearly altogether, from practical application. The three problems are as follows:—

I. *Given the fall, length, and diameter of a pipe or hydraulic mean depth of any channel, to find the discharge.*

Here all that is necessary is to find the mean velocity of discharge, which, multiplied by the area of the section (equal $d^2 \times .7854$ in a cylindrical pipe), gives the discharge sought. TABLE VIII. gives the velocity at once for long channels, according to Du Buat, or it can be found from equation (119A) by calculation. TABLE IX. gives the discharge in cubic feet per minute for different diameters of pipes, and velocities in inches per second, when found from TABLE VIII. or formula (119A). See also TABLES XI. and XII. For a pipe 6 inches in diameter, the velocity per second is practically equal to the discharge in cubic feet per minute. See also the tables, pp. 42, 43, 252, and 253.

II. *Given the discharge and cross section of a channel, to find the fall or hydraulic inclination.*

If the cross section be circular, as in most pipes, the hydraulic mean depth is one-fourth of the diameter; in other channels it is found by dividing the water and channel line of the section, wetted perimeter, or border, into the area. The velocity is found by dividing the area into the discharge, and reducing it to inches per second; then in TABLE VIII., under the hydraulic mean depth, find the velocity, corresponding to which the fall per mile will be found in the first column, and the hydraulic inclination in the second. This result can be corrected by trial and error to accord with formula (119A), and the table for the values of r 's and v , p. 220,

calculated from it. See also the tables, pp. 42, 43, 252, and 253.

III. *Given the discharge, length, and fall, to find the diameter of a pipe, or hydraulic mean depth and dimensions of a channel.*

This is the most useful problem of the three. Assume any mean radius r_a , and find the discharge D_a by Problem I. We shall then have for cylindrical pipes

$$r_a^{\frac{5}{2}} : r^{\frac{5}{2}} :: D_a : D :: 1 : \frac{D}{D_a};$$

and as r_a , D , and D_a are known, $r^{\frac{5}{2}}$ becomes also known, and thence r . TABLE XIII. will enable us to find r with great facility. Thus, if we had assumed $r_a = 1$ and found $D_a = 15$, D being 33, we then have

$$1 : r^{\frac{5}{2}} :: 1 : \frac{33}{15} :: 1 : 2.2, \text{ therefore } r^{\frac{5}{2}} = 2.2;$$

and thence by TABLE XIII., $r = 1.37$, the mean radius required, four times which is the diameter of the pipe. For other channels, the quantity thus found must be the hydraulic mean depth; and all channels, however varied in the cross section, will have the same velocity of discharge, when the fall, length, and hydraulic mean depth are constant. If r_a be assumed equal to $1\frac{1}{2}$ inch, the velocity found from TABLE VIII. will then be the discharge in cubic feet per minute nearly, and this "mean radius" can always be assumed for the first term of the proportion. See also the tables, pp. 42, 43, 252, and 253.

In order to find the dimensions of any polygonal channel whatever, which will give a discharge equal to D , we may assume any channel similar to that

proposed, one of whose known sides is s_n , and find the corresponding discharge, D_n , by Problem I., or from TABLES XI. and XII.; then, if we call the like side of the required channel, s , we shall have $s = s_n \left(\frac{D}{D_n} \right)^{\frac{2}{5}}$, and thence the numerical value from TABLE XIII. The result, as before, can be corrected to accord with any of our formulæ by the method of trial and error.

As it frequently happens that deposits in and encrustations on a pipe take place from time to time, which diminish the flowing section considerably, it is always prudent, when calculating the necessary diameter, to take the largest coefficient of friction, c , or to double its mean value, particularly for small pipes, when calculating the diameter from any of the formulæ. Some engineers, as D'Aubuisson, increase the quantity of water by one-half to find the diameter; but much must depend on the peculiar circumstances of each case, as sometimes less may be sufficient, or more necessary. The discharge increases in similar figures, nearly as $r^{\frac{5}{2}}$ or as $d^{\frac{5}{2}}$, that is, as the square root of the fifth power of the diameter, and the corresponding increase in the diameter for any given or allowed increase in the discharge can be easily found by means of TABLE XIII., as shown above. If we increase the dimensions by one-sixth, the discharge will be increased by one-half nearly, and by doubling them the discharge is increased in the proportion of $5\frac{2}{3}$ to 1.

For shorter pipes, we have to take into consideration the head due to the velocity and orifice of entry.

Taking the mean coefficient of velocity or discharge, we find from TABLE II. the head due to the velocity, if it be known; this subtracted from the whole head, H , leaves the head, h_v , due to the hydraulic inclination, which is that we must make use of in the table. If the velocity be not given, we can find it approximately; the head found for this velocity, due to the orifice of entry, when deducted, as before, will give a close value of h_v , from which the velocity may be determined with greater accuracy, and so on to any degree of approximation. In general, one approximation to h_v will be sufficient, unless the pipes be very short, in which case it is best to use equation (74). EXAMPLE VIII., p. 195, and the explanation of the use of the tables, SECTION I., may be usefully referred to.

TABLES XI., XII., and XIII. enable us to solve with considerable facility all questions connected with discharge, dimensions of channel, and the ordinary surface inclinations of rivers. The discharge corresponding to any intermediate channels or falls to those given in TABLES XI. or XII., will be found with abundant accuracy, by inspection and simple interpolation; and in the same manner the channels from the discharges. Rivers have seldom greater falls than those given in TABLE XII., but in such an event we have only to divide the fall by 4, then twice the corresponding discharge will be that required. TABLE XIII. gives the comparative discharging powers of all similar channels, whether pipes or rivers, and the comparative dimensions from the discharges. We perceive from it, that an increase

of one-third in the dimensions doubles, and a decrease of one-fourth reduces the discharge to one-half. By means of this table, we can determine by a simple proportion, the dimensions of any given form of channel when the discharge is known. See EXAMPLE 17, p. 30. See also the tables pp. 42, 43, 252 and 253.

The mean widths in TABLES XI. and XII. are calculated for rectangular channels, and those having side slopes of $1\frac{1}{2}$ to 1. Both these tables are, however, practically, equally applicable to any side slopes from 0 to 1 up to 2 to 1, or even higher, when the mean widths are taken and not those at top or bottom. A semihexagon of all trapezoidal channels of equal area has the greatest discharging power, and the semisquare and all rectangles exactly the same as channels of equal areas and depths with side slopes of $1\frac{1}{2}$ to 1. The maximum discharge is obtained between these for the semihexagon with side slopes, of nearly $\frac{1}{2}$ to 1, but for equal areas and depths *the discharge decreases afterwards as the slope flattens*. The question of "HOW MUCH?" is here, however, a very important one; for, as we have already pointed out in equations (28) and (31), the differences for any practical purposes may be immaterial. This is particularly so in the case of channels with different side slopes, if, instead of the top or bottom, we make use of the mean width to calculate from. We then have only to subtract the ratio of the slope multiplied by the depth to find the bottom, and add it to find the top. If the mean width be 50 feet, the depth 5 feet, and the side slopes

2 to 1, we get $50 - (2 \times 5) = 40$ for the bottom, and $50 + (2 \times 5) = 60$ for the top width.

Side slopes of 2 to 1 present a greater difference from the mean slope of $1\frac{1}{2}$ to 1, than any others in general practice when new cuts are to be made. A triangular channel having slopes of 2 to 1, and bottom equal to zero, differs more in its discharging power from the half square, equal to it in depth and area, than if the bottom in each was equally increased, yet even here it is easy to show that this maximum difference is only $5\frac{1}{2}$ per cent. If the bottom be increased so as to equal the depth, it is only $4\frac{1}{2}$ per cent.; when equal to twice the depth, 3.8 per cent.; and when equal to four times the depth, to 2 per cent.; while the differences in the dimensions taken in the same order are only 2.2, 1.8, 1.5, and 0.8 per cent. For greater bottoms in proportion to the depth the differences become of no comparative value. It therefore appears pretty evident, that TABLES XI. and XII. will be found *equally applicable to all side slopes from 0 to 1 up to 2 to 1, by taking the mean widths*. When new cuts are to be made, we see no reason whatever in starting from bottom rather than mean widths, to calculate the other dimensions; indeed, the necessary extra tables and calculations involved ought entirely to preclude us from doing so. Besides, the formulæ for finding the discharge vary in themselves, and for different velocities the coefficient of friction also varies.* Added to which

* The coefficient m in the formula $v = m (rs)^{\frac{1}{2}}$ in rivers for velocities from 3 inches to 3 feet per second, varies from about 72 to 103; yet, strange to say, most tables are calculated from

the inequalities in every river channel, caused by bends and unequal regimen, preclude altogether any regularity in the working slopes and bottom, though the mean width would continue pretty uniform under all circumstances.

The quantities in TABLE XII. are calculated, from the velocities found from TABLE VIII., to correspond to a channel 70 feet wide and of different depths, the equivalents to which are given in TABLE XI. In order to apply these tables generally to all open channels, the latter are to be reduced to rectangular ones of the same depth and mean width, or the reverse, as already pointed out. If the dimensions of the given channel be not within the limits of TABLE XI., divide the dimensions of the larger channels by 4, and multiply the corresponding discharge found in TABLE XII. by 32; for smaller channels, multiply the dimensions by 4, and divide by 32. In like manner, if the discharge be given and exceed any to be found in TABLE XIII., divide by 32, and multiply the dimensions of the suitable equivalent channel found in TABLE XI. by 4. If we wish to find equivalent channels of less widths than 10 feet for small discharges, multiply the discharge by 32, and divide the dimensions of the corresponding equivalent by 4. Many other multipliers and divisors as well as 4 and 32 may be found from TABLE XIII., such as 3 and

one coefficient alone; or, rather, from a formula equivalent to $94.17 (rs)^{\frac{1}{2}}$, which gives results suited only to a velocity of 16 inches. Dimensions of channels calculated by means of this formula are too small in one case, and too large in the other. In pipes the variation of the coefficients is shown in the small tables, pp. 214 and 217.

15.6, 6 and 88.2, 7 and 130, 9 and 243, 10 and 316, 12 and 499, &c. The differences indicated at pages 198 and 199, must be expected in the application of these rules, which will give, however, dimensions for new channels which can be depended on for doing duty.

It will be seen from TABLE XIII. that a very small increase in the dimensions increases the discharging power very considerably. TABLE XII. also shows that a small increase in the depth alone adds very much to the discharge. If we express in this latter case *a small increase* in the depth, d , by $\frac{d}{n}$, then it is easy to prove that the corresponding increase in the velocity, v , will be $\frac{v}{2n}$; and that in the

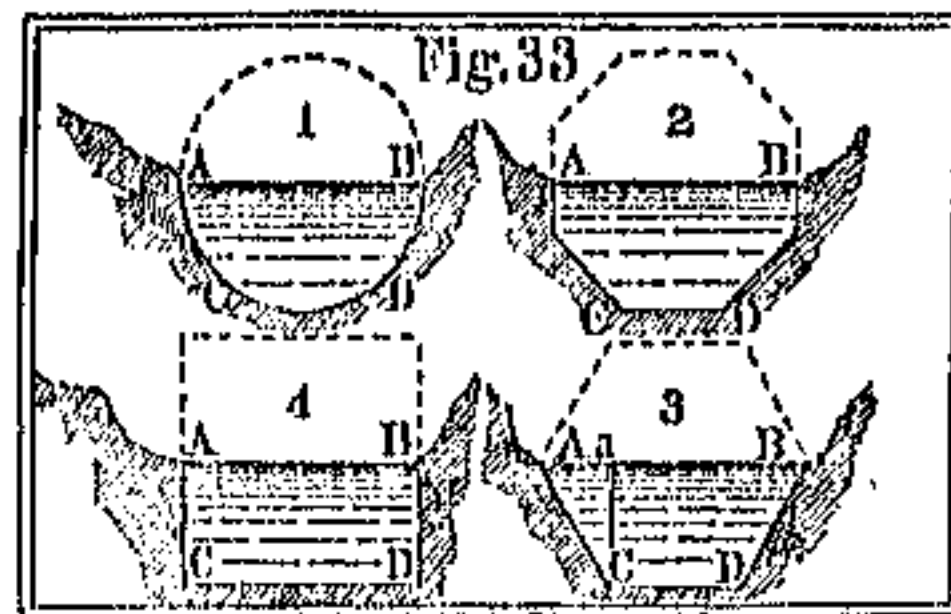
discharge D , $\frac{3D}{2n}$, if the surface inclination continue unchanged; but as it is always observable in rivers that the surface inclinations increase with floods, the differences in practice will be found greater than these expressions make it. As in a large river the surface inclination must be very small, four times the fall will add very little to the sectional area; yet this increase of fall will double the discharge, and we thence perceive how tributaries can be absorbed into the main channel without any great increase to its depth.

SECTION IX.

BEST FORMS OF THE CHANNEL.—REGIMEN.—VELOCITY.—
EQUALLY DISCHARGING CHANNELS.

We have seen above, that the determination of the hydraulic mean depth does not necessarily determine the section of the channel. If the form be a circle, the diameter is four times the mean radius; but, though this form be almost always adopted for pipes, the beds of rivers take almost every curvilineal and trapezoidal shape. Other things being the same, that form of a river channel, in which the area divided by the border is a maximum, is the best. This is a semicircle having the diameter for the surface line, and in the same manner, half the regular figures, an octagon, hex-

agon, and square, in Fig. 33, are better forms for the channel, the areas and side slopes being constant, than any others of the same number of



sides. Of all rectangular channels, Diagram 4, in which $ABCD$ is half a square, is the best cross section; and in Diagram 3, $ACDB$, half a hexagon, is the best trapezoidal form of cross section. When the width of the bottom, CD , Diagram 3, is given, and the slope

$\frac{An}{CA} = n$, then, in order that the discharge may be the greatest possible, we must have

$$oa = \left\{ 2 \frac{\Lambda}{(n^2 + 1)^{\frac{1}{2}} - n} \right\}^{\frac{1}{2}}, \text{ and}$$

$$od = \frac{\Lambda}{oa} - n \times oa$$

$$= \left\{ [2 (n^2 + 1)^{\frac{1}{2}} - n] \times \Lambda \right\}^{\frac{1}{2}} - n \left\{ \frac{\Lambda}{2 (n^2 + 1)^{\frac{1}{2}} - n} \right\}^{\frac{1}{2}},$$

in which Λ is the given area of the channel. As, however, we have never known a river in which the slope of the natural banks continued uniform, even though made so for any improvements, we consider it almost unnecessary to give tables for different values of n . If, notwithstanding, we put ϕ for the inclination of the slope ao , equal angle oaa , we shall find,

as $\cot. \phi = n$, and $\sqrt{n^2 + 1} = \frac{1}{\sin. \phi}$, that the foregoing equations become

$$(120.) \quad oa = \left\{ \frac{\Lambda \sin. \phi}{2 - \cos. \phi} \right\}^{\frac{1}{2}} = \frac{od}{2 \{ (n^2 + 1)^{\frac{1}{2}} - n \}};$$

and

$$(121.) \quad od = \frac{\Lambda}{oa} - oa \times \cot. \phi,*$$

which will give the best dimensions for the channel when the angle of the slope for the banks is known.

When the discharge from a channel of a given area, with given side slopes, is a maximum, it is easy to show that THE HYDRAULIC MEAN DEPTH MUST BE HALF OF THE CENTRAL OR GREATEST DEPTH. This simple principle enables us to construct the best form of channel with great facility. *Describe any circle on the drawing-board; draw the diameter and produce*

* When $od = o$. The channel is triangular; we get $\Lambda = oa^2 \times \cot. \phi$, and $oa = \left(\frac{\Lambda}{\cot. \phi} \right)^{\frac{1}{2}}$.

it on both sides, outside the circle ; draw a tangent to the lower circumference parallel to this diameter, and draw the side slopes at the given inclinations, touching the circumference also on each side and terminating on the parallel lines : the trapezoid thus formed will be the best form of channel, and the width at the surface will be equal to the sum of the two side slopes. It is easy to perceive that this construction may be, simply, extended for finding the best form of a channel having any polygonal border whatever of more sides than three and of given inclinations.

Commencing with the best discharging form of channel, which in practice will have the mean width, about double the depth ; an equally discharging section of double the width of the first will have the contents one-eleventh greater, and the depth less in the proportion of 1 to 1.85. A channel of double the mean width of the second must have the sectional area further increased by about one-fifth, and a further decrease in the depth from 1.67 to 1 nearly. The greater expense of the excavation at greater depths will, in general, more than counterbalance these differences in the contents of the channel. When the banks rise above the flood line, and are unequal in their section, the wider channel involves further upper extra cutting, but there is greater capacity to discharge extra and extraordinary flooding, the banks are less liable to slip or give way, the slopes may be less, and the velocity being also less, the regimen will, in general, be better preserved. The table of equally discharging channels, p. 252, will afford the means of calculating the difference of cubical contents.

TABLE OF THE RELATIVE DIMENSIONS OF MAXIMUM DISCHARGING CHANNELS.

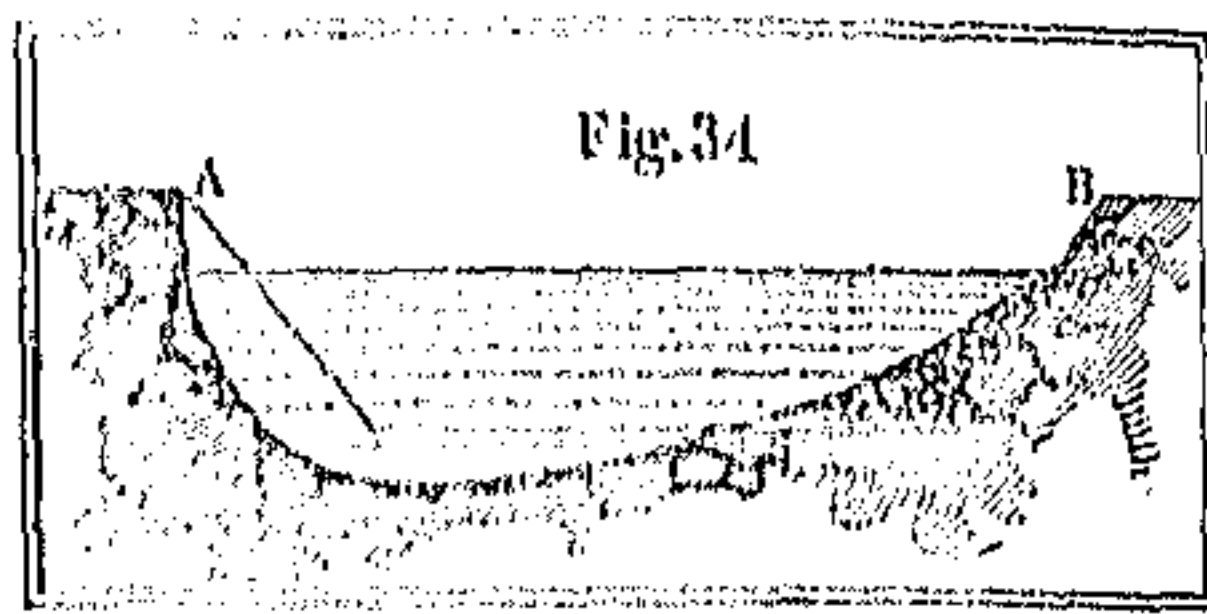
Angle of slope.	Engineering slope.	Depth in terms of the area.	Bottom in terms of the area.	Top in terms of the area.	Hydraulic mean depth in terms of the area.	Ratio of bottom to depth.	Ratio of top to depth.	Area in terms of the depth d .
90° 0'	0 to 1	$.707\sqrt{A}$	$1.414\sqrt{A}$	$1.414\sqrt{A}$	$.354\sqrt{A}$	2 to 1	2 to 1	$2d^2$
63 26	$\frac{1}{2}$ to 1	$.759\sqrt{A}$	$.938\sqrt{A}$	$1.697\sqrt{A}$	$.379\sqrt{A}$	1.236 to 1	2.236 to 1	$1.736d^2$
48 34½	$\frac{3}{4}$ to 1	$.748\sqrt{A}$	$.675\sqrt{A}$	$1.996\sqrt{A}$	$.374\sqrt{A}$.902 to 1	2.667 to 1	$1.784d^2$
55 0	1 to 1	$.740\sqrt{A}$	$.613\sqrt{A}$	$2.093\sqrt{A}$	$.370\sqrt{A}$.828 to 1	2.828 to 1	$1.828d^2$
36 52	$1\frac{1}{2}$ to 1	$.707\sqrt{A}$	$.471\sqrt{A}$	$2.357\sqrt{A}$	$.354\sqrt{A}$.667 to 1	3.333 to 1	$2d^2$
33 41½	$1\frac{1}{2}$ to 1	$.689\sqrt{A}$	$.417\sqrt{A}$	$2.484\sqrt{A}$	$.345\sqrt{A}$.605 to 1	3.605 to 1	$2.105d^2$
30 58	$1\frac{1}{2}$ to 1	$.671\sqrt{A}$	$.372\sqrt{A}$	$2.608\sqrt{A}$	$.336\sqrt{A}$.554 to 1	3.888 to 1	$2.221d^2$
26 34	2 to 1	$.636\sqrt{A}$	$.300\sqrt{A}$	$2.844\sqrt{A}$	$.318\sqrt{A}$.472 to 1	4.472 to 1	$2.472d^2$
Semicircle	Curved	$.798\sqrt{A}$.000	$1.596\sqrt{A}$	$.399\sqrt{A}$.000 to 1	2 to 1	$1.571d^2$
Circle	Curved	$1.128\sqrt{A}$.000	.000	$.282\sqrt{A}$.000 to 1	0 to d	$.785d^2$

NOTE.—Half the top will give the length of the side slope. The top itself is the sum of the side slopes and the border the sum of the top and bottom. The sum of the side slopes in any channel whatever is equal to the number corresponding to the slope in the eighth column multiplied by the depth; and the sum of the triangular side areas, corresponding to the slopes, is equal to the square of the depth multiplied by the numbers corresponding to the ratio in column 2. The bottom multiplied by the depth added to this gives the area of the section. The top of the best form of channel should be at the level of the extraordinary flood line, and not at that of the mean annual surface.

When the sectional area is given, the above table shows that the semicircle is the best discharging channel, and the complete circle the worst; the latter is so, however, only compared with the *open* channels given in the table, it being the very best form for an *enclosed* channel flowing full. *The best form of channel is particularly suited for new cuts in flat, marsh, callow, and fen lands, in which it is also often advisable to cut them with a level bed, up from the discharging point, in order to increase the hydraulic mean depth, and consequently the velocity and discharge.*

As the quantity of water coming down a river channel in a season varies very considerably,—we have observed it in one case to vary from one to thirty, and occasionally in the same channel from one to seventy-five,—the proportion of the water section to the channel itself must also vary, and those relations of the depth, sides, and width to each other, above referred to, cease to hold good and be the best under such circumstances. If the object be to construct a mill-race, temporary drain for unwatering a river, or other small channel, in which the depth remains nearly constant, channels of the form of a half hexagon, diagram 3, Fig. 33, will be, perhaps, the best, if the tenacity of the banks permit the slope; but rivers, in which the quantity of water varies considerably, require wider channels in proportion to the depth; and also, that the velocity be so proportioned to the tenacity of the soil, or as it is termed "*the regimen*," that the banks and bed

shall not vary from time to time to any injurious extent, and that any deposits made during their summer state, and during light freshes, shall be carried off periodically by floods. Another circumstance, also, modifies the effects of the water on the banks. It is this, that at curves, and turns, the current acts with greatest effect against the bank, concave to the direction in which it is moving; deepening the channel there; undermining also the bank,



as at A, Fig. 34; and raising the bed towards the opposite side B. The reflexion of the current to the opposite bank from A acts also in a similar manner, lower down, upon it; and this natural operation proceeds, until the number of turns, increased length of channel, and loss of head from reflexion and unequal depths, bring the currents into regimen with the bed and banks. At all bends it is, therefore, prudent to widen the channel on the convex side B, Fig. 34, in order to reduce the velocity of approach; and if the bed be here also sunk below its natural inclination, as we see it in most rivers at bends, the velocity will be farther reduced, and the permanence of the bed better established.

The circumstances to be considered in deciding on the dimensions and fall of a new river course, after the depth to which the surface of the water is to be brought has been decided on, are the following:—

The mean velocity must not be too slow, or aquatic plants will grow, and deposits take place, reducing the sectional area until a new and smaller channel is formed within the first with just sufficient velocity to keep itself clear. This velocity should not in general be less than from ten to fourteen inches per second. The velocity in a canal or river is increased very considerably by cutting or removing reeds and aquatic plants growing on the sides or bottom.*

The mean velocity must not be too quick, and should be so determined as to suit the tenacity and resistance of the channel, otherwise the bed and banks will change continually, unless artificially protected; it should not exceed

25	feet	per	minute	in	soft	alluvial	deposits.
40	"	"	"	"	"	clayey	beds.
60	"	"	"	"	"	sandy	and silty beds.
120	"	"	"	"	"	gravelly.	
180	"	"	"	"	"	strong	gravelly shingle.

* "M. Girard a fait observer, avec raison, que les plantes aquatiques, qui croissent toujours sur le fond et sur les berges des canaux, augmentent considérablement le périmètre mouillé, et par suite la résistance; il a rapellé que Du Bât, ayant mesuré la vitesse de l'eau dans le canal du Jard, avant et après la coupe des roseaux dont il était garni, avait trouvé un resultat bien moindre avant qu'après. En conséquence, il a presque doublé la pente donnée par le calcul . . ."—Traité d'Hydraulique, p. 185. When the fall does not exceed a few inches per mile, the velocity, as determined from the inclination, is very uncertain, and for this reason it is always prudent to increase the depths and sectional areas of channels in flat lands, as far as the regimen will permit. In such cases the section of the channel should approximate towards the best form. See p. 238.

240 feet per minute in shingly.

300 " " shingly and rocky.

400 and upwards in rocky and shingly.*

A velocity of 180 feet per minute will remove angular stones the size of an egg. Mr. Phillips, under the Metropolitan Commissioners of Sewers, states that $2\frac{1}{2}$ feet per second, or 150 feet per minute, is sufficient to prevent soil depositing in sewers.

The fall per mile should decrease as the hydraulic mean depth increases, and both be so proportioned that floods may have sufficient power to carry off the deposits, if any, periodically. The proportion of the width to the depth of the channel should not be derived, for new cuts or river courses, from any formula, but taken from such portions of the old channel as approximate in depth and in the inclination of the surface to that proposed. When the depth is nearly half the width, the formula shows, *cæteris paribus*, that the discharge will be a maximum; but as (altogether apart from the question

* TABLE OF VELOCITIES OF SOME MOVING BODIES COMPARED WITH THOSE OF RIVERS.

Objects in motion.	Miles per hour.	Feet per second.	Objects in motion.	Miles per hour.	Feet per second.
Current of slow rivers . .	6	2	Railway trains, French . .	27	39½
Currents of ordinary rivers, up to	13	3	" " German	24	33½
Currents of rapid rivers . .	11½	2½	Sound when atmosphere is at 52° Fahr.	749	1,000
Man walking	7	10½	Ditto 60° Fahr.	703	1,193
Horse trotting	8	4½	Air rushing into vacuum . .	850	1,247
Swiftest race-horse	7	10½	Ditto when the barometer stands at 30 inches . . .	917	1,314
Moderate winds	60	83	Common musket-ball . . .	850	1,247
Storms	7	10½	Rifle-ball	1,000	1,467
Hurricanes	96	52½	Cannon-ball	1,001	1,600
Swift English steamboats navigating the channels .	80	117½	Bullet discharged from air- gun, air being compressed into the hundredth part of its volume . . .		477
Swift American River steamers	14	20½	A point on earth's surface at the equator moving round the axis	1,040	1,525
Fast sailing vessels	18	25½	Earth moving round sun . .	68,182	100,000
Railway trains, English . .	12	17½			
" " American	82	47			
" " Belgian	18	26½			
	25	86½			

of expenso) the quantity of water discharged daily, at different seasons, may vary from one to seventy, or more, and "*the regimen*" has to be maintained, the best proportion between the width and depth of a new cut should be obtained, as we have stated, from some selected portion of the old channel, whose general circumstances and surface inclination approximate to those of the one proposed; and the side slopes of the banks must be such as are best suited to the soil. The resistance of the banks to the current being in general less than that of the beds, which get covered with gravel, and the necessary provision required for floods, appears to be the principal reason why rivers are in general so very much wider than about twice the depth, the relation which gives the minimum of friction.

The following table is given by Rennie, as an approximation, generally, to the actual state of rivers.* The surface inclinations, however, given in this table for the first and second classes, are very considerable for large rivers, and would give velocities which would effectually scour them. For a hydraulic mean depth of 12 feet, the velocity, with

a fall of $\frac{1}{12000}$, would be 2 feet 8 inches per second by Du Buat's formula; and 3.3 feet per second by our formula. The description, therefore, can only apply to small channels. In fact, 4 inches to a mile, or $\frac{1}{15740}$, is a considerable inclination for a large river.

* Report to the British Association 1884.

DISTINCTIVE ATTRIBUTES OF THE VARIOUS KINDS OF RIVERS.	Rank of rivers and flowing waters.	Comparative degrees of the mean velocities of currents.	Seconds of time in which currents run 20 fathoms.	Fathoms run by the current per minute of time	Ratios of decli- vity compared with horizontal length.	Fathoms of length for each one-twelfth inch of declivity.
Channels wherein the resistance from the bed, and other obstacles, equal the quantity of current acquired from the declivity; so that the waters would stagnate therein, were it not for the compression and impulsion of the upper and back waters . . .	1st.	0	0"	0	$\frac{1}{12000}$	14
Artificial canals in the Dutch and Austrian Netherlands . . .	2nd.	$\frac{1}{2}$	180	6 $\frac{1}{2}$	$\frac{1}{1000}$	8
Rivers in low flat countries, full of turns and windings, and of a very slow current, subject to frequent and lasting inundations	3rd.	1	120	10	$\frac{1}{5200}$	6
Rivers in most countries that are a mean between flat and hilly, which have good currents, but are subject to overflow; also the upper parts of rivers in flat countries	4th.	1 $\frac{1}{2}$	80	15	$\frac{1}{4000}$	4 $\frac{1}{2}$
Rivers in hilly countries with a strong current, and seldom subject to inundations; also all rivers near their sources have this declivity and velocity, and often much more	5th.	2 $\frac{1}{2}$	55	21 $\frac{1}{2}$	$\frac{1}{3200}$	3 $\frac{1}{2}$
Rivers in mountainous countries having a rapid current and straight course, and very rarely overflowing	6th.	3	40	30	$\frac{1}{2000}$	3
Rivers in their descent from among mountains down into the plains below, in which plains they run torrent-wise.	7th.	5	24	50	$\frac{1}{8000}$	2 $\frac{1}{2}$
Absolute torrents among mountains	8th.	8	15	80	$\frac{1}{1700}$	2

The following information with reference to the surface inclinations of the Thames, is from Rennie's Report on Hydraulics,* as a branch of engineering science.

Names of places.	Length.		Fall.		Fall in feet per mile.	Ratio of inclinations.
	Miles.	Fur.	Feet.	In.		
From Lechdale at St. John's Bridge to Oxford at Folly Bridge	28	0	47	0	1.68	$\frac{1}{3,143}$
From Oxford to Abingdon Bridge	9	0	13	11	1.73	$\frac{1}{3,052}$
From Abingdon to Wallingford Bridge	14	0	27	4	1.05	$\frac{1}{2,708}$
From Wallingford to Reading Bridge	18	0	24	1	1.31	$\frac{1}{4,030}$
From Reading to Henley Bridge	9	0	19	8	2.14	$\frac{1}{2,467}$
From Henley to Marlow Bridge	9	0	12	2	1.35	$\frac{1}{3,911}$
From Marlow to Maidenhead Bridge	8	0	15	1	1.86	$\frac{1}{2,839}$
From Maidenhead Bridge to Windsor Bridge	7	0	13	6	1.93	$\frac{1}{2,736}$
From Windsor to Staines Bridge	8	0	15	8	1.96	$\frac{1}{2,694}$
From Staines to Chertsey Bridge	4	6	6	6	1.44	$\frac{1}{3,867}$
From Chertsey to Teddington-Lock	13	6	19	8	1.45	$\frac{1}{3,411}$
From Teddington-Lock to London Bridge	19	0	2	9	.145	$\frac{1}{36,414}$
From London to Yanlet Creek	40	0	2	1	.052	$\frac{1}{101,537}$
From Lechdale to Yanlet Creek	180	4	218	0		
Deduct	40	0				
From Lechdale to London	146	4				

For enclosed channels, the circular form of sewer will have the largest scouring power, at a given hydraulic inclination. For the area of the sections being the same, the velocity in the circular channel will be a maximum. When the supply is intermittent, and the channel too large, the egg-shaped form

* Report, for 1834, of the British Association.

with the smaller end for the bottom,—or the sides vertical with an inverted ridge-tile or V bottom for drains,—will have a hydrostatic flushing power to remove soil and obstructions, which a cylindrical channel, only partly full, does not possess; because a given quantity of water rises higher against the same obstruction, or obstacle, to the flow in the pipe. It must be confessed, however, that for small drains and house-sewage, this gain is immaterial, and is at best but effected by a sacrifice of space, material, and friction in the upper part of drains, from 6 to 12 inches in diameter. Besides this, the mere hydrostatic pressure is only intermittent, and during an ordinary, or heavy, fall of rain, the hydrodynamic power is always more efficient in scouring properly-proportioned cylindrical drains; and the workmanship in the form and joints is less imperfect than for more compound forms, as those with egg-shaped and inverted tile bottoms. The moulds and joints of cylindrical stone-ware drains, exceeding 12 inches in diameter, are seldom, however, in large quantities perfect; and the expense will exceed that of brick, stone, or other sufficient drains in most localities.

As to the increased discharging power which it is asserted by some, stone-ware cylindrical drains possess over other ordinary drains, no doubt it is true for small sizes, because the form, jointing, and surface are in general more smooth and circular; and *for sewage matter*,* the friction and adherence to the sides and bottom is less; any advantage from these causes becomes, however, immaterial for the larger

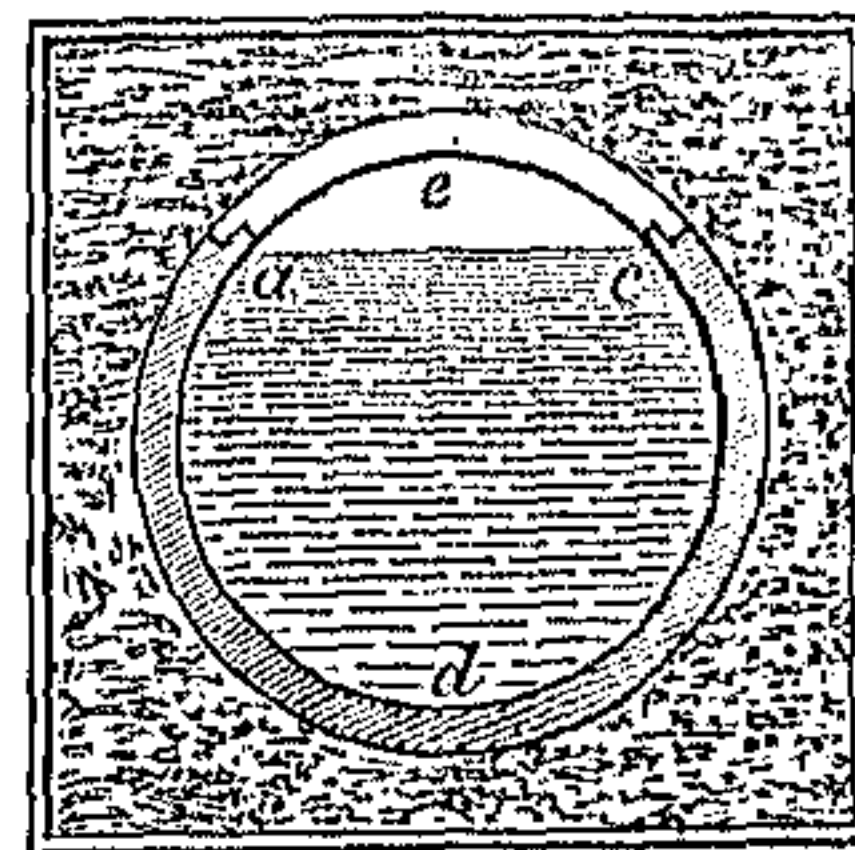
* Weisbach found the coefficient of resistance 1.75 times as great for small wooden as for metallic pipes. All permeable pipes

sizes, as these can be constructed of brick or stone abundantly perfect to any form, and sufficiently smooth for all practical purposes, for in the larger properly-proportioned sizes the same amount of surface roughness opposed to the sewage matter is, comparatively, of no effect. The judicious inclination and form of the bottom, and properly curved junctions, are the principal points to be attended to. Smaller drains tile-bottomed, with brick or stone sides, and flat-covered, have one great advantage over circular pipes.* They can be opened up, for examination and repairs at any time with facility, and at the smallest expense; but greater certainty must be attached to the working of *small* stone-ware drains than to equally-sized small brick or stone drains, and they will be found, in general, also cheaper. This, however will depend on the locality.

It may be observed in numerous experiments, that water flowing from a pipe does not entirely fill the orifice of exit, when the velocities are not considerable, and yet the results are found to be but slightly affected if a little more than three-fourths of the circumference be full. It is easy to demonstrate that the full circle does not give the maximum discharging velocity as has been generally believed, but present greater resistance than *impermeable* ones; hence the principal advantage derived from glazing.

* Half-socket joints at bottom would remedy this imperfection in small pipes, and they could be better laid and cemented. A semicircular flange laid on at top would effectually protect the joint on the upper side. Latterly Doulton has cut off an upper segment from the pipe, which can be removed for cleaning. And it may be demonstrated, that when this is a segment of $78\frac{1}{2}$ degrees, the lower portion will discharge more than a full pipe at the same inclination.

Fig. 84a.



when filled to the height of the chord ac of arc aec of $78\frac{1}{2}$ degrees, and where the velocity is $9\frac{1}{2}$ per cent. over that due to the full circle, for then the $\frac{\text{area } adc}{\text{arc } ade}$ is a maxi-

mum, and the length of the arc adc is equal to the tangent of the supplemental arc aec , as may be without difficulty demonstrated. The hydraulic mean depths of the circle and larger segment are to each other as $\cdot 5$ to $\cdot 6$, and their square roots, which are as the velocities or scouring powers, are as 1 to $1\cdot 095$. The discharging powers are to each other as $1 \times 3\cdot 1416$ to $1\cdot 095 \times 2\cdot 946$, or as 1 to $1\cdot 026$, which shews that the segment adc has also a greater discharging power than the whole circle of nearly three per cent. These facts, which were first pointed out by the author, are not unimportant in matters connected with pipe-drains and sewerage. The effects of greater velocity and discharge here pointed out, are sometimes increased, in short pipes, from the fall between the surface ac , and the surface from which the head is measured, being greater than the fall to the top of the pipe at e , or from the inclination of the surface of the water in the pipe being greater than the inclination of the pipe itself.

EQUALLY DISCHARGING CHANNELS.

In order that different channels should have the same discharging power, the inclination of the surface being the same, the areas must be inversely as

the square roots of the hydraulic mean depths. The channel $adcb$, Fig. 35, will have the same discharge as the channel $ADCB$ if they be to each other

as $\left\{ \frac{ADCB}{AD + DC + CB} \right\}^{\frac{1}{2}}$ to $\left\{ \frac{adcb}{ad + dc + cb} \right\}^{\frac{1}{2}}$,



and hence the square root of the cube of the channel area, divided by the border, must be constant. With a fall of one or more feet to a mile, two channels, one 70 feet wide and 1 foot deep, and the other 20 feet wide and $2\frac{1}{2}$ feet deep, will have the same discharge. If we put w for the width and d for the depth of any rectangular channel, then

$\left\{ \frac{w^3 d^3}{w + 2d} \right\}^{\frac{1}{2}} = m$; we therefore have the cubic equation

$$(122.) \quad d^3 - \frac{2m^2}{w^3}d = \frac{m^2}{w^2}$$

for finding the depth d of any other rectangular channel whose width is w , of the same discharging power. We have calculated the depths d for different widths of channel from this equation, assuming a width of 70 feet and different depths to find m from. The results are given in TABLE XI., which will be found sufficiently accurate for all practical purposes, when the banks are sloped, by taking the mean width. This table is equally applicable to any measures whatever, to their multiples, and sub-multiples.

If the hydraulic inclinations vary, then the \sqrt{rs} must be inversely as the areas of the channels when $\sqrt{rs} \times \text{channel}$ or the discharge is constant; and if

the area of the channel and discharge be each constant, r must vary inversely as s ; and $r s$ be also constant. For instance, a channel which has a fall of four feet per mile, and a hydraulic mean depth of one foot, will have the same discharge as another channel of equal area, having a hydraulic mean depth of four feet, and a fall per mile of only one foot. If in TABLE XII. we take the same discharge from the columns for different inclinations, we shall get the mean rectangular dimensions corresponding to them in the first column, and thereby be enabled to select an equally discharging channel from TABLE XI., suited to an increase or decrease of the hydraulic inclination.*

We have, however, calculated for this edition the table at p. 252, of equally discharging river channels, with a primary channel having a mean width of 100, instead of 70, as in TABLE XI.; and in the table at p. 253 we have given the discharges at different inclinations from this new primary channel, to find those from its equivalents. The tables at pp. 42, 43, 253, and TABLE VIII., have been calculated from Du Buât's formula. For slow velocity of only a few inches per second, the dimensions should be increased by about one-sixth, and the discharges by about one-half.

With reference to pipes, it is apparent that a given depth of roughness or contraction arising from any

* Tables similar to numbers XI., XII., and XIII., but on a much more extended scale, have been printed and published by Mr. WEALE, on a separate sheet for office use, and may be had from him.

cause will have a greater effect the smaller the diameter becomes. Now in practice, it is necessary to increase the diameter beyond what is found by calculation. For small service pipes half-an-inch is the smallest diameter in general use. For mains and sub-mains the value of c , in equation (74b), or at p. 214, should at least be doubled, or the discharge taken at one and a half times its amount to find the diameter. By enlarging the diameter by one-seventh, one-half the amount will be added to the discharge, very nearly; and by increasing the diameter by one-third, the discharge will be doubled. In a broad and practical sense, and considering the losses arising from depositions,* pipes under two inches should have one-third or more added to their calculated dimensions, and larger pipes from one-third to one-seventh—even after making allowance for junctions, bends, and contractions. For large conduits or channels the allowance need not be so large, if the maximum quantity to be conveyed be duly estimated.

* Mr. Bateman lately in giving evidence says:—"He wished to mention a circumstance which might be useful with regard to the spongillæ found in the Dublin water pipes. At Manchester, before the introduction of soft water, the city was supplied with hard water, which favoured the growth of a small fresh-water mussel, which thickly lined the reservoirs and pipes. There were myriads of them, and they lay in the pipes as thick as paving stones. These were caused by the large quantity of lime in the water. He was curious to see what would be the effect of passing water without lime. This was done ten or eleven years ago, and the result was that these mussels had entirely disappeared. There was no longer anything from which they could make their shells, and for years, on their discharge, the small pipes were found choked with them. If soft water were supplied to Dublin in place of the present hard water, which probably favoured the growth of spongillæ, they would probably disappear."

TABLE of mean widths and depths of equally discharging trapezoidal River-channels, or Sewers, with side slopes up to 2½ to 1. Practically all river-channels may be reduced to rectangular sections of equal areas and depths to find the discharge.

Primary Channel	Mean rectangular dimensions of equally discharging water-channels or sewers, in any measures whatever, inches, feet, yards, fathoms, or their aliquot parts, or multiples.										Primary Channel
Mean width 100	Mean width 90	Mean width 80	Mean width 70	Mean width 60	Mean width 50	Mean width 40	Mean width 30	Mean width 20	Mean width 15	Mean width 10	Mean width 100
.1	.11	.12	.13	.14	.16	.18	.22	.20	.35	.47	.1
.125	.13	.14	.16	.17	.20	.23	.28	.37	.45	.60	.125
.2	.21	.23	.25	.28	.32	.37	.45	.60	.73	.98	.2
.25	.27	.29	.32	.35	.40	.46	.50	.75	.92	1.25	.25
.3	.32	.35	.38	.42	.48	.56	.68	.90	1.11	1.52	.3
.375	.40	.44	.48	.53	.60	.70	.85	1.13	1.40	1.94	.375
.4	.43	.46	.51	.56	.64	.74	.91	1.21	1.50	2.08	.4
.5	.54	.58	.64	.71	.80	.93	1.14	1.53	1.90	2.67	.5
.6	.64	.70	.76	.85	.96	1.12	1.37	1.85	2.31	3.28	.6
.625	.67	.73	.79	.88	1.00	1.16	1.43	1.93	2.42	3.44	.625
.7	.75	.81	.89	.99	1.12	1.31	1.61	2.17	2.73	3.92	.7
.75	.80	.87	.95	1.06	1.20	1.41	1.73	2.34	2.95	4.25	.75
.8	.86	.93	1.02	1.13	1.29	1.50	1.85	2.51	3.17	4.59	.8
.875	.94	1.02	1.12	1.24	1.40	1.64	2.02	2.76	3.50	5.10*	.875
.9	.97	1.05	1.15	1.27	1.45	1.69	2.08	2.84	3.61	5.28	.9
1.0	1.07	1.16	1.27	1.42	1.61	1.88	2.32	3.18	4.07	5.99	1.0
1.125	1.21	1.31	1.43	1.60	1.81	2.13	2.63	3.62	4.64	6.92	1.125
1.2	1.29	1.40	1.53	1.70	1.94	2.27	2.81	3.88	5.00	7.50	1.2
1.25	1.35	1.46	1.60	1.78	2.02	2.37	2.94	4.00	5.24	7.80	1.25
1.3	1.40	1.51	1.66	1.85	2.10	2.47	3.06	4.24	5.48	8.29	1.3
1.375	1.48	1.60	1.76	1.96	2.23	2.62	3.25	4.51	5.85	8.89	1.375
1.4	1.50	1.63	1.79	1.99	2.27	2.66	3.31	4.60	5.97	9.10	1.4
1.5	1.61	1.75	1.92	2.14	2.43	2.86	3.56	4.97	6.47	9.92	1.5
1.6	1.72	1.86	2.05	2.28	2.60	3.06	3.81	5.34	6.98	10.78	1.6
1.625	1.75	1.89	2.08	2.32	2.64	3.11	3.87	5.43	7.11	11.00	1.625
1.7	1.83	1.98	2.17	2.43	2.76	3.26	4.06	5.72	7.50*	11.66	1.7
1.75	1.88	2.04	2.24	2.50	2.85	3.36	4.19	5.91	7.77	12.10	1.75
1.8	1.93	2.10	2.30	2.57	2.93	3.45	4.32	6.09	8.03	12.54	1.8
1.875	2.02	2.19	2.40	2.68	3.05	3.60	4.51	6.38	8.43	13.23	1.875
1.9	2.04	2.22	2.43	2.71	3.10	3.65	4.57	6.48	8.57	13.46	1.9
2.0	2.15	2.33	2.56	2.86	3.26	3.86	4.83	6.87	9.11	14.30	2.0
2.1	2.26	2.45	2.69	3.01	3.43	4.06	5.09	7.27	9.67	15.35	2.1
2.2	2.37	2.57	2.82	3.15	3.60	4.26	5.36	7.66	10.23	16.32	2.2
2.3	2.47	2.69	2.95	3.30	3.77	4.44	5.62	8.07	10.80	17.31	2.3
2.4	2.58	2.80	3.08	3.44	3.94	4.67	5.89	8.48	11.38	18.33	2.4
2.5	2.69	2.92	3.21	3.59	4.11	4.87	6.16	8.79	11.97	19.35	2.5
2.6	2.80	3.04	3.34	3.74	4.28	5.08	6.42	9.31	12.57	20.40	2.6
2.7	2.91	3.16	3.47	3.88	4.55	5.28	6.69	9.73	13.17	21.46	2.7
2.8	3.01	3.27	3.60	4.03	4.62	5.49	6.97	10.16*	13.78	22.52	2.8
2.9	3.12	3.39	3.73	4.18	4.79	5.70	7.24	10.59	14.40	23.63	2.9
3.0	3.23	3.51	3.86	4.32	4.96	5.91	7.52	11.02	15.03	24.75	3.0
3.1	3.34	3.63	3.99	4.47	5.13	6.12	7.79	11.46	15.68		3.1
3.2	3.45	3.75	4.13	4.62	5.30	6.33	8.07	11.90	16.32		3.2
3.3	3.55	3.86	4.26	4.77	5.48	6.54	8.35	12.35	16.97		3.3
3.4	3.66	3.98	4.39	4.92	5.65	6.75	8.64	12.80	17.63		3.4
3.5	3.77	4.10	4.52	5.06	5.82	6.96	8.92	13.26	18.29		3.5
3.6	3.88	4.22	4.65	5.21	6.00	7.18	9.21	13.71	18.96		3.6
3.7	3.99	4.34	4.78	5.36	6.17	7.39	9.49	14.18	19.65		3.7
3.8	4.09	4.46	4.91	5.51	6.35	7.60	9.78	14.65	20.34		3.8
3.9	4.20	4.58	5.05	5.66	6.52	7.82	10.07	15.12			3.9
4.0	4.31	4.69	5.18	5.81	6.70	8.04	10.36	15.59			4.0
4.1	4.42	4.81	5.31	5.96	6.87	8.25	10.66	16.07			4.1
4.2	4.53	4.93	5.44	6.11	7.05	8.47	10.95	16.55			4.2
4.3	4.64	5.05	5.57	6.26	7.23	8.69	11.25	17.04			4.3
4.4	4.74	5.17	5.71	6.41	7.40	8.91	11.55	17.53			4.4
4.5	4.85	5.29	5.84	6.56	7.58	9.13	11.85	18.02			4.5
4.6	4.96	5.41	5.97	6.72	7.76	9.35	12.15	18.52			4.6
4.7	5.07	5.53	6.10	6.87	7.94	9.57	12.45	19.02			4.7
4.8	5.18	5.64	6.24	7.02	8.12	9.79	12.75	19.53			4.8
4.9	5.29	5.76	6.37	7.17	8.29	10.02	13.06	20.04			4.9
5.0	5.40	5.88	6.50	7.32	8.47	10.24	13.37				5.0

* A semi-hexagon has the greatest discharge power of all trapezoids of equal depth and area. All rectangles have the same discharging power as trapezoids of the same depth and area, with side slopes of 1 to 1. The differences for other side slopes are immaterial in practice.

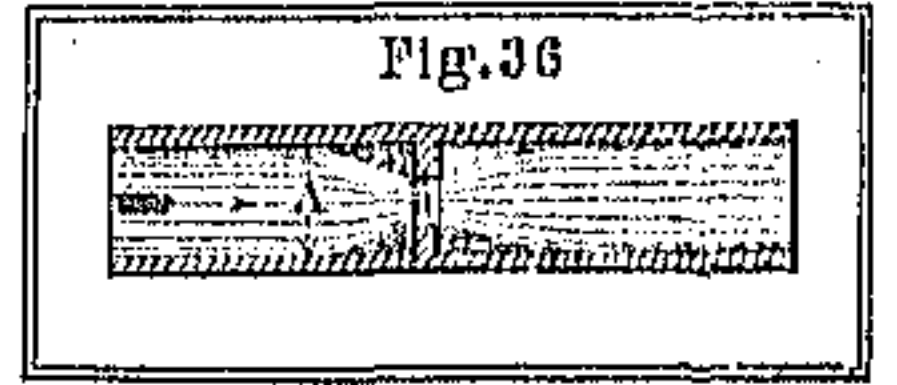
TABLE of the Discharges in cubic feet per minute from the primary Channel in the opposite page, taken in feet; and from the corresponding equivalent Channels, also taken in feet.

Discharges in cubic feet per minute. Interpolate for intermediate falls or inclinations; divide greater falls or inclinations by 4, and double the corresponding discharges. If the dimensions be in inches, divide the discharges here given by 64; if in tenths, by 100; if in eighths, by 128; if in sixths, by 16; if in fifths, by 25; if in quarters, by 32; if in thirds, by 48; and if in halves, by 96. Reverse the operation and multiply for like multiples.											
Falls or inclinations in feet.	Equivalent Channels in inches.										
	4 inches per mile 1 in 16840.	6 inches per mile 1 in 10500.	8 inches per mile 1 in 7010.	10 inches per mile 1 in 5250.	12 inches per mile 1 in 4375.	14 inches per mile 1 in 3500.	16 inches per mile 1 in 2812.5.	18 inches per mile 1 in 2240.	20 inches per mile 1 in 1792.	22 inches per mile 1 in 1418.	24 inches per mile 1 in 1100.
1	110	140	176	207	235	260	281	305	325	343	358
1.25	157	198	250	291	332	373	402	433	459	480	500
2	225	285	356	408	460	512	553	594	635	676	717
2.5	275	345	426	488	549	610	671	732	793	854	915
3	325	405	496	568	639	710	781	852	913	974	1035
3.75	375	465	566	638	709	780	851	922	983	1044	1105
4	425	525	626	708	779	850	921	992	1053	1114	1175
4.5	475	585	686	768	839	910	981	1052	1113	1174	1235
5	525	635	736	818	889	960	1031	1102	1163	1224	1285
5.25	575	685	786	868	939	1010	1081	1152	1213	1274	1335
5.5	625	735	836	918	989	1060	1131	1202	1263	1324	1385
5.75	675	785	886	968	1039	1110	1181	1252	1313	1374	1435
6	725	835	936	1018	1089	1160	1231	1302	1363	1424	1485
6.25	775	885	986	1068	1139	1210	1281	1352	1413	1474	1535
6.5	825	935	1036	1118	1189	1260	1331	1402	1463	1524	1585
6.75	875	985	1086	1168	1239	1310	1381	1452	1513	1574	1635
7	925	1035	1136	1218	1289	1360	1431	1502	1563	1624	1685
7.25	975	1085	1186	1268	1339	1410	1481	1552	1613	1674	1735
7.5	1025	1135	1236	1318	1389	1460	1531	1602	1663	1724	1785
7.75	1075	1185	1286	1368	1439	1510	1581	1652	1713	1774	1835
8	1125	1235	1336	1418	1489	1560	1631	1702	1763	1824	1885
8.25	1175	1285	1386	1468	1539	1610	1681	1752	1813	1874	1935
8.5	1225	1335	1436	1518	1589	1660	1731	1802	1863	1924	1985
8.75	1275	1385	1486	1568	1639	1710	1781	1852	1913	1974	2035
9	1325	1435	1536	1618	1689	1760	1831	1902	1963	2024	2085
9.25	1375	1485	1586	1668	1739	1810	1881	1952	2013	2074	2135
9.5	1425	1535	1636	1718	1789	1860	1931	2002	2063	2124	2185
9.75	1475	1585	1686	1768	1839	1910	1981	2052	2113	2174	2235
10	1525	1635	1736	1818	1889	1960	2031	2102	2163	2224	2285
10.25	1575	1685	1786	1868	1939	2010	2081	2152	2213	2274	2335
10.5	1625	1735	1836	1918	1989	2060	2131	2202	2263	2324	2385
10.75	1675	1785	1886	1968	2039	2110	2181	2252	2313	2374	2435
11	1725	1835	1936	2018	2089	2160	2231	2302	2363	2424	2485
11.25	1775	1885	1986	2068	2139	2210	2281	2352	2413	2474	2535
11.5	1825	1935	2036	2118	2189	2260	2331	2402	2463	2524	2585
11.75	1875	1985	2086	2168	2239	2310	2381	2452	2513	2574	2635
12	1925	2035	2136	2218	2289	2360	2431	2502	2563	2624	2685
12.25	1975	2085	2186	2268	2339	2410	2481	2552	2613	2674	2735
12.5	2025	2135	2236	2318	2389	2460	2531	2602	2663	2724	2785
12.75	2075	2185	2286	2368	2439	2510	2581	2652	2713	2774	2835
13	2125	2235	2336	2418	2489	2560	2631	2702	2763	2824	2885
13.25	2175	2285	2386	2468	2539	2610	2681	2752	2813	2874	2935
13.5	2225	2335	2436	2518	2589	2660	2731	2802	2863	2924	2985
13.75	2275	2385	2486	2568	2639	2710	2781	2852	2913	2974	3035
14	2325	2435	2536	2618	2689	2760	2831	2902	2963	3024	3085
14.25	2375	2485	2586	2668	2739	2810	2881	2952	3013	3074	3135
14.5	2425	2535	2636	2718	2789	2860	2931	3002	3063	3124	3185
14.75	2475	2585	2686	2768	2839	2910	2981	3052	3113	3174	3235
15	2525	2635	2736	2818	2889	2960	3031	3102	3163	3224	3285
15.25	2575	2685	2786	2868	2939	3010	3081	3152	3213	3274	3335
15.5	2625	2735	2836	2918	2989	3060	3131	3202	3263	3324	3385
15.75	2675	2785	2886	2968	3039	3110	3181	3252	3313	3374	3435
16	2725	2835	2936	3018	3089	3160	3231	3302	3363	3424	3485
16.25	2775	2885	2986	3068	3139	3210	3281	3352	3413	3474	3535
16.5	2825	2935	3036	3118	3189	3260	3331	3402	3463	3524	3585
16.75	2875	2985	3086	3168	3239	3310	3381	3452	3513	3574	3635
17	2925	3035	3136	3218	3289	3360	3431	3502	3563	3624	3685
17.25	2975	3085	3186	3268	3339	3410	3481	3552	3613	3674	3735
17.5	3025	3135	3236	3318	3389	3460	3531	3602	3663	3724	3785
17.75	3075	3185	3286	3368	3439	3510	3581	3652	3713	3774	3835
18	3125	3235	3336	3418	3489	3560	3631	3702	3763	3824	3885
18.25	3175	3285	3386	3468	3539	3610	3681	3752	3813	3874	3935
18.5	3225	3335	3436	3518	3589	3660	3731	3802	3863	3924	3985
18.75	3275	3385	3486	3568	3639	3710	3781	3852	3913	3974	4035
19	3325	3435	3536	3618	3689	3760	3831	3902	3963	4024	4085
19.25	3375	3485	3586	3668	3739	3810	3881	3952	4013	4074	4135
19.5	3425	3535	3636	3718	3789	3860	3931	4002	4063	4124	4185
19.75	3475	3585	3686	3768	3839	3910	3981	4052	4113	4174	4235
20	3525	3635	3736	3818	3889	3960	4031	4102	4163	4224	4285
20.25	3575	3685	3786	3868	3939	4010	4081	4152	4213	4274	4335
20.5	3625	3735	3836	3918	3989	4060	4131	4202	4263	4324	4385
20.75	3675	3785	3886	3968	4039	4110	4181	4252	4313	4374	4435
21	3725	3835	3936	4018	4089	4160	4231	4302	4363	4424	4485
21.25	3775	3885	3986	4068	4139	4210	4281	4352	4413	4474	4535
21.5	3825	3935	4036	4118	4189	4260	4331	4402	4463	4524	4585
21.75	3875	3985	4086	4168	4239	4310	4381	4452	4513	4574	4635
22	3925	4035	4136	4218	4289	4360	4431	4502	4563	4624	4685
22.25	3975	4085	4186	4268	4339	4410	4481	4552	4613	4674	4735
22.5	4025	4135	4236	4318	4389	4460	4531	4602	4663	4724	4785
22.75	4075	4185	4286	4368	4439	4510	4581	4652	4713	4774	4835
23	4125	4235	4336	4418	4489	4560	4631	4702	4763	4824	4885
23.25	4175	4285	4386	4468	4539	4610	4681	4752	4813	4874	4935
23.5	4225	4335	4436	4518	4589	4660	4731	4802	4863	4924	4985
23.75	4275	4385	4486	4568	4639	4710	4781	4852	4913	4974	5035
24	4325	4435	4536	4618	4689	4760	4831	4902	4963	5024	5085
24.25	4375	4485	4586	4668	4739	4810	4881	4952	5013	5074	5135
24.5	4425	4535	4636	4718	4789	4860	4931	5002	5063	5124	5185
24.75	4475	4585	4686	4768	4839	4910	4981	5052	5113	5174	5235
25	4525	4635	4736	4818	4889	4960	5031	5102	5163	5224	5285
25.25	4575	4685	4786	4868	4939	5010					

SECTION X.

EFFECTS OF ENLARGEMENTS AND CONTRACTIONS.—BACKWATER
WEIR CASE.—LONG AND SHORT WEIRS.

When the flowing section in pipes or rivers expands or contracts suddenly, a loss of head always ensues; this is probably expended in forming eddies at the sides, or in giving the water its new section. A side current, moving slowly *upwards*, may be frequently observed in the wide parts of rivers, when the channel is unequal, though the downward current, at the centre, be pretty rapid; and though we may assume generally that the velocities are inversely as the sections, when the channels are uniform, we cannot properly do so when they are not, and the motions so uncertain as those referred to. When a pipe is contracted by a diaphragm at the orifice of entry, Fig. 27, we have seen (equation 60), that the loss of head is,



$$(123.) \quad h = \frac{\left(1 - \frac{A^2}{C^2}\right)v^2 + \left(\frac{A}{a c_d} - 1\right)^2 v^2}{2g}$$

When the diaphragm is placed in a uniform pipe, Fig. 36, then $A = C$, and we get the loss of head

$$(124.) \quad h = \frac{\left(\frac{A}{a c_d} - 1\right)^2 v^2}{2g},$$

and the coefficient of resistance

$$(125.) \quad c_r = \left(\frac{A}{a c_d} - 1\right)^2,$$

as in equation (67). The coefficient of discharge c_d is

here equal to the coefficient of contraction c_c , or very nearly. Now we have shown in equation (45), and the remarks following it, that the value of the coefficient of discharge, c_d , varies according to the ratio of the sections, $\frac{A}{a}$,* and in TABLE V. we have calculated the new coefficients for different values of the ratios, and different values of the primary coefficient c_a . If we assume c_a , when A is very large compared with a , to be $\cdot 628$, we then find by attending to the remarks at pp. 109 and 128, that the different values of c_d corresponding to $\cdot 807 \times \frac{A}{a}$, taken from TABLE V., are those in columns Nos. 2 and 5 of the next small

TABLE OF COEFFICIENTS FOR CONTRACTION, BY A DIAPHRAGM IN A PIPE.

$\frac{a}{A}$	c_d	c_r	$\frac{a}{A}$	c_d	c_r
$\cdot 0$	$\cdot 028$	infinite	$\cdot 0$	$\cdot 713$	$1\cdot 700$
$\cdot 1$	$\cdot 030$	$221\cdot 2$	$\cdot 7$	$\cdot 759$	$\cdot 807$
$\cdot 2$	$\cdot 030$	$47\cdot 1$	$\cdot 8$	$\cdot 807$	$\cdot 301$
$\cdot 3$	$\cdot 047$	$17\cdot 2$	$\cdot 85$	$\cdot 815$	$\cdot 154$
$\cdot 4$	$\cdot 061$	$7\cdot 7$	$\cdot 9$	$\cdot 890$	$\cdot 002$
$\cdot 5$	$\cdot 083$	$3\cdot 7$	1	$1\cdot 000$	$\cdot 000$

* The general value of c_c , as given by Professor Rankine, is $\cdot 618$
 $c_c = \frac{\cdot 618}{\left(1 - \cdot 618 \frac{a^2}{A^2}\right)^{\frac{1}{2}}}$, which is equal to unity when $a = A$, as it should be; and equal to $\cdot 618$, when a is very small, compared with A , as it also should be when the diaphragm is a thin plate, *but not otherwise*. If the thickness of the diaphragm be twice the diameter of the orifice a , the coefficient of discharge would be $\cdot 815$; and if the higher arris be rounded, this would be increased to 1, in which cases the expression would clearly fail; the thickness of the diaphragm and the form of the aperture a must also be considered.

table, the values of the coefficient of resistance, in columns 3 and 6, being calculated from equation (125) for the respective new values of the coefficient of discharge thus found. The table shows that when the aperture in a diaphragm is $\frac{1}{16}$ ths of the section of the pipe, that 47 times the head due to the velocity is lost thereby. If the aperture in the diaphragm be rounded at the arrises, the loss will not be so great, as the primary coefficient c_d will then be greater than that due to an orifice in a thin plate: see coefficients, p. 174.

When there are a number of diaphragms in a tube, the loss of head for each must be found separately, and all added together for the total loss. If the diaphragms, however, approach each other, so that the water issuing from one of the orifices a , Fig. 36, shall pass into the next before it again takes the velocity due to the diameter of the pipe, the loss will not be so great as when the distance is sufficient to allow this change to take place. This view is fully borne out by the experiments of Eytelwein with tubes 1.03 inch in diameter, having apertures in the diaphragms of .51 inch in diameter.

Venturi's twenty-fourth experiment, with tubes varying from .75 inch to .934 inch in diameter at the junction with the cistern, so as to take the form of the contracted vein, and expanding and contracting along the length from .75 to 2 inches and from 2 inches to .75 inch alternately, shows the great loss of head sustained by successive enlargements and contractions of a channel, even when the junction of the parts is gradual. Calling the coefficient for the short

tube, with a junction of nearly the form of the contracted vein, 1, then the following coefficients are derivable from the experiment :—

Short tube with rounded junction	1.
One enlargement741
Three enlargements609
Five enlargements454
Simple tube with a rounded junction of the same length, 36 inches, as the tube with the five enlarged parts730

The head, in the experiment, was $32\frac{1}{2}$ inches. Venturi states that no observable differences occurred in the times of discharge when the enlarged portions were lengthened from $3\frac{1}{2}$ to $6\frac{1}{2}$ inches. See tables, pp. 152 and 191.

With reference to this experiment, so often quoted, it is necessary to remark that the diameters of the enlarged portions were 2 inches each, while the lengths varied only from $3\frac{1}{2}$ to $6\frac{1}{2}$ inches, and consequently were at most only $3\frac{1}{2}$ times the diameter. Now with

such a large ratio of the width to the length of the enlarged portions, *a A B b*, Fig. 37, it is pretty clear that a good deal of the head



is lost by the impact of the moving water on the shoulders *A* and *B*. That this is so is evident from the fact, stated by the experimenter, of the time of discharge remaining the same when *a A*, in five different enlargements, was increased from $3\frac{1}{2}$ to $6\frac{1}{2}$ inches; though this must have lengthened the whole

tube from 36 to 50 inches,* thereby increasing the loss from friction proportionately, but which happened to be compensated for by the reduction in the resistances from impact at A and B, and in the eddies, by doubling the lengths from a to A.

If, however, the length from a to A be very large compared with the diameter, and the junctions at a , A, B, and b , be well grafted, less loss will arise from the enlargement than if the smaller diameter continued all along uniform. The explanation is clear, as the resistance from friction is inversely as the square roots of the mean radii; and the length being the same, the loss must be less with a large than a small diameter.

These remarks, *mutatis mutandis*, apply equally to rivers as to pipes. We have already, pp. 140 and 147, pointed out the effects of submerged weirs and contracted river channels, and given formulæ for calculating them.

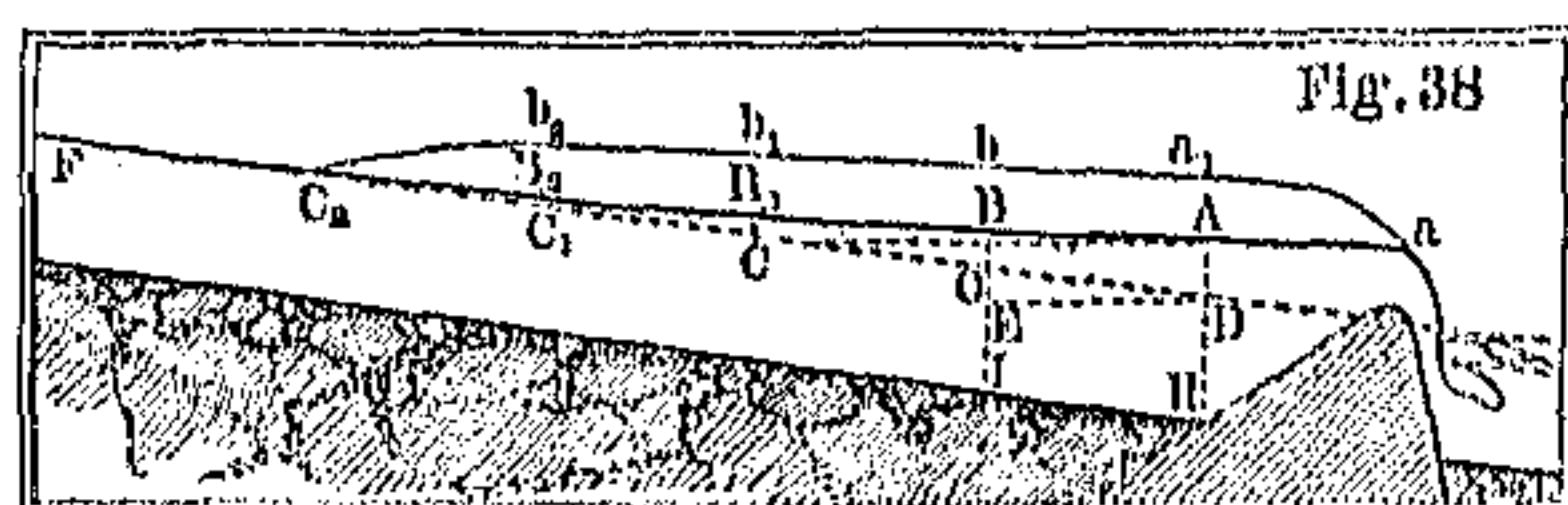
BACKWATER FROM CONTRACTIONS IN RIVERS.

A river may be contracted in width or depth, by jetties or by weirs; and when the quantity to be discharged is known, we have given, in formulæ (9), (55), and (57), equations from which the increase of head may be determined. The effect of a weir, jetty, or contracted channel of any kind, is to increase the depth of water above; and this is sometimes necessary for navigation purposes, or to obtain a head for mill power. When a weir is to rise over the surface, we can easily find, from the discharge and length, the discharge per minute over each foot of length, with which, and the coefficient due to the ratio of the

* The dimensions throughout this experiment are given as in the original, viz. in French inches.

sections, on and above the weir, found from TABLE V., we can find the head from TABLE VI. For submerged weirs and contracted widths of channel, the head can be best calculated, by approximation, from the equations above referred to.

The head once determined, the extent of the backwater is a question of some importance. If $F C O D$, Fig. 38, be the original surface of a river, and $a A B F$ the raised surface by backwater from the weir at a , then the extent $a F$ of this backwater, in a regular channel, will be from 1.5 to 1.9 times $a c$ drawn parallel to the horizon to meet the original surface in c . This rule



will be found useful for practical purposes; but in order to determine more accurately the rise for a given length, $B_1 B_2$ or $B_1 B$, of the channel, it is necessary to commence at the weir and calculate the heights from A to B , B to B_1 , and from B_1 to B_2 separately, the distance from A to B_2 being supposed divided into some convenient number of equal parts, so that the lengths AB , BB_1 , &c., may be considered free from curvature. Now, as the head AD is known, or may be calculated by some of the preceding formulæ, the section of the channel at the head of the weir also becomes known, and thence the mean velocity in it, by means of the discharge over the weir. Putting A for the area of the channel at AD , d for its depth AD , and v for the mean velocity; also A_1 for

the area of the channel at B I, d_1 for its depth, and v_1 for its mean velocity; b_m the mean border between the sections at A H and B I; r_m the mean hydraulic depth; $\frac{v + v_1}{2}$ the mean velocity; $AD = h$; $BO = h_1$; the sine of angle O D E $= s$; and the length AB $= DO$ nearly $= l$; we get $A \times v = A_1 \times v_1$ and $r_m = \frac{A + A_1}{2 b_m}$; but as, in passing from B to A, the velocity changes from v_1 to v , there is a loss of head equal $\frac{v_1^2 - v^2}{2g}$, and if c_f be the coefficient of friction, there is a loss of head from this cause equal $c_f \times \frac{l}{r_m} \times \frac{(\frac{v_1 + v}{2})^2}{2g}$; hence the whole change of head in passing from B to A is equal to $c_f \times \frac{l}{r_m} \times \frac{(v_1 + v)^2}{8g} - \frac{v_1^2 - v^2}{2g}$. But this change of head is equal to BE $- AD = BO + OE - AD = h_1 + ls - h$, whence we get

$$(126.) \quad h_1 - h = d_1 - d = c_f \times \frac{l}{r_m} \times \frac{(v_1 + v)^2}{8g} - \frac{v_1^2 - v^2}{2g} - ls;$$

or as $v_1 = \frac{A v}{A_1}$, and $r_m = \frac{A + A_1}{2 b_m}$, we get, by a few reductions and change of signs,

$$(127.) \quad h - h_1 = \left(s - c_f \times b_m \times \frac{A + A_1}{2 A_1^2} \times \frac{v^2}{2g} \right) l + \frac{A^2 - A_1^2}{A^2} \times \frac{v^2}{2g};$$

and therefore we get

$$(128.) \quad l = \frac{h - h_1 - \frac{A^2 - A_1^2}{A_1^2} \times \frac{v^2}{2g}}{s - c_t \times \frac{b_m \times (A + A_1)}{2 A_1^2} \times \frac{v^2}{2g}},$$

from which we can calculate the length l corresponding to any assumed change of level between A and B . Then, by a simple proportion we can find the change of level for any smaller length. To find the change of level directly from a given length does not admit of a direct solution, for the value of $h - h_1$ in equation (127) involves A_1 , which depends again on $h - h_1$, and further reduction leads to an equation of a higher order; but the length corresponding to a given rise, h_1 , is found directly by equation (128).

When the width of the channel, w , is constant, and the section equal to $w \times d$ nearly, the above equations admit of a further reduction for $A_1 = d_1 w$ and $A = d w$; by substituting these values in equation (127) it becomes, after a few reductions,

$$(129.) \quad h - h_1 = d - d_1 \\ = \left(s - c_t \times b_m \times \frac{d \times d_1}{2 d w} \times \frac{v^2}{2g} \right) l + \frac{d^2 - d_1^2}{d_1^2} \times \frac{v^2}{2g};$$

or, as it may be further reduced,

$$(130.) \quad h - h_1 = \frac{s - c_t \times \frac{b_m}{d_1 w} \times \frac{d + d_1}{2 d_1} \times \frac{v^2}{2g}}{1 - \frac{d + d_1}{d_1^2} \times \frac{v^2}{2g}} \times l.$$

Now, we may take in this equation for all practical purposes,

$$\frac{d + d_1}{2 d_1} \times \frac{b_m}{d_1 w} = \frac{b}{d w},$$

approximately, b being the border of the section at

AH; and also, $\frac{d + d_1}{d_1^2} = \frac{2}{d}$, approximately; therefore we shall have

$$(131.) \quad h - h_1 = \frac{s - c_1 \times \frac{b}{dw} \times \frac{v^2}{2g}}{1 - \frac{2}{d} \times \frac{v^2}{2g}} \times l;$$

and

$$(132.) \quad l = \frac{(h - h_1) \times \left(1 - \frac{2}{d} \times \frac{v^2}{2g}\right)}{s - c_1 \times \frac{b}{dw} \times \frac{v^2}{2g}}.$$

Now, as $\frac{b}{dw} = \frac{1}{r}$, $2g = 64.4$, and the mean value of the coefficient of friction for small velocities $c_1 = .0078$, we shall get

$$(133.) \quad h_1 = h - \frac{64.4 ds - .0078 \frac{d}{r} v^2}{64.4 d - 2 v^2} \times l;$$

and

$$(134.) \quad l = \frac{(h - h_1) \times (64.4 d - 2 v^2)}{64.4 ds - .0078 \frac{d}{r} v^2},$$

very nearly. Having by means of these equations found AB from BO or BE, and BO from AB, we can in the same manner proceed up the channel and calculate B₁C, B₂C₁, &c., until the points B, B₁, B₂ in the curve of the backwater shall have been determined, and until the last nearly coincides with the original surface of the river. When $h_1 = 0$, we shall have

$$h = \frac{64.4 ds - .0078 \frac{d}{r} v^2}{64.4 d - 2 v^2} \times l.$$

If we examine equation (134) it appears that when $64.4 \, d = 2 \, v^2$, l must be equal to zero; or when

$\frac{d}{2} = \frac{v^2}{64.4}$, equal the height due to the velocity v .

When l is infinite, $64.4 \, d$ must exceed $2 \, v^2$, and $64.4 \, d s$

equal to $.0078 \frac{d}{r} v^2$;

$$\text{or, } \frac{64.4 \, r \, s}{.0078} = v^2, \text{ and } v = 90.9 \sqrt{r \, s}.$$

This is the velocity due to friction in a channel of the depth d , hydraulic mean depth r , and inclination s ; and, as in wide rivers $r = d$ nearly, $v = 90.9 \sqrt{d \, s}$, but when the numerator was zero we had from it $v = \sqrt{32.2 \, d}$; equating these values of v , we get

$s = .0039 = \frac{1}{256}$ nearly: see p. 139. Now, the larger

the fraction s is, the larger will the velocity v become; and the larger v becomes, the more nearly, in all practical cases, will the terms

$$64.4 \, d - 2 \, v^2 \text{ and } 64.4 \, d \, s - .0078 \frac{d}{r} v^2,$$

in the numerator and denominator of equation (134), approach zero; when $64.4 \, d - 2 \, v^2$ becomes zero first,

$l = 0$; when $64.4 \, d \, s - .0078 \frac{d}{r} v^2$ becomes zero first,

l equals infinity; and when they both become zero

at the same time, $l = h - h_1$, and $s = \frac{1}{256}$, see p. 139;

if s be larger than this fraction, the numerator in equation (134) will generally become zero before the denominator, or negative, in which cases l will also be zero, or negative; and the backwater will take the

form $F c_2 b_2 b_1 b a_1 a$, Fig. 38, with a hollow at c_2 . Bidone first observed a hollow, as $F c_2 b_2$, when the inclination s was $\frac{1}{80}$. When the inclination of a river channel changes from greater to less, the velocity is obstructed, and a hollow similar to $F c_2 b_2$ sometimes occurs; when the difference of velocity is considerable, the upper water at b_2 falls backwards towards c_2 and F , and forms a *bore*, a splendid instance of which is the *pororoca*, on the Amazon, which takes place where the inclination of the surface changes from 6 inches to $\frac{1}{4}$ th of an inch per mile, and the velocity from about 22 feet to $4\frac{1}{2}$ feet per second.

WEIR CASE, LONG AND SHORT WEIRS.

When a channel is of very unequal widths, above a weir, we have found the following simple method of calculating the backwater sufficiently accurate, and the results to agree with observation. *Having ascertained the surface fall due to friction in the channel at a uniform mean section, add to this fall the height which the whole quantity of water flowing down would rise on a weir having its crest on the same level as the lower weir, and of the same length as the width of the channel in the contracted pass. The sum will be the head of water at some distance above such pass very nearly.* A weir was recently constructed on the river Blackwater, at the bounds of the counties Armagh and Tyrone, half a mile below certain mills, which, it was asserted, were injuriously affected by backwater thrown into the wheel-pits. The crest of the weir, 220 feet long, was 2 feet 6 inches below the

pit; the river channel between varied from 50 and 57 feet to 123 feet in width, from 1 foot to 14 feet deep; and the fall of the surface, with 3 inches of water passing over the weir and the sluices down, was nearly 4 inches in the length of half a mile. Having seen the river in this state in summer, the writer had to calculate the backwater produced by different depths passing over the weir in autumn and winter, which in some cases of extraordinary floods were known to rise to 3 feet. The width of the channel about 60 feet above the weir averaged 120 feet. The width, 2050 feet above the weir and 550 feet below the mills, was narrowed by a slip in an adjacent canal bank, to 45 feet at the level of the top of the weir, the average width at this place as the water rose being 55 feet. The channel above and below the slip widened to 80 and 123 feet. Between the mills and the weir there were, therefore, two passes; one at the slip, averaging 55 feet wide; another above the weir, about 120 feet wide. Assuming as above, that the water rises to the heights due to weirs 55 and 120 feet long, at these passes, we get, by an easy calculation, or by means of TABLE X., the heads in columns two and four of the following table, corresponding to the assumed ones on the weir, given in the first column.

As the length of the river was short, and the hydraulic mean depth pretty large, the fall due to friction for 60 feet above the weir was very small, and therefore no allowance was made for it; even the distance to the slip was comparatively short, being less than half a mile, and as the water approached it

TABLE OF CALCULATED AND OBSERVED HEIGHTS ABOVE M'KEAN'S
WEIR ON THE RIVER BLACKWATER.

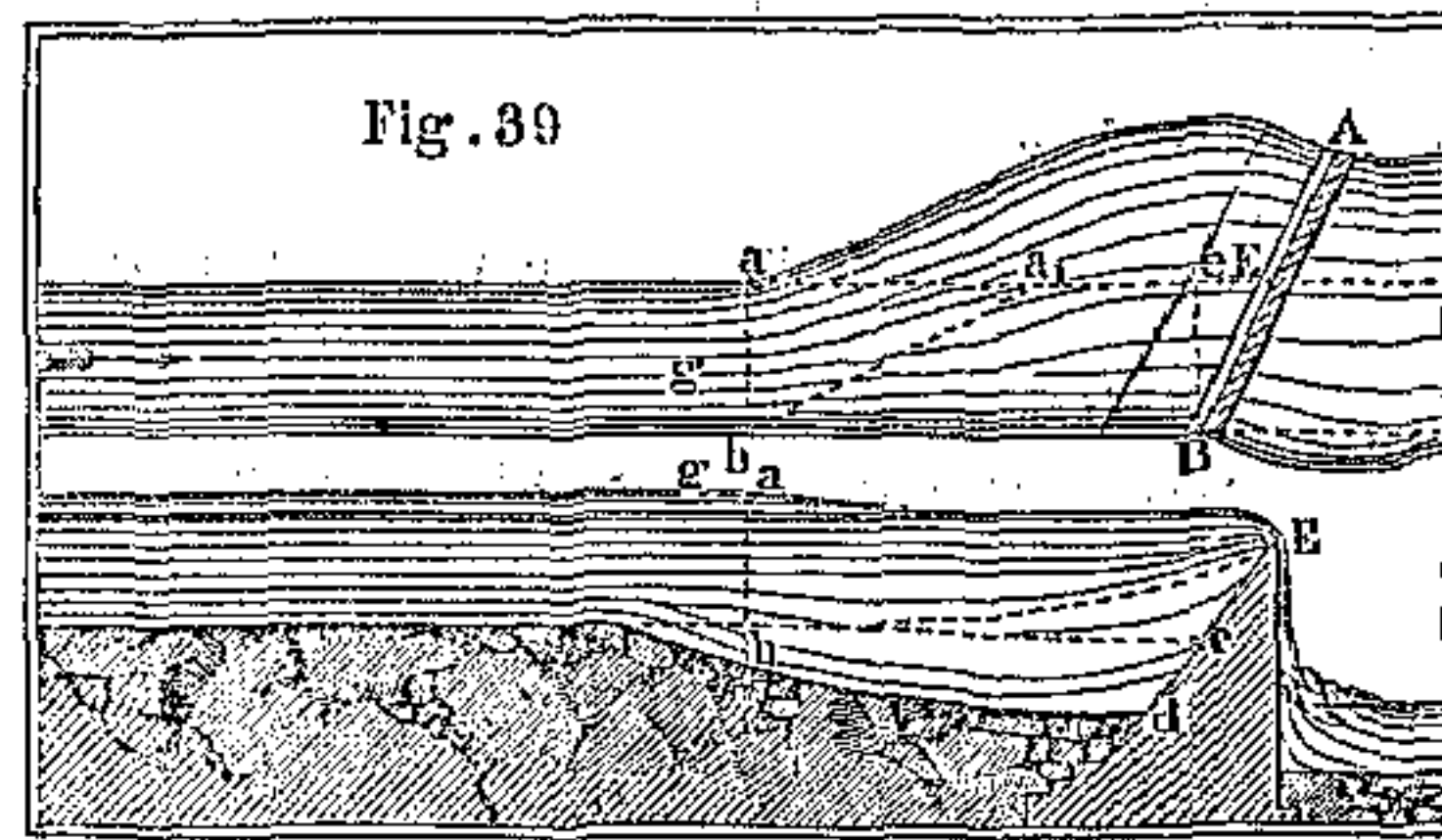
Heights at M'Kean's weir 220 feet long, in inches.	Heights 60 feet above the weir channel 120 feet wide.		Heights 2050 feet above the weir channel 55 feet wide; average.	
	Calculated inches.	Observed inches.	Calculated inches.	Observed inches.
1½	2½	2½	4½	5½
2
3	4½	..	7½	7
4	6	..	10	9
5	7½	..	12½	11½
6	9	9	15	10½
7	10½	10½	17½	18½
8	12	..	20	20½
9	13½	12½	22½	20½
10	15	..	24½	20
11	16½	..	27½	24
12	18	17	30½	31
13	19½	18½	32½	33
15	22½	21	37½	40
18	27	25	45½	46
21	31½	29½	53	54
24	36	34	60½	62

with considerable velocity, this was conceived, as the observations afterwards showed, to be a sufficient compensation for the loss of head below by friction. The observations were made by a separate party, over whom the writer had no control, and it is necessary to remark, that with the same head of water on the weir, they often differed more from each other than from the calculation. This, probably, arose from the different directions of the wind, and the water rising during one observation, and falling during another.

The true principle for determining the head at g , Fig. 39, apart from that due to friction, is that pointed out at pages 142 and 147; but when the passes are very near each other, or the depth d_2 , Fig. 23, is small, the effect of the discharge through d_2 is inconsiderable in reducing the head, as the contraction and loss of *vis-viva* are then large, and the head d_1 becomes that due to a weir of the width of the contracted channel at A , nearly. The reduction in the extent of the backwater, by lowering the head on a longer weir, is found by taking the difference of the amplitudes due to the heads at g , Fig. 39, in both cases, as determined from equations (56), (128), *et seq.* This will seldom exceed a mile up the river, as the surface inclination is found to be considerably greater than that due to mere friction and velocity, and hence the general failure of drainage works designed on the assumption that the lowering of the head below, by means of long weirs, extends its effects all the way up a channel. We must nearly treble the length of a weir before the head passing over can be reduced by one-half, TABLE X., even supposing the circumstances of approach to be the same: surely several engineering appliances for shorter weirs, during periods of flood, would be found more effective and far less expensive than this alternative, with its extra sinking and weir basin for drainage purposes.

The advocates for the necessity of weirs longer than the width of the channel, for drainage purposes, must show that the reduction of the head and extent of backwater above g , Fig. 39, is not small, and that the effects extend the whole way up the channel,

or at least as far as the district to be benefited. Practice has heretofore shown, that long weirs have failed (unless after the introduction of sluices or other appliances) in producing the expected arterial drainage results, notwithstanding the increased leakage from increased length, which must accompany their construction.



The deepening in the weir basin $a b B E A$ is mostly of use in reducing the surface inclination between $a b$ and $A B$ by increasing the hydraulic mean depth; but, thereby, the velocity of approach is lessened, and therefore the head at E increased. When the length of a weir basin $a E$ exceeds that point where these two opposite effects balance each other, there will be a gain by the difference of the surface inclinations in favor of the long weir: but unless $a E$ exceeds half a mile, this difference cannot amount to more than 3 or 4 inches, unless the river be very small indeed; and *if the channel be sunk for the long weirs $B A$ or $b a$, it should also be sunk to at least the same depth and extent for the short weirs $B e$, $b a$, otherwise there is no fair comparison of their separate merits.* The effect of the widening between $a b$ and $A B$, the depth being the same, is also to reduce the

surface inclination from α to E ; but, as before, unless αE be of considerable length, this gain will also be small. Now AB , at best, is but a weir the direct width of the new channel at A , and if the length αE be considerable, we have an entirely new river channel with a direct weir at the lower end, and the saving of head effected arises entirely from the larger channel, with a *direct* transverse weir at the lower end.

By referring to TABLE VIII., it will be found that for a hydraulic mean depth of 5 feet a fall of $7\frac{1}{2}$ inches per mile will give a velocity of 2 feet per second; if we double the depth, a fall of 4 inches will give the same velocity; and for a depth of only 2 feet 6 inches, a fall of $12\frac{1}{2}$ inches is necessary. This is a velocity much larger than we have ever observed in a weir basin, yet we easily perceive that the difference in the inclinations for a short distance, $E\alpha$ of a few hundred feet, must be small. If one section be double the other, the hydraulic mean depth remaining constant, the velocity must be one-half, and the fall per mile one-fourth, nearly. This would leave $7\frac{1}{2} - 2 = 5\frac{1}{2}$ inches per mile, or 1 inch per 1000 feet nearly, as the gain with a hydraulic mean depth of 5 feet for a double water channel. For greater depths the gain would be less, and the contrary for lesser depths.

Is the saving of head and amplitude of backwater we have calculated worth the increased cost of long weirs and the consequent necessity and expense of sinking and widening the channels for such long distances? We think not; indeed, *the sinking in the basin immediately at the weir is absolutely injurious,*

by destroying the velocity of approach, and increasing the contraction. The gradual approach of the bottom towards the crest, shown by the upper dotted line $b\ e$ in the section, Fig. 39, and a sudden overfall, will be found more effective in reducing the head, unless so far as leakage takes place, than any depth of sinking for nearly 80 or 100 feet above long weirs.

In most instances, the extra head will be only perceived by an increased surface inclination, which may extend for a mile or more up the channel, according to the sinking and widening.

It is a general rule that, for shorter weirs, the coefficients of discharge decrease; this arises from the greater amount of lateral contraction, and is more marked in notches or Poncelet weirs, than for weirs extending from side to side of the channel; but for weirs exceeding 10 feet in length the decrease in the coefficients from this cause is immaterial, unless the head passing over bear a large ratio to the length; and we even see from the coefficients, page 80, derived from Mr. Blackwell's experiments, that with 10 inches head passing over a 2-inch plank, the coefficient for a length of 3 feet is $\cdot 614$; for a length of 6 feet $\cdot 539$; and for a length of 10 feet $\cdot 534$; showing a decrease as the weir lengthens, but which may, in the particular cases, be accounted for. We have before referred to other circumstances which modify the coefficients, yet we may assume generally, without any error of practical value, that the coefficients are the same for different weirs extending from side to side of a river. If, then, we put w and w_1 for the lengths of two such weirs, we shall have the relation

of the heads d and d_1 for the same quantity of water passing over, as in the following proportion:—

$$d : d_1 :: w_1^{\frac{2}{3}} : w^{\frac{2}{3}};$$

and therefore

$$(135.) \quad d_1 = \left(\frac{w}{w_1}\right)^{\frac{2}{3}} \times d.$$

By means of this equation we have calculated TABLE X., the ratio $\frac{w}{w_1}$ being given in columns 1, 3,

5 and 7, and the value of $\left(\frac{w}{w_1}\right)^{\frac{2}{3}}$, or the coefficient by

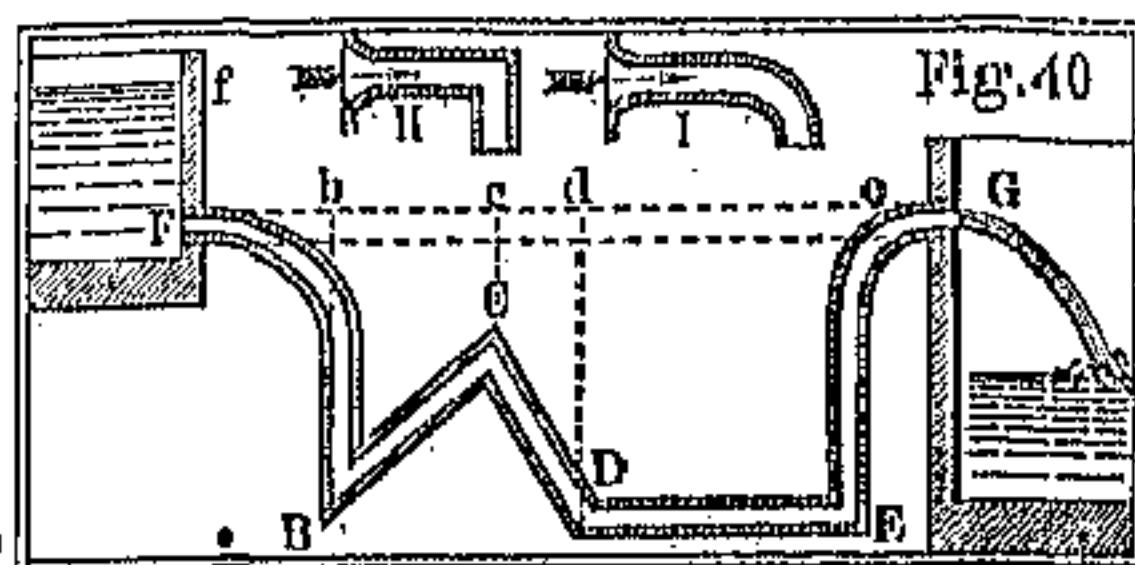
which d is to be multiplied, to find d_1 in columns 2, 4, 6 and 8. It appears also, that *if we take the heads passing over any weir in a river in an arithmetical progression, the heads then passing over any other weir in the same river must also be in arithmetical progression, unless the quantity flowing down varies from erogation or supply, such as drawing off by millraces, &c.* If c_d be the coefficient for a direct weir, $\cdot 94 c_d$ will answer for an obliquity of 45° , and $\cdot 91 c_d$ for an angle of 65° .

SECTION XI.

BENDS AND CURVES.—BRANCH PIPES.—DIFFERENT LOSSES OF HEAD.—GENERAL EQUATION FOR FINDING THE VELOCITY.—HYDROSTATIC AND HYDRAULIC PRESSURE.—PIEZOMETER.—CATCHMENT BASINS.—RAIN-FALL PER ANNUM.

The resistance or loss of head due to bends and curves has now to be considered. If we fix a bent pipe, $FBCDEG$, Fig. 40, between two cisterns, so as

to be capable of revolving round in collars at F and G, we shall find the time the water takes to sink a given distance from f



to F in the upper cistern the same, whether the tube occupy the position shown in the figure or the horizontal position shown by the dotted line $f b c d e g$. This shows that the resistances due to friction and to bends are independent of the pressure. If the tube were straight, the discharge would depend on the length, diameter, and difference of level between f and G , and may be determined from the mean velocity of discharge, found from TABLE VIII. or equation (79). Here, however, we have to take into consideration the loss sustained at the bends and curves, and our illustration shows that it is unaffected by the pressure.

The experiments of Bossut, Du Buât, and others, show that the loss of head from bends and curves—like that from friction—increases as the square of the velocity; but when the curves have large radii, and the bends are very obtuse, the loss is very small. With a head of nearly 3 feet, Venturi's twenty-third experiment, when reduced, gives—for a short straight tube 15 inches long, and $1\frac{1}{4}$ inch in diameter, having the junction of the form of the contracted vein, very nearly .873 for the coefficient of discharge. When of the same length and diameter, but bent as in Diagram I, Fig. 40, the coefficient is reduced to .785; and when bent at a right angle as at II, Fig. 40, the co-

efficient is further reduced to .560. In these respective cases we have therefore*

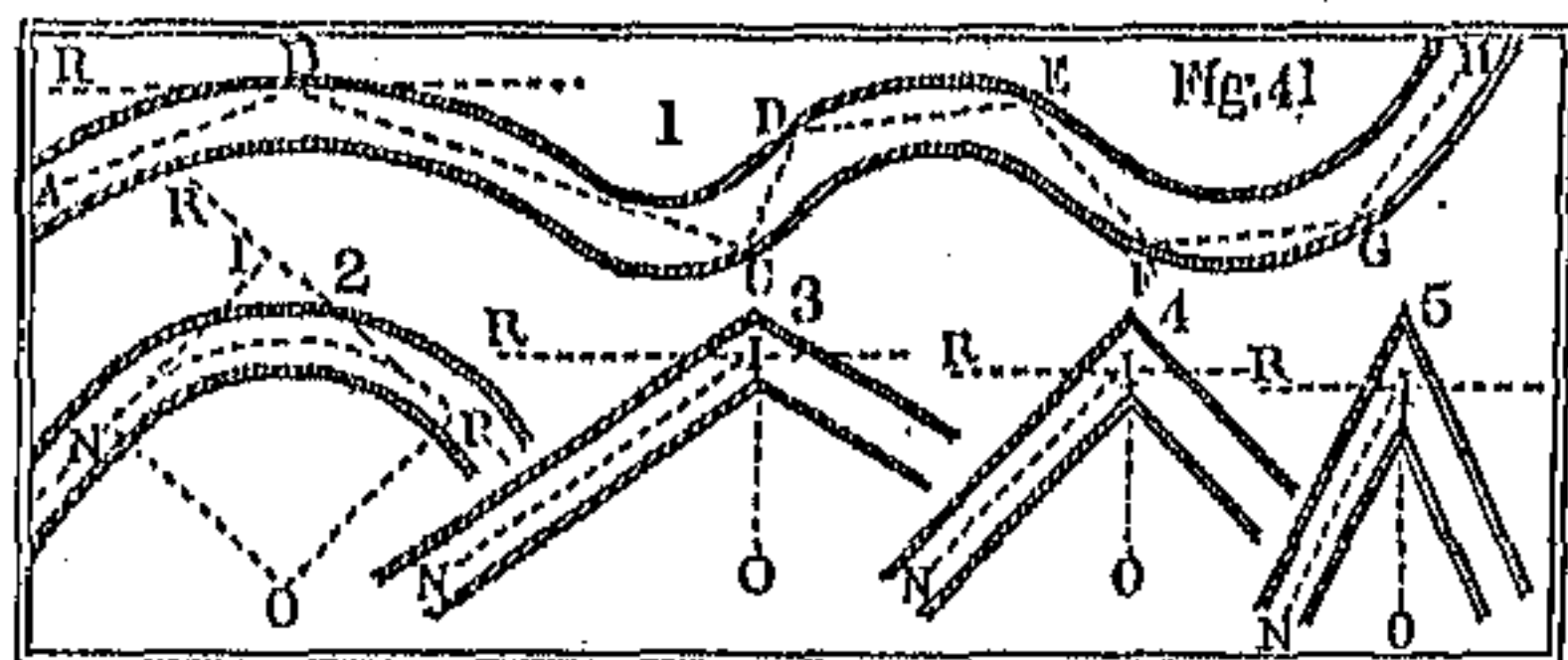
$$1. \quad v = .873 \sqrt{2gh}, \text{ and } h = 1.312 \times \frac{v^2}{2g};$$

$$2. \quad v = .785 \sqrt{2gh}, \text{ and } h = 1.623 \times \frac{v^2}{2g};$$

$$3. \quad v = .560 \sqrt{2gh}, \text{ and } h = 3.188 \times \frac{v^2}{2g};$$

showing that the loss of head in the tube H, Fig. 40, from the bend, is $1.876 \times \frac{v^2}{2g}$, or nearly double the theoretical head due to the velocity in the tube. The loss of head by the circular bend is only $.311 \frac{v^2}{2g}$, or not quite one-sixth of the other.

Du Buât deduced, from about twenty-five experiments, that the head due to the resistance in any bent tube A B C D E F G H, diagram 1, Fig. 41, depends



on the number of deflections between the entrance at A and the departure at H; that it increases at each

* It is stated that the time necessary for the discharge of a given quantity of water through a straight pipe being 1, the time for an equal quantity through a curve of 90° would be 1.11, with a right angle 1.57; two right angles would increase the time to 2.464, and two curved junctions to only 1.23. *Vide* REPORT ON THE SUPPLY OF WATER TO THE METROPOLIS, p. 287, APPENDIX No. 8.

reflection as the square of the sine of the deflected angle, A B R for instance, and as the square of the velocity; and that if $\phi, \phi_1, \phi_2, \phi_3, \&c.$, be the number of degrees in the angles of deflection at B, C, D, E, &c., then for measures in French inches the height h_v , due to the resistances from curves, is

$$(136.) \quad h_v = \frac{v^2(\sin.^2\phi + \sin.^2\phi_1 + \sin.^2\phi_2 + \sin.^2\phi_3 + \&c.)}{3000}$$

which for measures in English inches becomes

$$(137.) \quad h_v = \frac{v^2(\sin.^2\phi + \sin.^2\phi_1 + \sin.^2\phi_2 + \sin.^2\phi_3 + \&c.)}{3197}$$

and for measures in English feet,

$$(138.) \quad h_v = \frac{v^2(\sin.^2\phi + \sin.^2\phi_1 + \sin.^2\phi_2 + \sin.^2\phi_3 + \&c.)}{266.4}$$

or, as it may be more generally expressed for all measures,

$$(139.) \quad h_v = (\sin.^2\phi + \sin.^2\phi_1 + \sin.^2\phi_2 + \sin.^2\phi_3 + \&c.) \times \frac{v^2}{8.27g}, \text{ in which } \frac{v^2}{8.27g} = \frac{v^2}{266.4} = .00375 v^2 \text{ in feet.}$$

The angle of deflection, in the experiments from which equation (136) was derived, did not exceed 36° . We have already shown the loss of head from the circular bend in diagram I, Fig. 40, where the angle of deflection is nearly 45° , to be $.311 \frac{v^2}{2g} = .00483 v^2$, but as the $\sin. 45^\circ = .707$; $\sin.^2 45^\circ = .5$ we get $.00483 v^2 = .00966 v^2 \times \sin.^2 45^\circ$, or more than two and a half times as much as Du Buât's formula would give; and if we compare it with Rennie's experiments,* with a pipe 15 feet long, $\frac{1}{2}$ inch diameter, bent into fifteen curves, each $3\frac{1}{4}$ inches radius,

* Philosophical Transactions for 1831, p. 438.

we should find the formula gives a loss of head not much more than one half of that which may be derived from the observed change, 419 to 370 cubic feet per minute in the discharge. See p. 278.

Dr. Young* first perceived the necessity of taking into consideration the length of the curve and the radius of curvature. In the twenty-five experiments made by Du Buât, he rejected ten in framing his formula, and the remaining fifteen agreed with it very closely. Dr. Young finds

$$(140.) \quad h_b = \frac{.0000045 \phi \rho^{\frac{1}{2}} \times v^3}{\rho};$$

where ϕ is the number of degrees in the curve $N P$, diagram 2, Fig. 41, equal the angle $N O P$; $\rho = O N$ the radius of curvature of the axis; h_b the head due to the resistance of the curve, and v the velocity, all expressed in French inches. This formula reduced for measures in English inches is

$$(141.) \quad h_b = \frac{.0000044 \phi \rho^{\frac{1}{2}} \times v^3}{\rho};$$

and for measures in English feet,

$$(142.) \quad h_b = \frac{.000006 \phi \rho^{\frac{1}{2}} \times v^3}{\rho}.$$

Equation (140) agrees to $\frac{1}{25}$ th of the whole with twenty of Du Buât's experiments, his own formula agreeing so closely with only fifteen of them. The resistance must evidently increase with the number of bends or curves; but when they come close upon, and are grafted into each other, as in diagram 1, Fig. 41, and in the tube $F B C D E G$, Fig. 40, the motion in one bend or curve immediately affects those

* Philosophical Transactions for 1808, pp. 173—175.

in the adjacent bends or curves, and this law does not hold.

Neither Du Buât nor Young took any notice of the relation that must exist between the resistance and the ratio of the radius of curvature to the radius of the pipe. Weisbach does, and combining Du Buât's experiments with some of his own, finds for circular tubes,

$$(143.) \quad h_b = \frac{\phi}{180} \times \left\{ .131 + 1.847 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}} \right\} \times \frac{v^2}{2g};$$

and for quadrangular tubes,

$$(144.) \quad h_b = \frac{\phi}{180} \times \left\{ .124 + 3.104 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}} \right\} \times \frac{v^2}{2g};$$

in which ϕ is equal the angle $NOP = NIR$, diagram 2, Fig. 41; d the mean diameter of the tube, and ρ the

radius NO of the axis. When $\frac{d}{2\rho}$ exceeds .2, the value

of $.131 + 1.847 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}}$ exceeds $.124 + 3.104 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}}$,

and the resistance due to the quadrangular tube exceeds that due to the circular one. We have arranged and calculated the following table of the numerical values of these two expressions for the more easy application of equations (143) and (144). This table will be found of considerable use in calculating the values of equations (143) and (144), as the second and fifth columns contain the values of

$.131 + 1.847 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}}$, and the third and sixth columns

the values of $.124 + 3.104 \left(\frac{d}{2\rho} \right)^{\frac{7}{2}}$, corresponding to

different values of $\frac{d}{2\rho}$; and it is carried to twice the extent of those given by Weisbach.

TABLE OF THE VALUES OF THE EXPRESSIONS
 $\cdot 131 + 1\cdot 847 \left(\frac{d}{2\rho}\right)^{\frac{7}{2}}$ and $\cdot 124 + 3\cdot 104 \left(\frac{d}{2\rho}\right)^{\frac{7}{2}}$.

$\frac{d}{2\rho}$	Circular tubes.	Quadrangular tubes.*	$\frac{d}{2\rho}$	Circular tubes.	Quadrangular tubes.
·1	·131	·124	·0	·440	·643
·15	·133	·128	·65	·540	·811
·2	·138	·135	·7	·661	1·015
·25	·145	·148	·75	·800	1·258
·3	·158	·170	·8	·977	1·545
·35	·178	·203	·85	1·177	1·881
·4	·206	·250	·9	1·408	2·271
·45	·244	·314	·95	1·674	2·718
·5*	·294	·398	1·00	1·978	3·228

For bent tubes, diagrams 3, 4, and 5, Fig. 41, the loss of head is considerably greater than for rounded tubes. If, as before, we put the angle $\angle RIR = \phi$, IR being at right angles to IO the line bisecting the angle or bend, we shall find, by decomposing the motion, that the head $\frac{v^2}{2g}$ becomes $\frac{v^2}{2g} \times \cos.^2 \phi$ from the change of direction; and that a loss of head

$$(145.) \quad h_b = (1 - \cos.^2 2\phi) \frac{v^2}{2g} = \sin.^2 2\phi \frac{v^2}{2g}$$

must take place. When the angle is a right angle, as in diagram 4, $\cos. 2\phi = 0$, and $h_b = \frac{v^2}{2g}$; that is to say, the loss of head is exactly equal to the

* The values corresponding to $\frac{d}{2\rho} = \cdot 55$ are ·350 and ·507 for circular and quadrangular tubes.

theoretical head. When the angle or bend is acute, as in diagram 5, the loss of head is $(1 + \cos. 2\phi) \frac{v^2}{2g}$, for then $\cos. 2\phi$ becomes negative. Weisbach does not find the loss of head in a right angular bend greater than $.984 \frac{v^2}{2g}$; while Venturi's twenty-third experiment, made with extreme care, p. 273, shows the loss to be $1.876 \frac{v^2}{2g}$. When the pipes are long, however, the value of $\frac{v^2}{2g}$ is in general small, and this correction does not affect the final results in any material degree.

Rennie's experiments,* with a pipe 15 feet long, $\frac{1}{2}$ inch in diameter, and with 4 feet head, give the discharge per second

	Cubic feet.
1. Straight, see table, p. 15200699
2. Fifteen semicircular bends00617
3. One bend, a right angle $8\frac{1}{2}$ inches from the end of the pipe00556
4. Twenty-four right angles00253

From these data we find consecutively, the theoretical discharge being .021885 cubic feet per second,

and the theoretical head $H = \frac{v^2}{2g}$, that

1. $v = .319 \sqrt{2gH}$, and therefore $H = 9.82 \times \frac{v^2}{2g}$;
2. $v = .282 \sqrt{2gH}$, „ „ $H = 12.58 \times \frac{v^2}{2g}$;

* Philosophical Transactions for 1831, p. 438.

$$3. v = .254 \sqrt{2gH}, \text{ and therefore } H = 15.50 \times \frac{v^2}{2g};$$

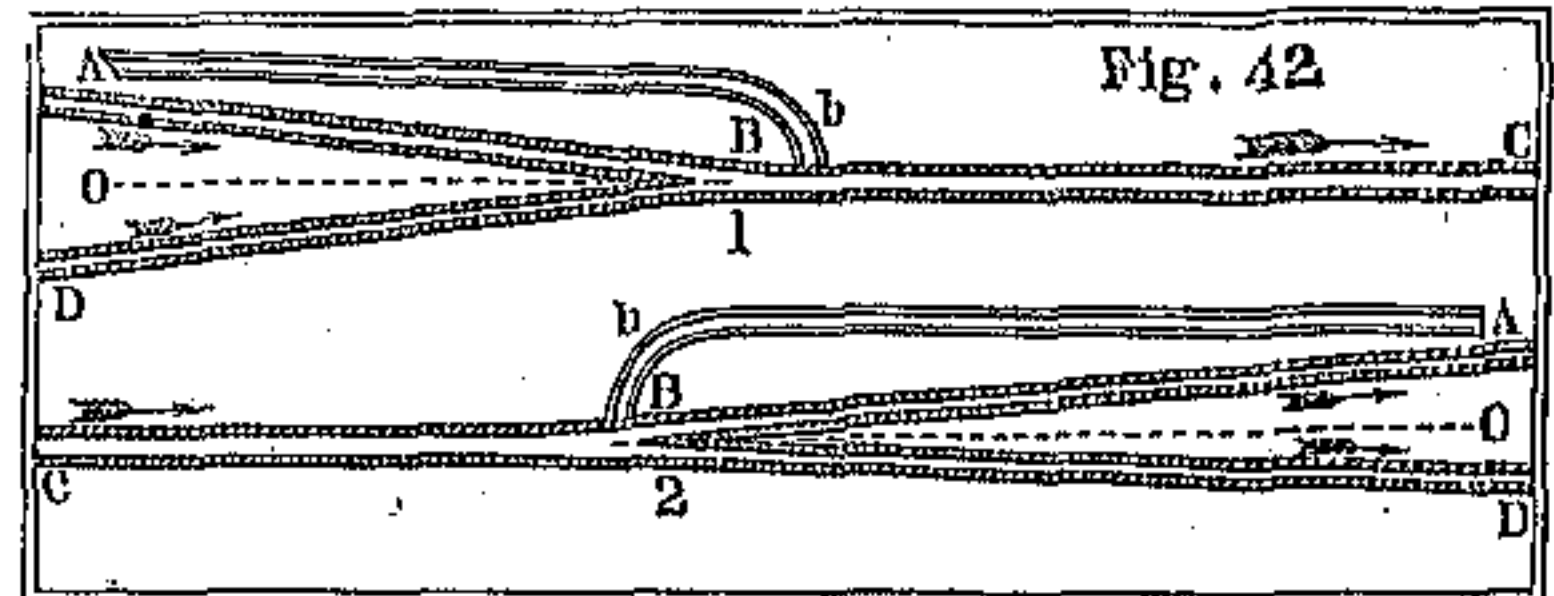
$$4. v = .116 \sqrt{2gH}, \text{ „ „ } H = 74.34 \times \frac{v^2}{2g}.$$

The loss of head, therefore, by the introduction of 15 semicircular bends, is $2.76 \frac{v^2}{2g}$; by the introduction of one right angle, $5.68 \frac{v^2}{2g}$; and by the introduction of 24 right angles, $64.52 \frac{v^2}{2g}$, or about 12 times the loss due to one right angle. This shows that the resistance does not increase as the number of bends, as we before remarked, p. 256, when they are close to each other. The loss of head from one right angle, $5.68 \frac{v^2}{2g}$, is more than double the loss from 15 semicircular bends, or $2.76 \frac{v^2}{2g}$. The loss of head for a right angular bend, determined from Venturi's experiment, is $1.876 \frac{v^2}{2g}$; formula (145) makes it $\frac{v^2}{2g}$; and Weisbach's empirical formula, $(.9457 \sin. \phi + 2.047 \sin.^4 \phi) \frac{v^2}{2g}$, makes it only $.984 \frac{v^2}{2g}$. The formulæ now in use give, therefore, results considerably under the truth. It appears to us, that the *velocity* of the water moving directly towards the bend must be taken into consideration, and also the loss of mechanical effect from contraction, and eddies at the bend, as

well as the loss arising from the mere change of direction.

BRANCH PIPES.

When a pipe is joined to another, the quantity of water flowing below the junction B, diagram 1, Fig. 42, must be equal to the sum of the quantities



flowing in the upper branches in the case of supply; and when the branch pipe draws off a portion of the water, as in diagram 2, the quantity flowing above the junction must be equal to the quantities flowing in the lower branches. Both cases differ only in the motion being *from* or *to* the branches, which, in pipes, are generally grafted at right angles to the main, for practical convenience, as shown at *bb*, and then carried on in any given direction. The loss of head arising from change of direction, equation (145), is $\sin.^2 2\phi \frac{v^2}{2g}$, in which $2\phi = \text{angle } \Lambda B O$; but as in general 2ϕ is a right angle for branches to mains, this source of loss becomes then simply $\frac{v^2}{2g}$. In addition to this, a loss of head is sustained at the junction, from a certain amount of force required to unite or separate the water in the new channel. In the case of drawing off, diagram 2, this loss was estimated by D'Aubuisson, from experiments by G nieys, to be about twice the theoretical head due

to the velocity in the branch, or $\frac{2v^2}{2g}$, so that the whole loss of head arising from the junction is $\frac{v^2}{2g} + \frac{2v^2}{2g} = \frac{3v^2}{2g}$, or three times the theoretical head due to the velocity. In the case of supply, the loss is probably nearly the same. The actual loss is, however, very uncertain; but, as was before observed when discussing the loss of head occasioned by bends, two or three times $\frac{v^2}{2g}$ is in general so comparatively small, that its omission does not materially affect the final results. A loss also arises from contraction, &c. See pp. 175, 176.

The calculations for mains and branches become often very troublesome, but they may always be simplified by rejecting at first any minor corrections for contraction at orifice of entry, bends, junctions, or curves. If, in diagram 2, Fig. 42, we put h for the head at B, or height of the surface of the reservoir over it; h_a for the fall from B to A; h_d for the fall from B to D; l equal the length of pipe from B to the reservoir; l_a equal the length BA; l_d equal the length BD; r equal the mean radius of the pipe BC; r_a the mean radius of the pipe BA; r_d the mean radius of BD; v the mean velocity in BC; v_a the velocity in BA; and v_d the velocity in BD, we then find, by means of equation (73), the fall from the reservoir to A equal to

$$(146.) \quad h + h_a = \left(c_r + c_f \times \frac{l}{r}\right) \frac{v^2}{2g} + \left(1 + c_f \times \frac{l_a}{r_a}\right) \frac{v_a^2}{2g};$$

the fall from the reservoir to D equal to

$$(147.) \quad h + h_d = \left(c_r + c_r \times \frac{l}{r}\right) \frac{v^2}{2g} + \left(1 + c_r \times \frac{l_d}{r_d}\right) \frac{v_d^2}{2g};$$

and, as the quantity of water passing from *c* to *B* is equal to the sum of the quantities passing from *B* to *A* and from *B* to *D*,

$$(148.) \quad v r^2 = v_a r_a^2 + v_d r_d^2.$$

By means of these three equations we can find any three of the quantities h , h_a , h_d , r , r_a , r_d , b , b_a , b_d , the others being given. Equations (146) and (147) may be simplified by neglecting c_r , the coefficient due to the orifice of entry from the reservoir, and 1, the coefficient of velocity. They will then become

$$(148A.) \quad h + h_a = c_r \times \left(\frac{l}{r} \times \frac{v^2}{2g} + \frac{l_a}{r_a} \times \frac{v_a^2}{2g}\right),$$

and

$$(149.) \quad h + h_d = c_r \times \left(\frac{l}{r} \times \frac{v^2}{2g} + \frac{l_d}{r_d} \times \frac{v_d^2}{2g}\right).$$

The mean value of c_r for a velocity of 4 feet per second is .005741, and of $\frac{c_r}{2g}$, .0000891. The values for any other velocities may be had from the table of coefficients of friction given at p. 214. When l , h , and r are given, the velocity v can be had from the equation, $v = \left(\frac{2g}{c_r} \times \frac{r h}{l}\right)$, or more immediately from TABLE VIII.

GENERAL EQUATION FOR MEAN VELOCITY.

We are now enabled to give a general equation for finding the whole head H , and the mean velocity v , in any channel; and to extend equations (73) and (74) so as to comprehend the corrections due to bends,

curves, &c. Designating, as before, the height due to the resistance at the orifice of entry by

h_r , and the corresponding coefficient by c_r ;

h_f the head due to friction, and c_f the coefficient of friction;

h_b the head due to bends, and c_b the coefficient of bends;

h_c the head due to curves, and c_c the coefficient of curves;

h_o the head due to orogation, and c_o the coefficient of orogation;

h_x the head due to other resistances, and c_x their mean coefficient;

then we get

$$(150.) \quad \Pi = h_r + h_f + h_b + h_c + h_o + h_x + \frac{v^2}{2g};$$

that is to say, by substituting for h_r , h_b , &c., their values as previously found,

$$\begin{aligned} \Pi = (1 + c_r) \frac{v^2}{2g} + c_f \frac{l}{r} \times \frac{v^2}{2g} + c_b \times \frac{v^2}{2g} \\ + c_c \times \frac{v^2}{2g} + c_o \times \frac{v^2}{2g} + c_x \times \frac{v^2}{2g}; \end{aligned}$$

or, more briefly,

$$(151.) \quad \Pi = \left(1 + c_r + c_f \times \frac{l}{r} + c_b + c_c + c_o + c_x\right) \frac{v^2}{2g};$$

from which we find

$$(152.) \quad v = \left\{ \frac{2g\Pi}{1 + c_r + c_f \times \frac{l}{r} + c_b + c_c + c_o + c_x} \right\}^{\frac{1}{2}}.$$

It is to be observed here, that for very long uniform channels, the value of the mean velocity will be found in general equal to $\left\{ \frac{2gr\Pi}{c_f l} \right\}^{\frac{1}{2}}$, as the other resistances and the head due to the velocity are all trifling compared with the friction, and may be rejected without error; but, as we before stated, it is advisable in practice, when determining the diameter of pipes, p. 229, to increase the value of c_r ,

table, p. 214, or to increase the diameter found from the formula by one-sixth, which will increase the discharging power by one half. (See TABLE XIII.)

In equations (74) and (151), the coefficient of friction c_r depends on the velocity v , and its value can be found from an approximate value of that velocity from the small table, p. 214. If, however, we use both powers of the velocity, as in equation (83), we shall get, when Π is the whole head, and h the head from the surface to the orifice of entry

$$(a v + b v^2) \frac{l}{r} + (1 + c_r) \frac{v^2}{2g} + h = \Pi,$$

a quadratic equation from which we find

$$v = \left\{ \frac{(\Pi - h) 2 g r}{(1 + c_r) r + 2 g b l} + \left(\frac{g a l}{(1 + c_r) r + 2 g b l} \right)^2 \right\}^{\frac{1}{2}} - \frac{g a l}{(1 + c_r) r + 2 g b l}$$

for a more general value of the velocity than that given in equation (74). If now we put $c_s = c_r + c_b + c_o + c_e + c_x$ in equation (151) we shall find

$$v = \left\{ \frac{(\Pi - h) 2 g r}{(1 + c_s) r + 2 g b l} + \left(\frac{g a l}{(1 + c_s) r + 2 g b l} \right)^2 \right\}^{\frac{1}{2}} - \frac{g a l}{(1 + c_s) r + 2 g b l}$$

for a more general expression of equation (152), when the simple power of the velocity, as in equation (83), is taken into consideration. For measures in English feet, we may take $a = .0000223$ and $b = .0000854$, which correspond to those of Eytelwein, in equation (97). The value of a is the same in English as in French measures, but the value of b in equation (83), for measures in metres, must be divided by 3.2809 to find its corresponding value for measures in English feet. In considering the head $\frac{v^2}{2g} c_r$, due to contraction at the orifice of entry as not implicitly comprised in the primary values of a and b , equation

(83), Eytelwein is certainly more correct than D'Aubuisson, *Traité d'Hydraulique*, pp. 223 et 224, as this head varies with the nature of the junction, and should be considered in connection with the head due to the velocity, or separately. It can never be correctly considered as a portion of the head due to friction. In all Du Buât's experiments, this head was considered as a portion of that due to the velocity, and the whole head, $(1 + c_r) \frac{v^2}{2g}$, deducted to find the head due to friction and thence the hydraulic inclination.

VALUES OF a AND b FOR MEASURES IN ENGLISH FEET.

	a .	b .
Equation (88.)	·0000445	·0000944
„ (90.)	·0000173	·0001061
„ (94.)	·0000243	·0001114
„ (98.)	·0000223	·0000854
„ (109.)	·0000189	·0001044
„ (111.)	·0000241	·0001114
„ (114.)	·0000035	·0001150
Mean values for all straight channels, pipes, or rivers .)	·0000221	·0001040

These mean values of a and b give the equation

$$rs = \cdot 0001040 v^2 + \cdot 0000221 v,$$

from which we find

$$9615 rs = v^2 + \cdot 21 v,$$

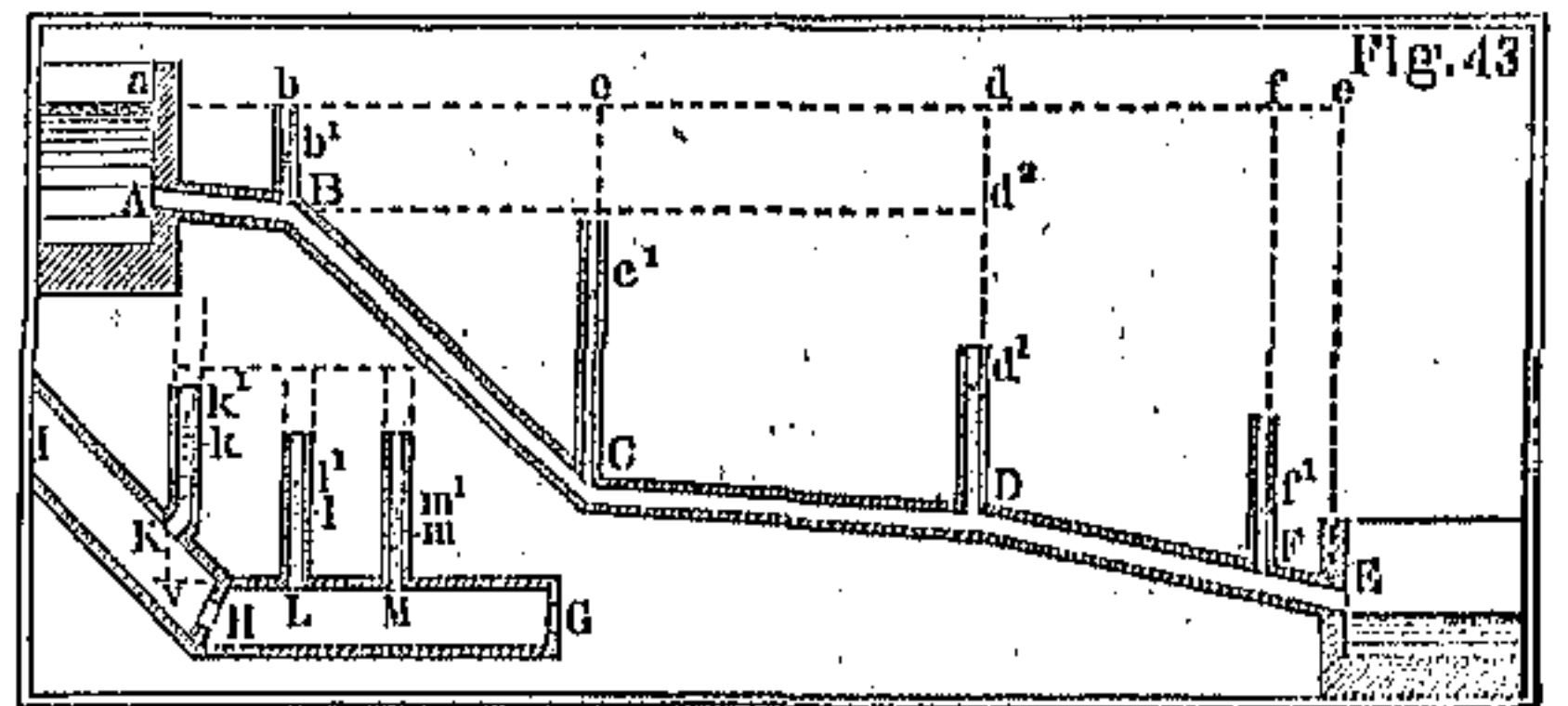
and thence

$$(153.) \quad v = (9615 rs + \cdot 011)^{\frac{1}{2}} - \cdot 105 = 98 \sqrt{rs} - \cdot 1,$$

very nearly, suited to velocities of about 2 feet, p. 217.

HYDROSTATIC AND HYDRAULIC PRESSURE.—PIEZOMETER.

When water is at rest in any vessel or channel, the pressure on a unit of surface is proportionate to the head at its centre,* measured to the surface, and is expressed in lbs. for measures in feet, by $62\frac{1}{2} h s$, in which h is the head, and s the surface exposed to the pressure, both in feet measures. This is the *hydrostatic pressure*. In the pipe A B C D F E, Fig. 43, the pressure at the points B, C, D, F, and E, on the sides of the tube will be respectively as the heads Bb, Cc, Dd, Ff, and Ee, if all motion in the tube be prevented by



stopping the discharging orifice at E. In this case the pressure is a maximum and hydrostatic; but if the discharging orifice at E be partially or entirely open, a portion of each pressure at B, C, D, F, &c., is absorbed in overcoming the different resistances of friction, bends, &c., between it and the orifice of entry at A, and also by the velocity in the tube, and the difference is the *hydraulic pressure*.

* This is only correct when the surface is small in depth compared with the head. If n be the depth of a rectangular surface in feet, and also the head of water measured to the lower horizontal edge, then the pressure in lbs. is expressed by $31\frac{1}{2} n^2$; and the centre of pressure is at $\frac{2}{3}$ rds of the depth.

Bernoulli first showed that *the head due to the pressure at any point, in any tube, is equal to the effective head at that point, minus the head due to the velocity*. When the resistances in a tube vanish, the effective head becomes the hydrostatic head, and by representing the former by h_{or} we shall have, adopting the notation in equation (150),

$$h_{\text{or}} = H - (h_r + h_f + h_b + \&c.),$$

and consequently the head due to the hydraulic pressure equal

$$(153A.) \quad h_p = h_{\text{or}} - \frac{v^2}{2g} = H - (h_r + h_f + h_b + \&c.) - \frac{v^2}{2g}.$$

If small tubes be inserted, as shown in Fig. 43, at the points B, C, D, and F, the heights B b^1 , C c^1 , D d^1 , F f^1 , which the water rises to, will be represented by the corresponding values of h_p in the preceding equation; and the difference between the heights C c^1 , F f^1 , at C and F, for instance, added to the fall from C to F will, evidently, express the head due to all the resistances between C and F. When $H = E e$, and the orifice at E is open, we have, from equation (150), $H = h_r + h_f + h_b + h_o + \&c. + \frac{v^2}{2g}$, and therefore $h_p = 0$, that is, the pressure at the discharging orifice is nothing.

The vertical tubes at B, C, D, F, when properly graduated, are termed *piëzometers* or *pressure gauges*; they not only show the actual pressure at the points where placed, but also the difference between any two; D d^1 — B b^1 , for instance, added to the difference of head between D and B, or D d^2 will give D d^1 — B b^1 + D d^2 for the head or pressure due to the resistances

between *b* and *d*. This instrument affords, perhaps, the very best means of determining the loss of head due to bends, curves, diaphragms, &c. The loss of head due to friction, bend, diaphragms, &c., between *k* and *l*, Fig. 43, is equal to $k\bar{k} - l\bar{l} + kv$. If *m* be the same distance from *l* as *k* is, $l\bar{l} - m\bar{m}$ will be the height due to the friction (*l* and *m* being on the same level); therefore $k\bar{k} - l\bar{l} + kv - l\bar{l} + m\bar{m} = k\bar{k} + kv + m\bar{m} - 2l\bar{l}$ is the head due to the diaphragm and bend both together. If the diaphragm be absent, we get the head due to the bend, and if the bend be absent, the head due to the diaphragm in like manner.

When the discharging orifice, as at *e*, is quite open, we have seen that the pressure there is zero; but when, as at *a*, it is only partly open, this is no longer the case, and the hydraulic pressure increases from zero to hydrostatic pressure, as the orifice decreases from the full section to one indefinitely small compared with it. A piëzometer, placed a short distance inside *a*, will give this pressure; and the difference between it and the whole head will be the head due to the resistances and velocity in the pipe: from which, and also the length and diameter, the discharge may be calculated as before shown. Again, by means of the head $m\bar{m}^1$, and that due to the velocity of approach, we can also find the discharge through the diaphragm *a*; see equation (45) and the remarks following it. This result must be equal to the other, and we may in this way test the formulæ anew or correct them by the practical results.

The velocity of discharge of the tube *a c d e*, may

be calculated by means of any piézometric height $c c^1$; for by putting the whole fall from c^1 to π equal

to πc^1 , we get, disregarding bends, $v = \left\{ \frac{2 g r H_0^1}{c_f L_0^1} \right\}^{\frac{1}{2}}$, in

which $L_0^1 = c \pi$. This is evident from equation (152), as we have supposed that no part of the head is absorbed in generating velocity, or in overcoming the resistance of bends. If the bend at π were taken

into consideration, then $v = \left\{ \frac{2 g H_0^1}{c_f \times \frac{L_0^1}{r} + c_b} \right\}^{\frac{1}{2}}$.

SECTION XII.

RAIN-FALL.—CATCHMENT BASINS.—DISCHARGE INTO CHANNELS.
—DISCHARGE FROM SEWERS.—LOSS FROM EVAPORATION, ETC.

A catchment basin is a district which drains itself into a river and its tributaries. It is bounded generally by the summits of the neighbouring hills, ridges, or high lands forming the water-shed boundary; and may vary in extent from a few square miles to many thousands; that of the Shannon is 4,544 square miles. The average quantity of water which discharges itself into a river will, *cæteris paribus*, depend on the extent of its catchment basin, and the whole quantity of rain discharged on the area of the catchment basin, including lakes and rivers.

The quantity of rain which falls annually varies with the district and the year; and it also varies at different parts of the same district. The average quantity in Ireland may be taken at about 34 inches

TABLE of some Catchment Basins in Ireland.

Names of Drainage-districts, or Rivers.	Counties or Towns.	Area of Catchment in acres.	Area of Catchment in square miles.
Avonmore	Wicklow and Wexford	128,000	200
Avoca River	Wicklow	170,810	281
Ballinasloe	Mayo	70,000	110
Barrow, Nore, and Suir	Waterford	2,170,000	3400
Blackwater and Boyne	Meath, &c.	695,040	1086
Blackwater	Waterford, Youghal	780,100	1210
Blackwater	Armagh	336,640	520
Blackwater	Meath and Kildare	50,000	78
Bandon River	Cork	145,020	228
Bann, Upper and Lower, and the Main	Down, Antrim	810,240	1200
Boyne	Meath, Westmeath, Kildare, and King's	304,130	478
Brusna (Ferbane)	King's	380,120	608
Ballyteigue	Wexford	20,752	41
Ballinamore and Ballycounel	Cavan, Fermanagh, Leitrim, and Roscommon	101,455	158
Breeogue	Sligo	180,408	282
Ballinhassig	Cork	23,500	37
Cappagh	Galway	34,850	54
Coolaney	Sligo	90,744	141
Camoge	Limerick	61,184	95
Dunmore	Galway, Mayo, and Roscom- mon	90,161	140
Dodder	Dublin	35,200	55
Deel	Meath and Westmeath	64,000	100
Dee	Louth and Meath	78,000	121
Erne	Belturbet, Enniskillen	1,014,400	1585
Foyle	Londonderry	944,040	1476
Fergus	Clare and Galway	134,400	210
Fane	Louth	87,400	136
Glyde	Louth, Meath, Monaghan, and Cavan	176,813	276
Inny	Meath, Westmeath, Long- ford, and Cavan	231,110	361
Kilbeggan	Westmeath and King's	88,030	137
Liffey and Tolka	Dublin, &c.	328,320	513
Lee	Cork	470,400	735
Lough Gara and Mantua ..	Roscommon, Mayo, and Sligo	128,000	200
Loughs Oughter and Gowna and River Erne	Cavan, Leitrim, and Longford	260,480	407
Lough Neagh	Londonderry, Antrim, Down, and Armagh	1,411,320	2205
Lough Mask and River Robe	Mayo and Galway	225,000	351
Loughs Corrib, Mask, and Carra	Galway and Mayo	780,000	1218
Longford	Longford	72,320	113
Moy	Mayo, Ballina	661,120	1033
Main	Antrim	37,000	60
Monivea	Galway	54,000	84
Maghera	Down	10,000	16
Nobber	Meath	40,000	62
Quoile	Down	57,000	89
Rinn and Black River	Leitrim and Longford	74,000	115
Strokestown	Roscommon	70,000	109
Shannon	Different Counties, Towns of Athlone, Limerick	2,008,160	4544
Slaney	Wexford	521,600	815

deep, that which falls in Dublin being 27 inches, and that in Cork 41 inches nearly. The average yearly fall in Dublin for seven years, ending with 1849, was 26.407 inches; and the maximum fall in any month took place in April 1846, being 5.082 inches. "The average fall in inches per month for seven years, ending with 1849, was as follows:—October, 3.060; August, 2.936; January, 2.544; April, 2.503; November, 2.300; July, 2.116; June, 2.005; December, 1.938; September, 1.860; May, 1.814; March, 1.739; February, 1.534."* A gauge at Londonderry, 1795 to 1801, gives 31 inches average; one at Belfast, from 1836 to 1841, gives 35 inches; at Mountjoy, Phoenix Park, 182 feet above low water, 1839 and 1840, there is an average of 33 inches; and at the College of Surgeons, 52 feet over low water, the average is 30 inches for the same two years. Sir Robert Kane assumes that 36 inches is the average fall in Ireland, and that out of that depth 12 inches, or one-third, passes on to the sea, two-thirds being evaporated and taken up by plants. The quantity varies a good deal with the altitude of the district. In parts of Westmoreland it rises sometimes to 140 inches; in London, an average of 20 years' observations, gives a fall of nearly 25 inches.

Forty years' observation at Greenwich, Kent, at 155 feet above the level of the sea, gives the following results:—

* Proceedings of the Royal Irish Academy, vol. v., p. 18.

Description of fall.	Winter.	Spring.	Summer.	Entire Years.
	inches.	inches.	inches.	inches.
Mean annual fall	7.86	7.25	10.47	{ 25.48 25.58
Maximum fall; being a mean of five of the wettest years during forty years	11.05	10.80	14.96	{ 34.00 30.87
Minimum fall; being a mean of five of the driest years during forty years	5.22	4.05	6.80	{ 18.40 16.07

In this table Winter comprises November, December, January, and February; Spring, the next four months; and Summer, the months of July, August, September, and October. The last column contains means of two classes of years: the first figures showing the ordinary years from January to December, and the second, under the first, years from November to October.* We see here that the mean maximum is fully double of the mean minimum, and about one-and-a-half times the mean annual fall, and therefore the necessity for calculating from the minimum fall for all water works in which it is an element, and from the maximum for sewerage works where it is not intended to pass off a portion on the surface or through other available channels.

In the district surrounding the Bann reservoirs in the County Down, the average fall has been so high as 72 inches. In Keswick, the average fall is said to be $67\frac{1}{2}$ inches, and in Upminster, Essex, only $19\frac{1}{2}$ inches. Indeed, it is requisite to obtain the fall from observation for any particular district, when it

* See Mr. James Simpson in the Metropolitan Main Drainage Report, 1857, p. 115.

is necessary to apply the results to scientific purposes; and not the mean average fall alone, but also the maximums and minimums in a series of years and months in each year.

Mr. Symons gives (see Builder for 1860, p. 230) the following heavy falls of rain during 1859:—*Wandsworth*, June 12th, 2·17 inches in two hours; *Manchester*, August 7th, 1·849 inches in twenty-four hours; *Southampton*, September 26th, 2·05 inches in two-and-a-quarter hours; *Truro*, October 25th, during the day, 2·4 inches. The mean falls in the *South Western* Counties were 39·1 inches; in the *South Eastern* Counties, 30·2 inches; in the *West Midland* Counties, 28 inches; in the *Eastern* Counties, 25·4 inches; in the *North Midland* Counties, 24 inches; in the *North Western* Counties, 39 inches; in the *Northern* Counties, 55 inches; and the average of all England, 31·857 inches.

As an instance of extraordinary rain-fall, in connexion with the sewage question, it is stated that 4 inches of rain fell in one hour in the Holborn and Finsbury sewers' district, on the 1st of August, 1846; at Highgate, 3·5 to 3·3 inches; and at Greenwich, 0·95 inches.*

In the upland districts about Manchester, Mr. Homersham† gives the result of observations at Fairfield, Bolton, Rocksdales, Marple, Comlis reservoir, Belmont, Chapel-en-le-Frith, and Whiteholme reservoir, for four years. These give a maximum fall of 61·4 inches at Belmont Sharples in 1847, and

* Metropolitan Main Drainage Report, p. 16.

† Report on the Supply of Water to Manchester.—WEALE.

a minimum of 24·8 at Whiteholme reservoir in 1844. The general average for the four years being 42·49 inches.

April is the driest month, and October, or about it, the wettest month, and the average fall during the year varies sometimes as two to one.

The proportion between the quantity which falls, and that which passes from a catchment basin into its river, also varies very considerably. When the sides of a catchment basin are steep, and the water passes off rapidly into the adjacent river or tributaries, there is less loss by evaporation and percolation than when they are nearly flat. The soil, subsoil, and stratification, have also considerable effect on the proportion. Reservoirs being generally constructed adjacent to steep side falls, give a much larger proportion of the quantity fallen than can be obtained from rivers in flatter districts; besides, the quantity of rain which falls on the high summits, near reservoirs, almost always considerably exceeds the average fall. As 640 acres is equal to one square mile, and one acre is equal to 43,560 square feet, a fall of one inch of rain is equal to 3,630 cubic feet per acre, and to $3,630 \times 640 = 2,323,200$ cubic feet per square mile: the proportion of this fall, for each acre, or square mile of the catchment basin, which enters the river, must depend entirely on the district and local circumstances, the full or maximum quantity being retained on lakes. A stream delivering 53 cubic feet per minute supplies an equivalent to 12 inches of rain-fall collected per square mile per annum.

It is too often taken for granted that the discharge from a catchment basin takes place, into the conveying channels, in nearly the same time that a given quantity of rain falls. Perhaps the largest registry on record in Great Britain is a fall of four inches in an hour. The maximum fall in any hour of any year seldom exceeds half of this amount, and then perhaps only once in several years. The quantity which falls will not be discharged into the channels in the same time. The quantity discharged,

QUANTITY PER ACRE FOR A GIVEN DEPTH OF FALL.

Fall in inches.	Cubic feet per acre.	Fall in inches.	Cubic feet per acre.	Fall in inches.	Cubic feet per acre.	Fall in inches.	Cubic feet per acre.
2	7200	$\frac{1}{2}$	1815	$\frac{1}{8}$	454	$\frac{1}{20}$	181
$1\frac{1}{4}$	6352	$\frac{3}{8}$	1361	$\frac{1}{9}$	403	$\frac{1}{30}$	121
$1\frac{1}{2}$	5445	$\frac{1}{4}$	907	$\frac{1}{10}$	363	$\frac{1}{40}$	91
$1\frac{3}{4}$	4537	$\frac{1}{5}$	720	$\frac{1}{12}$	302	$\frac{1}{50}$	73
1	3630	$\frac{1}{6}$	605	$\frac{1}{14}$	259	$\frac{1}{60}$	61
$\frac{3}{4}$	2723	$\frac{1}{7}$	510	$\frac{1}{16}$	227	$\frac{1}{70}$	52

and time, will depend a good deal on the season and district. The arterial channel receives the supply at different places and from different distances, and the water in passing into and from it does not encounter the same amount of resistance as if it all passed first into the upper end. Less sectional area is therefore necessary than if the whole discharge had to pass through the whole length of the channel and during the time of fall. The relation of the quantity of rain-fall to the portion which flows into the main channel, as well as the time which it takes to arrive at it, and the places of arrival, must be known

before the proper size of a new channel can be determined, particularly sewers in urban districts. A pipe sufficient to discharge the water from 200 acres need not be 20 times the discharging power of one exactly suited to 10 acres of the same district, for the discharge from the outlying 190 acres will not arrive at the main in the same time as that from the adjacent 10 acres.

The following table of rain-fall, at Athlone, central in Ireland, was furnished to the Royal Irish Academy by General Sir H. D. Jones, and is printed in the Proceedings.* The average for four years, gives 29 inches, and the effect on the Upper and Lower Sills of the Lock as affecting the rise and fall of the Shannon, affords valuable data, although not analysed. The rise and fall on the sills is the sum of the monthly risings and fallings for each year, and must be divided by 12 to get the average monthly rise and fall. In 1845 the greatest rise was in January, 2 feet 9 inches at the upper sill, and 3 feet 11½ inches at the lower sill. In 1846 the greatest rise was 2 feet 5 inches in October, at the upper sill; and 5 feet 6½ inches on the lower sill, in August.

	Upper Sill. Maximum rise in one month.	Lower Sill. Maximum rise in one month.
1845	2 ft. 9 in. January	3 ft. 11½ in. January.
1846	2 ft. 5 in. October	5 ft. 6½ in. January.
1847	3 ft. 1 in. November.	4 ft. 6 in. May.
1848	3 ft. 3 in. February	4 ft. 11 in. February.

The sum of the risings and fallings for each month, taken as a mean of four years, is nearly the same

* Vol. iv.

YEARS.	RAIN.				RIVER SHANNON.				WIND.										
	Quantity which fell each year.		Greatest fall in one night. Greatest dry and night.	Number of Days without rain.	Greatest number of successive fall for each month.		Total Rise and Fall of the Shannon during each year; being the sums of the rise or fall for each month.		Number of days in which the Wind was										
					With Rain.	Without Rain.													
	Days.	Nights.	Total.	Inches.	Inches.	No.	No.	No.	No.	No.	No.	No.	No.	No.	No.	No.	No.	No.	No.
Inches.	Inches.	Inches.	Inches.	No.	No.	No.	No.	Feet. inch.	Feet. inch.	Feet. inch.	Feet. inch.	No.	No.	No.	No.	No.	No.	No.	No.
1845	14-02	12-97	26-99	1-18	117	9	14	14 11	11 8½	24 4	18 9	60	20	24	47½	52½	31	41	89
1846	14-87	17-50	32-37	1-45	174	15	12	12 8½	15 0½	19 7	24 0	58	15	15	56	58	47	47	69
1847	12-85	10-65	23-50	0-89	193	11	8	17 2	14 8	22 11½	21 1½	51	21	22	49	67	39	49	67
1848	17-66	15-77	33-43	1-11	165	13	14	17 5½	19 0	24 0	27 4	45	27	16	43	67	48	57	63
Amount for four years.	59-40	56-89	116-29	1-45	709			62 3	60 5	90 10½	91 2½	214	83	77	195½	244½	165	194	288
Averages for one year.	14-85	14-22½	29-07½		177½			15 6½	15 1½	22 8½	22 9½	53½	20½	19½	48½	61½	41½	48½	72

at either sill. The general average of the rise and fall for the upper sill, is about 1 foot 3 $\frac{1}{2}$ inches each way, and 1 foot 10 $\frac{1}{2}$ inches at the lower sill. These would give 2 feet 7 inches for the average difference of level in the Shannon above, and 3 feet 9 $\frac{1}{2}$ inches for that in the Shannon below. In Lough Allen catchment of 146 square miles, the maximum rise was sometimes 6 inches in 24 hours, calculated at $\cdot 568$ inch of depth of rain, over the catchment area. Above Killaloe, the catchment is 3,611 square miles, and the floods about once a year rose 6 inches in 24 hours, or $\cdot 296$ inch in depth of rain over the catchment. Once, in 1840, it is reported to have risen 12 inches, or $\cdot 6$ inch of rain over the catchment in one day.

MAXIMUM DISCHARGES OF THE SHANNON AND ERNE, AND A TRIBUTARY OF THE LATTER, THE WOODFORD RIVER.

RIVERS IN IRELAND.	Extent of catchment, statute acres.	Square miles.	Maximum discharge per minute in cubic feet.	Cubic feet per minute from each acre.	Cubic feet per minute from each square mile.
Shannon, at Killaloe, measured previous to the commencement of Shannon Works, about	3,000,000	4087.5	1,000,000	0.33	211
Lower Erne, measured during the very high floods of Jan. 1851, at Belleek	974,000	1521.0	657,511	0.67	420.
Upper Erne, measured during the very high floods of Jan. 1851, at Belturbet	309,000	482.8	257,771	0.83	531
Woodford River, Counties of Leitrim and Cavan, measured during the very high floods of Jan. 1851, at Ballyconnell	90,000	140.6	101,035	1.12	717
Yellow River, or upper portion of the Woodford River, measured during the very high floods of Jan. 1851, Co. Leitrim	5,000	7.8	52,125	10.43	6075.

These results show how difficult it is to draw any inference from discharge and area of catchment alone, as the discharge, per minute per acre, must vary with the contour and elevation of the district in the same course; and with the climate also, in different countries. We have observed ourselves the maximum discharges to vary up to 6 cubic feet per minute per acre, the lesser maximums being due to broad flat districts, and the greater maximums to higher and steeper districts, near the sources. In the Proceedings of the Institution of Civil Engineers, Ireland, vol. iv., from which we have collected and arranged some of the foregoing information, it is stated, p. 96, that the ratio of the discharge to the rain-fall, on a catchment on the Glyde, of 79,433 acres, for three months, ending March 13th, 1851, was 1.49 to 1 up to January 13th; 1.39 to 1 up to February 13th; and 3.86 to 1 up to March 13th, making a general average of 1.59 to 1; the whole rain-fall for the three months being only 5.89 inches, while the discharge was 9.35 inches! We fancy there is a mistake here. The whole catchment of the Glyde is 176,813 acres, and there is no data to show the discharge previous to or after the rain-fall from which to calculate the difference due to it *per se* for the three months; nor is the place or method of gauging stated. The supply from springs and the actual discharge before and after rain-fall must be correctly gauged before the proportion passing into the main channel in a given time, can be properly estimated; the results just stated clearly contradict themselves. The following anomalous results

from p. 47 of the same work are also worthy of note. In five different districts the discharge is gauged, or estimated, greater than the fall, as shown in the following table. It is not stated, however, if

District.	River.	Catchment in Acres.	DECEMBER 1850.		JANUARY 1851.	
			Total fall of Rain by gauge in inches.	Total depth of discharge off catch- ment in inches.	Total fall of Rain by Castlebar gauge in inches.	Total depth of discharge off catch- ment in inches.
Saleen	Saleen	2,025	3.55	0.20	6.33	0.20
Lannagh	Castlebar ..	20,640		5.40		8.55
Balla	Manulla ...	33,500		5.40		8.18
Mask and Robe .	Robe	70,000		"		7.30
Dalla	Dalla	3,200	4.00	6.527	"	"
	Owenmore .	32,000		5.705	"	"

the depths passed off, estimated over the catchments, include the flow before the commencement of the rain. If so the results are so far useless; and if they do not include it, there must be an error somewhere. Indeed, in the Robe we have evidence that not more than 58 per cent. passed from the catchment to the river, from Mr. Betagh's paper, the results of which are arranged below. Also, in July 1850, it is shown that in the Lannagh district only .53 inch in depth passed off the catchment from a fall of 1.83 inches, or about one-third of the depth. The method of determining this was unobjectionable. Where such discrepancies as above exhibited exist, it is important that the method of gauging, and the whole calculation, should be shown, in order that other engineers should be able to judge

of their accuracy; otherwise the results should be rejected, no matter under whose authority they may be published.

The following information has been collected and arranged by us from a paper by Mr. Betagh, in the Proceedings of the Institution of Civil Engineers, Ireland, vol. iv. In January 1851, 3·41 inches of rain fell in seven days, producing the maximum discharge of 85,836 cubic feet; while in December 1852, 3·17 inches, also falling in seven days, produced 115,656 for the maximum. At the beginning of the first fall there was flowing 26,640 feet, leaving the effects of the seven days' rain $85,836 - 26,640 = 59,196$ cubic feet, while in the second year the quantity flowing at first was 75,360 cubic feet, leaving the effects of the seven days' rain-fall equal to $115,656 - 75,360 = 40,296$ cubic feet. The effect of the previous state of the weather on the catchment must always modify, to a considerable extent, the discharge from a given rain-fall, and this has more to do with the results than the effect of arterial drainage itself, unless so far as one is a result of the other. Taking the mean of 1851 and 1852, it appears that the evaporation in the Ballinrobe catchment was to the rain-fall as 41·6 to 98·7, or about 42 per cent. This is certainly, from the nature of the catchment, less than the average through Ireland, which cannot be less than 60 per cent. In high, steep districts, fully three-fourths or 75 per cent. of the rain-fall can be collected, and at times, when the catchment is saturated, nearly the whole; even in some few limited cases, when springs or hidden

TABLES, showing in detail, for the years 1851 and 1852, the Monthly Fall of Rain and the corresponding Discharge of the River Robe, at Ballinrobe, County Mayo; the catchment basin being 70,000 acres, or 110 square miles; the lower end 100 feet, the upper end 330 feet; and the average height of the surface about 180 feet above the level of the sea. The average fall of the river, not including the rapids, is from one to two feet per mile; the catchment is about 20 miles long, about one-tenth of the area bog or low marsh, and nine-tenths clayey and gravelly. The river is about 33 miles long.

OBSERVATIONS IN 1851.

MONTHS.	Rain-fall each Month in inches.	Discharge each Month of rain-fall in inches.	Discharge in cubic feet per minute, from a catchment of 70,000 acres, for each month.			Discharge in cubic feet per minute, per acre, for each month.		
			Maximum	Minimum.	Average.	Maximum.	Minimum.	Average.
January .	9.2	7.4	85,836	20,133	43,373	1.158	.287	.620
February .	6.8	4.7	72,448	18,420	30,410	1.034	.263	.434
March . .	4.4	3.6	49,137	10,800	20,045	.702	.155	.300
April . .	3.4	2.5	24,200	5,700	14,355	.345	.082	.205
May . . .	1.0	.8	5,820	4,125	5,001	.083	.059	.071
June . . .	3.8	.8	7,040	1,114	4,230	.100	.010	.000
July . . .	3.8	.5	4,920	1,500	2,558	.070	.021	.030
August . .	2.4	0.0	17,055	1,240	4,866	.243	.017	.000
September.	1.0	0.5	4,746	1,200	2,854	.067	.017	.040
October . .	5.0	1.6	23,080	6,940	12,588	.343	.090	.170
November .	1.3	1.2	12,852	6,000	7,827	.183	.085	.111
December .	2.0	2.5	44,715	6,210	14,373	.098	.088	.205
Total	45.6	27.	352,740	83,502	163,380	4.005	1.180	2.33

supplies are re-tapped, a larger discharge may take place than that due to the catchment and rain-fall; but these do not affect the general question.

“The future population of the suburbs of London is calculated at 30,000 inhabitants per square mile. According to the following data, some of the densest portions of our large towns have a population of 220 persons to an acre. The population on the

RIVER ROME OBSERVATIONS IN 1852. *Continued from last page.*

MONTHS.	Rain-fall each month in inches.	Discharge each month of rain-fall in inches.	Discharge in cubic feet per minute, from a catchment of 70,000 acres, for each month.			Discharge in cubic feet per minute, per acre, for each month.		
			Maximum.	Minimum.	Average.	Maximum.	Minimum.	Average.
January. .	7.5	5.2	41,600	12,852	28,730	.594	.189	.410
February .	4.8	4.3	50,400	8,100	25,200	.805	.117	.301
March . .	1.0	0.7	0,000	2,737	0,702	.137	.039	.095
April . .	1.1	0.5	3,031	1,408	2,477	.050	.020	.035
May . . .	1.0	0.4	3,031	1,050	1,801	.050	.015	.020
June . . .	0.0	1.2	22,704	1,400	0,547	.325	.020	.003
July . . .	2.5	1.0	15,430	3,172	0,057	.220	.045	.087
August . .	4.5	0.0	3,850	2,230	3,070	.055	.032	.043
September.	1.8	0.5	3,427	2,042	2,874	.048	.037	.041
October. .	3.0	1.0	32,040	1,114	5,093	.457	.010	.084
November .	5.5	5.2	45,300	17,000	30,742	.648	.242	.430
December .	12.0	0.5	115,050	23,232	54,840	1.057	.331	.783
Total	59.1	30.1	354,004	77,093	175,134	5.058	1.007	2.407

north side of the Thames is about 75 persons per acre, and on the south side 28 persons per acre. Taking the average density of population in our twenty-one principal towns, there appear to be 5045 inhabitants to the square mile; but, from the following table, extracted from Dr. Duncan's report on Liverpool, it will be seen that if we select five of our most populous cities, the average in these is much greater, while in others, it is equally certain that the crowding is far less than the general standard to which we have referred :—

Towns.	Inhabitants to a Square Mile.	
	Total Area.	Builded Area.
Leeds	20,892	87,256
London	27,423	50,000
Birmingham	33,669	40,000
Manchester	33,224	100,000
Liverpool	100,899	138,224

Dr. Duncan, however, states that there is a district in Liverpool containing 12,000 inhabitants crowded together on a surface of only 105,000 square yards, which gives a ratio of 460,000 inhabitants to the geographical square mile. In the East and West London Unions, Mr. Farr has estimated that there are nearly 243,000 inhabitants to a geographical square mile; but, great as this overcrowding is, the maximum density of Liverpool exceeds that of the metropolis by nearly double.”*

The amount of sewage from each person is calculated about FIVE CUBIC FEET PER PERSON, including the supply from manufactories, breweries, distilleries, &c. SEVEN FEET PER HEAD has been recommended as data to calculate from by Captain Galton, Messrs. Simpson and Blackwell, in their Report on the Main Drainage, and it has been found that about half of the estimated quantity of sewage would pass off in six or eight hours.

In calculating the size of sewers; however, the rain-fall must be provided for, in addition to the sewage matter from houses and public establishments. Mr. Bazalgette calculated this for the London sewerage at $\frac{1}{4}$ th of an inch fall in 24 hours in the urban districts, and $\frac{1}{8}$ th of an inch for the suburban districts. Captain Galton and the Messrs.

* Illustrated News, September 8th, 1855.

Simpson and Blackwell assumed $\frac{2}{3}$ ths of an inch fall during eight hours' maximum flow. This would be 1452 feet per acre. Assuming the highest data, we shall have to provide sewers to discharge in eight hours

1,452 cubic feet of rain water per acre,

$3\frac{1}{2}$ cubic feet of sewage nearly per person.

Assuming a population of 80 persons per acre, then these figures would become

1,452 cubic feet for rain, $\left\{ \begin{array}{l} \text{in eight hours, or} \\ \text{about } 3\frac{1}{2} \text{ cubic feet} \\ \text{per minute, per acre,} \end{array} \right.$
 280 cubic feet for sewage,

which shows that the sewage is not more than $\frac{1}{5}$ th of the rain water; and that, in calculations for the size of sewers, the surface water is the most important element to be considered. If we had assumed a larger fall of rain, the difference between sewage and rain would be greater. On the 20th June, 1857, the day after heavy rain, the referees on the Metropolitan Drainage question found the Norfolk-street sewer to discharge 3 feet; the Essex-street sewer $5\frac{1}{4}$ feet; the Northumberland-street sewer $3\frac{3}{4}$ feet; and the Savoy-street sewer $20\frac{1}{2}$ feet per minute per acre; but the last result has been controverted.

It appears that the daily amount of sewage varies from 4.8 cubic feet per head in the more thickly inhabited portions of London, occupied by a larger portion of the poorer classes, to 8 cubic feet per head in the western districts, where the value of water is more appreciated, and the cost less a matter of consideration; and the average of the whole

metropolitan districts appears to be 5·8 cubic feet per head per diem. If the day be divided into three periods of eight hours each, the amount of the maximum flow is between nine A.M. and five P.M. and 49 per cent. of the whole, whilst only 18 per cent. flows during the eight hours of minimum flow, which occur between eleven P.M. and seven A.M.* The advantage of storm flows in flushing is shown by the heavy rain which occurred on the 20th of June, causing a flow in the Savoy-street sewer which was equivalent to 20 times the ordinary flow at the time. This was six times the maximum flow, and *although the sewer had been scoured, to a considerable extent, by a heavy fall of rain on the previous night, the sample contained more than double the amount of total impurity contained in specimens of ordinary sewage.*

In a town district, such as that drained by the Savoy and Northumberland-street sewers, the quantity running off into sewers, within six hours after the fall, varies from 10 to 60 per cent. of the quantity fallen. Of the rain during the storm of the 20th June, 1857, nearly one inch-and-a-quarter in an hour, 65 per cent., ran off within 15 hours of the fall, viz. :—

46 per cent.	in 45 minutes after the rain ceased,
14 „	in the next 6¼ hours,
5 „	in the next 7½ hours.

In a suburban locality, such as the Counter Creek sewer drain, the quantity reaching the sewers would

* Metropolitan Main Drainage Report, pp. 15, 17.

vary from 0 to 30 or 40 per cent. in 24 hours after the rain.*

In the Holborn and Finsbury divisions Mr. Roe calculated that an 18-inch cylindrical pipe, laid at an inclination of 1 in 80, is sufficient for 20 acres of house-sewage, while a 5-inch pipe, laid at an inclination of 1 in 20, is necessary for 1 acre, and a 3-inch pipe, laid also at 1 in 20, for $\frac{1}{4}$ acre. A pipe 30" in diameter, laid with an inclination of 1 in 200, would discharge 1700 cubic feet per minute, and perfectly drain 200 acres of urban land covered with houses to the extent of 4000 or upwards, and each house having a water supply of 150 gallons per diem. In each of these cases, however, the discharge must depend on the head and length of the pipe as well as the inclination at which it is laid. Assuming the inclination of those pipes to correspond with the hydraulic inclination, we have calculated their discharging powers with water to be respectively 807, 72, 20, and 1700 cubic feet per minute, the areas to be drained being 20, 1, $\frac{1}{4}$, and 200 acres. *In all calculations of this kind it is necessary, for accuracy, to ascertain not only the maximum rain-fall per hour, but also the proportions discharged per hour, according to the season and district, into the main channel, as well as the junctions or places of arrival.* In urban districts, 1500, 2100, and sometimes 3600 cubic feet per hour per acre, have to be discharged after extraordinary rain-falls. These may be taken as maximum results. The

* Metropolitan Main Drainage Report, pp. 75, 78.

gaugings of the Westminster sewers in summer give 53 cubic feet per hour for the urban, and 17 feet for the suburban, according to Mr. Hawkins.

In urban districts, however, a much larger quantity of water is conveyed more rapidly, *cæteris paribus*, to the mains, than in suburban districts and catchment basins generally, in which the maximum discharge per acre per hour, even in the steeper and higher districts, seldom exceeds 700 cubic feet, and varies from about 20 cubic feet for the larger and flatter districts upwards. This arises from the impervious nature of the surfaces it falls upon in towns, and the lesser waste in passing to the drains, as well as a large portion of the supply being often artificial. From 70 to 90 cubic feet* per acre per hour, is generally taken for the maximum discharge from the average number of catchment basins; this is nearly equal to a supply of one-fiftieth part of an inch in depth from the whole area. Thorough-drainage increases the supply and discharge. *Every catchment basin has, however, its own peculiar data, and a knowledge of these is necessary before we can draw any correct conclusions for new waterworks in connection with it.* It may be remarked, however, that any conclusions drawn from experiments on the supply of tributaries, particularly in high districts, are wholly inapplicable to the main channel into which they flow. The flow into tributaries and mountain streams, or rivers, is always more rapid than into

* Some interesting observations on rain-fall and flood discharges are given in the Transactions of the Institution of Civil Engineers, Ireland, for 1851, pp. 19-33, and pp. 44-52.

main channels and rivers in flat districts, and the supply from springs often forms a large portion of the water flowing in them.

TABLE showing Summer Discharges of some English Rivers, as collected from various authorities, re-arranged, showing to some extent the effect of Springs in supplying Channels in different places.

NAMES OF RIVERS.	Height above sea in feet.		Catchment in square miles.	Discharge in cubic feet per minute.	Discharge per square mile in cubic feet per minute.	Representing inches of rain-fall per annum.	Total average rain-fall in inches per annum.
	Valley.	Hill.					
Gade, at Hunton Bridge, chalk . . .	150 to 500		69.5	2,500	36.2	8.19	..
Len, at Lea Bridge, chalk, (Rennie, April 1796) . . .	30 to 600		570.0	8,880	15.58	3.53	..
Loddon, (Feb. 1850,) green sand . . .	110 to 700		221.8	3,000	13.53	3.01	25.4
Medway, driest seasons, (Rennie 1787,) clay		481.5	2,209	4.59	1.04	..
Mimram, at Panshanger, chalk . . .	200 to 500		29.2	1,500	51.4	11.58	20.0
Medway, ordinary summer run, (Rennie, 1787,) clay		481.5	2,520	5.23	2.19	..
Nene, at Peterborough, oolites, Oxford clay, and lias.	10 to 600		620.0	5,000	8.45	1.88	23.1
Plym, at Sheepstor, granite	800 to 1,500		7.6	500	71.4	15.10	45.0
Severn, at Stonebench, silurian . .	400 to 2,600		3,900	33,111	8.49	1.98	..
Thames, at Staines, chalk, green sand, Oxford clay, oolites, &c.	40 to 700		3,086	40,000	12.98	2.03	24.5
Verulam, at Bushey Hall, chalk . . .	150 to 500		120.8	1,800	14.9	3.37	..
Wandle, below Carshalton, chalk . .	70 to 350		41.0	1,800	43.9	9.03	24.0
Trent, at its mouth, oolites and Oxford clay	100 to 600		3,921

The above information has been obtained from Mr. Beardmore and Mr. Hughes' books, and from Rennie's reports. The effect of the geology and fissures in the chalk and mountain limestone formations on the springs of a catchment basin, and on maintaining the summer discharge, should be carefully noted as one of the elements entering into catchment basin statistics. Indeed, the maximum and minimum discharges from catchments are of as much importance to the engineer as the averages, and, for many purposes, more important. There were abundant opportunities of acquiring this information for all our Irish rivers, but we are not aware if these were turned to account.

The effects of evaporation are very variable; sometimes 58 or 60 per cent. of the annual fall is carried off in this way from ordinary flat tillage soils, and other estimates are much higher; much, however, depends on the soil, subsoil, inclination, stratification, and season. The evaporation from water surfaces exceeds the annual fall in these countries by about one-third; and that from flat, marsh, and callow lands exceeds the evaporation from ordinary tillage, porous, and high lands. When the flat lands along the banks of rivers extend considerably on both sides, an extra fall is necessary into the main channel, along the normal drains, otherwise such lands must suffer from excessive evaporation as well as floods. Evaporation also varies with the climate, and in this country we may assume that one-third of the whole rain-fall passes on to the sea.

In a paper in the Journal of the Royal Agricultural

Society of England, vol. v, part 1, 1844, Mr. Josiah Parkes shows, that $42\frac{1}{2}$ per cent. of the whole annual rain of England filters through the soil, and $57\frac{1}{2}$ per cent. evaporated, being the mean results of eight years' observations, from 1836 to 1843, both included. The mean evaporation and filtration for each month during this period is shown and arranged by us in the following table:—

MONTHS.	Total falling.	Evaporated.		Remaining.		Deposited in Tons and Cubic feet per acre.	
	Inches.	Inches.	per cent.	Inches.	per cent.	Cubic feet	Tons.
January	1·847	·540	29·3	1·307	70·7	4,744	132
February	1·971	·424	21·6	1·547	78·4	5,616	156
March	1·617	·540	33·4	1·077	66·6	3,910	109
April	1·456	1·150	79·0	0·306	21·0	1,111	39
May	1·856	1·748	94·2	0·108	5·8	392	11
June	2·213	2·174	98·3	0·039	1·7	142	4
July	2·287	2·245	98·2	0·042	1·8	87	2·4
August	2·427	2·391	98·6	0·036	1·4	131	3·6
September	2·630	2·270	86·1	0·360	13·0	1,330	37
October	2·823	1·423	50·5	1·400	49·5	5,082	141
November	3·837	0·579	15·1	3·258	84·9	11,820	328
December	1·641	0·104	6·3	1·537	93·7	6,552	182
Yearly averages	26·614	15·320	57·6	11·294	42·4	40,932	1145

The maximum quantity, 32·10 inches, fell in 1841, and the minimum in 1837, 21·10 inches. The maximum quantity which fell in January was 3·95 inches, and the minimum ·31 inch; in February 2·85 and 1·02 inches; in March 3·65 and 0·34 inches; in April, 2·57 and ·34 inches; in May 5·00 and ·70

Experiments were made at York in 1832, 1833, and 1834, for the British Association, with three gauges,—the first placed on a large grass plot in the grounds of the Yorkshire Museum; the second at a higher elevation, 43 feet 8 inches, on the roof of the Museum; and the third on a pole 9 feet above the battlements of the great tower of the Minster, at an elevation over the gauge on the ground of 212 feet 10½ inches. The quantities received were as follows:—

	Depth for three years.	Average depth for one year.
First gauge . . .	64.430 inches . . .	21.477 inches . . .
Second gauge . . .	52.160 „ . . .	17.380 „ . . .
Third gauge . . .	38.972 „ . . .	12.991 „ . . .

Professor Phillips gives the following formula for calculating the difference between the ratios of rain falling on the ground and at any height h in the same place— t° the temperature of the season, and c a coefficient dependent upon it; then the difference d is

$$d = c h \frac{t^\circ}{110}.$$

The mean height at which rain begins to be formed by this formula is 1,747 feet over the ground; and at 356 feet high, the depth which falls is one-half of what falls on the ground.*

A discussion of the mean temperature in connexion with the fall of rain, has been made at Greenwich for the years 1852, 1853, and 1854; and at Oxford for the years 1855, 1856, and 1857. The result shows an average of 160.3 rainy days at Greenwich for each year, and 146.6 at Oxford. The difference of the mean temperatures of the day of rain and the day before is less than that of the day of rain and the day after.

* *Vide* Civil Engineer and Architect's Journal for 1860, p. 107.

	Mean tempera- ture, day before rain.	Mean tempera- ture, day of rain.	Mean tempera- ture, day after rain.
Greenwich observations	49.25°	49.27°	48.98°
Oxford do.	49.50	49.63	49.44

Dividing the winds into two groups, northerly and southerly, the Oxford observations give the direction for 218.5 days' fair weather. The wind was northerly for 131.5 days, and southerly for 87 days. For the remaining 146.5 rainy days, the wind was northerly for 64.5 days, and southerly for 82 days.

SECTION XIII.

WATER SUPPLY FOR TOWNS.—STRENGTH OF PIPES.—SEWERAGE ESTIMATES AND COST.—THOROUGH-DRAINAGE.—ARTERIAL DRAINAGE.

SUPPLY.—QUALITY.

The supply of water to towns has become latterly a subject of considerable importance. Three points have to be considered,—first, a sufficient supply at high pressure, when it can be obtained within a reasonable expenditure; secondly, the quality; and, thirdly, the cost. The advantages in towns of high pressure are now apparent to all in overcoming fire; fronts of houses and pavements may also be cleaned, and streets watered if the supply be abundant. The highest apartments can be supplied, and even mechanical power can be obtained for many purposes, as grinding coffee, at a reasonable cost. Mr. Glynn says,* “In many parts of London water is supplied at 4*d.* for 1000 gallons, at a pressure of 150 feet: a gallon of water weighs 10 lbs., so that 1000

* Power of Water.—WEALE.

gallons of water falling 150 feet, are equal to 1,500,000 lbs. falling one foot; and if 1500 gallons of water be used in one hour, they are equal to 37,500 lbs. falling one foot in one minute, or somewhat more than a horse's power, which is 33,000; therefore, it may be assumed, that the cost of a horse's power for an hour in such cases, is only 6d."

The number of gallons of water required for the supply of each person, including all collateral uses, has been differently estimated, and varies in almost every town, and even in the same city—London, for instance, when supplied by different companies and under different systems. 44 gallons per head, per diem, were supplied by the several companies of London in 1853, while evidence has been given to show that the actual average consumption for all purposes did not exceed 10 gallons per head, per diem; the remainder having been wasted under an imperfect system of distribution. It is asserted that when the supply is 25 gallons per head, per diem, that 5 gallons of it are used for purposes requiring filtration, 10 gallons for purposes not requiring filtration, and 10 gallons wasted, or two-fifths of the supply. As there must be a considerable loss under even the best system of supply, we may assume, with the Board of Health, that a *minimum supply* of 75 gallons per house, per diem, or 15 gallons per person, per diem, is necessary.

The following is an abstract of the average number of gallons of water furnished per diem, by different water companies in London, during the year 1853, to each house, including manufactories and public establishments as houses :—

	Gallons.	
	Per House.	Per Person.
New River Company	193	38·3-5
East London Water Works	187	37·4-5
West Middlesex Water Works	204	40·6-5
Grand Junction Water Works	{ 319 336	{ 63·4-5 67·1-5
Southwark and Vauxhall Companies' Houses	175	35
Ditto average houses, manufactories, public establishments	209	41·4-5
Chelsea Water Works	227	45·2-5
Hampstead Water Works	111	22·1-5
Kent Water Works	270	55
	2233	440·3-5
Mean Values	223·3-10	44·3-5

These quantities have been calculated from the parliamentary returns made in 1854; and if there be any truth in the calculations and returns of the quantities actually consumed per person—said to be 10 gallons—we get the proportion, as 10 is to 34 so is the quantity consumed to the quantity wasted. But, even assuming the quantity consumed to be 20 gallons per head, what an immense loss is here exhibited from want of a suitable system of distribution.

For large towns it is safe to provide for many purposes, besides mere personal or house wants; and it is safer, where it can be done without much cost, to provide for a supply of 40 gallons to each inhabitant, even if this quantity shall not be used or

raised. For high pressure, the supply required will generally vary from 15 to 42 gallons, or from 3 to 7 cubic feet to each inhabitant, or an average of about 30 gallons, including the supply to stables, offices, manufactories, and breweries.

The quality of water for drinking, washing, or cooking, is also an important element in selecting a source of supply. Hardness is measured by the number of grains of chalk or carbonate of lime to a gallon of water, each called a degree. The average hardness of spring water is about 26° , that is, 26 grains of carbonate of lime to one gallon of water. Rivers and brooks have an average hardness of 13° , and water derived from surface drainage 5° ; hence the great advantage of the latter kinds of water in washing. The average hardness of the London pipe waters is from 10° to 16° . The following report and analyses furnished to me, in 1855, by Professor Sullivan, of the Museum of Irish Industry, Dublin, will show what is generally required on this head:—

“On the annexed page you will find the numerical results of my analyses of the four samples of water which you left with me for examination. From the table you will perceive that the water of the Mattock River appears to be the purest, so far as the nature and the amount of the foreign substances held dissolved in it is concerned. The water of the Boyne comes next in quality to that of the Mattock River, the pump water being in every sense the worst, so far as amount of ingredients can be taken as a test of the quality of a water; in this respect, indeed, it resembles the water of the deep wells of London and elsewhere.

“As the ordinary mode in which the quality of a water, for drinking and for culinary and like purposes, is judged of is, by the comparative amount of organic matter, the total amount of dissolved matter, and its hardness, according to the ‘soap test,’ I shall give in the following table the numbers representing each of these qualities:—

TABLE showing the number of grains of Organic Matter, and the number of grains of Solid Matter, in an imperial gallon of

Water from	Number of Grains of Organic Matter, per Imperial Gal.	Number of Grains of Solid Matter, per Imperial Gal.	Degree of Hardness according to the Soap Test.
No. 1. Tullyescar . . .	8·975 grs.	31,175	15 8-10ths.
„ 2. River Mattock .	2 (about)	15,360	9 1-10th.
„ 3. River Boyne . . .	3·250	22,700	14 9-10ths.
„ 4. Burn's Pump . .	7·100	76,850	34 4-10ths.

“In order to render this table more instructive, it may be well to subjoin a few of the results obtained from the analyses of the waters of other localities.

TABLE showing the number of grains of Solid Matter contained in one gallon of the following Water:

Thames, at Greenwich	27·9 grains.
„ London	28·0 „
„ Westminster	24·4 „
„ Twickenham	22·4 „
„ Teddington	17·4 „
New River (London)	19·2 „
Lea „ „	23·7 „
Trafalgar Square Fountain, Deep Well	68·9 „
Well in St. Giles', Holborn	105·0 „
Artesian Well at Grenelle (Paris)	9·86 „

“The following are some of the results obtained from an examination of the waters in the neighbour-

hood of Dublin, or which have been proposed as a source of supply :*—

Locality from whence Water was obtained.	Total Number of Grains per Imperial Gallon.	Total Number of Grains of Organic Matter.	Degree of Hardness according to the Soap Test.
Royal Canal (12th Lock) . .	21·0	2·80	14·0 degs.
Grand Canal (7th Lock) . .	16·300	2·30	10 3-4ths.
River Liffey, at Kippure . .	3·525	1·90	0 2-10ths.
„ at Phoulaphouca . .	5·125	1·50	0 2-10ths.
Lough Dan, Co. Wicklow . .	2·800	1·225	0 8-10ths.
River Dodder, at City Weir .	8·350	1·625	1 8-10ths.
Lough Owel	10·225	1·550	6 7-10ths.

“The quality of a water for drinking purposes depends in a great degree upon the condition in which the organic matter is found, much more than upon its quantity. This is, however, a question outside of the domain of chemistry, and can only be solved by the aid of the microscope. I may, however, venture to remark that the organic matter contained in the water of the Boyne and the Mat-

* While these pages were passing through the press, Dr. Apjohn gave the following analyses :—

	Total matter dissolved.	Organic matter.	Hardness.
Grand Canal—mean of seven analyses .	20·78	·95	15·9
Royal Canal—mean of five analyses .	20·76	1·64	14·1
Liffey—mean of eleven analyses .	8·62	1·77	6·1
Analysis of the deposition on pipes from the Portobello basin :—			
Water		2·20	
Organic Matter		9·71	
Sand		10·20	
Per Oxide of Iron and Alumina		3·50	
Carbonate of Lime		74·20	
Carbonate of Magnesia		·19	

tock River is of vegetable origin, and would not, so far as I believe, be injurious to health.

“As a general rule, I believe that the water of clear flowing rivers, even though it may contain a large amount of solid matter, and even of organic matter, will be found wholesomer than well water, especially in towns.

“For certain manufacturing purposes, and for culinary purposes, too large an amount of lime is injurious, but I believe that a certain quantity present in water, is not only not injurious, but in my opinion is of the greatest utility, and renders the waters wholesome. I think the rage for extracting pure water containing only one grain of solid matter to the gallon, or thereabouts, for supplying towns, is carried too far. Such water is, no doubt, the best on a hill side; but, I question whether it is equally well adapted for resting in basins, tanks, pipes, &c., with that containing some lime. The River Dodder and Lough Owel waters appear to me the best adapted for city and town supplies. The River Mattock contains rather more than either, but it is decidedly better than the water of either of the canals from which our Dublin supply is drawn.

“Drogheda is rather badly situated for a supply of very soft water, as almost the whole drainage basin of the Boyne is either situated upon limestone, or the feeders of that river rise through the calcareous drift gravel which covers so much of the country. The water of the Boyne appears to be an excellent water for most purposes, and perhaps the difference between it and the Mattock River

*Tabular Results of the Special Analyses of Four Samples of Water
from the neighbourhood of Drogheda.*

Nature of dissolved matter.	No. 1. Tullyascar.	No. 2. Mattock River.	No. 3. Boyno River.	No. 4. Burns's pump water.	Observations.
Carbonate of lime . . .	0.350	7.802	11.048	21.475	Inclusive of a very small quantity of phosphate of lime and iron not separated from the lime.
Carbonate of magnesia .	0.420	0.610	0.888	0.585	
Sulphate of lime . . .	0.043	2.614	4.450	4.568	
Chloride of magnesium .	0.743	1.258	1.085	8.445	
Chloride of calcium	0.524	
Chloride of sodium	0.901	
Magnesia existing as crenate, &c., in the water.	0.404	
Lime do. do.	0.548	
Silica do. do. . .	0.027	..	0.322	2.212	
Potash and soda existing in water, as nitrates, crenates, and other organic salts	1.544	2.785	0.448	22.808	
Organic matter	8.976	..	3.250	7.100	
Total number of grains per Imperial gallon. .	31.176	15.300	22.700	70.850	

may in part be accounted for by its being taken near the banks, or more probably, perhaps, because it was above and close to where some small stream entered.

“The quantity of solid matter in it, however, was not more than I would expect considering the nature of the locality. I did not draw attention in my Report to a point of some importance—namely, the proportion of lime and magnesia existing as carbonates, and as sulphates, and chlorides. The whole

of the lime and magnesia existing as carbonates, and as sulphates, and chlorides, is precipitated by boiling, the water being thus proportionably rendered less hard; lime and magnesia existing as sulphates or chlorides, on the other hand, are not precipitated. This difference is of great consequence in culinary operations, as where boiled water is used, the carbonates of lime and magnesia are not injurious, and if no sulphates or chlorides be present, the water may be soft after boiling. The same observation applies to water applied to washing clothes when boiled. And lastly, sulphate of lime forms one of the worst elements of fur or deposits upon steam boilers."

The saving in soap effected by a reduction of 10 degrees in hardness, is found to be over 50 per cent.

Some of the metropolitan waters analyzed by Dr. Robert Dundas Thomson, F.R.S., were found, in May 1860, much more impure than others, the samples of which had been taken at the beginning of the month, before the impurities conveyed by the rains had contaminated them. The supply afforded by large and small rivers, as in London, in this table, contrasts most unfavourably with that afforded by the drainage of mountain ridges, as at Glasgow and Manchester. The specimens of water from the two latter cities were taken by the instructions of Mr. Bateman, F.R.S., the engineer, from the main pipes during the month. It should be the object of the London Companies to avoid pumping the water in its most impure state, and to store it when in the condition of the greatest purity.

	Total Impurity per gallon.	Organic Impurity per gallon.
	Grains, or °.	Grains, or °.
Distilled water	0·0	0·0
Loch Katrine water, new supply to Glasgow	3·16	0·96
Manchester water supply	4·32	0·64
THAMES COMPANIES:—Chelsea	17·84	1·48
Southwark	17·08	1·64
Grand Junction	20·72	2·00
West Middlesex	20·08	2·08
Lambeth	20·80	2·40
OTHER COMPANIES:—New River	18·52	1·56
East London	23·64	3·20
Kent	21·68	2·06

The table is read thus:—Loch Katrine water contains in the gallon 3·16 degrees or grains of foreign matter in solution, of which ·96 degrees or grains are of vegetable or animal origin.

Professor Apjohn gives the following analyses of waters furnished to the city of Dublin in 1860. It shows how necessary it is to distinguish the time of taking specimens for analysis, and the previous state of the weather as affecting the foreign matters in the water. The specimens were collected on the 5th and 19th of May, 1860. The quantity operated upon in each instance was an imperial gallon, or 277·273 cubic inches:—

CITY WATER COURSE, DODDER.

	5th May.	19th May.	
Carbonate of lime	4.050	7.308	Specific gravity of specimen (5th May)
Carbonate of magnesia			
Sulphate of lime and chlorides of sodium and magnesium	2.260	2.171	1.00011.
Silex			
Organic matter	1.811	1.101	Specific gravity of specimen (19th May) 1.00014.
	8.302	11.086	

PORTOBELLO BASIN.

Carbonate of lime	7.087	11.000	Specific gravity of specimen (5th May)
Carbonate of magnesia			
Sulphate of lime and chlorides of sodium and magnesium	4.058	3.761	1.00023.
Silex			
Organic matter	3.308	2.280	Specific gravity of specimen (19th May) 1.00031.
	15.123	18.038	

It will be observed that the quantities of saline and other ingredients found in specimens of same water collected at the two separate periods above mentioned are materially different; those obtained at the later date (May 19) containing the larger portion of foreign matters. The extent of this variation is very considerable, and it appears to Dr. Apjohn to have been the consequence of a very considerable fall of rain, which took place in the interval between the periods at which the specimens were taken up for analysis.

When the means of the preceding analyses are taken, we obtain the following results:—

	City Water Course.	Portobello Basin.
Mean amount of saline matter	8.508	14.004
„ „ organic matter	1.450	2.708

SOURCES AND GATHERING GROUNDS.

The sources from which a water supply for towns may be derived are lakes, rivers, and streams, springs, wells, and gathering grounds. Of the latter it may be said that, however ably put forward under the auspices of the Board of Health, it is far safer to resort to good river waters than trust to what has been termed, with some satirical truth, "new fangled schemes of pot-piped gathering grounds." Springs and wells afford, at best, but a partial supply unless for villages or manufactories; and we must almost always trust to lakes, rivers, or streams, with sometimes reservoirs, for stowage, for a sufficient supply for large towns. The Croton aqueduct, conveying water with an average of three degrees of hardness, to New York, is perhaps the noblest work for water supply of modern times. The length of the aqueduct is about 44 miles, with a channel inclination of about 15 inches per mile. The receiving reservoir is about two miles higher up the channel than the distributing reservoir, which latter is 115 feet over the level of the sea, and commands the highest buildings of the city. In the driest weather the supply is equal to 28,000,000 gallons.* The cost of the work, including the purchase of land and water rights, was 8,575,000 dollars, or £8 per lineal foot nearly. The cost of distributing pipes was 1,800,000 dollars. Latterly we have had the Loch Katrine and Glasgow aqueduct, also a noble work,

* Schramke's Croton Aqueduct, New York.

constructed after this model by Mr. Bateman, notwithstanding the previous supply of that city, or a portion of it, the Gorbals, from gathering grounds at a high level. It is, however, sometimes necessary to make use of such grounds, particularly when

TABLE showing the Quantities of Gathering Ground and Reservoir Room to supply a given population with 15, 30, and 40 gallons of water per head per diem. The reservoir room is calculated to hold 12 inches in depth of rain-fall per mile as a guide for lesser depths. For 4 inches the results are to be divided by 3; and for 6 inches by 2.

Population can be supplied at 15 gallons per head per diem.	Population can be supplied at 30 gallons per head per diem.	Population can be supplied at 40 gallons per head per diem.	Cubic feet per minute required for the supply. To find the net size and inclination of a pipe to convey it, see table p. 42.	Millions of gallons per diem required for the supply.	Gathering ground in square miles required for the supply for a depth of 12 inches collected, equal to a supply of 53 cubic feet per minute per mile.	Reservoir room required in millions of cubic feet per square mile of catchment, to hold a supply of 12 inches of rain-fall.
2,500	1,250	937	4.179	.0375	.0789	2.196
5,000	2,500	1,875	8.358	.075	.1577	4.393
7,500	3,750	2,812	12.536	.1125	.2366	6.589
10,000	5,000	3,750	16.715	.15	.3154	8.786
12,500	6,250	4,687	20.894	.1875	.3942	10.982
15,000	7,500	5,625	25.072	.225	.4731	13.179
17,500	8,750	6,562	29.251	.2625	.5519	15.375
20,000	10,000	7,500	33.430	.300	.6308	17.572
25,000	12,500	9,375	41.788	.375	.7885	21.965
30,000	15,000	11,250	50.145	.45	.9462	26.358
35,000	17,500	13,125	58.5	.525	1.1039	30.75
40,000	20,000	15,000	66.9	.6	1.2616	35.144
45,000	22,500	16,875	75.217	.675	1.4193	39.537
50,000	25,000	18,750	83.57	.75	1.577	43.93
55,000	27,500	20,625	91.932	.825	1.734	48.32
60,000	30,000	22,500	100.29	.9	1.8924	52.716
65,000	32,500	24,375	108.65	.975	2.0501	57.109
70,000	35,000	26,250	117.	1.05	2.2078	61.502
75,000	37,500	28,125	125.36	1.125	2.3655	65.895
80,000	40,000	30,000	133.72	1.2	2.5232	70.288
85,000	42,500	31,875	142.1	1.275	2.6809	74.681
90,000	45,000	33,750	150.435	1.35	2.8386	79.074
95,000	47,500	35,625	158.8	1.425	2.9970	83.467
100,000	50,000	37,500	167.15	1.5	3.154	87.86
105,000	52,500	39,375	175.5	1.57	3.311	92.25
110,000	55,000	41,250	183.86	1.65	3.469	96.64
115,000	57,500	43,125	192.22	1.72	3.62	101.10
120,000	60,000	45,000	200.58	1.8	3.785	105.48

flanking or lying above glens where an embankment may be easily thrown across, and the supply stored for use, which would otherwise pass quickly off. The table, page 327, gives the areas of reservoirs and gathering grounds according to a collection of one foot in depth from the catchment; it can be easily modified when the storeage or required supply exceeds or falls short of this depth. *One acre of gathering ground with a collection of twelve inches of rain-fall from it annually will give a daily supply of five cubic feet per head to twenty-five inhabitants.*

The next table will be of use in showing the actual quantities which have been collected, or could have been collected, for storeage. Homersham, Hughes, and Beardmore's books have been consulted in arranging it.

The various methods employed for purification may be classed under three heads: mechanical, by filtering or straining; chemical, or antiseptic media, such as peat and animal charcoal, and precipitation by the use of lime water; and the natural precipitation of impurities when the water is at rest, as well as the purification which takes place from oxidation and neutralization on thorough exposure by the ozone of the atmosphere. This latter plan has, however, been tried, and signally failed. Filter beds may be constructed to have a surface area of one square yard to every 1,000 gallons to be filtered in twenty-four hours. For executed works the proportions vary from 1 in 460 to 1 in 1140. The cost of filtering, in capital, may be said to vary from £30 to £70 for each million of gallons, or 30s. to 70s. annually.

TABLE showing information with reference to size of Reservoirs, Catchment Areas, &c., collected and arranged from various authorities. The first, fifth, and sixth columns contain information with reference to reservoirs and the collecting areas; the second, third, and fourth, show for different districts the whole rain-fall, and the portions or per centage flowing off and available.

Names of Drainage Areas and names of Reservoirs.	Drainage area in square miles.	Rain-fall in inches per annum.	Depth of rain in inches per annum flowing off the surface.	Per centage of rain-fall which flows off the surface.	Reservoir room per square mile in millions of cubic feet of water.	Contents of Reservoir in millions of cubic feet.
Ashton	50	40.0	15.6	39	21.0	12
Albany Works, U.S.	20	1.1	32
Ballinrobe, Ireland	11.0	40.3	28.5	58
Belmont (moorland, mean of four years)	2.81	54.5	30.0	72	20.8	75
Bolton	80	25.0	20
Bute (low country)	45.4	23.0	53
Bateman's Evidence on the drainage area of Longdendale:—						
First half of 1845, very dry	..	21.2	13.6	64
Second half of 1845	38.0	27.25	71
First half of 1846	22.6	17.6	78
Oct., Nov., and Dec., 1846	..	10.2	8.67	85
Bann Reservoir (moorland)	..	72	48.0	66
Drainage areas on south side of Longridge Fell, near Preston, May 1862, to April 1863	15.5	20
	..	64	18.0	33
	22.0	43
Dilworth Reservoir of Preston Works, Lancashire	0.02	64.0	5
Glencorse	0.00	37.0	22.3	60	7.06	46
Greenock	7.88	60.0	41.0	68	38	300
Homersham's estimate of 24,000 cubic feet of Reservoir to each acre of drainage	1	15.36	15.36
Longdendale	23.8	12.3	202
Proposed Reservoir for Wolverhampton Works	224	7	16
Rivington Pike	10.25	55.5	24.25	44	20.0	481
Sheffield	1.49	30.5	52
Turton and Entwistle ..	3.18	40.2	41.0	80	31.43	100

COST.

With reference to cost, the following tables, arranged by us from various sources, will afford information from works executed.

The actual cost of all works for house service varies very much in different towns, and with the quantities supplied, from a general average of 1*d.* per house per week, to 2*d.*, and from an annual rate of 9*d.* in the pound to 1*s.* 6*d.*, and higher. The cost of raising and supplying 1000 gallons from a height of 135 feet in Nottingham, is said to be 3*d.*, and the charge for house service to vary from 5*s.* to 60*s.* annually. In Rugby, the average cost per house is 19*s.* per year, 4½*d.* per week, or an annual charge of 3*s.* 3*d.* per year, or ¾*d.* per week per head of the population, and for a bare supply of 13 gallons. In Croydon, for a supply of only 14 gallons per head, the cost of works varied from 1½*d.* to 2½*d.* per house per week. The parliamentary returns, showing the number of houses supplied, and cost of supply, by different water companies of London, in 1834, give the following results:—

COMPANIES.	Number of Houses.	Daily average Supply in Gallons.	Height of Supply over Thames.	Amount of charge per Company.
New River	78,213	241	145	£ s. d. 1 0 0
Chelsea	13,801	108	135	1 13 3
West Middlesex	16,000	185	155	2 10 10
Grand Junction	11,140	350	152	3 8 6
East London	46,421	120	107	1 2 0
South London	12,010	100	80	0 16 0
Lambeth	10,082	124	185	0 17 0
Southwark	7,100	150	00	1 1 3

Cost of house apparatus for private supply from street mains, as averaged by the Board of Health, for first-rate houses, is £3 13s. 2d.; second-rate houses, £2 18s. 6d.; third-rate, £2 3s. 3d.; fourth-rate and cottages, 17s. 5d.; average cost for houses and cottages, £2 8s. 1d.

The actual cost of private works—to take water from mains for the supply of cottages—is shown in the following table :—

Work executed by	NAME OF PLACE.	Mean Expense of Private Works for each Cottage.	Annual Value of each Cottage.
Jan. 1852	Rugby, mean of 8 Cottages.....	£ s. d. 1 12 11	£ s. d. 5 10 0
Mar. 1852	Croydon..... 10 " 	2 0 0	4 0 0
" 1852	Barnard Castle 11 " 	1 18 1½	3 2 0
Aug. 1852	Tottenham.... 6 " 	2 11 10½	10 0 0
Mean values for each Cottage		2 0 0	5 13 1½

The water rate charged by the Local Board at Tottenham, is given as follows :—

	In the Special District Rate Amendment.		Water Rate per week.	Water Rate per annum.
	Above	And not exceeding		
On Premises assessed.	£ s. d. "	£ s. d. 10 0 0	£ s. d. "	£ s. d. 0 2 0
"	10 0 0	15 0 0	"	0 3 0
"	15 0 0	20 0 0	"	0 5 0
"	20 0 0	25 0 0	"	0 6 3
"	25 0 0	30 0 0	"	0 8 0
"	30 0 0	40 0 0	"	0 11 0
"	40 0 0	50 0 0	"	0 14 0

and 3s. for every additional rate of £10.

TABLE, SHOWING THE AVERAGE CHARGE PER ANNUM FOR HOUSE SUPPLY IN LONDON,
ARRANGED FROM THE PARLIAMENTARY RETURNS OF 1855.

No.	COMPANIES.	Houses containing 2 Rooms.	Houses containing 3 Rooms.	Houses containing 4 Rooms.	Houses containing 5 Rooms.	Houses containing 6 Rooms.	Houses containing 7 Rooms.	Houses containing 8 Rooms.	Houses containing 9 Rooms.	Houses containing 10 Rooms.	Houses containing 11 Rooms.	Houses containing 12 Rooms.	Houses containing 13 Rooms.	Houses containing 14 Rooms.	Houses containing 15 Rooms.	Houses containing 16 Rooms.	1st Rate Houses.	2nd Class Houses.	3rd Class Houses.	4th Class Houses.	5th Class Houses.	Horses.	Stables.	Water Closets.	Per centage paid on Capital expended.
1	New River	s. d. 10 0	s. d. 15 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0	s. d. 1 0
2	East London	1 0	0 1	12 0	3 3	0 3	18 0	4 13	0 4	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
3	Southwark & Vauxhall	0 14	0 1	11 0	2 10	0 3	3 0	5 1	0 3	6 8	1 5	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
4	West Middlesex
5	Lambeth	7 6	11 0	0 15	0 1	2 6	10 12
6	Chelsea	13 0	17 0	1 2	0
7	Grand Junction	4 15	0 1	5 0	2 2	0 2	15 0	4 4	0
8	Kent
9	Hampstead	10 0	15 0	1 0	0 1	10 0	2 0	0
	Average Annual Charge	10 12	14 6	0 18	0 1	7 5	1 15	0 2	8 3	3 5	4 12	8 3	10 6	2 12	5 16	6 1	5 10	0 12	8 14	3 9	3 12	6 4	2 6

PUBLIC WORKS OF WATER SUPPLY, PRESTON.

Yards.	Cost of Pipes.	£	s.	d.
44 of 2-in. iron pipes, including valves, fire-plugs, outlet-pipes, and all appurtenances, at 1s. 7d.		3	9	8
1,406 of 3-in. ditto, at 3s. 4d.		249	6	8
921 of 4-in. ditto, at 4s. 9d.		76	4	0
625 of 5-in. ditto, at 6s.		187	10	0
30 of 9-in. ditto, at 9s. 6d.		14	5	0
2,516		£530	16	1

Water Supply and its Cost for some Cities and Towns, from a Paper read to the British Association at Leeds, in 1858, by Dr. Strang, of Glasgow. Vide Builder, for 1858, p. 653.

TOWNS.	Population within bounds of Supply.	Daily Supply.	Daily Supply for each Inhabitant.	Cost of undertaking.	Daily Supply for every £1 expended.	Prospective Supply daily in addition.
		Gallons.	Gallons.	£	Gallons.	Gallons.
London	2,007,917	81,025,842	30.3	7,102,823	11.4	..
Paris	1,100,000	26,350,000	24.	800,000	33.	20,000,000
Hamburgh . .	160,000	5,000,000	31.25	170,000	29.50	..
New York . .	713,000	28,000,000	39.27	1,800,000	15.5	..
Manchester . .	500,000	11,000,000	22.	1,300,000	8.5	14,000,000
Liverpool . .	500,000	11,000,000	22.	1,040,000	7.	..
Leeds	153,000	1,850,000	12.	283,871	7.	..
Edinburgh . .	215,000	4,800,000	22.3	456,000	10.5	2,000,000
Aberdeen . .	65,000	1,200,000	18.4	50,000	24.	..
Dundee	66,000	1,750,000	18.2	130,000	12.5	..
Greenock . .	40,000	2,112,500	52.8	90,000	23.4	..
Paisley	48,000	1,021,452	21.	60,000	17.	..
Glasgow	420,000	16,710,000	39.8	651,199	26.	20,000,000

The cost of pumping varies with circumstances; we believe that pumping engines cannot be put down at less than from £60 to £100 per horse power, dependent

on the size of the engine, although the Board of Health adopted a standard of £50 per horse power. For the town of Drogheda we estimated for two engines at £75 per horse power. The following information has been furnished to us respecting the cost of the Waterworks, Cork, by Sir John Benson the engineer, who designed and carried out the works.

CORK WATER WORKS.

		£ s. d.
Steam engine 100-horse power.	Direct acting Cornish Engine with three cylindrical flue boilers, including engine and boiler house, setting boilers, chimnies, &c., &c., per horse power	55 0 0
Two 50-horse power Turbines.	Two Turbines completed with four 11 in. ram pumps on each, including buildings, cisterns, valves, gates, screens, per horse power	44 0 0
Reservoirs— One of 3,500,000 gallons. One of 500,000 gallons.	One reservoir on a level of 180 feet over weir One reservoir on a level of 300 feet over weir	1,000 0 0
Cost per head.	The inhabitants in 1851, 80,000	0 16 34
	The inhabitants in 1861, 100,000	0 13 0
Valuation standard per pound on the valuation.	City valuation, £112,000	0 11 7
Yearly cost per five inhabitants.	Distribution per house of every five persons	0 5 0
Water supplied.	Quantity supplied, including manufacteries, to one person per day	30 gallons.

The total estimated cost of engines, including pumps, engine houses, wells, &c., for raising the London sewage, is £70 per horse power, and the annual cost £20 per horse power.*

When coals are 10s. per ton, the cost of an engine

* Main Drainage Report, 1857, p. 29.

exceeding 100-horse power, single acting Cornish, working night and day, will be £10 per horse power; when coals are 15s. per ton, the cost would be £13 per horse power; when coals are 20s. per ton, the cost would be £16 per horse power; when coals are 25s. per ton, the cost would be £19 per horse power. These estimates have been given by Mr. Hughes, and include every expense of coals, wages, oil, tallow, materials for packing, cleaning, &c., but none for interest of capital or depreciation of machinery.*

At 16ly the cost of pumping is stated by a writer in the Builder to be as follows:—

To pump one million gallons 140 feet high, the old engine consumes—

	£	s.	d.
Four tons of coal, at 10s. per ton.....	3	4	0
Oil, tallow, and packing.....	0	12	0
Wages.....	0	9	0
Total cost of pumping one million gallons.....	4	5	0
which gives 1d. per 1,000 gallons pumped 140 feet high (not a very high price).			

The new engine requires—

Five and a half tons of coal at 10s.....	4	8	0
Oil, tallow, and packing.....	1	10	0
Wages.....	1	2	0
Total cost of pumping one million gallons 140 feet high	7	0	0
which is 65 per cent. more money than the old engine requires.			

While another writer in the same periodical states, that the cost of pumping 1,000,000 gallons with the old engine was £4 18s. 8½d., and with the new engine, £4 10s. 7d. In the following table, arranged from information in Mr. Hughes' book,† the estimated cost of pumping engines for various works, English and American, is given:—

* Main Drainage Report, 1857, p. 447. † WEALE, London.

ESTIMATED COST OF ENGINES FOR PUMPING, AND OTHER INFORMATION.

Description of Work, and Engineers' estimate, with short description of engine.	Millions of Gal. raised daily.	Water to be raised in feet.	Millions of Gal. to be raised 1 ft. in 24 hours.	Estimated cost in £s.	Work to be done in units of 33,000 lbs. raised 1 ft. high per minute, or horse power.	Cost for each unit of 33,000 lbs. raised 1 foot high per minute, or horse power, in £s.
Dagblin's estimate for Liverpool Works, exclusive of buildings, one engine and pumps	3	1000	600	8170	120-20	67-1
Ditto Two engines and pumps	3	1000	600	9850	120-20	78-0
Ditto For one engine and pump	1	75	75	1100	15-78	215-4
Homerham, for Watford supply	4	174	1003	85,367	902-02	120-7
Forster & Co.'s estimate for Liverpool engines, one or two, exclusive of buildings	3	1000	600	9700	120-20	76-8
Ditto ditto ditto	1	75	75	1200	15-78	202-8
Hurrey and Bateman's estimate, Wolverhampton Water Works	1 1/2	314	471	1128	840-11	101-4
Hurrey and West's for ditto, exclusive of buildings, 81 inch cylinder engine	1 1/2	314	471	11,000	93-50	118-9
Ditto ditto 61 inch cylinder engine	1 1/2	180	270	7000	50-82	123-2
Ditto ditto 58 inch cylinder engine	1 1/2	140	210	6500	44-10	147-1
Messrs. Hawthorne for ditto, double power expansive condensing beam engine, exclusive of engine	1 1/2	314	471	6055	99-11	61-1
Ditto	1 1/2	103	154 1/2	2530	51-45	48-0
Messrs. Hawthorne's high-pressure double acting horizontal engine, to be raised in 4 hours	1 1/2	33	103	1170	37-78	42-1
Ditto to be raised in 9 hours	1 1/2	33	103	780	22-73	81-3
Ditto to be raised in 12 hours	1 1/2	33	60	570	18-89	41-0
Mr. Hockley's estimate for Wolverhampton, single acting Cornish	1 1/2	500	750	20,800	157-83	181-8
Ditto exclusive of buildings	1 1/2	500	750	10,200	157-83	83-6
Ditto to be raised in 12 hours, by two 80 inch, two 45 inch, and one 30 inch cylinders	1 1/2	500	1500	33,000	315-05	103-3
Ditto exclusive of buildings	1 1/2	500	1500	21,800	315-05	60-0
Mr. Quake's estimate, for Grand Junction Works, one 64, or two 45 inch cylinder, single acting Cornish	5	40	230	7000	48-40	144-6
Ditto for Southwark and Vauxhall	4	40	220	7000	67-41	103-0
Ditto for West Middlesex	5	40	230	10,000	48-40	208-0
Sandys, Vicars & Co., two engines for Liverpool, exclusive of buildings	3	1000	600	8000	120-20	63-4
Ditto one engine, exclusive of buildings	1	75	75	1800	15-78	114-1
Seaward and Capel's, for Wolverhampton, exclusive of buildings	1 1/2	500	810	23,000	178-05	123-2
West's estimate, 65 inch cylinder, exclusive of buildings	1 1/2	314	471	6000	99-11	60-5
Ditto 80 inch cylinder, including buildings	1 1/2	100	150 1/2	7100	51-45	183-0
Ditto exclusive of buildings	1 1/2	100	150 1/2	6000	51-45	97-2
Ditto raised in 4 hours, by a 85 inch cylinder engine	1 1/2	33	103	4000	37-78	105-5
Ditto exclusive of buildings	1 1/2	33	103	3100	27-78	111-6
Ditto including duplicate engine	1 1/2	500	810	33,200	158-00	240-8
Ditto 65 inch cylinder engine, including buildings (?)	1 1/2	314	471	8400	99-11	84-8
AMERICAN ESTIMATES AND WORKS.						
Mr. McAlpine's, for Brooklyn, double acting, expansive, high-pressure, condensing engine, 79 inch cylinder engine, to be raised in 12 hours, exclusive of buildings	5	100	1950	18,000	300-80	45-0
Ditto non-condensing 80 inch cylinder engine, raised in 24 hours, exclusive of buildings	5	100	1935	1000	100-00	16-0
Ditto in 12 hours, exclusive of buildings	10	100	3900	30,000	700-00	48-7
Ditto exclusive of buildings	20	100	7600	60,000	1500-00	83-1
Ditto ditto	30	100	11,400	85,000	2300-00	83-4
Ditto Albany Water Works, 58 inch condensing beam engine, and one duplicate not condensing, working for day only	3 1/2	150	624	10,020	281-48	57-5
Ditto Chicago Works, as in list, including buildings, 48 inch cylinder condensing engine and duplicate, as in list	8	107	613	11,238	185-10	63-3
Ditto 48 inch cylinder, &c.	9	110	698	11,412	140-40	70-5

In EXAMPLE 28, pages 39 and 40, we have pointed out the method of calculating the increase of horse power required in raising water through pipes from friction, and also the great increase of this extra head if the velocity increases; the increase being nearly as the square of the velocity. In addition to this, an allowance of horse power must be made for bends, curves, junctions, and other obstructions, for the effects of which see SECTION XI. The more slowly the water is pumped, the less will the loss be from these causes through the same pipe. It is therefore, so far, advisable to give as large a diameter to the pipes supplying a reservoir from a pumping engine as other aspects of the question, cost, and engine power, will admit.

THICKNESS OF PIPES FOR WATER WORKS.

It is evident that the thickness of a pipe should be at least sufficient to bear the pressure of the atmosphere, and therefore the whole pressure in a pipe is best expressed by a determinate number of pressures, each equal to that of a column of water 33 feet high. If n be the number of such pressures, or the number of units each equal 33 feet high, d the diameter of the pipe in inches, and t the thickness, also in inches, we shall have for

(A).	1.	Iron pipes, plate	$t = .0000 nd + .19.$
	2.	Iron pipes cast horizontally	$t = .0024 nd + .33.$
	3.	Iron pipes cast vertically	$t = .0010 nd + .32.$
	4.	Copper pipes, plate	$t = .0015 nd + .16.$
	5.	Lead pipes	$t = .0024 nd + .19.$
	6.	Zinc pipes	$t = .0051 nd + .16.$
	7.	Artificial stone	$t = .0054 nd + 1.00.$

For cast-iron pipes the engineer of the Paris water works, M. Dupuis, adopted in his practice a formula which is equivalent to

$$(B.) \quad t = .0016 n d + .32 + .013 d$$

in the foregoing measures. This formula may also be expressed as follows :—

$$(C.) \quad t = (.0016 n + .013) d + .32.$$

If d be 12 inches, and $n = 9$, corresponding to a pressure of 297 feet, we shall find from the last equation, $t = (.0144 + .013) \times 12 + .32 = .3336 + .32 = .6536$ inch. All pipes should however be proved with ten atmospheres, or 330 feet, and in practically applying the above formulæ in equation (A), for finding the thickness of pipes, the value of n should always have 10 added to it. Hence, applying formula (A), No. 3, to our example, we get $t = .0016 \times 19 \times 12 + .32 = .6848$ inch, which is the same practically as found from equation (C).

SEWERAGE COST.

As for water works, the minimum rain-fall of a district should be calculated upon, so the maximum fall must be considered in sewerage and drainage. We have already shown, page 305, that for a population of 80 persons per statute acre, and a discharge of two-fifths of an inch in eight hours, sewers should be calculated to discharge about $3\frac{1}{2}$ cubic feet per minute, the rain supply being about seven times the house supply, or sewage, including house water supply. Instances are quoted in which the discharge, after a heavy rain-fall, amounted to

20½ cubic feet per minute per acre, as in the Savoy-street sewer, which of course was principally surface water, as the sewage of 80 persons at 7 cubic feet per person, one-half of which, if discharged in

eight hours, would only be $\frac{80 \times 7}{8 \times 2} = 35$ cubic feet

per hour, or $\frac{35}{60} = .59$ feet nearly per minute, which

is only about the thirty-third part of 20½ feet. In other words, the storm waters were thirty-three times the amount of house sewage. It would be waste to provide drainage for so much surface water considered in itself, where it can be passed off from the surface channels. But sewage is not water, and it is essential, in the greater number of cases, that sewers should be flushed occasionally. It is absurd to calculate the size of sewers, as if the sewage matter were thoroughly diluted or passed off like water. In fact, the sewage in part lies at the bottom of the sewer, or is deposited there in nine cases out of ten, while the house supply of water passes on and escapes over it, removing only diluted and detached portions. It is, therefore, of importance, where artificial flushing and cleansing out are not provided, that storm waters should occasionally pass through and flush a system of sewers, particularly the main or arterial lines. An engineer must be guided, in calculating the dimensions, &c., of main sewers, by the circumstances of each case. The inclinations to be obtained, the form of the bottom or invert, the rain-fall, the amount of sewage which will not affect the size to any considerable

extent, the material and the cost consistent with permanency.

The discharging power of a water channel is more than doubled by increasing its dimensions by one-third; and it is increased in the proportion of 5·7 to 1 by doubling the dimensions. By giving four times the fall, the same channel will only double the discharge. Now a pipe 2 feet in diameter with a fall of 1 in 200, would discharge fully 1000 cubic feet of water flowing full with a velocity of 5·4 feet per second; at $3\frac{1}{2}$ cubic feet per minute per acre, for a population of 80 to the acre, the thoroughly diluted sewage of 280 acres would be passed off by one such pipe; that is, the sewage from 20,400 persons, on 280 acres, and also two-fifths of an inch of rain falling for eight hours, can be conveyed by a 2 feet pipe, with a fall of 1 in 200. But as this rain supply is about seven times the house supply, passing $2\frac{1}{2}$ feet off per person in eight hours, made up of fæces and used-up water supply. It is apparent that such a pipe would convey about eight times the sewage alone of the district, if flowing as water; but, under any circumstances, would be abundantly large for the duty, even when assuming the whole quantity to pass in at the upper end. For a fall of 1 in 800, two such pipes would be required, or one pipe 32 inches in diameter; for a fall of 1 in 3200, four 2 feet pipes would be required, or one pipe 3 feet 6 inches.

2. House drains should not be less than 6 inches in diameter, and should have facilities for being cleaned, either by using half-flange joints, or by having a

moveable upper segment. The inclination for these drains should be uniform, but the amount is not so important as some appear to think, if proper provision be made for cleaning. Where flushing is used, cast-iron pipes are the best, but they are also the most expensive. House drains of brick with a *v* tile bottom covered with flags or bricks are perhaps the best, as the capacity can be considerably augmented by adding to the height of the sides, and they can be at all times easily opened and cleaned. If inclinations from 1 in 50 to 1 in 20 can be had, so much the better. The following items as to cost have been selected from the Builder:—

COST OF SEWERS, NEWPORT, MONMOUTHSHIRE.

Total lengths.	Average depths.	Sizes of sewers.	Thick- nesses.	Cost per foot lineal.
1,322 ft.	15 ft. 6 in.	ft. in. ft. in. 4 6 by 3 6	9 in.	s. d. 11 8
2,317 ft.	13 ft. 0 in.	4 6 by 3 0	9 in.	10 1½
6,110 ft.	12 ft. 0 in.	3 0 by 2 2	9 in.	7 7½
12,354 ft.	11 ft. 8 in.	3 0 by 2 2	6 in.	5 3¾
1,953 ft.	9 ft. 3 in.	2 6 by 1 10	6 in.	4 7
9,663 ft.	10 ft. 0 in.	2 6 by 1 10	4½ in.	3 8½
690 ft.	10 ft. 2 in.	2 3 by 1 0	4½ in.	3 5½
3,264 ft.	8 ft. 6 in.	1 2 diameter	4½ in.	2 4¾

COST OF SEWERS AND PIPES IN PRESTON.

The following extract from the recently published summary of public works executed during the year ending April 30th, 1859, contains some useful information:—

Yards.	£	s.	d.	£	s.	d.
60 of Brick Sowers, 2ft. 6in. diameter, at 7s.....	21	0	0			
538 3ft. by 2ft., at 17s. 6d.....	470	14	8			
204 3ft. 6in. by 2ft. 4in., at 28s.	412	12	11			
372 3ft. 6in. by 2ft. 6in., at 28s.....	520	10	0			
250 4ft. 3in. by 2ft. 10in., at 41s. 9d. .	521	16	8			
56 4ft. 6in. by 3ft., at 75s. 7d.....	211	13	4			
66 4ft. 6in. diameter, at 40s. 9d.....	134	0	6			
				2,203	2	8
1,078						
42 of Cast-iron Sower, 2ft. diameter, at 30s.				75	12	0
22 of Earthenware Pipe Sower, 6in. diameter, at 4s.	4	8	0			
1,120 9in. diameter, at 7s. 5d.	418	13	5			
565 12in. diameter, at 8s. 9d.....	247	2	0			
88 15in. diameter, at 11s. 3d.	49	10	0			
98 18in. diameter, at 13s.....	63	14	0			
145 21in. diameter, at 18s. 6d.	134	2	0			
				017	10	8
2,047						
Total, including superintendence, also man-holes, street gullies, and all appurtenances	£3,280	5	4			

TABLE showing the prices of Tubular Drains as made by the Board of Health in 1852, fifty per cent. being added for profit, &c. ; and the sale prices in the market.

Diameter in inches.	Lengths.	Red earthenware pipes made by the Board.	Red pipes at sale prices.	Stoneware glazed at sale prices.	Assumed gain.			
					On red ware pipes.		Over glazed stoneware pipes.	
		£ s. d.	£ s. d.	£ s. d.	£ s. d.	£ s. d.	£ s. d.	£ s. d.
5	For 1,000 feet	6 15 0	20 10 8	25 0 0	14 1 8	18 5 0		
6	For 1,000 feet	9 14 0	25 0 0	29 3 4	15 0 0	19 0 4		
9	For 1,000 feet	15 1 6	37 10 0	50 0 0	22 8 6	34 18 6		

Did the Board of Health here add the cost of their own establishment and staff to the cost of production? The manufacturer must surely live.

ESTIMATE FOR SEWERS AT BRIGHTON.

Description of Sewers.	Length in yards.		Price per yard.	Amount.
<i>Brick Sewers:—</i>				
Diameter.			£ s.	£ s.
6ft.	4,850		3 10	16,075 0
4ft. 6in.	350		2 10	875 0
4ft. 6in. by 3ft.	3,000		2 8	6,600 0
3ft. 6in. by 2ft. 6in.	1,800		2 2	3,960 0
3ft. by 2ft.	2,820		1 16	5,076 0
2ft. 9in. by 1ft. 6in.	8,580		0 18	7,722 0
Total brick sewers.....		22,400		
<i>Earthenware Pipe Sewers:—</i>			£ s. d.	£ s. d.
16 inches diameter	9,400		13 6	6,380 11
12 " "	44,430		10 0	22,215 0
Total earthenware pipe sewers.		53,830		
<i>Cast-iron Pipe Sewers:—</i>			£ s.	£ s.
9in. diameter	750		7 0	5,250 0
1ft. 6in. "	1,260		3 0	3,780 0
Total cast-iron pipe sewers		2,010		
Total length of sewers.....		78,380		
Or 44 miles 950 yards.				
Man-holes and ventilating shafts		Number		
Lamp-holes		600	20 0	12,000 0
Gullies		600	4 0	2,400 0
Outlet works, overflows, and extra work on steep gradients, &c.		3,000	3 10	10,500 0
Contingencies, including repairs, &c. of existing sewers, 10 per cent.....				5,000 0
				11,178 0
Total.....				122,030 0

The following estimates have been made for laying pipes at Tottenham, not including their cost:—

Diameter of pipe in inches.	Depth 6 feet.	Depth 8 feet.	Depth 10 feet.
6	8½d.	11d.	12d.
9	9½d.	14½d.	16½d.
12	11½d.	16½d.	19½d.

The cost of laying alone at St. Thomas's, Exeter, was—

6 inch pipes	5 <i>d.</i> per foot lineal	3 to 4 feet deep.
9 "	5 <i>d.</i> " "	3 to 4 feet deep.
12 "	8 <i>d.</i> " "	5 feet deep.
15 "	9 <i>d.</i> " "	5 feet deep.
18 "	11 <i>d.</i> " "	5 feet deep.
2 <i>d.</i> per foot lineal for relaying pitching; 4 <i>d.</i> for macadamized roads; and 6 <i>d.</i> for pavements.		

The author has constructed a large quantity of main sewers, from 18 inches to 2 feet and 2 feet 6 inches wide, and 4 feet 6 inches high; the side walls built with rubble masonry, 9-inch segment invert laid with 4½-inch courses in cement; the top sometimes flagged, when flags of sufficient length could be procured, and sometimes arched with rough rubble arches. The invert was laid on, well bedded, well rammed, rubble to prevent subsidence, and preserve the bottom inclination uniform. The cost, at an average depth of about 9 feet, was 9*s.* per running foot, the side walls being about 18 inches thick. Upright side walls, where rubble is cheap, have many advantages in giving a considerable increase of capacity for a small outlay. The tenement and house drains were of earthenware pipes. Cast-iron gully grates and traps, weighing 3 cwt., cost 30*s.* each; the grate fastened by a wrought-iron chain.

The following regulations have been laid down for Cambridge and Carlisle :—

STIPULATIONS FOR CAMBRIDGE DRAINAGE.

" Water from the rear of premises should not be conveyed to the front under the basement floor.

" Rain-water from the roofs should not be conveyed into the basement, but conducted into the sewer by shallow drains.

" Cast-iron pipes may be used for basement drains in some instances.

" The scullery sink should be kept as high as possible, and approached by a step. A flap trap should be fixed between the sink and sewer.

" There should be no water-closet on the basement floor; if it cannot be arranged elsewhere, the soil-pipe should have a flap trap, or similar contrivance, to prevent the influx of sewage water."

FOR CARLISLE DRAINAGE.

" STIPULATION 1.—If water-closets are to be generally used, the description of such to be sanctioned by the Board, the same to be fixed to the satisfaction of the Surveyor.

" 2.—All down-spouts to be connected with the sewers where it may be proper to connect the same; in all cases where they are not connected with the sewer they are to be connected with the channel.

" 3.—All stench traps to be similar to samples furnished by the Surveyor, or others approved by him, and properly fixed to his satisfaction.

" 4.—All sewers to water-closets not to be less than six inches diameter.

" 5.—All sewers to yards, stables, kitchens, and sculleries, not to be less than four inches diameter.

" 6.—In every case the whole of the fall to be made available from the junction with the main sewer to the end of the private drain, that is to say, only one inclination to be used from the junction with the public sewer to the end of the private drain; and all branches from the private drain to sinks, water-closets, &c., to have one inclination from the junction of such drain. None of the above instructions to be departed from without the express sanction of the Surveyor.

" 7.—In no case must a private drain be put in with a less fall than one in fifty, without the sanction of the Surveyor.

" 8.—No pipes, water-closets, stench traps, gullies, kitchen sinks, bends, junction or tapering pipes, to be used without being approved by the Surveyor.

" 9.—All ash pits and dung depôts to be raised to the level of the adjoining ground, to be properly paved and drained as the Surveyor may direct.

"10.—All buildings, outhouses, &c., to be properly spouted, and the water conveyed into the sewers where approved of by the Surveyor."

THOROUGH LAND DRAINAGE.

The following instructions and general specifications, have been prepared by the Commissioners of Public Works in Ireland, for the use of the District Inspectors, and persons reporting on thorough-drainage. The drains are made in general parallel, and to suit the fall of the ground. The depths must alter in order that the bottoms should have an uninterrupted fall, and may vary from 2 feet to 4 feet 6 inches in practice, averaging, say about 3 feet 6 inches, but dependent on circumstances. The portions printed in italics are from specifications prepared by officers of the Board, and are varied according to each particular case:—

GENERAL OBSERVATIONS.

"No drainage works should be undertaken until it has been clearly ascertained that the surface level of the maximum floods in the main drain can be discharged at a level that will admit of the submain drains venting the waters from the lowest point of the lands proposed to be thorough-drained, at a level sufficiently below the surface of such land, that the highest floods shall not prevent the free discharge of such submain.

"When sufficient out-fall can be obtained, no open main drain should be of a less depth than five feet, and in all cases a greater depth is desirable, in order to insure a permanent and efficient drainage, and at the same time to prevent cattle, &c., from crossing.

"As it has been found by practical experiments on different varieties of soils, that deep drains, say from four to five feet deep, are more effective than shallow ones: they should always be estimated for, when the open main drains admit of their being

cut to that depth, or when, by a moderate outlay per acre, the main drains can be cut to a sufficient depth; the distance between the parallel drains must necessarily vary with the texture of the soil,—forty feet may be taken as a general rule.

OPEN MAIN DRAINS.

“ Main drains should have gradients of such inclination, and be sunk to a depth that will admit of the above stipulations, as to the discharge of the submain drains being carried out. They should have such width at bottom and side slopes as may be necessary; and be free of sharp angles, projecting stones, and other impediments to the quick discharge of the waters.

“ The spoil or material raised in sinking and improving the drains, where not available for filling up useless holes or drains, should be removed to a proper distance from the edge of the main drains, and dressed off in a workmanlike manner.

“ The abutments and piers of such bridges as have sufficient breadth of water-way, should, if necessary, be carefully underpinned; and those bridges which are insufficient to discharge floods, should be taken down and rebuilt of suitable dimensions.

COVERED MAIN DRAINS.

“ Whenever, from the nature of the lands, the extent of the district under drainage, and the quantity of water to be voided, it may be necessary to form covered main drains to receive the water discharged from the submains, their dimensions must be proportional to the amount of water to be voided, well flagged or paved at bottom, the sides built of stone or brick, and covered with a flag or arch at top.

SUBMAINS.

“ The submains to be of such depth and width at top and bottom as may be necessary. The fall in each to be as great as the above-described main drainage of the district will allow, and not to be allowed to run beyond a suitable length without discharging itself into a covered or open main drain.

THE MINOR DRAINS

"To be of such depth, width at top and bottom, and at such distance apart, as will secure the perfect drainage of the land, to be run in a straight direction parallel to each other, directly up and down the declivity, unless where the declivity happens to be very steep, and then to be carried across the fall at such an angle as to secure a free discharge for the water. The fall in each minor drain to be as great as the main drainage and submain drainage, previously described, will admit.

"In filling in the stones, great care should be taken that the bottom of the drain be clean, and that no clay or dirt be put in with them; a sod, grass side down, or a few inches of tough clay, to be placed on the surface of the stones, and trodden firmly. The drain should then be filled up with the stuff previously shovelled out, observing to keep the active soil for the top. The putting in of the stones to be commenced at the highest part or head of the drain.

"In using draining pipes or other tiles, care should be taken that they be laid firmly on the bottom for their entire length, so as to prevent them being deranged by the filling of the drain, and that the joints be fitted as closely together as possible.

"In cases of unfavourable ground, caused by running sand or otherwise, whereby the level of the conduit might be deranged, collared pipe tiles offer considerable advantages in the way of remedy.

"When grips may be necessary on the sides of farm roads, they should be on the field side of the fences."

SPECIFICATION FOR MAIN DRAINAGE.

OPEN MAIN DRAINS.

"The deepening and improving of the main drain, No. —, is to be commenced at the point — on the accompanying map, and from thence a gradient carried up to the point —, having an inclination of at least — feet per statute mile, and sunk to the depth of — feet. It shall be — feet wide at bottom, and the side slopes shall average — at least, unless in rock cutting, when the side slopes may be diminished to six inches to one foot; all sharp angles, projecting stones, and other impediments to the

free discharge of the water, must be carefully removed. The spoil or material raised in sinking and improving the drain, when not immediately used for top-dressing the adjoining lands, or for filling useless holes or drains, is to be removed to a distance of ——— feet from the edge of the main drain, and dressed off in a workmanlike manner.

“The bridge marked at the point ——— on the accompanying map to be ———,

“The whole to be executed in a proper and workmanlike manner, and the works to be maintained in good order for so long as any interest shall be payable for the money advanced on account of its execution.”

SPECIFICATION FOR THOROUGH-DRAINAGE (WITH TILES).

COVERED MAIN DRAINS.

“These shall be cut *fifty-four* inches deep, *thirty-six* inches wide at top, *twenty-four* inches wide at bottom; the materials used in them shall be *double row of three-inch pipe tiles*.

“The side walls shall be ——— inches in height, ——— inches thick, and well ——— at bottom. They shall be covered with a flag not less than ——— in thickness.

SUBMAINS.

“These shall be cut *fifty* inches deep, *thirty* inches wide at top, *eighteen* inches wide at bottom. They shall be carried along the low side of the fields, or portions of land to be drained, at a distance from the fence of *fifteen* feet, and through natural hollows where necessary. No submain to be allowed to run beyond a length of *two hundred* yards without discharging itself into a covered or open main drain.

MINOR DRAINS.

“These shall be cut *forty-eight* inches deep, *eighteen* inches wide at top, *five* inches wide at bottom, and at a distance of *forty* feet apart. They shall be run in a straight direction, parallel to each other, directly up and down the declivity (when possible). No minor drain to be allowed to run beyond a length of *two hundred* yards without discharging itself into a submain.

FILLING IN.

"All the drains (or a large number of them) having been opened and cut in a workmanlike manner, and it being ascertained that no water is standing in any of them, the filling in may be commenced.

MINOR DRAINS.

"Into each minor drain shall be put *pipe tiles twelve inches in length, one-and-a-half inch in the ope, for one hundred yards, commencing from the upper end of the drain, and pipe tiles twelve inches in length, one-and-three-quarter inch in the ope, in continuation from thence to the submains.*

SUBMAINS.

"Into each submain shall be put *pipe tiles twelve inches in length, two inches in the ope, for one hundred yards, commencing from the upper end of the drain, and pipe tiles twelve inches in length, three inches in the ope, in continuation to the end or point where they discharge themselves.*

GENERAL RULES.

"All tiles to be of good sound material, and well burned. The tiles shall be laid firmly on the bottoms of the drains for their entire length; the joints fitted as closely as possible, they shall be carefully covered with a *thin grassy sod or screen*. The stuff previously taken out of the drains shall then be returned, observing to keep the active soil uppermost.

"The mouths of the covered main or submain drains shall be built about with solid masonry set in mortar, carried up with the same slope as the sides of the open main drain, into which they discharge themselves.

"Before laying the tiles, great care must be taken that the bottom of the drains be clean. The putting in of the tiles to be commenced at the highest point or head of the drains.

"In case of an entire field being thorough-drained, a drain shall be cut at the top of it, parallel to the fence, and running at a distance from it equal to one-half of the distance between each of the minor drains, into one or more of which (as may be necessary) it shall discharge itself. The remainder of the minor drains to be discontinued at a distance from this drain equal to

one-half the entire distance between each of the minor drains; this drain to be of the same dimensions, and filled with the same materials, and in like manner, as the above described.

"No open drain shall run into a closed one.

"In passing through unfavourable ground, caused by running sand or otherwise, whereby the level of the conduit might be deranged, and where pipe tiles are the materials used for forming the conduit, collars must be used, so as to connect the ends of the tiles, and they must be fitted as closely as possible.

"Soles must, in all cases, be used when laying single D tiles, and they must be so laid that the ends of the tiles shall rest equally on them; when inverted D tiles are used, they shall also be connected from end to end by placing one-half of the upper tiles on one-half of the adjoining tiles below them.

"The whole to be executed in a proper and workmanlike manner; and the work to be maintained in like good order as when approved of at its completion, for so long as any interest shall be payable for the money advanced on account of its execution." [*Collars for up to 4-inch pipes can be had at the Florence Court Tillery.*]

SPECIFICATION FOR THOROUGH-DRAINAGE (WITH BROKEN STONES).

COVERED MAIN DRAINS.

"These shall be cut *forty-two* inches deep, *thirty* inches wide at top, *twenty-four* inches wide at bottom; the materials used in them shall be ———.

"The side walls in them shall be twelve inches in height, six inches thick, and well ——— at bottom. They shall be covered with ———.

SUBMAINS.

"These shall be cut *forty-two* inches deep, *eighteen* inches wide at top, *fourteen* inches wide at bottom. They shall be carried along the low side of the fields, or portions of land to be drained, at a distance from the fences of *thirteen* feet, and through natural hollows, where necessary. No submain to be allowed to run beyond the length of *one hundred and fifty* yards, without discharging itself into a covered or open main drain.

MINOR DRAINS.

" These shall be cut *thirty-six* inches deep, *fifteen* inches wide at top, *four* inches wide at bottom, and at a distance of *twenty-six* feet apart. They shall be run in a straight direction, parallel to each other, directly up and down the declivity (when possible). No minor drain to be allowed to run beyond a length of *two hundred* yards without discharging itself into a submain.

FILLING IN.

" All the drains (or a large number of them) having been opened and cut in a workmanlike manner, and it being ascertained that no water is standing in any of them, the filling in may be commenced.

MINOR DRAINS.

" Into each minor drain shall be put *ten* inches of broken stones in depth, the stones having been broken to a size not exceeding *two-and-a-half* inches in diameter. Great care should be taken that the bottom of the drain be clean, and that no clay or dirt be put in along with the stones; a sod (or clay, as may be convenient) *three* inches thick shall be placed carefully on top, and the whole trampled upon or rammed hard. The drain shall then be filled up with the stuff previously shovelled out, observing to keep the active soil for covering the top. The putting in of the stones shall invariably be commenced at the highest part or head of the drain.

FILLING IN SUBMAINS.

" In each submain a conduit shall be formed of *six* inches in height, *four* inches wide, and the filling in completed as above described.

GENERAL RULES.

" The mouths of the covered main or submain drains shall be built about with solid masonry set in mortar, carried up with the same slope as the sides of the open main drain into which they discharge themselves.

" Before filling in the stones, great care must be taken that the bottom of the drains be clean, and that no clay or dirt be put in

along with them. The putting in of the stones to be commenced at the highest part or head of the drains.

"In case of an entire field being thorough-drained, a drain shall be cut at the top of it, parallel to the fence, and running at a distance from it equal to one-half the distance between each of the minor drains, into one or more of which (as may be necessary) it shall discharge itself. The remainder of the minor drains to be discontinued at a distance from this drain, equal to one-half the entire distance between each of the minor drains; this drain to be of the same dimensions, to be filled with the same material, and in like manner, as the above described.

"No open drain shall run into a closed one.

"The whole to be executed in a proper and workmanlike manner, and the work to be maintained in like good order as when approved of at its completion, for so long as any interest shall be payable for the money advanced on account of its execution."

One of the officers of the Commissioners of Public Works, Ireland; the Inspector of Drainage for Roscommon, a gentleman residing in that county, writes to me as follows, with reference to tile and broken stone drains on the carboniferous formation :—

"With respect to tile drainage my experience has not been very extensive, as the proprietors of the district, with scarcely any exception, give a decided preference to broken stones; but from what I have seen, I am very much inclined to prefer good well-burnt pipes to any other draining material, provided that collars be used, but not otherwise. As to the best diameters, I have found the 1½" collared pipes of the Clonbrock Tile Works (now closed) very satisfactory; but when the length of minor drains exceeded 100 yards, I should like an increase to 1¾ or 1¾. For submains (say 150 or 180 yards long) I have recommended pipes of 2 inches, 2½, and 3 inches in succession, all of which were to be had

with collars: if 4-inch pipes were to be had with collars, I should have recommended longer submains. The larger-sized pipes are not provided with collars in our present tileries, *and on this account I generally put a note on the margin of the printed form, suggesting that a stone duct of the ordinary size of submain, say 6 inches in height and 4 inches wide, be substituted for the tile filling.*

“ I decidedly prefer an open duct to broken stone filling; and in nine-tenths of my own drainage I have made the minor drains on the same plan as the submain, with an open stone conduit; the only difference being, that the minor drains are a few inches shallower, with a smaller duct. The increase of expense is a mere trifle, and when the substratum (as very frequently occurs here) is a fine calcareous gravel, containing 40 to 60 per cent. carbonate of lime, the additional spoil is a very cheap fertilizer for the land.

“ With respect to depths and distances apart, the two most commonly used in my specification are $3\frac{1}{2}$ feet deep, 33 feet apart,—and 4 feet deep, 42 feet apart. These arrangements will not suit all cases, and I vary accordingly. Thus, in one case of exceedingly retentive land of peculiar texture, 4 feet drains, 27 feet apart, produced the required result, while in another, $3\frac{1}{2}$ feet drains, 66 feet apart, effected all that was required. In the latter case there was a mixed soil, which might be described as “half wet;” yet the water lingered sufficiently long to make the land unsound for sheep, and greatly to injure the crops in quality as well as quantity.”

Mr. Josiah Parkes says, in 1843:—“ Experiment

and experience have rapidly induced the adoption of a system of parallel drains, considerably deeper, and less frequent, than those commonly advocated by professed drainers, or in general use. I gave several instances of this practice in Kent in the Report of last year, 1843, already alluded to, and it is rapidly extending. Mr. Hammond stated to you that he drained 'stiff clays 2 feet deep, and 24 feet between the drains, at £3 4s. 3d. per acre, and porous soils 3 feet deep, 33½ feet asunder, at £2 5s. 2d. per acre.' I now find him continuing his drainage at 4 feet deep, wherever he can obtain the outfall, from a conviction founded on the experience of a cautious progressive practice as to the depth and distance, that depth consists with economy of outlay as well as with superior effect. He has found 4-foot drains to be efficient, at 50 feet asunder, in soils of varied texture—not uniform clays—and executes them at a cost of about £2 5s. per acre, being 18s. 4d. for 871 pipes, and £1 6s. 6d. for 53 rods of digging. Communications have been recently made to me by several respectable Kentish farmers, of the satisfactory performance of drains deeply laid in the Weald clays, at distances ranging from 30 to 40 feet, but I have not had the opportunity of personally inspecting these drainages.

“The following little table shows the actual and respective cost of the above three cases of under-draining, calculated on the effects really produced, that is, on the masses of earth effectively relieved of their surplus water at an equal expense. I conceive this to be the true expression of the work

done, as a mere statement of the cost of drainage per acre of surface conveys but an imperfect, indeed a very erroneous, idea of the substantive and useful expenditure on any particular system. This will be apparent on reference to the two last columns of the table, which give the cost in cubic yards and square yards of soil drained for one penny, at the above mentioned prices, depths, and distances.

<i>Depth of drains, in feet.</i>	<i>Distance between the drains, in feet.</i>	<i>Mass of soil drained per acre, in cubic yards.</i>	<i>Mass of soil drained for one penny, in cubic yards.</i>	<i>Surface of soil drained for one penny, in square yards.</i>
2	24	3220½	4·1	6·27
3	33½	4840	8·93	8·93
4	60	6453	12·00	8·93

“I may here observe, that Mr. Hammond, when draining tenacious clays, chooses the month of February for the work, when he lays his pipes (just covering them with clay to prevent crumbs from getting in), and leaves the trenches open through March, if it be drying weather, by which means he finds the cracking of the soil much accelerated, and the complete action of the drains advanced a full season. The process of cracking may, doubtless, be hastened both by a choice of the period of the year in which drains are made, and by such a management of the surface as to expose it to the full force of atmospheric evaporation.”

With reference to drains, we have known a case in the Queen's County in which inch pipes had to be taken up, and pipes of 2½-inch bore substituted.

The drains were 40 feet apart, and 4 feet deep, and the pipes had collars. The minor drains should discharge into submains at convenient distances, say 100 yards, on flat grounds. Small pipes will choke unless the velocity in them be sufficient to carry off deposits, and the diameters should vary according to the inclinations of the ground, and distance apart of the drains.

Mr. Mechi, in 1844, lays down the following rules :—

“1st.—That it is not the size or form of the drains that regulate perfect drainage; but the *depth at which they are placed*. The depth also governs the distances at which the drains should be cut according to the quality of the soil.

“2nd.—The pipes of 1-inch bore, without stones, are amply sufficient, placed at 4 feet deep and 30 feet wide in dense soils, and the same depth and 50 feet wide in mixed soils.

“3rd.—The deep drains receive more water than shallow ones, and consequently lay dry a greater extent of ground.

“4th.—The deep drains *begin and end running sooner than shallow ones*, and carry off more water in a given time.

“5th.—That where shallow drains are made and deep ones cut below them, the shallow ones no longer act, all the water passing to the deeper drains.

“6th.—That when round stones are used as well as pipes, the latter should always be placed at the bottom, as I find, practically, water flows more quickly through pipes than amongst stones.

"Before persons begin draining, I would recommend their porusing attentively the facts developed by Mr. Parkes, at pages 39 and 40, and my remarks at page 36 of Letters on Agricultural Improvements.

"Pipes made to socket into each other (by Ford's

TABLE showing a Return of the number of Acres thorough-drained in the years 1843 and 1844, by the different Competitors for Sir Richard O'Donnell's Gold Medal, together with the Average Prices per Perch, and Cost per Acre respectively. (Given in Reports to the Royal Agricultural Improvement Society of Ireland.)

Competitors.	Number of statute acres.	Number of statute perches.	Average price per statute perch.	Average price per statute acre.	Cost to the tenants.	Rate of wages per day and by task work.
Marquess of Waterford	A. R. P. 501 2 15	P. 60,000	0d. in 1842, reduced to 5d. in 1845.	£5 in 1842, reduced to £3 2s. 8d. in 1845.	5 per cent. charged to the rent.	..
Viscount Templetown	504 0 25	54,351	4½d.	£1 14 4	£500 0 0	..
Sir R. O'Donnell, Bart.	551 0 0	53,478	4d.	1 12 4	408 0 0	8d. a day.
The Earl of Caledon	321 0 10	47,183	4d.	2 0 5	300 0 0	10d. to 1s. a day.
J. L. W. Naper, Esq.	254 1 20	34,433	0¾d.	5 8 3	700 7 4	7d. a day in Winter, and 10d. in Summer.
Lord Blayney . . .	201 2 30	34,634	5d.	3 12 0	427 18 0	10d. to 1s. a day.

TABLE showing a Return of the number of Acres thorough-drained by Proprietors, for the Society's Gold Medal, and the Average Prices per Perch and per Acre respectively.

The Earl of Erno . .	A. R. P. 110 0 37	P.	£ s. d. 3 12 3
Lord Dufferin and Glanlogh . . .	203 1 0	20,478	10¾d.	6 11 0	..	1s. per day.
Messrs. Andrews . .	117 1 4	10,614	9¾d.	5 15 0	..	13d. per day.
Dr. O'Neill . . .	115 0 12	2 10 3	..	7d. to 8d. a day.

Patent Socketing Machine) are best adapted to loose or mixed soils."

Pipes laid, however, too near the surface, are frequently choked with the roots of plants. The principal advantage of submains alongside open mains is, that the mouths of the minor drains should not be choked from vegetation, and that the water from them, flowing into and taken up by this submain, may be discharged by a few apertures only, and thereby keep themselves open, or as much so as the nature of the case will admit. The foregoing tables show the cost per statute acre, in Ireland, of thorough-drainage, which must vary with circumstances, locality, and the value of labour.

The average cost per statute acre for Sir Richard O'Donnell's Gold Medal, was £3 5s. 7d., and £4 13s. 10d. for the Society's Gold Medal; average of both, £4 per statute acre nearly.

The average number of acres now annually improved in Ireland, is about 5530, at an average cost per acre of £4 17s.

In Ireland, thorough-drainage is almost generally carried out by loan, under the Commissioners of Public Works, and there is no branch of the public service has given more satisfaction to owners of property. The works are, we believe, always executed within the estimates, and the owner having the expenditure in his own hands, can satisfy himself of its proper application. No loans are made unless where immediately, or prospectively, a return of 6½ per cent. is estimated on the expenditure, a rent charge for this amount being made for 22 years.

TABLE showing Estimates of the Quantities and General Cost for the Thorough-Drainage of a Statute Acre of Land, with Broken Stones or Tiles, with the distances apart for different class soils.

Description of land, section of parallel drain, and cost.	Distance between the parallel drains.	Lineal perches of drains per statute acre.	Cubic yards per acre.		Expense per statute acre.			Number of feet of tiles per statute acre.
			Of clay excav.	Of broken stones.	£	s.	d.	
Hard subsoil stiff and sandy clay drains, from 12 to 20 feet apart, at 8d. per lineal perch.	Feet.	Perches.	Cu. yds.	Cu. yds.				Number
	12	220	277	38.6	7	0	8	3,630
	13	203	255½	35.5	6	15	4	3,351
	14	188½	237	33.0	6	5	8	3,111
	15	176	221	30.8	5	17	4	2,904
	16	165	207	28.9	5	10	0	2,722
	17	155½	195½	27.2	5	3	6½	2,562
	18	146½	184	25.9	4	17	6½	2,420
	19	139	175	24.3	4	12	0	2,293
	20	133	166½	23.1	4	8	0	2,178
Freestone bottom drains, from 20 to 30 feet apart, at 9d. per lineal perch	21	125½	182½	29.3	4	14	6	2,074
	22	120	174½	28.1	4	10	0	1,971
	23	115	167	26.9	4	6	3	1,894
	24	110	159½	25.7	4	2	6	1,815
	25	105½	153½	24.7	3	19	1½	1,742
	26	101½	147½	23.7	3	16	1½	1,675
	27	97½	142	22.0	3	13	4	1,613
	28	94	136½	22.0	3	10	6	1,556
	29	91	132½	21.3	3	8	3	1,502
	30	88	127½	20.6	3	6	0	1,452
Beds of gravel, sand, and rocky stratification, from 30 to 100 feet apart, at 10d. per lineal perch	31	85	121½	24.7	3	10	10	1,405
	32	82½	117	24.0	3	8	0	1,361
	33	80	112½	23.2	3	6	8	1,320
	34	77½	108½	22.6	3	4	8½	1,280
	35	75½	104	21.9	3	2	9½	1,245
	36	73½	100½	21.3	3	1	1½	1,210
	37	71	120½	20.6	2	10	2	1,177
	38	69	123	20.0	2	17	6	1,146
	39	67½	119½	19.6	2	16	1½	1,117
	40	65½	116½	19.1	2	14	8½	1,089
	100	27	47	8.0	1	2	6	436

ARTERIAL DRAINAGE.

The effect of thorough-drainage on the arterial channels of a district, is to discharge the rain-fall into the main channels in a shorter time than before, particularly during wet seasons. This frequently causes floods to rise higher as well as more rapidly. During dry seasons the supply is less, and so far, when it is limited, an injury is done to the adjacent districts requiring it for use. The effect of obstructions in the main channel is to impound the upland water, sometimes made available for water power or navigation purposes, but in general, to the injury of the drainage of adjacent lands, and the regimen of the river, particularly in flat districts. The arterial drainage in Ireland has effected a vast amount of good, but up to 1853 the estimates appear to have been usually doubled; the estimates for eleven of these works being £124,647, and the expenditure, £293,532. The average cost per acre, on the land improved by these projects, varied from £1 19s. 8d. to £5 17s. 7d., the average of the eleven districts being £4 3s., which is about the average for thorough-drainage.*

The following table affords valuable information of the cost of arterial drainage works in Ireland; it is extracted from the Report of the Commissioners of Inquiry, presented to the House of Commons June 16th, 1853.

* See Parliamentary Report, by Sir Richard Griffith, Sir W. Cubitt, and Jas. M. Rendel, June 16th, 1853.

NAME OF WORK	Quantity of Earth-work Gravel	Cost.		Quantity of Rock-work	Cost.	Cost of Unwatering.				Cost of Superintendence Plant, Tools, and Materials not used in the Works.				Analysis of Cost per Cube Yard in Pence.								
		£ s. d.				£ s. d.		£ s. d.		£ s. d.		Gravel	Rock	Un-watering	Imple-ments.	Superin-tendence	Total.					
		£	s.			d.	£	s.	d.	£	s.							d.	Pence.	Pence.	Pence.	Pence.
Kilcolgan..	50,992	1,984	9	11	22,572	868	7	4	11	500	2	10	374	18	8	5-88	9-23	0-30	0-57	1-15	8-20	11-53
Craughwell	51,491	600	18	10	2,066	100	0	0	8	5	2	123	0	0	88	0	0	0-04	0-39	0-55	3-75	12-93
Strongfort.	17,475	285	12	8	400	12	0	0	13	17	11	55	0	0	38	0	0	0-18	0-51	0-73	5-42	8-62
Ballymore.	75,610	1,771	16	11	9,350	503	1	0	7	5	10	360	0	0	255	0	0	0-18	0-72	1-01	7-38	14-75
Raford.....	64,100	1,151	10	2	800	60	9	0	47	18	3	215	0	0	150	0	0	0-18	0-56	0-80	5-55	19-54
Ballykeeran	17,033	285	9	9½	60	9	10	0	17	5	6	55	0	0	33	0	0	0-24	0-53	0-77	5-54	39-54
Dunsandle.	35,724	669	15	9½	3,526	114	5	0	35	12	2	130	0	0	89	0	0	0-21	0-54	0-79	6-04	9-31
St. Clerans.	40,842	745	8	7	850	44	0	0	29	9	5½	147	15	0	101	0	0	0-17	0-58	0-85	5-98	14-02
Loughbrea..	89,296	1,512	12	8	2,700	180	18	4	12	19	0	297	10	0	205	0	0	0-03	0-53	0-77	5-39	17-41
Monksfield.	61,122	594	2	8	320	33	14	0	9	18	7	157	15	0	110	0	0	0-04	0-42	0-61	3-40	26-32
Lackafinna.	48,790	864	6	9	522	53	14	0	15	19	1	108	0	0	77	0	0	0-08	0-37	0-52	5-22	23-66
Carra.....	74,850	1,186	5	6½	7,900	444	0	0	25	8	2	198	0	0	136	0	0	0-07	0-39	0-57	4-53	14-51
Raruddy ..	28,854	388	9	8½	3,060	113	12	0	4	3	7	79	0	0	56	10	0	0-03	0-42	0-59	4-36	9-94
	686,179	12,041	0	0	54,066	2,543	10	9	372	5	7½	2,426	2	10	1,718	8	8	0-12	0-55	0-78	5-66	12-74

The abstract of 84 arterial drainage awards, made by the Commissioners of Public Works in Ireland, in 1854, gives for different years, 1849 to 1854.—

Number of districts drained.	Total combined catchment average of districts.	Area of flooded lands.	Average cost of arterial drainage per acre improved by drainage.		
			£	s.	d.
12 districts	90,332	9,453	3	14	2
27 „	95,583	11,579	3	16	7
19 „	237,400	13,707	4	17	3
16 „	374,427	29,453	3	13	4
2 „	49,840	3,275	5	0	0
8 „	260,420	21,033	3	0	4
84 „	1,114,067	88,561	3	17	7

The last line gives the general average, and shows that in these 84 districts, about 1 acre in 13 is the average of flooded lands to the catchment area, or 8 per cent. nearly.

SECTION XIV.

WATER AND HORSE POWER.—FRICTION BRAKE, OR DYNAMOMETER. — CALCULATION OF THE EFFECTIVE POWER OF WATER WHEELS.—OVERSHOT, UNDERSHOT, AND BREAST VERTICAL WHEELS.—HORIZONTAL WHEELS AND TURBINES. —HYDRAULIC RAM.—WATER ENGINE.

Taking the representative of a horse's power at 33,000 foot-pounds* or 33,000 lbs. raised one foot

* 16,500 foot-pounds, or one half of the above, is much nearer the average power of a horse, working for 10 hours only, as the work is ordinarily done through the country; 33,000 lbs. raised one foot per minute, is equivalent to 884 tons, nearly, raised one foot in an hour. Therefore, a river discharging 884 tons, over a fall one foot high in an hour, or 884 tons, over a fall 24 feet high

high in one minute, the theoretical horse-power of an overfall is expressed by the fall in feet, multiplied by the discharge in cubic feet per minute, the product multiplied by 62½ (the weight in lbs., nearly, of a cubic foot of water), and divided by 33,000. The following table gives the weight in air of a cubic foot of pure water at different temperatures, Fahrenheit's thermometer.

WEIGHT OF A CUBIC FOOT OF WATER.

The weight of 36 cubic feet of water is one ton, nearly.

Temperature, in degrees.	Weight of a cubic foot of water. Pounds Avordupois.	Temperature, in degrees.	Weight of a cubic foot of water. Pounds Avordupois.	Temperature, in degrees.	Weight of a cubic foot of water. Pounds Avordupois.
32	62·375	51	62·365	69	62·278
33	62·377	52	62·363	70	62·272
34	62·378	53	62·360	71	62·264
35	62·379	54	62·356	72	62·257
36	62·380	55	62·352	73	62·249
37	62·381	56	62·349	74	62·242
38	62·381	57	62·345	75	62·234
39	62·382	58	62·340	76	62·225
40	62·382	59	62·336	77	62·217
41	62·381	60	62·331	78	62·208
42	62·381	61	62·326	79	62·199
43	62·380	62	62·321	80	62·190
44	62·379	63	62·316	81	62·181
45	62·378	64	62·310	82	62·172
46	62·376	65	62·304	83	62·162
47	62·375	66	62·298	84	62·152
48	62·373	67	62·292	85	62·142
49	62·371	68	62·285	86	62·132
50	62·368			87	62·122

The effective power of a fall depends on the nature, in 24 hours, has also a horse power. The drainage of 10 square miles, with an average collection of 12 inches annually of rain in depth, will give an annual unceasing one-horse power for each foot of fall in a receiving channel; or five square miles will give the same result, if the collection amounts to 24 inches in depth. The collection of 10 square miles, one foot deep, yearly, is nearly equal to the delivery of 580 cubic feet per minute, for the same period.

proportions, and construction of the wheel or machine, and also upon the manner in which the theoretical power is applied. When the velocity of a stream acting on a wheel only is known, the theoretical head, h , due to it is found in feet from the formula $h = .0155 v^2$, v being the velocity in feet per second.

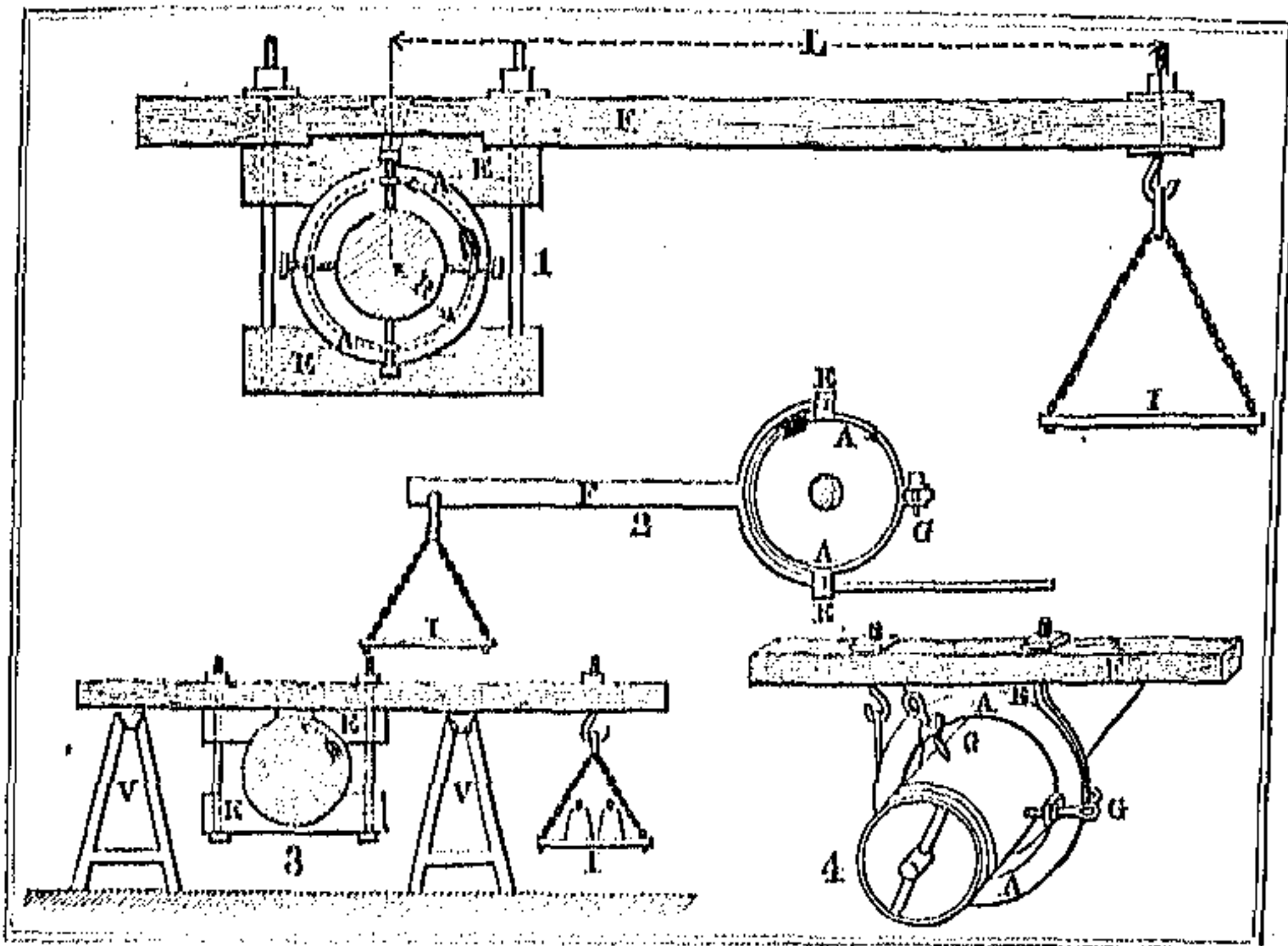
In order to gauge the quantity of water applied to a wheel, and thereby determine with accuracy its effective power, the water used must be passed through a notch, orifice, or over a weir, the coefficients for which had been previously ascertained from experiment. Greater accuracy can be obtained from gaugings through thin plates, or planks having the down-stream arrises chamfered, than with any other form of orifice or notch; and when it can be effected, the channel above should be sufficiently enlarged to prevent the effects of an approaching current. We have already in the body of this work dwelt in detail on the various formulae required for gauging under different circumstances. The accuracy of the results showing the effective powers of wheels depends in the first place, on the accuracy of the gaugings or estimates of the quantity of water used, and next on the fall employed.

FRICTION BRAKE, OR DYNAMOMETER.

The power applied to a revolving shaft through a water wheel of any construction, is the weight of water multiplied by the fall. It is evident that the portion of this power available to turn a shaft and machinery, or the effective power, must depend on

the construction of the wheel, as a portion of the theoretical power is lost mechanically, in applying it; in changes of direction, friction, eddies, and discharging currents. The greater the effective power conveyed to a shaft, the greater becomes the power of the wheel, or medium through which the original power is transmitted. The mechanical effect produced by a revolving shaft is best measured by a friction brake, the principle of which is as follows. In diagram 1, Fig. 44, let the friction pulley $\Lambda\Lambda$ be

FIG. 44.
FRICTION BRAKES, OR DYNAMOMETERS.



firmly fixed to the revolving shaft or axis of the wheel ; \mathbb{E} and \mathbb{E} , two wooden clamps grasping the friction pulley by means of the screw bolts, delineated, which can be tightened on the axis, and also to the arm \mathbb{F} , by means of suitable nuts. The more tightly the bolts are screwed, the greater will be the friction

between the friction pulley AA , and the clamping pieces EE . If, while the axis and friction pulley AA , are revolving in the direction indicated by the arrow, a weight be applied in the scale at r , so that the arm r shall not be carried round, but remain fixed; it is clear that the work done by the revolving shaft in one revolution, will be measured by the circumference of the friction pulley, multiplied by the friction due to the pressure on it, or by its equivalent, the weight in the scale r , multiplied by the circumference of a circle whose radius is r , or by $2\pi \times 3.14159 \times w$, in which expression w is the weight in lbs. in the scale r . If n be the number of revolutions in a given time, say one minute, we shall therefore have the useful effect of the wheel on the shaft in foot-pounds per minute, equal to

$$2\pi \times 3.14159 \times w \times n.$$

We have also the power of the water acting on the wheel, equal to

$$h \times D \times 62.37,$$

in which h is the head and D the discharge in cubic feet per minute; therefore, we shall have for the ratio of the effect to the power the expression

$$\frac{2\pi \times 3.14159 \times w \times n}{h \times D \times 62.37}.$$

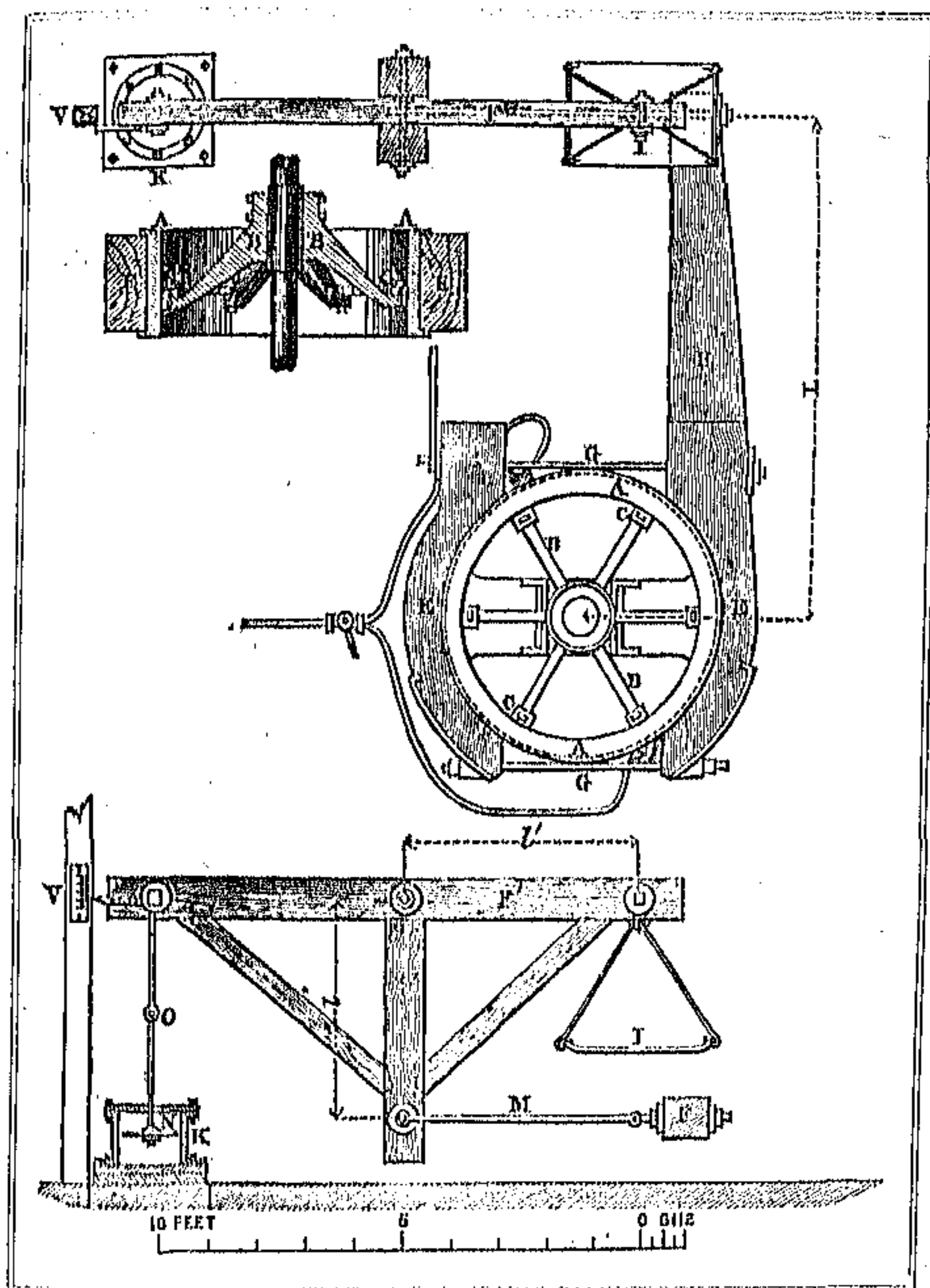
If the revolving shaft be horizontal, the weight of the arm r , acting at its centre of gravity and reduced to the length r , where the weight w is suspended, will have to be included in the weight w . If the weight w be suspended at the end of a connexion of levers, or other mechanical powers, the length r will have to be determined accordingly. Diagram 2, Fig. 44,

shows the Armstrong-brake; Diagram 3, the common form; and Diagram 4, Egen's brake.

Fig. 45, is a general representation of the brake

FIG. 45.

DYNAMOMETER FOR DETERMINING THE USEFUL EFFECT OF THE
TREMONT TURBINE.



used by Francis, in the Lowell experiments. The length of the arm of the brake L , was 9.745 feet; the length of the vertical arm Z of the bell crank

feet; and the length of the horizontal arm 7 5 feet. The following detailed description is by Francis:—

“*The Friction Pulley A* is of cast-iron, 5·5 feet in diameter, two feet wide on the face, and three inches thick. It is attached to the vertical shaft by the spider B, the hub of which occupies the place on the shaft intended for the bevel gear.

“The friction pulley has, cast on its interior circumference, six lugs, *o o*, corresponding to the six arms of the spider. The bolt holes in the ends of the arms are slightly elongated in the direction of the radius, for the purpose of allowing the friction pulley to expand a little as it becomes heated, without throwing much strain upon the spider. When the spider and friction pulley are at the same temperature, the ends of the arms are in contact with the friction pulley. The friction pulley was made of great thickness for two reasons. When the pulley is heated, the arms cease to be in contact with the interior circumference of the pulley, consequently they would not prevent the pressure of the brake from altering the form of the pulley. This renders great stiffness necessary in the pulley itself. Again, it is found that a heavy friction pulley insures more regularity in the motion, operating, in fact, as a fly-wheel, in equalizing small irregularities.

“*The brakes E and F* are of maple wood; the two parts are drawn together by the wrought-iron bolts *o o*, which are two inches square.

“*The bell crank R'*, carries at one end the scale *r*, and at the other the piston of the hydraulic regulator *K*; this end carries also the pointer *r*, which indicates

the level of the horizontal arm. The vertical arm is connected with the brake *r*, by the link *m*.

“*The hydraulic regulator* *κ*, shown in the figures, is a very important addition to the Prony dynamometer, first suggested to the author by Mr. Boyden, in 1844. Its office is to control and modify the violent shocks and irregularities, which usually occur in the action of this valuable instrument, and are the cause of some uncertainty in its indications.

“The hydraulic regulator used in these experiments, consisted of the cast-iron cylinder *κ*, about 1·5 feet in diameter, with a bottom of plank, which was strongly bolted to the capping stone of the wheel pit, as represented in figure 1. In this cylinder, moves the piston *π*, formed of plate-iron 0·5 inches thick, which is connected with the horizontal arm of the bell crank by the piston rod *ο*. The circumference of the piston is rounded off, and its diameter is about $\frac{1}{16}$ inch less than the diameter of the interior of the cylinder. The action of the hydraulic regulator is as follows. The cylinder should be nearly filled with water, or other heavy inelastic fluid. In case of any irregularity in the force of the wheel, or in the friction of the brake, the tendency will be, either to raise or lower the weight, in either case the weight cannot move, except with a corresponding movement of the piston. In consequence of the inelasticity of the fluid, the piston can move only by the displacement of a portion of the fluid, which must evidently pass between the edge of the piston and the cylinder; and the area of this space being very small, compared to the area of the piston, the motion of the latter must be slow,

giving time to alter the tension of the brake screws before the piston has moved far. It is plain that this arrangement must arrest all violent shocks, but, however violent and irregular they may be, it is evident, that, if the mean force of them is greater in one direction than in the other, the piston must move in the direction of the preponderating force, the resistance to a slow movement being very slight. A small portion of the useful effect of the turbine must be expended in this instrument, probably less, however, than in the rude shocks the brake would be subject to without its use.

“For the purpose of ascertaining the velocity of the wheel, a counter was attached to the top of the vertical shaft, so arranged, that a bell was struck at the end of every fifty revolutions of the wheel.

“To lubricate the friction pulley, and at the same time to keep it cool, water was let on to its surface in four jets, two of which are shown. These jets were supplied from a large cistern, in the attic of the neighbouring cotton mill, kept full, during the working hours of the mill, by force pumps. The quantity of water discharged by the four jets was, by a mean of two trials, 0.0288 cubic feet per second.

“In many of the experiments with heavy weights, and consequently slow velocities, oil was used to lubricate the brake, the water, during the experiment, being shut off. It is found, that, with a small quantity of oil, the friction between the brake and the pulley is much greater than when the usual quantity of water is applied; consequently, the requisite tension of the brake screws is much less

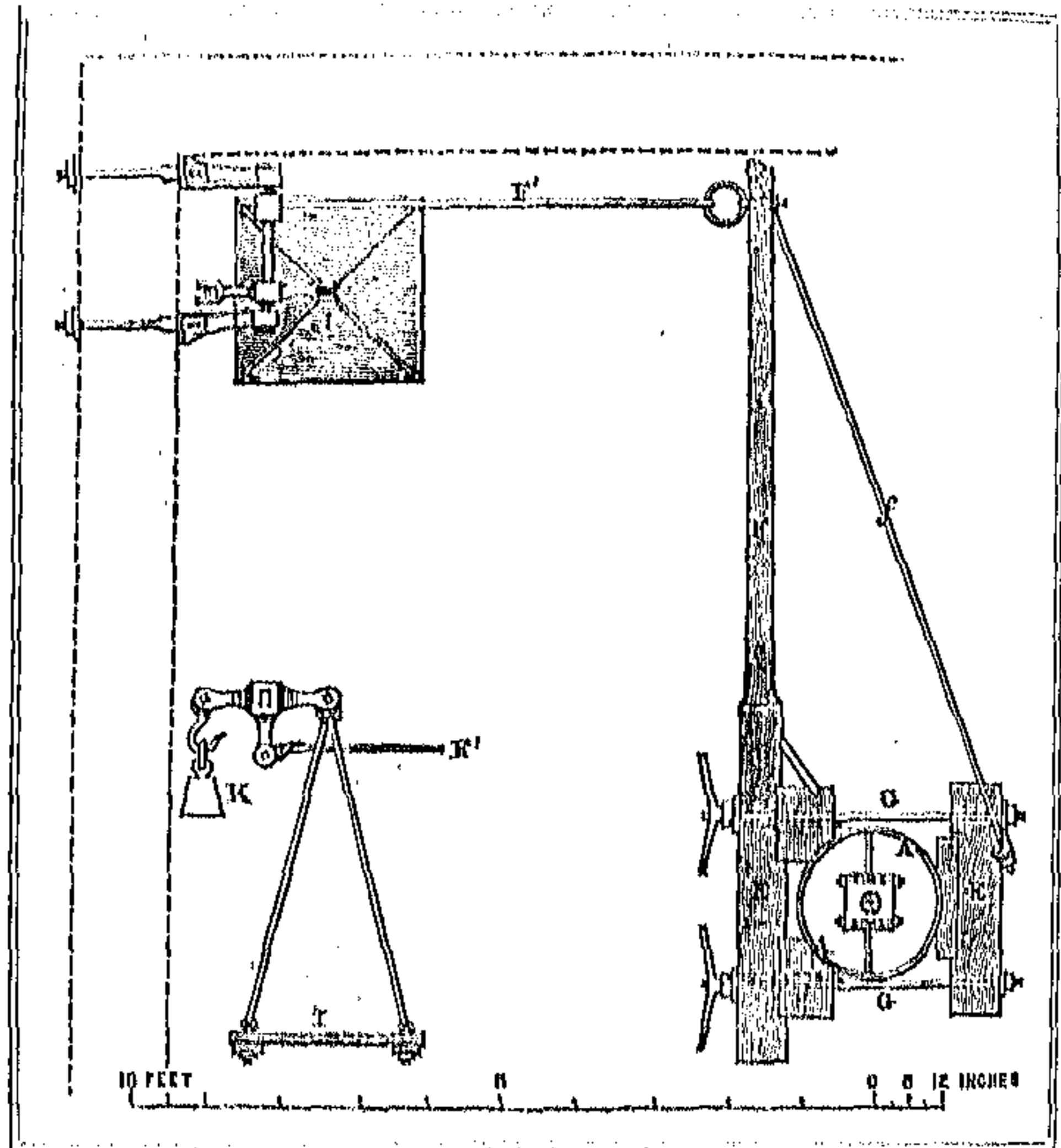
with the oil, as a lubricator, than with water. This may not be the whole cause of the phenomenon, but, whatever it may be, the ease of regulating in slow velocities is incomparably greater with oil as a lubricator, than with water applied in a quantity sufficient to keep the pulley cool. The oil was allowed to flow on in two fine continuous streams; it did not, however, prevent the pulley from becoming heated sufficiently to decompose the oil, after running some time, which was distinctly indicated by the smoke and peculiar odour. When these indications became very apparent, the experiment was stopped, and water let on by the jets, until the pulley was cooled. As the pulley became heated, the brake screws required to be gradually slackened. Water, linseed oil, and resin oil, were each used for lubrication."

Fig. 46 is a representation of a brake used by Professor Thomson, at Crawford and Lindsay's mill, to determine the power of a turbine put up there, by Mr. Gardner, of Armagh. One of the common causes of the swinging or vibratory jumps of the arms *r*, in Figs. 44, 45, and 46 is, that very often the friction pulley, or drum *A, A*, must be made in two parts, so as to be fixed to its place on the shaft. This fixing is liable to give an oval shape, and causes an irregular action with the clamps *E, E*. Oil gives greater regularity of motion than water, but without the use of the latter abundantly, the friction pulley would usually get too much heated. The following calculations from practical operations will point out pretty clearly the use of the brake, and the manner

of determining the useful effect in the tables of experiments, by Francis and Thomson, pp. 383 and 392:—

FIG. 46.

FRICTION BRAKE, USED AT CHAWFORD AND LINDRAY'S MILL, BY PROFESSOR JAMES THOMSON, TO DETERMINE THE POWER OF THE TURBINE.



Length of the brake, v , Fig. 46, adjusted . . . 9.745 feet.

Effective length of vertical arm l . . . 4.5 "

Effective length of horizontal arm h . . . 5.0 "

The effective length of the brake was therefore

$$\frac{9.745 \times 5}{4.5} = 2 \times 10.827778 \text{ feet; and the circumference}$$

of a circle of this radius $= 10.827778 \times 3.14159 = 68.0329$ feet.

In the first experiment on the Tremont turbine,

page 393, the number of revolutions of the wheel per second was $\cdot 89374$, and the weight in the scale $1443\cdot 34$ lbs. The useful effect of the brake was therefore in foot-pounds per second $68\cdot 0329 \times \cdot 89374 \times 1443\cdot 34 = 87680\cdot 3$ lbs. raised one foot per second. The quantity of water which passed the gauge-weir in cubic feet per second was $139\cdot 4206$, and the total fall acting on the wheel $12\cdot 864$ feet; therefore, the total power of the water acting on the wheel was $12\cdot 864 \times 139\cdot 4206 \times 62\cdot 375 = 111870$ foot-pounds per second, $62\cdot 375$ being taken as the weight in lbs. of a cubic foot of water at 32° Fahrenheit. The ratio of the useful effect, at the given velocity of the wheel (viz. 450 revolutions in 503·5 seconds), to the power expended, is therefore $\frac{87680\cdot 3}{111870} = \cdot 784$, or about $78\frac{1}{2}$ per cent. The effect in the experiments generally appears to have been a maximum, when the velocity of the interior circumference of the wheel was about 66 per cent. of the velocity due to the fall; and this was about half of the maximum velocity, which was 1·333 times that due to the fall alone, when the turbine was doing no work.

In Thomson's brake for determining the useful effect of the vortex turbine, erected from his designs at Ballysillan, Ireland, $r = 4$ ft. 2 in., and the circumference of a circle that would be described by the arm $3\cdot 14159 \times 8$ feet 4 inches. $= 26\cdot 18$ feet. In the first experiment, taken from the tabulated results, page 383, we get $26\cdot 18$, the circumference multiplied by $46\cdot 31$, the weight in lbs., and the product by $323\cdot 3$, the number of revolutions per minute, equal

to 391,967 foot-pounds, for the effect transmitted from the turbine or work done. We have also 354.4, the number of cubic feet of water passed to the wheel per minute, multiplied by 62.37, the weight of a cubic foot of water in lbs., multiplied by 23.73 feet, the available fall, equal to 524,526 foot-pounds:

therefore $\frac{391,967}{524,526} = .747$ is the useful effect, that in the table being .7481, which probably arose from taking a different weight per cubic foot for the water. Of course the difference is immaterial. The drum attached to the vortex wheel shaft for fixing the brake to, was in two parts, bolted together, and firmly enclosing the shaft. It was of cast-iron, 20 inches diameter, and 8 inches wide; the shaft to which it was attached was 2½ inches diameter. The arm of the brake was 5'4" × 6" × 4"½, of timber, and extending 1 foot 2 inches beyond the centre of the shaft and drum. The clamping pieces were about 2 feet 5 inches long externally.

For overshot wheels the ratio of the power to the effect may be taken as 3 to 2, and therefore the effective horse-power, — taking 33,000 foot-pounds per minute as a standard, — will be 49,500 lbs. of water falling one foot in one minute. The maximum effect varies with the construction of the wheel. Smeaton found it .76 times the theoretical power; Weisbach .78 times for the wheel of a Stamp Mill at Frieberg, which was 23 feet high, 3 feet wide, carrying 48 buckets.* To find the effective horse-

* Some valuable experiments on the power of water wheels are given by Rennie, in Weale's Quarterly Papers of Engineering, vol. vi. They however require reduction.

power, the theoretical horse-power must be here divided by the coefficient of effect $\cdot 76$ or $\cdot 78$, which will give 43,600 foot-pounds, or 43,300 foot-pounds per minute. The following experimental results from a model wheel are by Smeaton.

TABLE containing the Result of Sixteen Experiments, on a Model Overshot Wheel, by Smeaton.

No.	Whole descent, in inches.	Water expended in a minute, in lbs.	Turns at the maximum in a minute.	Weight raised at the maximum in lbs.	Power of the whole descent.	Power of the Wheel.	Effect.	Ratios of the whole effect to the power.	Ratios of the effect to the power or coefficient of effect.	Mean Ratios.
1	27	30	10	61	810	720	656	$\cdot 80$	$\cdot 77$	$\cdot 81$
2	27	56 $\frac{1}{2}$	16 $\frac{1}{2}$	14 $\frac{1}{2}$	1580	1300	1000	$\cdot 69$	$\cdot 78$	
3	27	50 $\frac{1}{2}$	20 $\frac{1}{2}$	12 $\frac{1}{2}$	1630	1300	1107	$\cdot 70$	$\cdot 84$	
4	27	63 $\frac{1}{2}$	20 $\frac{1}{2}$	13 $\frac{1}{2}$	1710	1524	1245	$\cdot 73$	$\cdot 82$	
5	27	70 $\frac{1}{2}$	21 $\frac{1}{2}$	16 $\frac{1}{2}$	2070	1840	1500	$\cdot 73$	$\cdot 82$	
6	28 $\frac{1}{2}$	73 $\frac{1}{2}$	18 $\frac{1}{2}$	17 $\frac{1}{2}$	2000	1764	1476	$\cdot 70$	$\cdot 84$	$\cdot 82$
7	28 $\frac{1}{2}$	96 $\frac{1}{2}$	20 $\frac{1}{2}$	20 $\frac{1}{2}$	2765	2320	1868	$\cdot 68$	$\cdot 80$	
8	30	90	20	10 $\frac{1}{2}$	2700	2160	1755	$\cdot 65$	$\cdot 81$	$\cdot 82$
9	30	103 $\frac{1}{2}$	20 $\frac{1}{2}$	20 $\frac{1}{2}$	2000	2320	1914	$\cdot 66$	$\cdot 82$	
10	30	113 $\frac{1}{2}$	21	23 $\frac{1}{2}$	3400	2720	2221	$\cdot 65$	$\cdot 82$	
11	33	50 $\frac{1}{2}$	20 $\frac{1}{2}$	13 $\frac{1}{2}$	1870	1300	1230	$\cdot 66$	$\cdot 90$	$\cdot 85$
12	33	100 $\frac{1}{2}$	22 $\frac{1}{2}$	21 $\frac{1}{2}$	3520	2500	2153	$\cdot 61$	$\cdot 84$	
13	33	140 $\frac{1}{2}$	23	27 $\frac{1}{2}$	4840	3520	2846	$\cdot 69$	$\cdot 81$	
14	35	65	10 $\frac{1}{2}$	16 $\frac{1}{2}$	2275	1500	1460	$\cdot 65$	$\cdot 95$	$\cdot 85$
15	35	120	21 $\frac{1}{2}$	25 $\frac{1}{2}$	4200	2880	2407	$\cdot 59$	$\cdot 86$	
16	35	163 $\frac{1}{2}$	26	26 $\frac{1}{2}$	5728	3024	2081	$\cdot 52$	$\cdot 76$	

In this table the effective power of the water must be reckoned upon the whole descent, because it must be raised that height, in order to be in a condition of producing the same effect a second time.

The ratios between the *powers* so estimated, and the *effects* at the *maximum*, deduced from the several sets of experiments, are exhibited at one view, in column 9, of Table II. ; and from hence it appears, that those ratios differ from that of 10 to 7·6 to that of 10 to 5·2, that is, nearly from 4 to 3 to 4 to 2. In those experiments where the heads of water and quantities expended are least, the proportion is nearly as 4 to 3, but where the heads and quantities are greatest, it approaches nearer to that of 4 to 2 ; and by a medium of the whole, the ratio is that of 3 to 2, nearly. We have seen before, in our observations upon the effects of undershot wheels, that the general ratio of the power to the effect, when greatest, was 3 to 1 ; *the effect, therefore, of overshot wheels, under the same circumstances of quantity and fall, is at a medium double to that of the undershot ; and, as a consequence thereof, that non-elastic bodies, when acting by their impulse or collision, communicate only a part of their original power ; the other part being spent in changing their figure, in consequence of the stroke.*

The powers of water, computed from the height of the wheel only, compared with the effects as in column 10, appear to observe a more constant ratio : for, if we take the medium of each class, which is set down in column 11, we shall find the extremes to differ no more than from the ratio of 10 to 8·1 to that of 10 to 8·5 ; and as the second term of the ratio gradually increases from 8·1 to 8·5, by an increase of head from 3 inches to 11, the excess of 8·5 above 8·1 is to be imputed to the superior impulse of the water

at the head of 11 inches, above that of 3 inches : so that if we reduce 8.1 to 8, on account of the impulse of the 3-inch head, *we shall have the ratio of the power, computed upon the height of the wheel only, to the effect at a maximum, as 10 to 8 or as 5 to 4, nearly*; and from the equality of the ratio between power and effect subsisting, where the constructions are similar, we must infer, *that the effects, as well as the powers, are as the quantities of water and perpendicular heights, multiplied together respectively.*

FOR BREAST WHEELS, the ratio of the theoretical power to the effective power must vary considerably, the mean value being about 1 to .5 and, therefore, the effective horse-power would be 66,000 foot-pounds in one minute. Morin gives an efficiency of from .52 to .7. Egen, with a wheel 23 feet in diameter, $4\frac{1}{2}$ feet wide, having 69 ventilated buckets, very well constructed, found at best an efficiency of only .52, under ordinary circumstances .48, the mean amount being .5. Very wide wheels give a larger effect, sometimes as high as .7; but a great deal depends on the manner of bringing on the water and the construction of the wheel and buckets.

FOR UNDERSHOT WHEELS the mean effect may be taken at one-third or .33, or 100,000 foot-pounds in one minute for an effective horse's power; a maximum effect of .5 is sometimes approached, and a minimum of .26 or less. The following results, obtained from a model, are given by Smeaton. The *virtual* or *effective* head is here termed the theoretical head due to the velocity of the wheel, at the circumference, which was 75 inches girth.

TABLE containing the Result of Twenty-seven Experiments, on a Model Undershot Wheel, by Smeaton.

	Height of water in the cistern, in inches.	Turns of the wheel unloaded.	Virtual head deduced therefrom in inches.	Turns at the maximum.	Load at the equilibrium, in lbs. and ozs.	Load at the maximum, in lbs. and ozs.	Water expended in a minute.	POWER.	EFFECT.	Ratio of the effect to the power or coefficient of effect.	Ratio of the velocity of the wheel and water.	Ratio of the load at the equilibrium, to the load at the maximum.
1	33	88	15.85	30.	13 10	10 0	275.	4358	1411	.324	.340	.775
2	30	86	15.0	30.	13 10	0 0	261.7	3970	1200	.320	.350	.740
3	27	83	13.7	28.	11 2	8 6	243.	3320	1044	.315	.340	.750
4	24	78	12.3	27.7	9 10	7 5	235.	2890	901.4	.312	.355	.753
5	21	75	11.4	25.0	8 10	6 5	214.	2439	735.7	.303	.345	.732
6	18	70	9.95	23.5	6 10	5 5	199.	1970	561.8	.285	.330	.802
7	15	65	8.54	23.4	5 2	4 4	178.5	1524	442.5	.260	.360	.830
8	12	60	7.20	22.	3 10	3 5	161.	1173	328	.280	.377	.910
9	9	52	5.47	19.	2 12	2 8	134.	733	213.7	.260	.365	.910
10	6	42	3.55	16.	1 12	1 10	114.	404.7	117.0	.282	.380	.930
11	24	84	14.2	30.75	13 10	10 14	343.	4890	1505	.307	.300	.700
12	21	81	13.5	29.	11 10	0 6	297.	4000	1223	.301	.362	.805
13	18	73	10.5	26.	9 10	8 7	285.	3993	975	.325	.300	.875
14	15	60	9.0	25.	7 10	6 14	277.	2059	774	.302	.302	.900
15	12	63	8.0	25.	5 10	4 14	234.	1872	549	.304	.307	.870
16	9	50	6.37	23.	4 0	3 13	201.	1280	300	.305	.410	.950
17	6	46	4.35	21.	2 8	2 4	167.5	712	212	.308	.455	.900
18	16	72	10.5	29.	11 10	9 6	357.	3748	1210	.323	.402	.805
19	12	60	8.75	26.75	8 10	7 6	330.	2887	878	.305	.405	.810
20	9	58	6.8	24.5	5 8	5 0	255.	1734	541	.301	.422	.910
21	6	48	4.7	23.5	3 2	3 0	228.	1064	317	.300	.490	.960
22	12	68	9.3	27.	9 2	8 0	350.	3338	1006	.302	.397	.917
23	9	58	6.8	26.25	6 2	5 13	332.	2257	680	.304	.452	.950
24	6	48	4.7	24.5	3 12	3 8	262.	1291	385	.313	.510	.935
25	9	60	7.29	27.3	9 12	0 0	355.	2588	785	.303	.455	.945
26	6	50	5.03	24.0	4 6	4 1	307.	1514	450	.292	.490	.930
27	6	50	5.03	26.	4 15	4 9	300.	1811	531	.295	.520	.925

Smeaton derived the following "maxims" from the foregoing experiments. Their truth, independent of any experiment, will be apparent:—

I.—That the virtual or effective head being the same, the effect will be nearly as the quantity of water expended.

II.—That the expence of water being the same, the effect will be nearly as the height of the virtual or effective head.

III.—That the quantity of water expended being the same, the effect is nearly as the square of the velocity.

IV.—The aperture being the same, the effect will be nearly as the cube of the velocity of the water.

FOR TURBINES OR HORIZONTAL WHEELS, a useful effect of two-thirds or $\cdot 67$ may be assumed, or 49,500 foot-pounds in a minute for a horse-power, and the efficiency varies from $\cdot 5$ to $\cdot 8$, or less.* Poncelet's turbine gives an efficiency of $\cdot 5$ to $\cdot 6$. Floating wheels $\cdot 38$, impact wheels from $\cdot 16$ to $\cdot 4$, and Barker's mill from $\cdot 16$ to $\cdot 35$. We believe that the efficiency of the turbine has been too often over-estimated, and that the great advantage of this wheel, as a medium of power, is derived from its capability of employment for all falls, whether large or small, without any considerable loss of effect. In Ireland, Mr. Gardner, of Armagh, was amongst the first, if not the first, to apply this wheel to practical purposes; and Professor Thomson has, in his

* In our first edition we gave an efficiency of $\cdot 821$, on the authority of a paper by Dr. Robinson, Armagh, in the Proceedings of the Royal Irish Academy, vol. iv., p. 214. On again glancing over this paper, we believe there are mistakes, which vitiate the results there given; first, in the formula for calculating the discharge over the weir, and next, in the formula for finding the effect of the brake. Francis gives an efficiency of $\cdot 88$, p. 8, his book.

vortex wheels, produced, we believe, the highest efficiencies which have yet been obtained in practice. In the experiments on the Ballysillan wheel, higher efficiencies would probably have been attained with a supply pipe of larger diameter. It will be seen from the remarks, at pp. 171 and 172, and the tables, at pp. 152 and 191, that quite apart from bends, &c., a loss of mechanical power always results from the passage through orifices and pipes; and that it is necessary to take this loss into account, before the head acting on the wheel can be accurately used to determine its effective power. The table, next page, contains the experiments on the Ballysillan turbine.

The following remarks on the vortex turbine, read at the meeting of the British Association at Belfast, in 1852, are also by Professor Thomson :—

“Numberless are the varieties, both of principle and of construction, in the mechanisms by which motive power may be obtained from falls of water. The chief modes of action of the water are, however, reducible to three, as follows :—First, the water may act directly by its weight on a part of the mechanism which descends while loaded with water, and ascends while free from load. The most prominent example of the application of this mode is afforded by the ordinary bucket water wheel. Secondly, the water may act by fluid pressure, and drive before it some yielding part of a vessel by which it is confined. This is the mode in which the water acts in the water pressure engine, analogous to the ordinary high-pressure steam-engine. Thirdly, the water, having been brought to its place of action subject to the pressure

TABULATED STATEMENT of Experiments made on March 29th, 1854, by Professor James Thomson, C.E., Queen's College, Belfast; on the Vortex Wheel at Ballysillan, to determine its efficiency. The Radius of the Friction Brake was 4 feet 2 inches. The corresponding Circumference is 26.18 feet. In this Table, the quantity of Water passing over the Weir is calculated by means of M.M. Poncelet and Lesbros' Coefficients for Notches.

Number of Experiments.	Total weight of cord of Brake in pounds.	Time of each Experiment.	Number of Revolutions during Experiment.	Number of Revolutions per minute.	Total height of Fall in feet.	Height in feet of Water over Weir, length of Weir being 3 feet.	Number of cubic feet of Water per minute over Weir.	Number of cubic feet of Water per minute supplied to the Wheel, supposed being due to weir.	Work done out of Friction Brake, in foot pounds per minute.	Horse Power given out by the Wheel.	Work due to the Fall of Water in foot-pounds.	Efficiency of the Wheel or Work done out of the Water being 100.	Remarks entered at the time of the experiments, and relating to the supposed accuracy of the experiments.
1	46-31	5 34	1800	323-3	23-73	.718	355-8	354-4	392000	11-88	523900	74-81	Satisfactorily accurate.
2	46-31	5 38	1800	319-5	23-71	.718	355-8	354-4	387400	11-73	523500	73-99	Satisfactorily accurate.
3	53-31	6 11	1800	291-	23-72	.718	355-8	354-4	406100	12-31	523700	77-55	Good.
4	60-31	7 9	1800	251-7	23-71	.718	355-8	354-4	397400	12-04	523500	75-91	Water supposed running over.
5	53-31	6 12	1800	290-3	23-71	.718	355-8	354-4	405100	12-27	523500	77-39	One of the best, no water lost.
6	39-31	5 26	1800	351-2	23-21	.718	355-8	354-4	340800	10-33	512400	66-51	Some water lost at water case, and water too low part of time.
7	46-31	6 9	1800	292-7	23-77	.665	318-1	316-7	354900	10-75	469000	75-66	Good.
8	46-31	6 13	1800	289-5	23-75	.665	318-1	316-7	351000	10-63	468600	74-90	Good, one of the best.
9	32-31	5 8	1800	350-6	23-75	.665	318-1	316-7	296600	8-99	468600	63-28	Very good.
10	36-31	5 24	1800	333-3	23-75	.665	318-1	316-7	316800	9-59	468600	67-61	Very good.
11	36-31	5 21	1800	336-4	23-75	.665	318-1	316-7	319800	9-69	468600	68-24	Excellent.
12	53-31	7 5	1800	254-1	23-75	.663	316-7	315-3	354600	10-74	466500	75-01	Arm not very steady.
13	53-31	7 14	1800	248-8	23-75	.663	316-7	315-3	347200	10-52	466500	74-43	Arm not very steady.

Total Height of Fall = 24 feet. Standard quantity of Water = 420 cubic feet per minute. Calculated speed 292.5 revolutions per minute. Effective Standard Power at an efficiency of 75 per cent = 16.25 Horse Power. Supply pipe, 21 inch bore. Diameter of Wheel, 1 foot 8½ inches.

REMARKS.—It is to be observed, that as the experiments happened to be made in dry weather, the stream in none of them supplied the standard quantity of water for which the wheel was more particularly adapted. Even with the diminished quantity of water, the efficiencies experimentally found were very high. It is also to be observed, that in some of the experiments the resistance to the wheel was purposely made less than what was most suitable for the water supply. The wheel thus ran considerably too fast in those experiments, and consequently the efficiencies in them came to be not so high as in the others that were more properly arranged. Also, from information received after the time of the experiments, it appeared that during the experiments the Joint Rings of the Vortex were not screwed properly close to the rings of the Water Wheel; and that on their being afterwards screwed, the power of the wheel was sensibly increased. It is therefore probable that still higher efficiencies are attainable than those shown by the above experiments.

due to the height of fall, may be allowed to issue through small orifices with a high velocity, its inertia being one of the forces essentially involved in the communication of the power to the moving part of the mechanism. Throughout the general class of water wheels called turbines, which is of wide extent, the water acts according to some of the variations of which this third mode is susceptible. In our own country, and more especially on the Continent, turbines have attracted much attention, and many forms of them have been made known by published descriptions. The subject of the present communication is a new water wheel, which belongs to the same general class, and which has recently been invented and brought successfully into use by the author.

“In this machine the moving wheel is placed within a chamber of a nearly circular form. The water is injected into the chamber tangentially at the circumference, and thus it receives a rapid motion of rotation. Retaining this motion it passes onwards towards the centre, where alone it is free to make its exit. The wheel, which is placed within the chamber, and which almost entirely fills it, is divided by thin partitions into a great number of radiating passages. Through these passages the water must flow on its course towards the centre; and in doing so it imparts its own rotatory motion to the wheel. The whirlpool of water acting within the wheel chamber, being one principal feature of this turbine, leads to the name *Vortex* as a suitable designation for the machine as a whole.

“The vortex admits of several modes of construction, but the two principal forms are the one adapted

for high falls and the one for low falls. The former may be called the High-pressure Vortex, and the latter the Low-pressure Vortex. Examples of these two kinds are in operation at two mills near Belfast.

“The height of the fall for the first vortex is about 37 feet, and the standard or medium quantity of water, for which the dimensions of the various parts of the wheel and case are calculated, is 540 cubic feet per minute. With this fall and water-supply the estimated power is 28 horse-power, the efficiency being taken at 75 per cent. The proper speed of the wheel, calculated in accordance with its diameter and the velocity of the water entering its chamber, is 355 revolutions per minute. The diameter of the wheel is $22\frac{1}{8}$ inches, and the extreme diameter of the case is 4 feet 8 inches.

“In the second vortex, the fall being taken at 7 feet, the calculated quantity of water admitted, at the standard opening of the guide-blades, is 2,460 cubic feet per minute. Then, the efficiency of the wheel being taken at 75 per cent., its power will be 24 horse-power. Also, the speed at which the wheel is calculated to revolve is 48 revolutions per minute.

“The two examples which have now been described of vortex water wheels, adapted for very distinct circumstances, will serve to indicate the principal features in the structural arrangements of these new machines in general. Respecting their principles of action some further explanations will next be given. In these machines the velocity of the circumference is made the same as the velocity of the entering water, and thus there is no impact between the water and the

wheel ; but, on the contrary, the water enters the radiating conduits of the wheel gently, that is to say, with scarcely any motion in relation to their mouths. In order to attain the equalization of these velocities, *it is necessary that the circumference of the wheel should move with the velocity which a heavy body would attain, in falling through a vertical space equal to half the vertical fall of the water, or in other words, with the velocity due to half the fall ;* and that the orifices through which the water is injected into the wheel-chamber should be conjointly of such area that when all the water required is flowing through them, it also may have a velocity due to half the fall. Thus one-half only of the fall is employed in producing velocity in the water ; and, therefore, the other half still remains acting on the water within the wheel-chamber at the circumference of the wheel, in the condition of fluid pressure. Now, with the velocity already assigned to the wheel, it is found that this fluid pressure is exactly that which is requisite to overcome the centrifugal force of the water in the wheel, and to bring the water to a state of rest at its exit ; the mechanical work due to both halves of the fall being transferred to the wheel during the combined action of the moving water and the moving wheel. In the foregoing statements, the effects of fluid friction, and of some other modifying influences, are, for simplicity, left out of consideration ; but in the practical application of the principle, the skill and judgment of the designer must be exercised in taking all such elements, as far as possible, into account. To aid in this, some practical rules, to which the author as yet closely adheres,

were made out by him previously to the date of his patent. These are to be found in the specification of the patent, published in the *Mechanics' Magazine* for January 18 and January 25, 1851 (London).

"In respect to the numerous modifications of construction and arrangement which are admissible in the vortex, while the leading principles of action are retained, it may be sufficient here merely to advert,—first, to the use of straight instead of curved radiating passages in the wheel; secondly, to the employment, for simplicity, of invariable entrance orifices, or of fixed instead of moveable guide-blades; and lastly, to the placing of the wheel at any height, less than about thirty feet, above the water in the tail-race, combined with the employment of suction pipes descending from the central discharge orifices, and terminating in the water of the tail-race, so as to render available the part of the fall below the wheel.

"In relation to the action of turbines in general, the chief and most commonly recognized conditions, of which the accomplishment is to be aimed at, are that the water should flow through the whole machine with the least possible resistance, and that it should enter the moving wheel without shock, and be discharged from it with only a very inconsiderable velocity. The vortex is in a remarkable degree adapted for the fulfilment of these conditions. The water moving centripetally (instead of centrifugally, which is more usual in turbines), enters at the period of its greatest velocity (that is, just after passing the injection orifices) into the most rapidly moving part of the wheel, the circumference; and, at the period

when it ought to be as far as possible deprived of velocity, it passes away by the central part of the wheel, the part which has the least motion. Thus, in each case, that of the entrance and that of the discharge, there is an accordance between the velocities of the moving mechanism and the proper velocities of the water.

“The principle of injection from without inwards, adopted in the vortex, affords another important advantage in comparison with turbines having the contrary motion of the water; as it allows ample room, in the space outside of the wheel, for large and well-formed injection channels, in which the water can be made very gradually and regularly to converge to the most contracted parts, where it is to have its greatest velocity. It is as a concomitant also of the same principle, that the very simple and advantageous mode of regulating the power of the wheel, by the moveable guide-blades already described, can be introduced. This mode, it is to be observed, while giving great variation to the areas of the entrance orifices, retains at all times very suitable forms for the converging water channels.

“Another adaptation in the vortex is to be remarked as being highly beneficial, that, namely, according to which, by the balancing of the contrary fluid pressures due to half the head of water and to the centrifugal force of the water in the wheel, combined with the pressure due to the ejection of the water backwards from the inner ends of the vanes of the wheel when they are curved, only one-half of the work due to the fall is spent in communicating *vis viva* to the water,

to be afterwards taken from it during its passage through the wheel; the remainder of the work being communicated through the fluid pressure to the wheel, without any intermediate generation of *vis viva*. Thus the velocity of the water, where it moves fastest in the machine, is kept comparatively low; not exceeding that due to half the height of the fall, while in other turbines the water usually requires to act at much higher velocities. In many of them it attains at two successive times the velocity due to the whole fall. The much smaller amount of action, or agitation, with which the water in the vortex performs its work, causes a material saving of power by diminishing the loss necessarily occasioned by fluid friction.

“In the vortex, further, a very favourable influence on the regularity of the motion proceeds from the centrifugal force of the water, which, on any increase of the velocity of the wheel, increases, and so checks the water supply; and on any diminution of the velocity of the wheel, diminishes, and so admits the water more freely; thus counteracting, in a great degree, the irregularities of speed arising from variations in the work to be performed. When the work is subject to great variations, as for instance in saw-mills, in bleaching works, or in forges, great inconvenience often arises with the ordinary bucket water-wheels and with turbines which discharge at the circumference, from their running too quickly when any considerable diminution occurs in the resistance to their motion.

“The first vortex which was constructed on the

large scale was made in Glasgow, to drive a new beetling-mill of Messrs. O. Hunter and Co., of Dunadry, in County Antrim. It was the only one in action at the time of the meeting of the British Association in Belfast; but the two which have been particularly described in the present article, and one for an unusually high fall, 100 feet, have since been completed, and brought into operation. There are also several others in progress; of which it may be sufficient to particularize one of great dimensions and power, for a new flax-mill at Ballyshannon in the West of Ireland. It is calculated for working at 150 horse-power, on a fall of 14 feet, and it is to be impelled by the water of the River Erne. This great river has an ample reservoir in the Lough of the same name; so that the water of wet weather is long retained, and continues to supply the river abundantly even in the driest weather. The lake has also the effect of causing the floods to be of long duration, and the vortex will consequently be, through a considerable part of the year, and for long periods at a time, deeply submerged under back-water. The water of the tail-race will frequently be seven feet above its ordinary summer level; but as the water of the head-race will also rise to such a height as to maintain a sufficient difference of levels, the action of the wheel will not be deranged or impeded by the floods. These circumstances have had a material influence in leading to the adoption in the present case of this new wheel in preference to the old breast or undershot wheels."

The next tables have been arranged by us from

Mr. Francis' valuable experiments. They show the ratio of the effect to the power in two wheels, the first a centre-vent wheel, erected at the Boott Cotton Mills, and the second a turbine, erected at the Tremont Mills, Lowell, Massachusetts.

The maximum effect $\cdot 794$ was obtained from the Tremont turbine experiments, when the velocity of the interior circumference of the wheel was to that due to the whole fall as $\cdot 63$ to 1; and an effect of $\cdot 78$ per cent. was obtained when these velocities were as $\cdot 51$ to 1. In the Boott centre-vent wheel the maximum effect $\cdot 797$ was obtained when the velocity of the exterior circumference was to that due to the fall as $\cdot 64$ to 1; and a like effect was produced when this ratio was $\cdot 708$ to 1. Indeed, between these ratios the useful effect was nearly the same; an effect of $\cdot 78$ to $\cdot 79$ was obtained for all such ratios between limits of $\cdot 50$ and $\cdot 71$ to 1, averaging a ratio of $\cdot 65$ to 1. If a turbine have a variable fall, say from 2 to 1, and be of sufficient capacity to give the required power always, the dimensions should be determined from the lesser fall, and if correctly so determined, it will not have sufficient velocity for the greater fall. When the fall is greatest the quantity in the same place is generally least, giving thereby a lessened effect when most is required. For such cases two turbines may be used with advantage.

TABLE showing the Results of Experiments upon a Model of a Centre-vent Water Wheel, an also upon a Centres-vent Water Wheel at the Hunt Cotton Mills, Lowell, Massachusetts arranged from Mr. Francis' valuable Experiments. Diameter of Wheel to the outside of the Buckets, about 19 feet. Depth of External Guide Curves about 7½ feet. External height of Wheel about 16 feet. Number of Buckets, 40. Mean height of the Orifices between the blades, 1 foot. Diameter of Supply Pipe, 8 feet. The first seven Experiments were made on a Model, the Exterior Diameter of the Wheel being 23½ inches, Interior Diameter 10½ inches, height between the Crowns 2½ inches, and the number of Buckets 31. The construction of the Wheel is shown in Mr. Francis' Book, and all necessary Detail. The general principle of the Centre-vent Wheel of Francis' and Thomson's Porter Wheel appears to be the same; the Guide Blades being fewer in the latter, and capable of adjustment.

Numbers of the Experiments.	Rate of Flow of the Water.	Depth of the Water in feet.	Circle feet of Water striking the Wheel per second.	Number of Pounds of Water striking the Wheel per second.	Revolutions of the Wheel per second.	Weight in the same in Pounds avoirdupois.	Ratio of the force to the power, calculated by means of Prony's Brake.	Velocity due to the fall acting on the Wheel, in feet per second.	Velocity of the outside circumference of the Wheel, in feet per second.
1	2.62	3.65	2.15	337.7	1.14	16	.679	12.73	6.83
2	2.46	3.66	2.16	331.5	1.34	14	.711	12.58	8.03
3	2.50	3.67	2.17	338.0	1.54	12.5	.716	12.08	9.23
4	2.60	3.72	2.21	358.2	1.70	12	.705	12.03	10.03
5	2.60	3.73	2.22	361.3	1.73	11.5	.692	12.05	10.36
6	2.60	3.73	2.22	359.6	1.71	11.5	.680	12.03	10.26
7	2.00	3.74	2.23	360.8	1.90	10.0	.648	12.93	11.15
8	14.00	1.296	67.53	61493.4	1.18	390.9	.377	30.65	17.39
9	14.67	1.262	64.89	59364.4	1.20	776.6	.203	30.72	25.77
10	14.57	1.282	66.43	60347.0	1.14	963.3	.332	30.61	21.21
11	14.16	1.284	66.61	58831.6	1.08	1069	.382	30.18	15.97
12	14.20	1.290	67.03	59351.7	1.02	1150.8	.381	30.22	14.65
13	14.14	1.288	66.89	59002.2	.96	1213.0	.373	30.16	13.18
14	14.24	1.294	67.37	59858.1	.93	1295.6	.269	30.27	7.46
15	14.30	1.311	61.08	54486.4	.90	1345.5	..	30.33	29.75
16	14.29	1.513	85	75732.8	.86	1396.1	.303	30.32	29.63
17	14.23	1.531	86.35	76603.0	.83	1444.0	.376	30.25	28.16
18	14.20	1.539	87.08	77093.3	.80	1494.7	.413	30.22	27.36
19	14.19	1.547	87.68	77607.2	.77	1648.7	.444	30.21	26.43
20	14.19	1.554	88.28	78143.8	.71	1667.0	.489	30.22	25.12
21	13.78	1.676	90.17	77480.4	1.55	..	.582	29.77	20.37
22	13.61	1.694	91.70	77812.1	1.35	816.0	.596	29.58	17.64
23	13.64	1.418	77.11	67076.7	1.22	610.7	..	29.9	34.61
24	13.62	1.643	95.76	80736.3	1.10	720.3	.245	29.49	32.44
25	13.87	1.673	98.49	83145.3	1.00	832.3	.437	29.33	29.14
26	13.87	1.695	100.42	83728.3	.93	934.7	.576	29.32	26.43
27	13.40	1.718	102.42	85671.4	.84	1038.3	.657	29.35	24.20
28	13.38	1.723	102.82	85800.0	.77	1116.0	.690	29.34	23.07
29	13.34	1.731	103.62	86138.0	.77	1116.0	.749	29.30	21.89
30	13.32	1.731	103.77	86218.8	.69	1201.8	.720	29.27	21.26
31	13.33	1.733	103.70	86229.3	.62	1278.0	.728	29.29	20.86
32	13.30	1.739	104.23	86483.7	.46	1482.6	.731	29.25	20.50
33	13.70	1.698	92.02	78618.0	.86	1482.6	..	29.70	36.78
34	13.40	1.832	112.62	94057.5	.82	1544.9	.707	29.36	20.81
35	13.43	1.837	112.99	94662.2	.79	1604.8	.796	29.39	20.51
36	13.33	1.832	112.66	93603.9	1.25	..	.797	29.28	19.93
37	13.38	1.837	113.00	94206.4	1.15	118.6	.797	29.33	19.71
38	13.39	1.838	113.07	94476.2	.96	325.4	.796	29.34	19.36
39	13.38	1.839	113.16	94471.1	.77	619.9	.796	29.34	19.03
40	13.36	1.838	113.09	94219.0	.67	612.2	.797	29.31	18.67
41	13.38	1.844	113.67	94881.9	.66	704.4	.791	29.34	18.32
42	13.40	1.851	114.29	95571.3	.46	777.6	.787	29.36	18.00
43	13.32	1.848	113.97	94703.1	.30	882.1	.781	29.27	17.88
44	13.54	1.869	119.45	99250.1	.62	148.6	..	29.51	..
45	13.57	1.807	110.32	93422.4	.69	73.1	..	29.55	..
46	13.60	1.988	99.79	84635.0	.39	296.4	..	29.57	37.70

TABLE showing the Results of Experiments upon the Turbine at Tremont Mills, Lowell, Massachusetts, arranged from Mr. Francis' valuable Experiments. Diameter, measured to the Exterior Circumference of Crowns of the Wheel, 8.333 feet. Height of Buckets from top of the Disc to the bottom of the Garniture, .97 feet. Number of Buckets, 44. Width of the Buckets 8 foot, nearly. Width of Guide Curves, 2.8 feet, nearly. Number of Ditto, 33. A Double Weir with 4 end constructions and 16.98 feet long, used for gauging the Water the Crest being 6.5 feet above the floor of the Wheel Pit. The Falls show the difference of heads in the Forebay and Wheel Pit. For further details, see Francis' Lowell Hydraulic Experiments, pp. 1 to 43. The supply pipe is fully a quadrant, and varies from 6 to 9 feet in diameter.

Numbers of the Experiments.	Falls acting on the Wheel.	Depth on the Weir in feet.	Cubic feet of Water acting on the Wheel per second.	Number of pounds avoirdupois, if raised one foot per second.	Number of revolutions of the Wheel per second.	Weight in the scale in pounds avoirdupois.	Ratio of the effect to the power, calculated by means of Prony's brake or dynamometer.	Velocity due to the fall acting on the Wheel, in feet per second.	Velocity of the interior circumference of the Wheel, in feet per second.
1	12.804	1.88	139.42	111870.0	.894	1443.34	.784	28.76	18.95
2	12.860	1.88	130.47	111951.2	.892	1443.34	.784	28.77	18.96
3	12.611	2.02	154.40	121444.2	.893	1443.34	.353	28.48	32.49
4	12.600	1.97	149.40	118363.5	1.596	307.03	.507	28.58	29.32
5	12.777	1.94	146.02	110373.2	1.532	411.48	.622	28.67	26.40
6	12.810	1.92	143.91	115087.3	1.461	519.77	.703	28.71	28.86
7	12.856	1.91	142.52	114284.2	1.382	638.36	.735	28.76	22.63
8	12.888	1.90	142.04	114187.1	1.309	750.42	.750	28.79	21.71
9	12.890	1.90	141.28	113640.9	1.245	854.87	.766	28.80	20.57
10	12.883	1.89	140.08	112568.7	1.184	957.35	.779	28.79	19.57
11	12.899	1.88	139.00	112563.3	1.125	1057.49	.781	28.80	19.13
12	12.905	1.88	139.01	111893.5	1.784	..	.789	28.81	18.92
13	12.899	1.88	139.03	111850.4	1.784	..	.788	28.80	18.77
14	12.902	1.87	138.85	111740.3	1.087	1156.27	.790	28.81	18.06
15	12.906	1.87	138.51	111504.9	1.024	1229.41	.792	28.81	18.37
16	12.915	1.87	138.27	111384.0	.990	1269.42	.794	28.82	17.73
17	12.934	1.87	138.23	111521.1	.970	1319.22	.792	28.84	17.25
18	12.939	1.86	137.71	111139.7	.945	1359.23	.783	28.85	15.74
19	12.940	1.84	135.14	109077.1	.923	1397.12	.770	28.85	13.69
20	12.963	1.84	135.34	109433.4	.911	1416.70	.769	28.88	13.72
21	12.977	1.83	133.75	108265.8	.902	1433.43	.725	28.89	11.29
22	12.948	1.82	133.43	107704.7	.897	1443.00	.679	28.86	9.63
23	12.954	1.87	138.02	112000.3	1.781	..	.791	28.86	17.64
24	12.932	1.87	138.50	111720.6	.892	1454.24	.791	28.84	17.32
25	12.951	1.87	138.37	111777.2	.885	1464.80	.783	28.86	16.74
26	12.758	1.92	143.33	114080.7	.875	1474.37	.346	28.65	31.45
27	12.909	1.86	137.75	110917.6	.880	1474.37	.675	28.82	24.21
28	12.950	1.86	137.00	110864.7	.873	1485.63	.725	28.86	21.73
29	12.965	1.84	135.10	109252.2	.866	1498.00	.750	28.88	19.74
30	12.999	1.82	133.30	108082.5	.851	1524.07	.760	28.92	18.35
31	13.026	1.81	131.99	107240.3	1.790	..	.763	28.95	17.05
32	13.028	1.80	130.80	106366.6	.830	1552.44	.750	28.95	15.01
33	13.077	1.68	118.55	96699.5	.813	1597.08	.300	29.00	28.66
34	13.134	1.60	116.10	95112.0	.784	1648.87	.453	29.06	25.84
35	13.215	1.63	113.24	93346.0	.742	1724.49	.609	29.15	21.28
36	13.282	1.59	109.71	90893.3	.695	1816.71	.652	29.23	17.88
37	13.310	1.58	107.95	89620.7	.646	1911.45	.656	29.26	16.44
38	13.302	1.54	103.85	86555.6	1.786	..	.619	29.32	13.06
39	12.883	1.80	137.30	110380.8	.647	1911.45	.781	28.79	18.12
40	12.896	1.86	136.07	110176.6	.600	2011.52	.784	28.80	17.42
41	12.912	1.85	136.55	109973.0	.532	2167.38	.783	28.82	16.73
42	13.309	1.27	78.84	65748.8	.454	2307.88	.141	29.32	24.30
43	13.305	1.25	76.62	64018.1	..	2213.38	.332	29.35	20.86
44	13.435	1.22	74.05	62062.0	..	3046.38	.440	29.40	16.37
45	13.478	1.19	71.87	60424.6	1.783	..	.463	29.44	14.25
46	13.513	1.17	70.01	59006.4	.832	1585.21	.455	29.48	11.87
47	13.550	1.11	64.50	54554.2	.817	1590.50	.330	29.53	6.36
48	13.985	.78	38.22	33940.8	.803	1814.79	.150	29.99	13.14
49	14.001	.78	38.57	33683.5	.780	1841.34	.102	30.01	14.61
50	14.020	.76	37.17	32508.0	.769	1679.62	.240	30.03	8.22

TABLE for Turbines of different Diameters, modified from Francis, operating with different Falls; assuming the useful effect is seventy-five per cent. of the power expended, that the Velocity of the Interior Circumference is fifty-six per cent. of the Velocity due to the Fall; and that also, the Height between the Crowns is one-tenth of the Outside Diameter.

Fall in feet.	Outside diameter 2 ft. Inside 1.50 Number of buckets 30.			Outside diameter 3 ft. Inside 2.25 Number of buckets 36.			Outside diameter 4 ft. Inside 3.24 Number of buckets 42.			Outside diameter 5 ft. Inside 4.11 Number of buckets 45.		
	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.
5	4.6	1.9	123	10.1	4.3	80	17.88	7.6	59	27.9	11.9	47
6	4.9	2.5	135	11.0	5.6	88	19.0	10.0	65	30.0	15.6	51
7	5.3	3.1	146	11.9	7.1	95	21.17	12.6	70	33.1	19.7	55
8	5.7	3.8	156	12.7	8.7	102	22.03	15.1	75	35.3	24.0	59
9	6.0	4.6	165	13.5	10.3	108	24.00	18.1	79	37.5	28.7	63
10	6.3	5.4	174	14.2	12.1	114	25.30	21.5	84	39.5	33.0	66
11	6.6	6.2	183	14.9	13.0	119	26.53	24.8	88	41.5	38.8	69
12	6.9	7.1	191	15.6	15.9	125	27.71	28.3	92	43.3	44.2	72
13	7.2	8.0	199	16.3	17.9	130	28.81	31.9	95	45.1	49.8	75
14	7.5	8.9	206	16.8	20.0	136	29.93	35.6	99	46.8	55.7	78
15	7.7	9.9	213	17.4	22.2	139	30.98	39.5	103	48.4	61.7	81
16	8.0	10.9	220	18.0	24.5	144	32.00	43.5	106	50.0	68.0	83
17	8.2	11.9	227	18.5	26.8	148	32.99	47.7	109	51.5	74.5	86
18	8.5	13.0	234	19.1	29.2	153	33.94	51.9	113	53.0	81.1	88
19	8.7	14.1	240	19.6	31.7	157	34.87	56.3	115	54.5	88.0	91
20	8.9	15.2	247	20.1	34.2	161	35.78	60.8	118	55.9	95.0	93
21	9.2	16.4	253	20.6	36.8	165	36.66	65.4	121	57.3	102.2	95
22	9.4	17.5	259	21.1	39.5	169	37.52	70.2	124	58.6	109.6	98
23	9.6	18.7	264	21.6	42.2	172	38.37	75.0	127	59.9	117.2	100
24	9.8	20.0	270	22.0	45.0	176	39.19	79.9	130	61.2	124.9	102
25	10.0	21.2	276	22.5	47.8	180	40.00	85.0	133	62.5	132.8	104
26	10.2	22.5	281	22.9	50.7	183	40.79	90.1	135	63.7	140.9	106
27	10.4	23.8	286	23.4	53.7	187	41.57	95.4	138	65.0	149.1	108
28	10.6	25.2	292	23.8	56.7	190	42.33	100.7	140	66.1	157.4	110
29	10.8	26.5	297	24.2	59.7	194	43.08	106.2	143	67.3	165.9	112
30	10.9	27.9	302	24.6	62.8	197	43.82	111.7	145	68.5	174.6	114
31	11.1	29.3	307	25.0	66.0	200	44.54	117.4	147	69.6	183.4	116
32	11.3	30.8	312	25.5	69.2	203	45.25	123.1	150	70.7	192.3	118
33	11.5	32.2	317	25.8	72.5	207	45.90	128.9	152	71.8	201.4	120
34	11.7	33.7	321	26.2	75.8	210	46.65	134.8	154	72.9	210.6	122
35	11.8	35.2	326	26.6	79.2	213	47.33	140.8	157	73.9	220.0	123
36	12.0	36.7	331	27.0	82.6	216	48.00	146.9	159	75.0	229.5	125
37	12.2	38.3	335	27.4	86.1	219	48.66	153.0	161	76.0	239.1	127
38	12.3	39.8	340	27.7	89.6	222	49.32	159.3	163	77.0	248.9	129
39	12.5	41.4	344	28.1	93.2	225	49.96	165.6	165	78.1	258.8	130
40	12.6	43.0	349	28.5	96.8	227	50.60	172.0	167	79.1	268.8	132

TABLE of Turbines of different Diameters, modified from Francis, operating with different Falls; assuming the useful effect is seventy-five per cent. of the power expended, that the Velocity of the Interior Circumference is fifty-six per cent. of the Velocity due to the Fall, and that also, the Height between the Crowns is one-tenth of the Outside Diameter.

Fall in feet.	Outside diameter 6 ft. Inside 5 ft. Number of buckets 18.			Outside diameter 7 ft. Inside 5-90 Number of buckets 21.			Outside diameter 8 ft. Inside 6-81 Number of buckets 24.			Outside diameter 10 ft. Inside 8-67 Number of buckets 30.		
	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.	Water discharged in cubic feet per second.	Horse-power.	Revolutions per minute.
5	40.2	17.1	38	54.8	23.3	32.5	71.5	30.4	28.1	111.8	47.5	22.1
6	44.1	22.5	42	60.0	30.6	35.6	78.4	40.0	30.8	122.5	62.5	24.2
7	47.6	28.3	45	64.8	38.6	38.4	84.7	50.4	33.3	132.3	78.7	26.2
8	50.9	34.6	48	69.3	47.1	41.1	90.5	61.5	35.6	141.4	96.2	28.0
9	54.0	41.3	51	73.5	56.2	43.6	96.0	73.4	37.8	150.0	114.7	29.7
10	56.9	48.4	54	77.5	65.9	46.0	101.2	86.0	39.8	158.1	134.4	31.3
11	59.7	55.8	57	81.3	76.0	48.2	106.1	99.2	41.7	165.8	155.0	32.8
12	62.4	63.6	59	84.9	86.6	50.3	110.8	113.1	43.6	173.2	176.7	34.3
13	64.9	71.7	62	88.3	97.6	52.4	115.4	127.5	45.4	180.3	199.2	35.7
14	67.3	80.1	64	91.7	109.1	54.4	119.7	142.5	47.1	187.1	222.6	37.0
15	69.7	88.9	66	94.9	121.0	56.3	123.9	158.0	48.7	193.6	246.9	38.3
16	72.0	97.9	69	98.0	133.3	58.1	128.0	174.1	50.3	200.0	272.0	39.6
17	74.2	107.2	71	101.0	146.0	59.9	131.9	190.6	51.9	206.2	297.9	40.8
18	76.4	116.8	73	103.9	159.0	61.7	135.8	207.7	53.4	212.1	324.6	42.0
19	78.5	126.7	75	106.8	172.5	63.3	139.5	225.3	54.9	217.9	352.0	43.1
20	80.5	136.8	77	109.6	186.3	65.0	143.1	243.3	56.3	223.6	380.1	44.3
21	82.5	147.2	79	112.3	200.4	66.6	146.6	261.7	57.7	229.1	409.0	45.4
22	84.4	157.9	80	114.9	214.9	68.2	150.1	280.7	59.0	234.5	438.5	46.4
23	86.3	168.8	82	117.5	229.7	69.7	153.5	300.0	60.4	239.8	468.8	47.5
24	88.2	179.9	83	120.0	244.8	71.2	156.8	319.8	61.7	244.9	499.7	48.5
25	90.0	191.2	85	122.5	260.3	72.7	160.0	340.0	62.9	250.0	531.2	49.5
26	91.8	202.8	87	124.9	276.1	74.1	163.2	360.6	64.2	254.9	563.4	50.5
27	93.5	214.6	89	127.3	292.2	75.5	166.3	381.6	65.4	259.8	596.3	51.4
28	95.2	226.7	91	129.6	308.5	76.9	169.3	403.0	66.6	264.6	629.7	52.4
29	96.9	238.9	92	131.9	325.2	78.3	172.3	424.8	67.8	269.3	663.7	53.3
30	98.6	251.1	94	134.2	342.2	79.6	175.3	446.9	68.9	273.9	698.3	54.2
31	100.2	264.1	95	136.4	359.4	80.9	178.2	469.5	70.1	278.4	733.5	55.1
32	101.8	277.0	97	138.6	377.0	82.2	181.0	492.4	71.2	282.8	769.3	56.0
33	103.4	290.0	98	140.7	394.8	83.5	183.8	515.6	72.3	287.2	805.7	56.9
34	105.0	303.3	100	142.9	412.9	84.7	186.6	539.2	73.4	291.5	842.6	57.7
35	106.5	316.8	101	144.9	431.2	86.0	189.3	563.2	74.5	295.8	880.0	58.5
36	108.0	330.5	103	147.0	449.8	87.2	192.0	587.5	75.5	300.0	918.0	59.4
37	109.5	344.3	104	149.0	468.7	88.4	194.6	612.2	76.6	304.1	956.5	60.2
38	111.0	358.4	105	151.0	487.8	89.6	197.3	637.1	77.6	308.2	995.5	61.0
39	112.4	372.6	107	153.0	507.2	90.8	199.8	662.5	78.6	312.2	1035.1	61.8
40	113.8	387.1	108	154.9	526.8	91.9	202.4	688.1	79.6	316.2	1075.2	62.6

THE HYDRAULIC RAM has been applied with advantage in raising water to a considerable height by the momentum of a larger quantity at a lower level. The shock of the valves, and vibration of the machine, require heavy and strong setting, and considerable strength in all the parts. This limits its application, and prevents its use for raising large quantities of water. The work done by the ram, in over one thousand experiments by Eytelwein, did not exceed in any of them 1480 lbs. raised one foot in one minute; and in France, the ram put up by the younger Montgolfier, said to be the largest constructed, raised only 7400 lbs. one foot high per minute, and had a useful effect, it is reported, of .65. This ram was put up at Mello, near Clermont-sur-Oise. Its dimensions were—

Length of the body pipe or injection pipe	108 feet.
Diameter	4.3 inches.
Weight of body pipe	8190 lbs.
Weight of head	440 lbs.
Contents of air-chamber	1½ gallons.

This ram worked under a head of 37 feet, discharging in use 31½ gallons each minute, and raising 3.85 gallons a height of 195 feet.

The largest ram employed by Eytelwein in his experiments had the following dimensions—

Length of the body pipe or injection pipe	43 feet 9 inches.
Diameter of ditto	9 feet 2½ inches.
Contents of air-chamber	1.94 gallons.
Area of tail or escape valve	3.74 square inches;

and his experiments led to the following practical formula by D'Aubuisson—

$$\frac{d h'}{D h} = 1.42 - .28 \sqrt{\frac{h'}{h}} :$$

in which D is the water used per minute in gallons, d the quantity raised in gallons, h the head used, and h' the lift of the quantity d . By a slight reduction we get

$$d h' = 1.42 D (h - .28 \sqrt{h h'})$$

for the effect produced, which is reduced nearly one-sixth for practical application, giving the formula

$$d h' = 1.2 D (h - .2 \sqrt{h h'})$$

for the work done.

EXPERIMENTAL RESULTS.—HYDRAULIC RAM.

Number of strokes per minute.	Height in feet of		Ratio of heights.	Gallons of water per minute.		Ratio $\frac{Dh}{dh}$		Ratio $\frac{D}{d}$
	Fall h	Elevation h'		Expended D	Raised d	Experiments.	Formula.	
66	Ft. In. 10' 0"	Ft. In. 20' 4"	2.03	10.06	3.30	.0	.07	2.02
64	10 3	33 4	3.18	13.07	3.83	.873	.02	3.07
60	9 11	38 8	3.0	12.01	2.022	.85	.87	4.68
52	8 0	33 4	4	8.10	1.087	.817	.86	4.72
45	8 0	38 8	4.4	10.86	2.00	.815	.84	5.2
43	7 6	38 8	5.21	9.02	1.5	.787	.78	6.02
36	6 0	38 8	6.5	8.80	1.05	.764	.71	8.02
20	4 0½	33 4	7.2	5.23	.406	.072	.07	10.7
31	5 0	38 7	7.7	8.05	.704	.007	.05	11.54
23	4 1	38 8	9.47	11.11	.640	.548	.60	17.2
17	3 0	32 2	10.7	10.8	.470	.473	.61	22.0
16	3 3	38 8	11.0	12.34	.363	.352	.45	33.8
14	3 0	38 8	15.6	11.05	.22	.284	.92	54.0
10	1 11½	38 8	19.3	9.81	.088	.181	.18	100.6

Eytelwein recommends, that the length of the body pipe should not be less than three-fourths of the height to which the water is to be raised; its diameter in inches equal $\cdot58 \sqrt{h}$; the diameter of the rising pipe $\cdot3 \sqrt{h}$; and the contents of the air-chamber equal to that of the rising pipe.

The following table gives the result of experiments made by Montgolfier and his son:—

TABLE OF EXPERIMENTAL RESULTS—HYDRAULIC RAY.

Height.		Water per Minute.		$\frac{d}{D}$	Mean Ratio $\frac{dh}{Dh}$
Fall h	Elevation h	Expended D	Delivered d		
Ft. In.	Ft. In.	Gallons.	Gallons.		
8' 0"	52' 8"	16	1.37	.57	..
97	195	31	3.86	.663	..
94	111	19.5	3.74	.651	.66
9	14	407	60.10	.620	..
93	100	290	0.23	.671	..

Latterly, the Messrs. Easton and Amos have patented improvements in this machine, and have raised water to a height of 330 feet. The injection pipe is laid by them at an inclination of about one in four for high falls, and varies down to one in eighteen for smaller falls. The quantities raised in their practice vary up to six gallons per minute.

WATER PRESSURE ENGINES give a useful effect varying up to 70 per cent. for the best constructed. An immense amount of mechanical skill and invention has been brought to bear on their construction,

and in Weisbach's book* a useful effect of 83 per cent. has been calculated; this is, however, a result seldom obtained in practice, where two-thirds, or 66 per cent., is nearer to the general efficiency. Jordan got a maximum efficiency of .66 from one of the Clausthal engines, making four strokes per minute, and .71 making three strokes per minute. These results were for the combined engine and pumps, from which it was calculated that the efficiency of the engine alone, was in the first case .83, and in the second .85. It would be a great mistake to calculate on such high efficiencies.

CORN MILLS will grind about a bushel of corn per horse-power per hour, but much depends on the state of the stones and of the grain. The value of the work done in an hour being once known, the value of the standard horse-power can be determined accordingly.

* Vol. ii., p. 342.

TABLE I.—Coefficients of Discharge from Square and different proportioned Rectangular Lateral Orifices in thin Vertical Plates arranged from Poncelet and Lesbros.

Heads of water measured to the center of the orifice in English inches.	Ratio of the head to the depth of the orifice.	Square orifice $B^2 \times B^2$, Ratio of the sides 1 to 1.		Rectangular orifice $B^2 \times 4B$, Ratio of the sides 4 to 1.		Rectangular orifice $B^2 \times 2B$, Ratio of the sides 2 to 1.	
		Heads taken back from the orifice.	Heads taken at the orifice.	Heads taken back from the orifice.	Heads taken at the orifice.	Heads taken back from the orifice.	Heads taken at the orifice.
0.000			.610		.607		.713
0.197	.025		.597		.600		.698
0.394	.050		.595		.618	.607	.642
0.591	.075		.594	.593	.616	.612	.639
0.787	.100	.572	.594	.590	.614	.615	.638
1.181	.150	.578	.593	.600	.613	.620	.637
1.575	.200	.582	.593*	.603	.612	.623	.636
1.969	.250	.585	.593	.605	.612*	.625	.636
2.362	.300	.587	.594	.607	.613	.627	.635
2.756	.350	.588	.594	.609	.613	.628	.635
3.150	.400	.589	.594	.610	.613	.629	.635
3.545	.450	.591	.595	.610	.614	.629	.634
3.937	.500	.592	.595	.611	.614	.630	.634
4.724	.600	.593	.596	.612	.614	.630	.633
5.512	.700	.595	.597	.613	.614	.630	.632
6.299	.800	.596	.597	.614	.615	.631*	.631
7.087	.900	.597	.598	.615	.615	.630	.631
7.874	1.000	.598	.599	.615	.615	.630	.630
8.661	1.250	.599	.600	.616	.616	.630	.630
11.811	1.500	.600	.601	.616	.616	.629	.629
16.748	2.000	.602	.602	.617	.617	.628	.629
19.685	2.500	.603	.603	.617*	.617*	.628	.628
22.622	3.000	.604	.604	.617	.617	.627	.627
27.560	3.500	.604	.604	.616	.616	.627	.627
31.497	4.000	.605	.605	.616	.616	.627	.627
35.434	4.500	.605*	.605*	.615	.615	.626	.626
39.371	5.000	.605	.605	.615	.615	.626	.626
43.307	5.500	.604	.604	.614	.614	.625	.625
47.245	6.000	.604	.604	.614	.614	.624	.624
51.182	6.500	.603	.603	.613	.613	.622	.622
55.119	7.000	.603	.603	.612	.612	.621	.621
59.056	7.500	.602	.602	.611	.611	.620	.620
62.993	8.000	.602	.602	.611	.611	.618	.618
66.930	8.500	.602	.602	.610	.610	.617	.617
70.867	9.000	.601	.601	.609	.609	.615	.615
74.805	9.500	.601	.601	.608	.608	.614	.614
78.742	10.000	.601	.601	.607	.607	.613	.614
118.112	15.000	.601	.601	.603	.603	.609	.609

See pages 71, 72, and 73.

TABLE I.—Coefficients of Discharge from Square and differently proportioned Rectangular Lateral Orifices in thin Vertical Plates, arranged from Poncelet and Lesbros.

Rectangular orifice 8" X 1.18". Ratio of the sides 7 to 1 nearly.		Rectangular orifice 8" X 0.8". Ratio of the sides 10 to 1.		Rectangular orifice 8" X 0.4". Ratio of the sides 20 to 1.		Ratio of the head to the length of the orifice.	Heads of water measured to the upper sides of the orifices in English inches.
Heads taken back from the orifice.	Heads taken at the orifice.	Heads taken back from the orifice.	Heads taken at the orifice.	Heads taken back from the orifice.	Heads taken at the orifice.		
	.766		.783		.795		
	.725		.750	.705	.778	.025	0.197
.680	.687	.600	.720	.701	.762	.050	0.304
.632	.674	.600	.707	.697	.745	.075	0.591
.634	.668	.650	.697	.694	.729	.100	0.787
.638	.659	.659	.685	.688	.708	.150	1.181
.640	.654	.658	.678	.683	.695	.200	1.575
.640*	.651	.658	.672	.679	.686	.250	1.969
.640	.647	.657	.668	.676	.681	.300	2.362
.639	.645	.656	.665	.673	.677	.350	2.756
.638	.643	.656	.662	.670	.675	.400	3.150
.637	.641	.655	.659	.668	.672	.450	3.543
.637	.640	.654	.657	.666	.669	.500	3.937
.636	.637	.653	.655	.663	.665	.600	4.724
.635	.636	.651	.653	.660	.661	.700	5.512
.634	.635	.650	.651	.658	.659	.800	6.299
.634	.634	.649	.650	.657	.657	.900	7.087
.633	.633	.648	.649	.655	.656	1.000	7.874
.632	.632	.646	.646	.653	.653	1.250	9.843
.632	.632	.644	.644	.650	.651	1.500	11.811
.631	.631	.642	.642	.647	.647	2.000	15.748
.630	.630	.640	.640	.644	.645	2.500	19.685
.630	.630	.638	.638	.642	.643	3.000	23.622
.629	.629	.637	.637	.640	.640	3.500	27.560
.629	.629	.636	.636	.637	.637	4.000	31.497
.628	.628	.634	.634	.635	.635	4.500	35.434
.628	.628	.633	.633	.632	.632	5.000	39.371
.627	.627	.631	.631	.629	.629	5.500	43.307
.626	.626	.628	.628	.626	.626	6.000	47.245
.624	.624	.625	.625	.622	.622	6.500	51.182
.622	.622	.622	.622	.618	.618	7.000	55.119
.620	.620	.619	.619	.615	.615	7.500	59.056
.618	.618	.617	.617	.613	.613	8.000	62.993
.616	.616	.615	.615	.612	.612	8.500	66.930
.615	.615	.614	.614	.612	.612	9.000	70.867
.613	.613	.612	.612	.611	.611	9.500	74.805
.612	.612	.612	.612	.611	.611	10.000	78.742
.608	.608	.610	.610	.609	.609	15.000	118.112

See pages 71, 72, and 73.

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TABLE II.—For finding the Velocities from the Altitudes, or Altitudes from the Velocities.

Altitudes 0 feet 0 $\frac{1}{16}$ inch to 0 feet 3 $\frac{1}{4}$ inches.

Altitudes in feet and inches	Coefficients of velocity, and the corresponding velocities of discharge in inches per second.					
	1. Values of $\sqrt{2gH}$ Coefficient of velocity	2. Values of $\sqrt{2gH}$ Coefficient of velocity	3. Values of $\sqrt{2gH}$ Coefficient of velocity	4. Values of $\sqrt{2gH}$ Coefficient of velocity	5. Values of $\sqrt{2gH}$ Coefficient of velocity	6. Values of $\sqrt{2gH}$ Coefficient of velocity
0 0 $\frac{1}{16}$	2.78	2.71	2.66	2.60	2.57	2.5
0 0 $\frac{1}{8}$	3.48	3.38	3.32	3.26	3.23	3.15
0 0 $\frac{3}{16}$	4.05	3.93	3.87	3.80	3.77	3.68
0 0 $\frac{1}{4}$	4.820	4.67	4.60	4.53	4.50	4.40
0 0 $\frac{5}{16}$	12.038	11.72	11.61	11.50	11.46	11.35
0 0 $\frac{3}{8}$	13.400	13.04	12.92	12.80	12.76	12.65
0 0 $\frac{7}{16}$	16.541	16.14	16.00	15.86	15.82	15.70
0 0 $\frac{1}{2}$	17.924	17.50	17.37	17.23	17.19	17.05
0 0 $\frac{9}{16}$	18.388	17.94	17.80	17.66	17.62	17.48
0 0 $\frac{5}{8}$	19.658	19.19	19.04	18.90	18.86	18.72
0 0 $\frac{11}{16}$	20.850	20.37	20.22	20.07	20.03	19.88
0 0 1	21.978	21.47	21.32	21.17	21.13	20.98
0 0 $1 \frac{1}{16}$	23.051	22.54	22.39	22.24	22.20	22.05
0 0 $1 \frac{1}{8}$	24.070	23.55	23.40	23.25	23.21	23.06
0 0 $1 \frac{3}{16}$	25.050	24.51	24.36	24.21	24.17	24.02
0 0 $1 \frac{1}{2}$	26.005	25.53	25.38	25.23	25.19	25.04
0 0 $1 \frac{5}{8}$	26.917	26.52	26.37	26.22	26.18	26.03
0 0 $1 \frac{7}{8}$	27.800	27.38	27.23	27.08	27.04	26.89
0 1 0	28.486	28.05	27.90	27.75	27.71	27.56
0 1 $\frac{1}{16}$	31.081	30.63	30.48	30.33	30.29	30.14
0 1 $\frac{1}{8}$	32.508	32.04	31.89	31.74	31.70	31.55
0 1 $\frac{3}{16}$	34.048	33.57	33.42	33.27	33.23	33.08
0 1 $\frac{1}{2}$	35.438	34.95	34.80	34.65	34.61	34.46
0 1 $\frac{5}{8}$	36.770	36.27	36.12	35.97	35.93	35.78
0 1 $\frac{3}{4}$	38.007	37.49	37.34	37.19	37.15	37.00
0 1 $\frac{7}{8}$	39.216	38.69	38.54	38.39	38.35	38.20
0 2 0	40.625	39.97	39.82	39.67	39.63	39.48
0 2 $\frac{1}{16}$	41.700	40.92	40.77	40.62	40.58	40.43
0 2 $\frac{1}{8}$	42.843	41.73	41.58	41.43	41.39	41.24
0 2 $\frac{3}{16}$	43.956	42.81	42.66	42.51	42.47	42.32
0 2 $\frac{1}{2}$	45.041	43.87	43.72	43.57	43.53	43.38
0 2 $\frac{5}{8}$	46.101	44.90	44.75	44.60	44.56	44.41
0 2 $\frac{3}{4}$	47.137	45.90	45.75	45.60	45.56	45.41
0 2 $\frac{7}{8}$	48.151	46.90	46.75	46.60	46.56	46.41
0 3 0	49.144	47.87	47.72	47.57	47.53	47.38
0 3 $\frac{1}{16}$	50.117	48.81	48.66	48.51	48.47	48.32
0 3 $\frac{1}{8}$	51.072	49.74	49.59	49.44	49.40	49.25
0 3 $\frac{3}{16}$	52.000	50.66	50.51	50.36	50.32	50.17
0 3 $\frac{1}{2}$	52.930	51.55	51.40	51.25	51.21	51.06
0 3 $\frac{5}{8}$	53.834	52.43	52.28	52.13	52.09	51.94
0 3 $\frac{3}{4}$	54.726	53.30	53.15	52.99	52.95	52.80

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 0 feet 0 $\frac{1}{100}$ inch to 0 feet 3 $\frac{1}{8}$ inches.

Coefficients of velocity, and the corresponding velocities of discharge in inches per second.						Altitudes <i>h</i> in feet and inches.
7. Values of $v = 19.46 \sqrt{h}$ Coefficient .780.	8. Values of $v = 18.515 \sqrt{h}$ Coefficient .686.	9. Values of $v = 17.458 \sqrt{h}$ Coefficient .628.	10. Values of $v = 17.153 \sqrt{h}$ Coefficient .617.	11. Values of $v = 16.847 \sqrt{h}$ Coefficient .606.	12. Values of $v = 15.935 \sqrt{h}$ Coefficient .584.	
1.95	1.85	1.75	1.72	1.68	1.62	0 0 $\frac{1}{100}$
2.43	2.31	2.18	2.15	2.11	2.03	0 0 $\frac{1}{4}$
4.87	4.63	4.36	4.29	4.21	4.06	0 0 $\frac{1}{2}$
6.88	6.55	6.17	6.03	5.96	5.74	0 0 $\frac{3}{8}$
8.43	8.02	7.56	7.43	7.29	7.03	0 0 $\frac{1}{2}$
9.73	9.26	8.73	8.58	8.42	8.12	0 0 $\frac{5}{8}$
10.88	10.35	9.76	9.59	9.42	9.08	0 0 $\frac{3}{4}$
11.92	11.24	10.69	10.50	10.32	9.94	0 0 $\frac{7}{8}$
12.87	12.25	11.55	11.35	11.14	10.74	0 0 1
13.76	12.97	12.34	12.13	11.91	11.48	0 0 1 $\frac{1}{8}$
14.60	13.89	13.09	12.86	12.64	12.18	0 0 1 $\frac{1}{4}$
15.38	14.64	13.80	13.56	13.32	12.84	0 0 1 $\frac{1}{2}$
16.14	15.35	14.48	14.22	13.97	13.46	0 0 1 $\frac{3}{8}$
16.85	16.03	15.12	14.85	14.59	14.06	0 0 1 $\frac{1}{2}$
17.54	16.69	15.74	15.46	15.19	14.63	0 0 1 $\frac{5}{8}$
18.20	17.32	16.33	16.04	15.76	15.09	0 0 1 $\frac{3}{4}$
18.84	17.93	16.90	16.61	16.31	15.72	0 0 1 $\frac{7}{8}$
19.46	18.51	17.46	17.15	16.85	16.24	0 1 0
20.04	19.04	18.52	18.19	17.87	17.22	0 1 1
21.76	20.70	19.62	19.18	18.84	18.15	0 1 1 $\frac{1}{8}$
22.82	21.71	20.47	20.11	19.75	19.04	0 1 1 $\frac{1}{4}$
23.85	22.69	21.38	21.01	20.63	19.88	0 1 1 $\frac{1}{2}$
24.81	23.60	22.26	21.87	21.48	20.70	0 1 1 $\frac{3}{8}$
25.74	24.49	23.10	22.69	22.29	21.48	0 1 1 $\frac{1}{2}$
26.65	25.35	23.91	23.49	23.07	22.23	0 1 1 $\frac{5}{8}$
27.52	26.18	24.69	24.26	23.82	22.96	0 1 2
28.37	26.99	25.45	25.00	24.50	23.67	0 1 2 $\frac{1}{8}$
29.19	27.77	26.19	25.73	25.27	24.35	0 1 2 $\frac{1}{4}$
29.99	28.53	26.91	26.43	25.96	25.02	0 1 2 $\frac{1}{2}$
30.77	29.27	27.60	27.12	26.64	25.67	0 1 2 $\frac{3}{8}$
31.53	30.00	28.29	27.79	27.29	26.30	0 1 2 $\frac{1}{2}$
32.27	30.70	28.95	28.44	27.94	26.92	0 1 2 $\frac{5}{8}$
33.00	31.39	29.60	29.08	28.57	27.53	0 1 2 $\frac{3}{4}$
33.71	32.07	30.24	29.71	29.18	28.12	0 1 3
34.40	32.73	30.86	30.32	29.78	28.70	0 1 3 $\frac{1}{8}$
35.08	33.38	31.47	30.92	30.37	29.27	0 1 3 $\frac{1}{4}$
35.75	34.01	32.07	31.51	30.95	29.83	0 1 3 $\frac{1}{2}$
36.41	34.64	32.66	32.09	31.52	30.37	0 1 3 $\frac{3}{8}$
37.05	35.25	33.24	32.66	32.08	30.91	0 1 3 $\frac{1}{2}$
37.68	35.85	33.81	33.22	32.62	31.44	0 1 3 $\frac{5}{8}$
38.31	36.45	34.37	33.77	33.16	31.96	0 1 3 $\frac{3}{4}$

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 0 foot 4 inches to 1 foot.

Altitudes in feet and inches.	Coefficients of velocity, and the corresponding velocities of discharge in inches per second.					
	1. Values of $v = 87.5 \sqrt{h}$ for clay in inches.	2. Values of $v = 87.5 \sqrt{h}$ Coefficient 57.4.	3. Values of $v = 87.5 \sqrt{h}$ Coefficient 50.2.	4. Values of $v = 87.5 \sqrt{h}$ Coefficient 50.1.	5. Values of $v = 87.5 \sqrt{h}$ Coefficient 50.1.	6. Values of $v = 87.5 \sqrt{h}$ Coefficient 50.2.
0 4	55.000	54.15	53.15	47.82	45.31	44.48
0 4 1	56.402	54.60	53.60	48.50	46.02	45.17
0 4 2	57.811	55.05	54.05	49.20	46.71	45.85
0 4 3	59.226	55.50	54.50	50.01	47.30	46.52
0 4 4	60.648	55.95	54.95	50.72	48.00	47.18
0 4 5	62.076	56.40	55.40	51.42	48.73	47.83
0 4 6	63.510	56.85	55.85	52.11	49.38	48.47
0 4 7	64.950	57.30	56.30	52.78	50.02	49.00
0 4 8	66.396	57.75	56.75	53.46	50.66	49.73
0 4 9	67.848	58.20	57.20	54.12	51.29	50.35
0 5 0	69.306	58.65	57.65	54.78	51.91	50.96
0 5 1	70.770	59.10	58.10	55.43	52.53	51.56
0 5 2	72.240	59.55	58.55	56.07	53.14	52.16
0 5 3	73.716	60.00	59.00	56.70	53.74	52.75
0 5 4	75.198	60.45	59.45	57.33	54.33	53.33
0 5 5	76.686	60.90	59.90	57.95	54.92	53.91
0 5 6	78.180	61.35	60.35	58.56	55.50	54.48
0 5 7	79.680	61.80	60.80	59.17	56.08	55.05
0 5 8	81.186	62.25	61.25	59.77	56.65	55.60
0 5 9	82.698	62.70	61.70	60.36	57.22	56.15
0 6 0	84.216	63.15	62.15	60.95	57.79	56.70
0 6 1	85.740	63.60	62.60	61.53	58.35	57.25
0 6 2	87.270	64.05	63.05	62.11	58.90	57.78
0 6 3	88.806	64.50	63.50	62.68	59.45	58.31
0 6 4	90.348	64.95	63.95	63.25	60.00	58.84
0 6 5	91.896	65.40	64.40	63.82	60.55	59.37
0 6 6	93.450	65.85	64.85	64.39	61.10	59.89
0 6 7	95.010	66.30	65.30	64.96	61.65	60.41
0 6 8	96.576	66.75	65.75	65.53	62.20	60.93
0 6 9	98.148	67.20	66.20	66.10	62.75	61.45
0 7 0	99.726	67.65	66.65	66.67	63.30	61.97
0 7 1	101.310	68.10	67.10	67.24	63.85	62.49
0 7 2	102.900	68.55	67.55	67.81	64.40	63.01
0 7 3	104.496	69.00	68.00	68.38	64.95	63.53
0 7 4	106.098	69.45	68.45	68.95	65.50	64.05
0 7 5	107.706	69.90	68.90	69.52	66.05	64.57
0 7 6	109.320	70.35	69.35	70.09	66.60	65.09
0 7 7	110.940	70.80	69.80	70.66	67.15	65.61
0 7 8	112.566	71.25	70.25	71.23	67.70	66.13
0 7 9	114.198	71.70	70.70	71.80	68.25	66.65
0 8 0	115.836	72.15	71.15	72.37	68.80	67.17
0 8 1	117.480	72.60	71.60	72.94	69.35	67.69
0 8 2	119.130	73.05	72.05	73.51	69.90	68.21
0 8 3	120.786	73.50	72.50	74.08	70.45	68.73
0 8 4	122.448	73.95	72.95	74.65	71.00	69.25
0 8 5	124.116	74.40	73.40	75.22	71.55	69.77
0 8 6	125.790	74.85	73.85	75.79	72.10	70.29
0 8 7	127.470	75.30	74.30	76.36	72.65	70.81
0 8 8	129.156	75.75	74.75	76.93	73.20	71.33
0 8 9	130.848	76.20	75.20	77.50	73.75	71.85
0 9 0	132.546	76.65	75.65	78.07	74.30	72.37
0 9 1	134.250	77.10	76.10	78.64	74.85	72.89
0 9 2	135.960	77.55	76.55	79.21	75.40	73.41
0 9 3	137.676	78.00	77.00	79.78	75.95	73.93
0 9 4	139.398	78.45	77.45	80.35	76.50	74.45
0 9 5	141.126	78.90	77.90	80.92	77.05	74.97
0 9 6	142.860	79.35	78.35	81.49	77.60	75.49
0 9 7	144.600	79.80	78.80	82.06	78.15	76.01
0 9 8	146.346	80.25	79.25	82.63	78.70	76.53
0 9 9	148.100	80.70	79.70	83.20	79.25	77.05
1 0 0	149.860	81.15	80.15	83.77	79.80	77.57

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 0 foot 4 inches to 1 foot.

Coefficients of velocity, and the corresponding velocities of discharge in inches per second.						Altitudes h in feet and inches.
7. Values of $v = 19.45 \sqrt{h}$ Coefficient 701.	8. Values of $v = 18.515 \sqrt{h}$ Coefficient 686.	9. Values of $v = 17.458 \sqrt{h}$ Coefficient 668.	10. Values of $v = 17.158 \sqrt{h}$ Coefficient 617.	11. Values of $v = 16.847 \sqrt{h}$ Coefficient 606.	12. Values of $v = 15.985 \sqrt{h}$ Coefficient 634.	
38.02	37.03	34.02	34.31	33.00	32.47	0 4
39.52	37.00	35.46	34.84	34.22	32.97	0 4½
40.12	38.17	35.00	35.30	34.73	33.47	0 4¾
40.70	38.73	36.52	35.88	35.24	33.96	0 4⅝
41.28	39.28	37.03	36.30	35.74	34.44	0 4½
41.85	39.82	37.55	36.80	36.23	34.92	0 4⅝
42.41	40.35	38.05	37.38	36.72	35.38	0 4¾
42.90	40.87	38.54	37.80	37.19	35.84	0 4¾
43.51	41.40	39.04	38.35	37.67	36.30	0 5
44.05	41.91	39.52	38.83	38.14	36.75	0 5½
44.50	42.42	40.00	39.30	38.60	37.20	0 5½
45.12	42.92	40.48	39.77	39.06	37.64	0 5½
45.64	43.42	40.94	40.23	39.51	38.07	0 5½
46.15	43.91	41.41	40.68	39.96	38.51	0 5½
46.66	44.40	41.86	41.13	40.40	38.93	0 5½
47.17	44.88	42.32	41.58	40.83	39.35	0 5½
47.67	45.35	42.76	42.02	41.27	39.77	0 6
48.05	46.20	43.65	42.88	42.12	40.60	0 6½
49.01	47.20	44.51	43.73	42.96	41.39	0 6½
50.56	48.10	45.30	44.50	43.77	42.18	0 6¾
51.40	48.99	46.10	45.38	44.57	42.95	0 7
52.40	49.85	47.01	46.18	45.36	43.71	0 7½
53.29	50.70	47.81	46.97	46.14	44.46	0 7½
54.17	51.54	48.60	47.75	46.90	45.20	0 7¾
55.04	52.37	49.38	48.51	47.65	45.92	0 8
55.89	53.18	50.15	49.27	48.39	46.63	0 8½
56.74	53.98	50.90	50.01	49.12	47.33	0 8½
57.56	54.77	51.64	50.74	49.88	48.02	0 8¾
58.38	55.54	52.38	51.46	50.64	48.71	0 9
59.19	56.31	53.10	52.17	51.24	49.38	0 9½
59.98	57.07	53.81	52.87	51.93	50.04	0 9½
60.76	57.81	54.51	53.56	52.60	50.69	0 9¾
61.54	58.55	55.22	54.24	53.27	51.34	0 10
62.30	59.28	55.89	54.92	53.94	51.98	0 10½
63.06	60.00	56.57	55.58	54.59	52.61	0 10½
63.80	60.70	57.24	56.24	55.24	53.23	0 10¾
64.54	61.41	57.90	56.89	55.87	53.85	0 11
65.27	62.10	58.56	57.53	56.51	54.45	0 11½
65.99	62.79	59.20	58.17	57.13	55.06	0 11½
66.71	63.47	59.84	58.80	57.75	55.65	0 11¾
67.41	64.14	60.48	59.42	58.36	56.24	1 0

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 1 foot 0 $\frac{1}{8}$ inch to 5 feet 8 inches.

Altitudes in feet and inches.	Coefficients of velocity, and the corresponding velocities of discharge in inches per second.					
	1. Values of $v = \pi \sqrt{2gh}$ Coefficient 1.00.	2. Values of $v = \pi \sqrt{2gh}$ Coefficient .974.	3. Values of $v = \pi \sqrt{2gh}$ Coefficient .952.	4. Values of $v = \pi \sqrt{2gh}$ Coefficient .931.	5. Values of $v = \pi \sqrt{2gh}$ Coefficient .910.	6. Values of $v = \pi \sqrt{2gh}$ Coefficient .890.
1 0 $\frac{1}{8}$	98.288	95.73	93.00	91.53	89.10	78.03
1 1	100.234	97.03	95.82	93.20	91.00	80.10
1 1 $\frac{1}{2}$	102.144	99.40	97.05	95.84	93.25	81.71
1 2	104.018	101.31	99.44	98.40	95.77	83.21
1 2 $\frac{1}{2}$	105.859	103.11	101.20	99.01	96.28	84.60
1 3	107.660	104.87	102.93	99.60	97.75	86.14
1 3 $\frac{1}{2}$	109.440	106.60	104.63	99.13	99.20	87.56
1 4	111.200	108.31	106.31	99.63	99.69	88.90
1 4 $\frac{1}{2}$	112.924	109.99	107.99	99.11	99.03	90.34
1 5	114.622	111.42	109.58	98.58	99.42	91.70
1 5 $\frac{1}{2}$	116.296	113.27	111.18	99.01	99.78	93.04
1 6	117.945	114.78	112.76	99.43	99.13	94.36
1 7	119.177	116.03	115.85	99.21	98.70	95.04
1 8	121.325	121.00	118.80	99.02	99.33	96.40
1 9	127.390	124.08	121.70	99.66	99.83	99.02
1 10	130.301	127.00	124.00	112.14	99.27	104.31
1 11	133.324	129.80	127.40	114.00	99.00	106.00
2 0	136.102	132.05	130.20	117.12	99.00	108.05
2 1 $\frac{1}{2}$	140.383	136.73	134.21	120.73	114.41	112.31
2 3	144.453	140.70	138.10	124.23	117.73	115.50
2 4 $\frac{1}{2}$	148.411	144.55	141.98	127.61	120.00	118.73
2 6	152.267	148.31	145.57	130.05	124.10	121.81
2 7 $\frac{1}{2}$	156.027	151.97	149.10	134.18	127.10	124.82
2 9	159.690	155.55	152.07	137.34	130.15	127.70
2 10 $\frac{1}{2}$	163.288	159.04	156.10	140.43	133.80	130.03
3 0	166.800	162.40	159.40	143.46	136.94	133.44
3 1 $\frac{1}{2}$	170.240	165.81	162.76	146.41	138.75	136.10
3 3	173.611	169.10	165.97	149.31	141.40	138.80
3 4 $\frac{1}{2}$	176.918	172.32	169.13	152.15	144.10	141.53
3 6	180.165	175.48	172.24	154.94	146.83	144.13
3 7 $\frac{1}{2}$	183.354	178.50	175.20	157.68	149.43	146.68
3 9	186.488	181.61	178.28	160.38	151.90	149.10
3 10 $\frac{1}{2}$	189.571	184.64	181.23	163.03	154.50	151.00
4 0	192.604	187.60	184.13	165.64	156.97	154.08
4 2	196.570	191.40	187.03	168.00	160.21	157.20
4 4	200.469	195.23	191.05	172.40	163.38	160.37
4 6	204.287	198.98	195.30	175.69	166.40	163.43
4 8	208.030	202.63	198.88	178.91	169.55	166.43
4 10	211.718	206.21	202.40	182.08	172.55	169.37
5 0	215.358	209.74	205.80	185.10	175.50	172.27
5 3	220.050	214.02	210.05	189.70	179.83	176.52

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 1 foot 0 $\frac{1}{2}$ inch to 5 feet 3 inches.

Coefficients of velocity, and the corresponding velocities of discharge in inches per second.						Altitudes h in feet and inches.
7. Values of $v = 19.46 \sqrt{h}$ Coefficient .700.	8. Values of $v = 18.515 \sqrt{h}$ Coefficient .666.	9. Values of $v = 17.458 \sqrt{h}$ Coefficient .633.	10. Values of $v = 17.153 \sqrt{h}$ Coefficient .617.	11. Values of $v = 16.847 \sqrt{h}$ Coefficient .606.	12. Values of $v = 15.935 \sqrt{h}$ Coefficient .584.	
68.80	65.46	61.72	60.04	59.50	57.40	1 0 $\frac{1}{2}$
70.16	66.76	62.95	61.84	60.74	58.54	1 1
71.50	68.03	64.15	63.02	61.90	59.65	1 1 $\frac{1}{2}$
72.81	69.28	65.32	64.18	63.03	60.75	1 2
74.10	70.50	66.48	65.32	64.15	61.82	1 2 $\frac{1}{2}$
75.37	71.71	67.62	66.43	65.25	62.88	1 3
76.61	72.89	68.73	67.53	66.33	63.92	1 3 $\frac{1}{2}$
77.84	74.06	69.83	68.61	67.34	64.94	1 4
79.05	75.21	70.92	69.67	68.49	65.95	1 4 $\frac{1}{2}$
80.24	76.34	71.98	70.72	69.46	66.94	1 5
81.41	77.45	73.03	71.75	70.48	67.92	1 5 $\frac{1}{2}$
82.56	78.55	74.07	72.77	71.47	68.88	1 6
84.82	80.70	76.10	74.77	73.43	70.77	1 7
87.03	82.80	78.08	76.71	75.34	72.61	1 8
89.18	84.85	80.00	78.60	77.20	74.40	1 9
91.28	86.84	81.89	80.45	79.02	76.15	1 10
93.33	88.79	83.73	82.26	80.79	77.86	1 11
95.33	90.70	85.53	84.03	82.53	79.54	2 0
98.27	93.50	88.16	86.02	85.07	81.98	2 1 $\frac{1}{2}$
101.12	96.21	90.72	89.13	87.54	84.36	2 3
103.80	98.84	93.20	91.57	89.94	86.67	2 4 $\frac{1}{2}$
106.50	101.41	95.62	93.95	92.27	88.92	2 6
109.22	103.91	97.99	96.27	94.55	91.12	2 7 $\frac{1}{2}$
111.70	106.36	100.20	98.53	96.78	93.26	2 9
114.30	108.75	102.54	100.75	98.95	95.36	2 10 $\frac{1}{2}$
116.76	111.00	104.75	103.02	101.08	97.41	3 0
119.17	113.38	106.91	105.04	103.17	99.42	3 1 $\frac{1}{2}$
121.53	115.62	109.03	107.12	105.21	101.39	3 3
123.84	117.83	111.10	109.16	107.21	103.32	3 4 $\frac{1}{2}$
126.12	119.99	113.14	111.16	109.18	105.22	3 6
128.35	122.11	115.15	113.13	111.11	107.08	3 7 $\frac{1}{2}$
130.54	124.20	117.11	115.06	113.01	108.91	3 9
132.70	126.25	119.15	116.97	114.88	110.71	3 10 $\frac{1}{2}$
134.82	128.27	120.96	118.84	116.72	112.48	4 0
137.00	130.92	123.45	121.20	119.12	114.80	4 2
140.33	133.51	125.89	123.60	121.48	117.07	4 4
143.00	136.06	128.29	126.05	123.80	119.30	4 6
145.63	138.55	130.65	128.36	126.07	121.49	4 8
148.20	141.00	132.96	130.63	128.30	123.64	4 10
150.74	143.42	135.23	132.86	130.49	125.76	5 0
154.40	146.00	138.57	136.14	133.72	128.86	5 3

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 6 feet 6 inches to 17 feet.

Altitudes h in feet and inches.	Coefficients of velocity, and the corresponding velocities of discharge in inches per second.					
	1. Values of $c = 27.5 \sqrt{h}$, the theoretical velocity in inches.	2. Values of $c = 27.5 \sqrt{h}$, Coefficient $\cdot 974$.	3. Values of $c = 27.5 \sqrt{h}$, Coefficient $\cdot 962$.	4. Values of $c = 27.5 \sqrt{h}$, Coefficient $\cdot 950$.	5. Values of $c = 27.5 \sqrt{h}$, Coefficient $\cdot 938$.	6. Values of $c = 27.5 \sqrt{h}$, Coefficient $\cdot 909$.
6 0	225.848	219.08	216.91	194.23	184.07	180.68
6 3	230.921	224.02	220.70	198.50	188.20	184.74
6 6	236.801	229.76	225.51	202.87	192.25	188.71
6 9	240.756	234.50	230.16	207.05	196.22	192.60
7 0	246.624	239.14	234.72	211.15	200.10	196.42
7 3	250.200	243.69	239.10	215.17	203.91	200.16
7 6	254.701	248.17	243.58	219.12	207.65	203.83
7 9	259.301	252.56	247.89	222.90	211.33	207.44
8 0	263.734	256.88	252.13	226.81	214.94	210.99
8 3	268.003	261.12	256.30	230.66	218.50	214.47
8 6	272.383	265.30	260.40	234.25	221.99	217.91
8 9	276.607	269.41	264.44	237.88	225.43	221.20
9 0	280.760	273.47	268.41	241.46	228.82	224.61
9 3	284.65	277.46	272.33	244.98	232.17	227.80
9 6	288.900	281.39	276.19	248.46	235.46	231.12
9 9	292.801	285.28	280.00	251.80	238.71	234.31
10 0	296.823	289.11	283.76	255.27	241.91	237.46
10 3	300.703	292.88	287.47	258.60	245.07	240.56
10 6	304.534	296.62	291.13	261.90	248.19	243.63
10 9	308.317	300.30	294.75	265.15	251.28	246.65
11 0	312.054	303.94	297.92	268.37	254.32	249.64
11 3	315.747	307.54	301.85	271.64	257.33	252.60
11 6	319.398	311.00	305.34	274.08	260.31	255.52
11 9	323.007	314.41	308.70	277.70	263.25	258.41
12 0	326.576	318.00	312.21	280.80	266.16	261.26
12 3	330.107	321.52	315.68	283.80	269.04	264.09
12 6	333.600	324.93	318.92	286.90	271.88	266.88
12 9	337.057	328.29	322.23	289.87	274.70	269.65
13 0	340.479	331.63	325.50	292.81	277.49	272.38
13 3	343.807	334.93	328.74	295.73	280.25	275.09
13 6	347.222	338.19	331.94	298.61	282.99	277.78
13 9	350.545	341.43	335.12	301.47	285.69	280.44
14 0	353.830	344.64	338.27	304.30	288.38	283.07
14 3	357.097	347.81	341.39	307.10	291.03	285.68
14 6	360.329	350.96	344.47	309.88	293.67	288.26
14 9	363.707	354.17	347.57	312.67	296.27	290.87
15 0	367.076	357.28	350.57	315.40	298.83	293.38
15 3	370.141	360.28	353.46	318.00	301.36	295.81
15 6	375.208	365.19	358.20	321.28	303.84	298.17
15 9	379.181	368.01	360.97	323.42	306.28	300.44
16 0	381.181	370.74	363.50	325.47	308.61	302.65

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 5 feet 6 inches to 17 feet.

Coefficients of velocity, and the corresponding velocities of discharge in inches per second.						Altitudes h in feet and inches.
7. Values of $v = 19.46 \sqrt{h}$ Coefficient .769.	8. Values of $v = 19.515 \sqrt{h}$ Coefficient .769.	9. Values of $v = 17.455 \sqrt{h}$ Coefficient .683.	10. Values of $v = 17.153 \sqrt{h}$ Coefficient .677.	11. Values of $v = 16.847 \sqrt{h}$ Coefficient .666.	12. Values of $v = 15.935 \sqrt{h}$ Coefficient .584.	
158.00	150.41	141.83	139.35	136.80	131.90	5 6
161.05	153.80	145.02	142.48	139.94	134.86	5 9
165.12	157.10	148.14	145.55	142.95	137.76	6 0
168.53	160.34	151.10	148.55	145.90	140.60	6 3
171.87	163.52	154.10	151.49	148.70	143.39	6 6
175.14	166.63	157.13	154.37	151.62	146.12	6 9
178.35	169.60	160.01	157.21	154.40	148.80	7 0
181.51	172.60	162.84	159.99	157.14	151.43	7 3
184.61	175.65	165.62	162.72	159.82	154.02	7 6
187.67	178.55	168.36	165.41	162.46	156.57	7 9
190.67	181.41	171.06	168.06	165.06	159.07	8 0
193.62	184.22	173.71	170.67	167.62	161.54	8 3
196.54	186.99	176.32	173.23	170.14	163.97	8 6
199.41	189.72	178.90	175.76	172.63	166.36	8 9
202.23	192.41	181.43	178.26	175.08	168.72	9 0
205.02	195.07	183.94	180.71	177.49	171.05	9 3
207.78	197.68	186.40	183.14	179.87	173.34	9 6
210.49	200.27	188.84	185.53	182.23	175.61	9 9
213.17	202.82	191.25	187.90	184.55	177.85	10 0
215.82	205.34	193.62	190.23	186.84	180.00	10 3
218.44	207.83	195.97	192.54	189.10	182.24	10 6
221.02	210.29	198.29	194.82	191.34	184.40	10 9
223.58	212.72	200.58	197.07	193.55	186.53	11 0
226.10	215.12	202.85	199.30	195.74	188.64	11 3
228.60	217.50	205.09	201.50	197.91	190.72	11 6
231.07	219.85	207.31	203.68	200.04	192.78	11 9
233.52	222.18	209.50	205.83	202.16	194.82	12 0
235.94	224.48	211.67	207.96	204.26	196.84	12 3
238.34	226.76	213.82	210.08	206.33	198.84	12 6
240.71	229.02	215.95	212.17	208.38	200.82	12 9
243.06	231.25	218.06	214.24	210.42	202.78	13 0
245.38	233.46	220.14	216.30	212.43	204.72	13 3
247.69	235.65	222.21	218.32	214.42	206.64	13 6
249.97	237.83	224.26	220.33	216.40	208.54	13 9
252.23	239.98	226.29	222.32	218.36	210.43	14 0
254.47	242.13	228.29	224.29	220.22	212.30	14 3
256.70	244.23	230.26	226.26	222.06	214.16	14 6
258.91	246.40	232.23	228.13	223.90	216.00	14 9
261.08	248.40	234.23	230.00	225.70	217.82	15 0
263.24	250.51	236.10	231.83	227.44	219.62	15 3
265.38	252.55	237.91	233.67	229.14	221.40	15 6
267.50	254.55	239.61	235.40	230.84	223.16	15 9
269.61	256.53	241.30	237.06	232.50	224.90	16 0
271.71	258.44	242.96	238.71	234.14	226.62	16 3
273.78	260.34	244.60	240.35	235.76	228.32	16 6
275.84	262.23	246.23	241.98	237.36	229.99	16 9
277.89	264.14	247.83	243.60	238.94	231.64	17 0

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 17 feet 0 inches to 40 feet.

Altitudes in feet and inches	Coefficient of velocity, and the corresponding velocities of discharge in inches per second.					
	1. Values of $\sqrt{2gH}$ Coefficient of velocity	2. Values of $\frac{H}{V^2}$ Coefficient of velocity	3. Values of $\frac{H}{V^2}$ Coefficient of velocity	4. Values of $\frac{H}{V^2}$ Coefficient of velocity	5. Values of $\frac{H}{V^2}$ Coefficient of velocity	6. Values of $\frac{H}{V^2}$ Coefficient of velocity
17 0	402.800	302.300	385.13	340.40	328.33	322.20
18 0	408.575	307.95	390.00	351.37	332.40	326.86
19 0	414.211	313.44	395.00	359.22	337.58	331.37
20 0	419.772	318.80	401.30	366.00	342.11	335.82
21 0	425.258	324.20	408.55	372.72	346.59	340.21
22 0	430.676	329.48	414.73	379.38	351.00	344.54
23 0	436.020	334.60	421.84	385.98	355.30	348.82
24 0	441.311	339.64	428.80	392.53	359.50	353.05
25 0	446.534	344.62	435.60	398.92	363.60	357.23
26 0	451.697	349.55	442.32	405.26	367.60	361.36
27 0	456.801	354.42	448.97	411.53	371.50	365.44
28 0	461.848	359.24	455.56	417.75	375.30	369.48
29 0	466.841	364.00	462.00	423.91	379.00	373.47
30 0	471.782	368.72	468.32	429.93	382.60	377.43
31 0	476.671	373.38	474.50	435.90	386.10	381.34
32 0	481.510	378.00	480.52	441.94	389.50	385.21
33 0	486.301	382.50	486.40	447.82	392.80	389.04
34 0	491.040	387.00	492.14	453.63	396.00	392.81
35 0	495.736	391.40	497.83	459.34	399.10	396.53
36 0	500.380	395.70	503.38	464.98	402.10	400.22
37 0	505.012	400.00	508.80	470.53	405.00	403.87
38 0	509.602	404.20	514.10	476.00	407.80	407.47
39 0	514.142	408.30	519.30	481.40	410.60	411.02
40 0	518.632	412.40	524.40	486.70	413.30	414.53
41 0	523.072	416.40	529.40	491.90	415.90	418.00
42 0	527.462	420.40	534.30	497.00	418.50	421.43
43 0	531.802	424.30	539.10	502.00	421.00	424.82
44 0	536.092	428.20	543.80	506.90	423.40	428.17
45 0	540.332	432.00	548.40	511.70	425.70	431.48
46 0	544.522	435.80	552.90	516.40	427.90	434.75
47 0	548.662	439.50	557.30	521.00	430.10	437.98
48 0	552.752	443.20	561.60	525.50	432.20	441.17
49 0	556.792	446.80	565.80	530.00	434.30	444.32
50 0	560.882	450.40	570.00	534.40	436.30	447.43
51 0	564.922	454.00	574.10	538.70	438.30	450.50
52 0	568.912	457.50	578.10	543.00	440.30	453.53
53 0	572.852	461.00	582.10	547.20	442.20	456.53
54 0	576.742	464.50	586.00	551.40	444.10	459.50
55 0	580.582	468.00	590.00	555.50	445.90	462.43
56 0	584.372	471.40	593.90	559.50	447.70	465.32
57 0	588.112	474.80	597.80	563.40	449.50	468.17
58 0	591.802	478.20	601.60	567.30	451.30	471.00
59 0	595.442	481.50	605.40	571.10	453.00	473.79
60 0	599.032	484.90	609.10	574.90	454.70	476.53

TABLE II.—For finding the Velocities from the Altitudes, and the Altitudes from the Velocities.

Altitudes 17 feet 0 inches to 40 feet.

Coefficients of velocity, and the corresponding velocities of discharge in inches per second.						Altitudes h in feet and inches.	
7. Values of $v = 19.45 \sqrt{h}$ Coefficient .760.	8. Values of $v = 19.515 \sqrt{h}$ Coefficient .699.	9. Values of $v = 17.455 \sqrt{h}$ Coefficient .628.	10. Values of $v = 17.153 \sqrt{h}$ Coefficient .617.	11. Values of $v = 16.247 \sqrt{h}$ Coefficient .606.	12. Values of $v = 15.935 \sqrt{h}$ Coefficient .584.		
282.00	268.30	253.00	248.50	244.13	235.27	17	0
286.00	272.11	256.50	252.00	247.00	238.61	18	0
289.95	275.80	260.12	255.57	251.01	241.90	18	6
293.84	279.57	263.32	259.00	254.38	245.14	19	0
297.08	283.22	267.06	262.38	257.71	248.35	19	6
301.47	286.83	270.40	265.73	260.99	251.51	20	0
305.22	290.30	273.82	269.03	264.23	254.64	20	6
308.92	293.91	277.08	272.23	267.37	257.67	21	0
312.67	297.30	280.42	275.51	270.60	260.78	21	6
316.19	300.83	283.07	278.70	273.73	263.70	22	0
319.76	304.23	286.87	281.85	276.82	266.77	22	6
323.29	307.60	290.04	284.96	279.88	269.72	23	0
326.70	310.92	293.18	288.04	282.91	272.64	23	6
330.25	314.21	296.28	291.09	285.90	275.52	24	0
333.67	317.46	299.35	294.11	288.86	278.38	24	6
337.03	320.69	302.30	297.09	291.80	281.20	25	0
340.41	323.88	305.40	300.05	294.70	284.00	25	6
343.73	327.04	308.38	302.98	297.57	286.77	26	0
347.02	330.17	311.33	305.87	300.42	289.52	26	6
350.28	333.13	314.25	308.75	303.24	292.23	27	0
353.51	336.34	317.15	311.59	306.04	294.93	27	6
356.71	339.38	320.02	314.41	308.81	297.60	28	0
359.88	342.40	322.86	317.20	311.55	300.24	28	6
363.02	345.39	325.68	319.98	314.27	302.86	29	0
366.14	348.35	328.48	322.72	316.97	305.46	29	6
369.23	351.29	331.25	325.45	319.65	308.04	30	0
372.29	354.21	334.00	328.15	322.30	310.60	30	6
375.33	357.10	336.73	330.83	324.93	313.13	31	0
378.35	359.97	339.43	333.48	327.54	315.60	31	6
381.34	362.81	342.11	336.12	330.13	318.14	32	0
384.30	365.64	344.78	338.74	332.70	320.62	32	6
387.25	368.44	347.42	341.33	335.25	323.08	33	0
390.17	371.22	350.04	343.91	337.78	325.51	33	6
393.07	373.98	352.64	346.47	340.29	327.93	34	0
395.95	376.72	355.23	349.00	342.78	330.34	34	6
398.81	379.44	357.79	351.52	345.26	332.72	35	0
401.47	381.82	360.27	354.01	347.73	335.08	35	6
404.05	384.13	362.87	356.43	350.18	337.44	36	0
406.55	386.37	365.41	358.88	352.61	339.78	36	6
409.08	388.64	367.98	361.31	355.03	342.10	37	0
411.55	390.84	370.51	363.73	357.44	344.41	37	6
414.05	393.07	373.01	366.13	359.83	346.70	38	0
416.55	395.23	375.48	368.51	362.21	348.98	38	6
419.08	397.44	377.93	370.88	364.58	351.25	39	0
421.55	399.64	380.35	373.23	366.93	353.51	39	6
424.05	401.84	382.75	375.57	369.27	355.76	40	0

TABLE III.—*Square Roots for finding the effects of the Velocity, Approach when the Orifice is small in proportion to the Head. Also for finding the Increase in the Discharge from an Increase of Head. (See p. 101.)*

No.	Square root.	No.	Square root.	No.	Square root.	No.	Square root.
1.000	1.0000	1.115	1.0559	1.475	1.2141	1.975	1.4059
1.001	1.0005	1.120	1.0583	1.480	1.2207	1.980	1.4107
1.002	1.0010	1.125	1.0607	1.485	1.2247	2.000	1.4142
1.004	1.0020	1.130	1.0630	1.490	1.2288	2.010	1.4177
1.005	1.0025	1.135	1.0654	1.495	1.2349	2.025	1.4230
1.006	1.0030	1.140	1.0677	1.500	1.2410	2.040	1.4283
1.008	1.0040	1.145	1.0700	1.505	1.2450	2.050	1.4318
1.009	1.0044	1.150	1.0723	1.510	1.2490	2.060	1.4353
1.010	1.0050	1.155	1.0747	1.515	1.2550	2.075	1.4405
1.011	1.0055	1.160	1.0770	1.520	1.2570	2.080	1.4457
1.012	1.0060	1.165	1.0794	1.525	1.2610	2.100	1.4491
1.014	1.0070	1.170	1.0817	1.530	1.2640	2.110	1.4526
1.015	1.0075	1.175	1.0840	1.535	1.2680	2.125	1.4577
1.016	1.0080	1.180	1.0863	1.540	1.2748	2.140	1.4629
1.018	1.0090	1.185	1.0886	1.545	1.2800	2.150	1.4663
1.019	1.0095	1.190	1.0909	1.550	1.2845	2.160	1.4697
1.020	1.0100	1.195	1.0932	1.555	1.2884	2.175	1.4748
1.0225	1.0112	1.200	1.0954	1.560	1.2942	2.190	1.4799
1.025	1.0124	1.210	1.1000	1.565	1.3000	2.200	1.4832
1.0275	1.0137	1.220	1.1045	1.570	1.3038	2.210	1.4866
1.030	1.0149	1.230	1.1091	1.575	1.3077	2.225	1.4916
1.0325	1.0161	1.240	1.1136	1.580	1.3134	2.240	1.4967
1.035	1.0174	1.250	1.1180	1.585	1.3191	2.250	1.5000
1.0375	1.0186	1.260	1.1225	1.590	1.3220	2.260	1.5033
1.040	1.0198	1.270	1.1269	1.595	1.3267	2.275	1.5083
1.0425	1.0210	1.280	1.1314	1.600	1.3323	2.290	1.5133
1.045	1.0223	1.290	1.1358	1.605	1.3370	2.300	1.5166
1.0475	1.0235	1.300	1.1402	1.610	1.3416	2.310	1.5199
1.050	1.0247	1.310	1.1446	1.615	1.3454	2.325	1.5248
1.055	1.0271	1.325	1.1511	1.620	1.3500	2.340	1.5297
1.060	1.0296	1.340	1.1576	1.625	1.3546	2.350	1.5330
1.065	1.0320	1.350	1.1610	1.630	1.3591	2.360	1.5362
1.070	1.0344	1.360	1.1662	1.635	1.3638	2.375	1.5411
1.075	1.0368	1.375	1.1726	1.640	1.3683	2.390	1.5460
1.080	1.0392	1.380	1.1780	1.645	1.3748	2.400	1.5492
1.085	1.0416	1.390	1.1832	1.650	1.3784	2.410	1.5524
1.090	1.0440	1.400	1.1874	1.655	1.3820	2.425	1.5572
1.095	1.0464	1.425	1.1937	1.660	1.3875	2.440	1.5621
1.100	1.0488	1.440	1.2000	1.665	1.3928	2.450	1.5652
1.105	1.0512	1.450	1.2042	1.670	1.3964	2.460	1.5684
1.110	1.0536	1.460	1.2083	1.675	1.4000	2.475	1.5732

TABLE III.—Square Roots for finding the effects of the Velocity of Approach when the Orifice is small in proportion to the Head. Also for finding the Increase in the Discharge from an Increase of Head. (See p. 101.)

No.	Square root.	No.	Square root.	No.	Square root.	No.	Square root.
2.40	1.5780	3.0000	1.7321	4.5	2.1213	26	5.0990
2.5	1.5811	3.025	1.7393	5.0	2.2301	27	5.1962
2.51	1.5843	3.05	1.7464	5.5	2.3452	28	5.2915
2.525	1.5890	3.075	1.7536	6.0	2.4495	29	5.3852
2.54	1.5937	3.1	1.7607	6.5	2.5495	30	5.4772
2.55	1.5980	3.125	1.7678	7.0	2.6458	31	5.5678
2.56	1.6000	3.15	1.7748	7.5	2.7386	32	5.6569
2.575	1.6047	3.175	1.7819	8.0	2.8284	33	5.7446
2.59	1.6093	3.2	1.7889	8.5	2.9155	34	5.8310
2.6	1.6125	3.225	1.7958	9.0	3.0000	35	5.9161
2.61	1.6155	3.25	1.8028	9.5	3.0822	36	6.0000
2.625	1.6202	3.275	1.8097	10.0	3.1623	37	6.0828
2.64	1.6248	3.3	1.8166	10.5	3.2404	38	6.1644
2.65	1.6279	3.325	1.8235	11.0	3.3166	39	6.2450
2.66	1.6310	3.35	1.8303	11.5	3.3912	40	6.3246
2.675	1.6355	3.375	1.8371	12.0	3.4641	41	6.4031
2.69	1.6401	3.4	1.8439	12.5	3.5355	42	6.4807
2.7	1.6432	3.425	1.8507	13.0	3.6056	43	6.5574
2.71	1.6462	3.45	1.8574	13.5	3.6742	44	6.6332
2.725	1.6508	3.475	1.8641	14.0	3.7417	45	6.7082
2.74	1.6553	3.5	1.8708	14.5	3.8079	46	6.7823
2.75	1.6583	3.525	1.8775	15.0	3.8730	47	6.8557
2.76	1.6613	3.55	1.8841	15.5	3.9370	48	6.9282
2.775	1.6658	3.575	1.8908	16.0	4.0000	49	7.0000
2.79	1.6703	3.6	1.8974	16.5	4.0620	50	7.0711
2.8	1.6733	3.625	1.9039	17.0	4.1231	51	7.1414
2.81	1.6763	3.65	1.9105	17.5	4.1833	52	7.2111
2.825	1.6808	3.675	1.9170	18.0	4.2426	53	7.2810
2.84	1.6852	3.7	1.9235	18.5	4.3012	54	7.3485
2.85	1.6882	3.725	1.9300	19.0	4.3589	55	7.4162
2.86	1.6912	3.75	1.9365	19.5	4.4159	56	7.4833
2.875	1.6956	3.775	1.9429	20.0	4.4721	57	7.5498
2.89	1.7000	3.8	1.9494	20.5	4.5277	58	7.6158
2.9	1.7029	3.825	1.9558	21.0	4.5826	59	7.6811
2.91	1.7059	3.85	1.9621	21.5	4.6368	60	7.7460
2.925	1.7103	3.875	1.9685	22.0	4.6904	61	7.8102
2.94	1.7146	3.9	1.9748	22.5	4.7434	62	7.8740
2.95	1.7176	3.925	1.9812	23.0	4.7958	63	7.9373
2.96	1.7205	3.95	1.9875	23.5	4.8477	64	8.0000
2.975	1.7248	3.975	1.9938	24.0	4.8990	65	8.0623
2.99	1.7292	4.0	2.0000	25.0	5.0000	66	8.1240

TABLE IV.—For finding the Discharge through Rectangular Orifices; in which $n = \frac{h}{d}$. Also for finding the effects of the Velocity of Approach to Weirs, and the Depression on the Crest, (See p. 101.)

$1+n$	n^3	$(1+n)^3$	$(1+n)^3 - n^3$	$1+n$	n^3	$(1+n)^3$	$(1+n)^3 - n^3$
1.000	.0000	1.0000	1.0000	1.115	.0030	1.4774	1.4884
1.001	.0000	1.0015	1.0015	1.120	.0040	1.4853	1.4937
1.002	.0001	1.0030	1.0029	1.125	.0042	1.4932	1.4991
1.004	.0003	1.0060	1.0057	1.13	.0060	1.5012	1.5043
1.005	.0004	1.0075	1.0071	1.135	.0066	1.5092	1.5106
1.006	.0005	1.0090	1.0085	1.14	.0084	1.5172	1.5186
1.008	.0007	1.0120	1.0113	1.145	.0092	1.5251	1.5265
1.009	.0009	1.0135	1.0127	1.15	.0100	1.5332	1.5346
1.010	.0010	1.0150	1.0140	1.155	.0110	1.5413	1.5427
1.011	.0012	1.0165	1.0154	1.16	.0120	1.5494	1.5508
1.012	.0013	1.0181	1.0169	1.165	.0130	1.5574	1.5588
1.014	.0017	1.0211	1.0197	1.17	.0140	1.5655	1.5669
1.015	.0018	1.0226	1.0207	1.175	.0150	1.5737	1.5750
1.016	.0020	1.0241	1.0221	1.18	.0160	1.5818	1.5832
1.018	.0024	1.0271	1.0247	1.185	.0170	1.5900	1.5914
1.019	.0026	1.0286	1.0260	1.19	.0180	1.5981	1.5995
1.020	.0028	1.0301	1.0273	1.195	.0190	1.6063	1.6077
1.0225	.0034	1.0339	1.0306	1.2	.0200	1.6145	1.6159
1.025	.0040	1.0377	1.0338	1.21	.0210	1.6227	1.6241
1.0275	.0046	1.0415	1.0370	1.22	.0220	1.6309	1.6323
1.03	.0052	1.0453	1.0401	1.23	.0230	1.6391	1.6405
1.0325	.0059	1.0491	1.0433	1.24	.0240	1.6473	1.6487
1.035	.0066	1.0530	1.0464	1.25	.0250	1.6555	1.6569
1.0375	.0073	1.0568	1.0495	1.26	.0260	1.6637	1.6651
1.04	.0080	1.0606	1.0526	1.27	.0270	1.6719	1.6733
1.0425	.0088	1.0644	1.0557	1.28	.0280	1.6801	1.6815
1.045	.0095	1.0683	1.0587	1.29	.0290	1.6883	1.6897
1.0475	.0104	1.0721	1.0617	1.30	.0300	1.6965	1.6979
1.05	.0112	1.0759	1.0648	1.31	.0310	1.7047	1.7061
1.055	.0129	1.0836	1.0707	1.325	.0343	1.7252	1.7266
1.06	.0137	1.0873	1.0736	1.34	.0383	1.7512	1.7526
1.065	.0160	1.0961	1.0825	1.35	.0407	1.7600	1.7614
1.07	.0165	1.1008	1.0883	1.36	.0430	1.7688	1.7702
1.075	.0205	1.1146	1.0940	1.375	.0463	1.7823	1.7837
1.08	.0220	1.1224	1.0997	1.39	.0496	1.7958	1.7972
1.085	.0248	1.1302	1.1051	1.40	.0520	1.8046	1.8060
1.09	.0270	1.1380	1.1110	1.41	.0544	1.8134	1.8148
1.095	.0293	1.1458	1.1168	1.425	.0577	1.8271	1.8285
1.1	.0310	1.1537	1.1221	1.44	.0610	1.8408	1.8422
1.105	.0340	1.1616	1.1275	1.45	.0634	1.8496	1.8510
1.110	.0365	1.1695	1.1330	1.46	.0658	1.8584	1.8598

Values of n from 0 to .40.

[Continued on next page.]

TABLE IV.—For finding the Discharge through Rectangular Orifices; in which $n = \frac{h}{d}$. Also for finding the effects of the Velocity of Approach to Weirs, &c. (See p. 101.)

$1+n$	n^2	$(1+n)^2$	$(1+n)^2 - n^2$	$1+n$	n^2	$(1+n)^2$	$(1+n)^2 - n^2$
1.475	.3274	1.7914	1.4640	1.975	.9627	2.7756	1.8128
1.49	.3430	1.8188	1.4758	1.99	.9850	2.8072	1.8222
1.5	.3536	1.8371	1.4836	2.	1.0000	2.8284	1.8284
1.51	.3642	1.8555	1.4913	2.01	1.0150	2.8497	1.8346
1.525	.3804	1.8832	1.5028	2.025	1.0377	2.8816	1.8439
1.54	.3968	1.9111	1.5143	2.04	1.0606	2.9137	1.8531
1.55	.4079	1.9297	1.5218	2.05	1.0759	2.9352	1.8592
1.56	.4191	1.9484	1.5294	2.06	1.0913	2.9567	1.8653
1.575	.4360	1.9766	1.5406	2.075	1.1146	2.9890	1.8744
1.58	.4417	1.9860	1.5443	2.09	1.1380	3.0215	1.8835
1.59	.4532	2.0049	1.5517	2.10	1.1537	3.0432	1.8895
1.6	.4648	2.0239	1.5591	2.11	1.1695	3.0650	1.8955
1.61	.4764	2.0429	1.5664	2.125	1.1932	3.0977	1.9046
1.625	.4941	2.0715	1.5774	2.14	1.2172	3.1306	1.9134
1.64	.5120	2.1002	1.5882	2.15	1.2332	3.1525	1.9193
1.65	.5240	2.1195	1.5954	2.16	1.2494	3.1745	1.9252
1.66	.5362	2.1388	1.6026	2.175	1.2737	3.2077	1.9340
1.675	.5546	2.1678	1.6132	2.19	1.2981	3.2409	1.9428
1.69	.5732	2.1970	1.6238	2.2	1.3145	3.2631	1.9486
1.7	.5857	2.2165	1.6309	2.21	1.3310	3.2854	1.9544
1.71	.5983	2.2361	1.6379	2.225	1.3558	3.3189	1.9631
1.725	.6173	2.2656	1.6483	2.24	1.3808	3.3525	1.9717
1.74	.6368	2.2952	1.6586	2.25	1.3975	3.3760	1.9775
1.75	.6495	2.3150	1.6655	2.26	1.4143	3.3975	1.9832
1.76	.6626	2.3349	1.6724	2.275	1.4397	3.4314	1.9917
1.775	.6823	2.3648	1.6826	2.29	1.4652	3.4654	2.0002
1.79	.7022	2.3949	1.6927	2.3	1.4822	3.4881	2.0059
1.80	.7155	2.4150	1.6994	2.31	1.4994	3.5109	2.0115
1.81	.7290	2.4351	1.7061	2.325	1.5252	3.5451	2.0200
1.825	.7493	2.4654	1.7161	2.34	1.5512	3.5795	2.0284
1.84	.7699	2.4959	1.7260	2.35	1.5686	3.6025	2.0339
1.85	.7837	2.5163	1.7326	2.36	1.5860	3.6255	2.0395
1.86	.7975	2.5367	1.7392	2.375	1.6123	3.6601	2.0478
1.875	.8185	2.5674	1.7490	2.39	1.6388	3.6948	2.0561
1.89	.8396	2.5983	1.7587	2.4	1.6565	3.7181	2.0616
1.9	.8538	2.6190	1.7652	2.41	1.6743	3.7413	2.0670
1.91	.8681	2.6397	1.7716	2.425	1.7011	3.7763	2.0752
1.925	.8896	2.6709	1.7813	2.44	1.7280	3.8114	2.0834
1.94	.9114	2.7021	1.7907	2.45	1.7460	3.8349	2.0888
1.95	.9259	2.7230	1.7971	2.46	1.7641	3.8584	2.0942
1.96	.9406	2.7440	1.8034	2.475	1.7914	3.8937	2.1023

Values of n from .475 to 1.475.

[Continued on next page.]

TABLE IV.—For finding the Discharge through Rectangular Orifices; in which $n = \frac{h}{d}$. Also for finding the effects of the Velocity of Approach to Weirs, &c. (See p. 101.)

$1+n$.	n^2 .	$(1+n)^2$.	$(1+n)^2 - n^2$.	$1+n$.	n^2 .	$(1+n)^2$.	$(1+n)^2 - n^2$.
2.40	1.8188	3.0202	2.1104	3.	2.8284	5.1062	2.3077
2.5	1.8371	3.0528	2.1157	3.025	2.8810	5.2012	2.3700
2.61	1.8555	3.0700	2.1211	3.05	2.9352	5.3020	2.3914
2.625	1.8832	4.0123	2.1201	3.075	2.9800	5.3022	2.4032
2.64	1.9111	4.0481	2.1370	3.1	3.0432	5.4581	2.4149
2.65	1.9207	4.0720	2.1423	3.125	3.0977	5.5249	2.4200
2.66	1.9484	4.0000	2.1470	3.15	3.1525	5.5907	2.4382
2.675	1.9700	4.1321	2.1554	3.175	3.2077	5.6574	2.4497
2.69	2.0040	4.1082	2.1633	3.2	3.2631	5.7243	2.4612
2.7	2.0230	4.1024	2.1685	3.225	3.3189	5.7915	2.4726
2.71	2.0420	4.2100	2.1737	3.25	3.3750	5.8590	2.4840
2.725	2.0716	4.2530	2.1815	3.275	3.4314	5.9268	2.4953
2.74	2.1002	4.2805	2.1893	3.3	3.4881	5.9947	2.5066
2.75	2.1195	4.3130	2.1944	3.325	3.5451	6.0630	2.5179
2.76	2.1388	4.3383	2.1996	3.35	3.6025	6.1315	2.5290
2.775	2.1678	4.3751	2.2073	3.375	3.6601	6.2003	2.5401
2.79	2.1970	4.4110	2.2140	3.4	3.7181	6.2693	2.5512
2.8	2.2165	4.4300	2.2200	3.425	3.7763	6.3380	2.5623
2.81	2.2361	4.4612	2.2251	3.45	3.8349	6.4081	2.5732
2.825	2.2550	4.4883	2.2327	3.475	3.8937	6.4779	2.5842
2.84	2.2742	4.5355	2.2403	3.5	3.9528	6.5470	2.5951
2.85	2.2936	4.5604	2.2453	3.525	4.0123	6.6182	2.6059
2.86	2.3130	4.5853	2.2504	3.55	4.0720	6.6887	2.6167
2.875	2.3340	4.6227	2.2570	3.575	4.1321	6.7605	2.6274
2.89	2.3540	4.6602	2.2654	3.6	4.1924	6.8305	2.6381
2.9	2.3740	4.6853	2.2703	3.625	4.2530	6.9018	2.6488
2.91	2.3951	4.7104	2.2753	3.65	4.3130	6.9733	2.6594
2.925	2.4054	4.7482	2.2827	3.675	4.3751	7.0451	2.6700
2.94	2.4250	4.7861	2.2902	3.7	4.4300	7.1171	2.6805
2.95	2.4450	4.8114	2.2951	3.725	4.4883	7.1899	2.6910
2.96	2.4650	4.8367	2.3000	3.75	4.5504	7.2618	2.7015
2.975	2.4854	4.8748	2.3074	3.775	4.6227	7.3346	2.7119
2.99	2.5083	4.9130	2.3147	3.8	4.6853	7.4070	2.7223
3.	2.5190	4.9386	2.3196	3.825	4.7482	7.4808	2.7326
3.01	2.5307	4.9641	2.3244	3.85	4.8114	7.5542	2.7429
3.025	2.5408	5.0026	2.3317	3.875	4.8748	7.6270	2.7531
3.04	2.5521	5.0411	2.3389	3.9	4.9386	7.7010	2.7634
3.05	2.5630	5.0668	2.3438	3.925	5.0026	7.7761	2.7735
3.06	2.5740	5.0926	2.3480	3.95	5.0668	7.8505	2.7837
3.075	2.5775	5.1313	2.3558	3.975	5.1313	7.9251	2.7938
3.09	2.5872	5.1702	2.3630	4.	5.1962	8.	2.8038

Values of n from .475 to 1.475.

[Continued on next page.]

For finding the Discharge through Rectangular Orifices in which $n = \frac{h}{d}$. Also for finding the effects of the g of Approach to Weirs, &c. (See p. 101.)

	$(1+n)^2$	$(1+n)^2 - n^2$	$1+n$	n^2	$(1+n)^2$	$(1+n)^2 - n^2$
70	0.5450	2.0080	20	125.0000	132.5745	7.5745
60	11.1803	3.1803	27	132.5745	140.2961	7.7216
50	12.8086	3.3527	28	140.2961	148.1621	7.8660
40	14.0000	3.5166	29	148.1621	156.1698	8.0077
30	16.5718	3.6732	30	156.1698	164.3168	8.1470
20	18.5203	3.8234	31	164.3168	172.6007	8.2839
18	20.5300	3.9678	32	172.6007	181.0193	8.4186
13	22.6274	4.1071	33	181.0193	189.5700	8.5513
10	24.7815	4.2410	34	189.5700	198.2524	8.6818
7.4	27.0000	4.3720	35	198.2524	207.0628	8.8104
15	29.2810	4.4995	36	207.0628	216.0000	8.9372
10	31.6228	4.6228	37	216.0000	225.0622	9.0622
10	34.0239	4.7420	38	225.0622	234.2477	9.1855
28	36.4829	4.8601	39	234.2477	243.5549	9.3072
30	38.9084	4.9745	40	243.5549	252.9822	9.4273
20	41.5002	5.0863	41	252.9822	262.5281	9.5459
44	44.1042	5.1958	42	262.5281	272.1911	9.6630
12	46.8722	5.3030	43	272.1911	281.9699	9.7788
12	49.6022	5.4080	44	281.9699	291.8630	9.8931
22	52.3832	5.5110	45	291.8630	301.8692	10.0062
22	55.2144	5.6122	46	301.8692	311.9872	10.1180
12	58.0047	5.7116	47	311.9872	322.2168	10.2286
14	61.0236	5.8092	48	322.2168	332.5538	10.3380
17	64	5.9053	49	332.5538	343.0000	10.4462
10	67.0247	6.0011	50	343.0000	353.5534	10.5534
	70.0028	6.0928	51	353.5534	364.2128	10.6594
17	73.2078	6.1831	52	364.2128	374.9773	10.7645
24	76.3075	6.2747	53	374.9773	385.8458	10.8685
28	79.5715	6.3637	54	385.8458	396.8173	10.9715
25	82.8101	6.4510	55	396.8173	407.8900	11.0736
5	86.1007	6.5382	56	407.8900	419.0650	11.1747
11	89.4427	6.6236	57	419.0650	430.3406	11.2750
12	92.8177	6.7080	58	430.3406	441.7148	11.3742
17	96.2341	6.7914	59	441.7148	453.1876	11.4728
7	99.6914	6.8737	60	453.1876	464.7580	11.5704
1	103.1802	6.9551	61	464.7580	476.4252	11.6672
4	106.7200	7.0355	62	476.4252	488.1885	11.7633
2	110.3041	7.1149	63	488.1885	500.0470	11.8585
10	113.9205	7.1933	64	500.0470	512.0000	11.9530
1	117.5755	7.2714	65	512.0000	524.0468	12.0468
5	125	7.4245	66	524.0468	536.1865	12.1397

Values of n from 3.5 to 66.

TABLE V.—Coefficients of Discharge for Different Ratios of the Channel to the Orifice.

Coefficients for Heads in still water .550 and .573.

Ratio of the channel to the orifice.	Coefficient .550 for heads in still water.			Coefficient .573 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centre.	Coefficients for weirs; the heads measured to the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centre.	Coefficients for weirs; the heads measured to the full depth.
30	.000	.550	.550	.000	.573	.573
20	.001	.550	.551	.001	.573	.574
16	.001	.550	.551	.001	.573	.574
10	.003	.551	.552	.003	.574	.575
9	.004	.551	.553	.004	.574	.576
8	.005	.551	.553	.005	.574	.577
7	.006	.552	.555	.007	.575	.578
6	.008	.552	.557	.009	.576	.580
5.5	.010	.553	.558	.011	.576	.582
5.0	.012	.553	.559	.013	.577	.584
4.5	.015	.554	.562	.016	.578	.586
4.0	.019	.555	.565	.021	.579	.589
3.75	.022	.556	.566	.024	.580	.592
3.50	.025	.557	.569	.028	.581	.594
3.25	.029	.558	.572	.032	.582	.598
3.0	.036	.559	.575	.038	.584	.602
2.75	.042	.561	.580	.045	.586	.607
2.50	.051	.564	.586	.053	.589	.614
2.25	.061	.567	.594	.069	.593	.623
2.00	.082	.572	.600	.089	.598	.636
1.95	.086	.573	.600	.094	.599	.639
1.90	.091	.575	.602	.100	.601	.643
1.85	.097	.576	.603	.106	.603	.647
1.80	.103	.578	.605	.113	.604	.651
1.75	.110	.579	.607	.120	.606	.655
1.70	.117	.581	.609	.128	.608	.660
1.65	.125	.583	.612	.137	.611	.666
1.60	.134	.586	.617	.147	.614	.671
1.55	.144	.588	.623	.158	.617	.678
1.50	.155	.591	.630	.171	.620	.685
1.45	.168	.594	.638	.185	.624	.694
1.40	.183	.598	.647	.201	.628	.703
1.35	.199	.602	.657	.220	.633	.713
1.30	.218	.607	.668	.241	.638	.724
1.25	.240	.612	.680	.266	.645	.737
1.20	.265	.619	.693	.295	.652	.753
1.15	.297	.626	.707	.330	.661	.770
1.10	.333	.635	.721	.372	.671	.791
1.05	.378	.646	.736	.424	.684	.816
1.00	.434	.659	.757	.480	.699	.845

See the auxiliary table, p. 130.

TABLE V.—Coefficients of Discharge for different Ratios of the Channel to the Orifice.

Coefficients for heads in still water .584 and .595.

Ratio of the channel to the orifice.	Coefficient .584 for heads in still water.			Coefficient .595 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centres.	Coefficients for weirs: the heads measured the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centres.	Coefficients for weirs: the heads measured the full depth.
30.	.000	.584	.584	.000	.595	.595
20.	.001	.584	.585	.001	.595	.596
15.	.002	.584	.585	.002	.595	.596
10.	.003	.585	.587	.004	.596	.598
9.0	.004	.585	.588	.004	.596	.599
1.0	.005	.586	.588	.006	.597	.600
7.0	.007	.586	.590	.007	.597	.601
6.0	.010	.587	.592	.010	.598	.603
5.5	.011	.587	.593	.012	.599	.605
5.0	.014	.588	.595	.014	.599	.607
4.5	.017	.589	.598	.018	.600	.610
4.0	.022	.590	.601	.023	.602	.613
3.75	.025	.591	.604	.026	.603	.616
3.50	.029	.592	.606	.030	.604	.619
3.25	.033	.594	.610	.035	.605	.622
3.0	.039	.595	.614	.041	.607	.627
2.75	.047	.598	.620	.049	.609	.633
2.50	.058	.601	.627	.060	.613	.641
2.25	.072	.605	.637	.075	.617	.651
2.0	.093	.611	.651	.097	.623	.666
1.95	.099	.612	.654	.103	.625	.669
1.90	.104	.614	.656	.109	.627	.673
1.85	.111	.615	.659	.115	.628	.678
1.80	.118	.617	.663	.123	.630	.682
1.75	.125	.620	.671	.131	.633	.687
1.70	.134	.622	.676	.140	.635	.693
1.65	.143	.624	.682	.149	.638	.699
1.60	.154	.627	.689	.160	.641	.706
1.55	.166	.631	.696	.173	.644	.713
1.50	.179	.634	.703	.187	.648	.721
1.45	.194	.638	.712	.202	.652	.730
1.40	.211	.643	.722	.220	.657	.741
1.35	.230	.648	.732	.241	.663	.752
1.30	.253	.654	.745	.265	.669	.765
1.25	.279	.661	.759	.293	.677	.780
1.20	.310	.669	.775	.325	.685	.797
1.15	.348	.678	.794	.366	.695	.818
1.10	.393	.689	.816	.414	.707	.842
1.05	.448	.703	.842	.473	.722	.870
1.00	.518	.719	.874	.548	.740	.905

See the auxiliary table, p. 136.

TABLE V.—*Coefficients of Discharge for different Ratios of the Channel to the Orifice.*

Coefficients for heads in still water .005 and .017.

Ratio of the channel to the orifice.	Coefficient .005 for heads in still water.			Coefficient .017 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centre.	Coefficients for weirs: the heads measured to the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centre.	Coefficients for weirs: the heads measured to the full depth.
30	.000	.000	.000	.000	.017	.017
20	.001	.000	.007	.001	.017	.018
15	.002	.007	.007	.002	.018	.019
10	.004	.007	.009	.004	.018	.020
9.0	.005	.007	.010	.005	.018	.021
8.0	.006	.008	.011	.006	.019	.022
7.0	.008	.008	.012	.008	.019	.024
6.0	.010	.009	.015	.011	.020	.026
5.5	.012	.010	.016	.013	.021	.028
5.0	.015	.011	.019	.016	.022	.030
4.5	.018	.012	.021	.019	.023	.033
4.0	.023	.013	.025	.024	.024	.037
3.75	.027	.014	.028	.028	.026	.040
3.50	.031	.015	.031	.032	.027	.043
3.25	.036	.017	.035	.037	.028	.047
3.00	.043	.019	.040	.044	.030	.053
2.75	.051	.021	.046	.053	.033	.060
2.50	.062	.025	.054	.065	.037	.068
2.25	.078	.029	.065	.084	.042	.079
2.00	.101	.036	.084	.105	.049	.096
1.95	.107	.038	.086	.111	.050	.100
1.90	.113	.040	.090	.118	.052	.104
1.85	.119	.041	.093	.125	.054	.109
1.80	.128	.044	.098	.133	.057	.114
1.75	.136	.046	.103	.142	.060	.120
1.70	.146	.049	.109	.152	.062	.126
1.65	.156	.052	.116	.163	.065	.133
1.60	.167	.055	.123	.175	.069	.141
1.55	.180	.058	.131	.188	.073	.149
1.50	.195	.062	.139	.204	.077	.159
1.45	.212	.067	.149	.221	.081	.168
1.40	.231	.072	.160	.241	.087	.180
1.35	.252	.078	.172	.264	.094	.193
1.30	.278	.085	.186	.291	.101	.208
1.25	.307	.093	.203	.322	.109	.225
1.20	.342	.102	.221	.359	.119	.245
1.15	.384	.113	.243	.404	.131	.268
1.10	.430	.126	.268	.459	.145	.295
1.05	.490	.142	.298	.527	.163	.328
1.00	.580	.162	.336	.615	.184	.369

See the auxiliary table, p. 130.

TABLE V.—Coefficients of Discharge for different Ratios of the Channel to the Orifice.

Mean Coefficient .628.

Coefficients for heads in still water .628 and .630.

Ratio of the channel to the orifice.	Coefficient .628 for heads in still water.			Coefficient .630 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centres.	Coefficients for weirs; the heads measured the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centres.	Coefficients for weirs; the heads measured the full depth.
30.	.000	.628	.628	.000	.630	.630
20.	.001	.628	.629	.001	.630	.640
15.	.002	.629	.630	.002	.640	.641
10.	.004	.629	.632	.004	.640	.643
9.0	.005	.630	.632	.005	.641	.644
8.0	.006	.630	.634	.006	.641	.645
7.0	.008	.631	.635	.008	.642	.647
6.0	.011	.631	.638	.011	.643	.649
5.5	.013	.632	.640	.014	.643	.651
5.0	.016	.633	.642	.017	.644	.654
4.5	.020	.634	.645	.021	.646	.657
4.0	.025	.636	.649	.026	.647	.662
3.75	.029	.637	.652	.030	.648	.665
3.50	.033	.638	.656	.034	.650	.668
3.25	.039	.639	.659	.040	.652	.673
3.0	.046	.642	.666	.048	.654	.678
2.75	.055	.645	.672	.057	.657	.686
2.50	.067	.649	.682	.070	.661	.695
2.25	.084	.654	.694	.088	.666	.708
2.0	.109	.661	.711	.114	.674	.727
1.95	.116	.663	.715	.120	.676	.731
1.90	.123	.665	.720	.128	.679	.736
1.85	.130	.668	.725	.135	.681	.741
1.80	.139	.670	.731	.144	.684	.747
1.75	.148	.673	.737	.154	.686	.753
1.70	.158	.676	.743	.165	.690	.760
1.65	.169	.679	.750	.176	.693	.768
1.60	.182	.683	.758	.190	.697	.776
1.55	.196	.687	.767	.205	.701	.786
1.50	.213	.692	.777	.222	.706	.796
1.45	.231	.697	.788	.241	.712	.808
1.40	.252	.703	.800	.262	.718	.820
1.35	.276	.709	.814	.289	.725	.836
1.30	.304	.717	.830	.319	.734	.853
1.25	.338	.726	.846	.354	.743	.872
1.20	.377	.734	.866	.396	.755	.895
1.15	.425	.750	.894	.447	.769	.921
1.10	.484	.765	.924	.509	.785	.953
1.05	.557	.784	.959	.588	.805	.991
1.00	.651	.807	1.002	.690	.831	1.038

See the auxiliary table, p. 136.

TABLE V.—Coefficients of Discharge for Different Ratios of the Channel to the Orifice.

Coefficients for heads in still water .050 and .067.

Ratio of the channel to the orifice.	Coefficient .050 for heads in still water.			Coefficient .067 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centres.	Coefficients for weirs; the heads measured to the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices; the heads measured to the centres.	Coefficients for weirs; the heads measured to the full depth.
30	.000	.050	.050	.000	.067	.067
20	.001	.050	.051	.001	.067	.068
15	.002	.051	.052	.002	.067	.069
10	.004	.051	.054	.004	.068	.071
8	.005	.052	.055	.006	.069	.072
7	.007	.052	.056	.007	.069	.073
6	.009	.053	.058	.009	.070	.075
5	.012	.054	.061	.012	.071	.078
4	.014	.055	.063	.015	.072	.080
3	.017	.056	.065	.018	.073	.082
2	.021	.057	.068	.022	.074	.087
1	.027	.059	.074	.030	.076	.092
0.75	.031	.060	.077	.033	.078	.096
0.50	.036	.062	.081	.038	.079	.100
0.25	.042	.063	.083	.044	.081	.105
0	.049	.066	.086	.052	.084	.111
2.75	.050	.066	.086	.062	.087	.120
2.50	.073	.073	.100	.077	.092	.131
2.25	.091	.079	.123	.096	.098	.145
2	.118	.087	.142	.125	.107	.160
1.75	.125	.089	.147	.132	.109	.171
1.50	.133	.092	.152	.140	.112	.177
1.25	.141	.094	.158	.149	.115	.183
1	.150	.097	.164	.159	.118	.190
0.75	.160	.100	.171	.170	.121	.197
0.50	.172	.104	.179	.182	.125	.205
0.25	.184	.107	.186	.195	.129	.214
0	.198	.111	.195	.210	.133	.223
1.55	.213	.116	.205	.227	.138	.233
1.50	.231	.121	.216	.246	.144	.246
1.45	.251	.127	.228	.268	.151	.259
1.40	.275	.134	.242	.293	.158	.274
1.35	.302	.142	.258	.322	.166	.288
1.30	.339	.151	.276	.356	.176	.311
1.25	.371	.161	.296	.398	.188	.334
1.20	.415	.173	.320	.446	.202	.361
1.15	.469	.188	.349	.500	.218	.392
1.10	.537	.206	.383	.580	.238	.430
1.05	.621	.228	.424	.675	.263	.476
1.00	.732	.255	.474	.800	.304	.533

See the auxiliary table, p. 136.

TABLE V.—Coefficients of Discharge for different Ratios of the Channel to the Orifice.

Coefficients for heads in still water $\sqrt{h} = .7071$ and 1.

Ratio of the channel to the orifice.	Coefficient .7071 for heads in still water.			Coefficient 1.000 for heads in still water.		
	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centres.	Coefficients for weirs: the heads measured to the full depth.	Ratio of the height due to the velocity of approach to the head.	Coefficients for orifices: the heads measured to the centres.	Coefficients for weirs: the heads measured to the full depth.
30.	.001	.707	.708	.001	1.001	1.002
20.	.001	.708	.708	.003	1.001	1.004
15.	.001	.708	.709	.005	1.002	1.006
10.	.005	.709	.712	.010	1.005	1.014
9.	.006	.709	.713	.013	1.006	1.017
8.	.008	.710	.714	.016	1.008	1.021
7.	.010	.711	.717	.021	1.010	1.028
6.	.014	.712	.721	.029	1.014	1.038
5.5	.017	.713	.723	.034	1.017	1.045
5.0	.020	.714	.727	.041	1.021	1.056
4.5	.025	.716	.731	.052	1.026	1.067
4.0	.032	.718	.737	.067	1.033	1.084
3.75	.037	.720	.742	.077	1.038	1.096
3.50	.043	.722	.747	.089	1.044	1.110
3.25	.050	.724	.753	.105	1.051	1.127
3.00	.059	.728	.760	.125	1.061	1.149
2.75	.071	.732	.770	.152	1.073	1.178
2.50	.087	.737	.783	.190	1.091	1.216
2.25	.110	.745	.801	.240	1.116	1.269
2.00	.143	.759	.826	.333	1.155	1.347
1.95	.151	.760	.832	.356	1.165	1.367
1.90	.161	.762	.839	.383	1.176	1.389
1.85	.171	.765	.846	.412	1.188	1.413
1.80	.182	.769	.854	.446	1.203	1.441
1.75	.195	.773	.863	.484	1.218	1.471
1.70	.209	.778	.873	.529	1.237	1.505
1.65	.225	.783	.883	.579	1.257	1.543
1.60	.243	.788	.895	.641	1.281	1.589
1.55	.263	.795	.908	.711	1.308	1.638
1.50	.286	.802	.923	.800	1.342	1.699
1.45	.312	.810	.939	.903	1.379	1.767
1.40	.342	.819	.958	1.042	1.429	1.854
1.35	.378	.830	.980	1.216	1.489	1.958
1.30	.421	.842	1.003	1.449	1.565	2.088
1.25	.471	.857	1.033	1.778	1.667	2.259
1.20	.532	.875	1.066	2.273	1.810	2.499
1.15	.608	.897	1.107	3.100	2.025	2.844
1.10	.704	.923	1.155	4.762	2.400	3.440
1.05	.830	.957	1.210	9.756	3.280	4.803
1.00	1.000	1.000	1.293	infinite.	infinite.	infinite.

See the auxiliary table, p. 136, also p. 138.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depth $\frac{1}{2}$ inch to 10 inches. Coefficients .007 to .017.

GREATER COEFFICIENTS.

The Formule at the heads of the Columns give the Value of the Discharge, D , in Cubic feet per minute, when l , the length of the Weir, is taken in feet, and the head, h , in inches. For $l\sqrt{h^3}$ we may substitute $lh\sqrt{h}$, retaining the same standards.

Heads in inches.	Theoretical Discharge, $D = 7.73 l\sqrt{h^3}$.	Coefficient .007, $D = 6.15 l\sqrt{h^3}$.	Coefficient .008, $D = 5.02 l\sqrt{h^3}$.	Coefficient .009, $D = 4.03 l\sqrt{h^3}$.	Coefficient .010, $D = 3.85 l\sqrt{h^3}$.	Coefficient .017, $D = 4.76 l\sqrt{h^3}$.
.25	.005	.004	.027	.017	.006	.000
.5	2.730	1.821	1.775	1.744	1.714	1.084
.75	5.018	3.345	3.200	3.205	3.160	3.005
1.	7.722	5.151	5.010	4.934	4.840	4.704
1.25	10.702	7.108	7.015	6.890	6.777	6.650
1.5	14.186	9.402	9.291	9.085	8.968	8.753
1.75	17.877	11.924	11.620	11.423	11.227	11.030
2.	21.842	14.560	14.107	13.657	13.717	13.477
2.25	26.082	17.383	16.940	16.454	16.307	16.080
2.5	30.524	20.300	19.841	19.505	19.100	18.833
2.75	35.215	23.480	22.890	22.503	22.115	21.728
3.	40.125	26.703	26.081	25.640	25.100	24.767
3.25	45.244	30.178	29.408	28.911	28.413	27.915
3.5	50.569	33.720	32.800	32.310	31.754	31.197
3.75	56.077	37.403	36.450	35.833	35.216	34.590
4.	61.777	41.205	40.153	39.470	38.796	38.113
4.25	67.668	45.129	43.978	43.233	42.480	41.745
4.5	73.744	49.167	47.914	47.103	46.202	45.482
4.75	79.992	53.321	51.962	51.083	50.203	49.324
5.	86.395	57.585	56.118	55.108	54.218	53.200
5.25	92.901	61.959	60.379	59.357	58.335	57.314
5.5	99.504	66.436	64.743	63.647	62.551	61.450
5.75	106.212	71.017	69.207	68.030	66.864	65.693
6.	113.041	75.698	73.769	72.521	71.272	70.024
6.25	120.057	80.478	78.427	77.106	75.772	74.445
6.5	127.200	85.355	83.180	81.772	80.305	78.957
6.75	135.422	90.329	88.024	86.536	85.045	83.555
7.	143.815	95.401	92.960	91.397	89.813	88.240
7.25	150.744	100.540	97.983	96.325	94.607	93.000
7.5	158.008	105.792	103.095	101.350	99.400	97.801
7.75	160.004	111.125	108.292	106.460	104.457	102.705
8.	174.731	116.540	113.575	111.653	109.731	107.800
8.25	182.084	122.051	118.940	116.927	114.914	112.901
8.5	191.365	127.640	124.387	122.282	120.177	118.072
8.75	199.800	133.313	129.915	127.710	125.518	123.310
9.	208.400	139.067	135.522	133.220	130.935	128.642
9.25	217.243	144.901	141.207	138.818	136.428	134.030
9.5	226.111	150.816	146.972	144.485	141.907	139.510
9.75	235.003	156.807	152.810	150.225	147.030	145.053
10.	244.103	162.877	158.725	156.030	153.953	150.600

See pp. 111 to 127.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depth 10.25 inches to 32 inches. Coefficients .007 to .617.

GREATER COEFFICIENTS.

The Formulas at the heads of the Columns give the Value of the Discharge D , in Cubic feet per minute, when l , the length of the Weir, is taken in feet, and the head, h , in inches. For $l\sqrt{h^3}$ we may substitute $lh\sqrt{h}$, retaining the same standards.

Head in inches.	Theoretical discharge $D = 7.73 l \sqrt{h^3}$.	Coefficient .007. $D = 5.15 l \sqrt{h^3}$.	Coefficient .050. $D = 5.92 l \sqrt{h^3}$.	Coefficient .080. $D = 4.93 l \sqrt{h^3}$.	Coefficient .028. $D = 4.85 l \sqrt{h^3}$.	Coefficient .617. $D = 4.76 l \sqrt{h^3}$.
10.25	253.407	160.023	164.715	101.927	159.140	156.352
10.5	262.734	175.244	179.777	107.887	164.997	162.107
10.75	272.173	181.540	176.913	173.910	170.925	167.931
11.	281.723	187.900	183.120	180.021	176.922	173.823
11.25	291.382	194.352	189.398	186.103	182.988	179.782
11.5	301.148	200.806	195.746	192.434	189.121	185.808
11.75	311.024	207.451	202.164	198.743	195.321	191.900
12.	321.	214.107	208.650	205.110	201.568	198.057
12.5	341.275	227.628	221.826	218.072	214.313	210.564
13.	361.050	241.421	235.268	231.286	227.305	223.323
13.5	383.031	255.482	248.970	244.757	240.543	236.330
14.	404.507	269.806	262.930	258.480	254.030	249.581
14.5	426.368	284.387	277.130	272.440	267.759	263.060
15.	448.611	299.223	291.597	286.662	281.728	276.793
15.5	471.228	314.309	306.298	301.115	295.931	290.748
16.	494.212	329.630	321.238	315.801	310.365	304.929
16.5	517.558	345.211	336.413	330.720	325.026	319.333
17.	541.261	361.021	351.820	345.866	339.912	333.958
17.5	565.315	377.065	367.455	361.236	355.018	348.799
18.	589.715	393.340	383.315	376.828	370.341	363.854
18.5	614.443	409.833	399.388	392.629	385.870	379.111
19.	639.533	426.569	415.696	408.662	401.627	394.592
19.5	664.944	443.518	432.214	424.899	417.585	410.270
20.	690.682	460.685	448.943	441.346	433.748	426.151
20.5	716.737	478.064	465.879	457.995	450.111	442.227
21.	743.125	495.664	483.031	474.857	466.683	458.508
21.5	769.823	513.472	500.385	491.917	483.440	474.981
22.	796.832	531.487	517.931	509.176	500.410	491.645
22.5	824.151	549.709	535.698	526.632	517.507	508.501
23.	851.775	568.134	553.654	544.284	534.915	525.545
23.5	879.700	586.760	571.805	562.128	552.452	542.775
24.	907.925	605.586	590.151	580.164	570.177	560.190
25.	965.253	643.824	627.414	616.797	606.179	595.561
26.	1023.748	682.840	665.436	654.175	642.914	631.653
27.	1083.375	722.611	704.194	692.277	680.360	668.442
28.	1144.116	763.125	743.675	731.090	718.505	705.920
29.	1205.959	804.360	783.868	770.602	757.337	744.071
30.	1268.804	846.332	824.762	810.864	796.847	782.889
31.	1332.833	889.000	866.341	851.680	837.019	822.358
32.	1397.842	932.361	908.597	893.221	877.845	862.469

See pp. 111 to 127.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depth 33 inches to 72 inches. Coefficients .667 to .617.

GREATER COEFFICIENTS.

The Formula at the heads of the Columns give the Value of the Discharge, D , in Cubic feet per minute, when L , the length of the Weir, is taken in feet, and the head, h , in inches. For $L\sqrt{h^3}$ we may substitute $Lh\sqrt{h}$, retaining the same standards.

Heads in inches.	Theoretical discharge $D = 7.75 L\sqrt{h^3}$	Coefficient .667, $D = 6.16 L\sqrt{h^3}$	Coefficient .659, $D = 6.02 L\sqrt{h^3}$	Coefficient .655, $D = 5.91 L\sqrt{h^3}$	Coefficient .650, $D = 5.85 L\sqrt{h^3}$	Coefficient .617, $D = 4.76 L\sqrt{h^3}$
33	1403.875	970.405	951.510	935.410	919.314	903.211
34	1500.017	1021.122	995.090	978.250	961.410	944.570
35	1598.051	1069.500	1039.318	1021.730	1004.141	986.553
36	1697.984	1112.532	1084.177	1065.820	1047.481	1029.134
37	1797.943	1150.208	1120.403	1110.540	1091.428	1072.311
38	1898.876	1200.520	1175.700	1165.871	1145.974	1116.070
39	1880.740	1254.458	1222.485	1201.707	1181.108	1160.420
40	1958.544	1303.014	1269.804	1248.315	1220.826	1205.337
41	2027.258	1352.181	1317.718	1295.418	1273.118	1250.818
42	2101.873	1401.951	1366.210	1343.000	1319.978	1298.857
43	2177.387	1452.317	1415.302	1391.350	1367.300	1345.448
44	2255.783	1503.279	1464.050	1440.107	1415.370	1390.584
45	2331.052	1554.812	1513.184	1489.542	1463.901	1438.260
46	2409.183	1606.926	1563.900	1539.408	1512.907	1486.403
47	2488.170	1659.600	1617.311	1589.911	1562.571	1535.201
48	2568	1712.850	1669.200	1640.952	1612.704	1584.450
49	2648.000	1766.000	1721.030	1692.498	1663.302	1634.327
50	2730.100	1821.021	1774.004	1744.572	1714.540	1684.509
51	2812.474	1876.920	1828.108	1797.171	1766.234	1735.200
52	2895.507	1931.303	1882.138	1850.283	1818.435	1786.583
53	2979.626	1987.343	1936.091	1903.910	1871.142	1838.307
54	3064.243	2043.857	1991.704	1958.058	1924.951	1890.014
55	3149.756	2100.887	2047.341	2012.603	1978.940	1943.900
56	3236.060	2158.445	2103.493	2067.830	2032.230	1998.043
57	3323.117	2216.510	2160.923	2123.472	2086.917	2053.363
58	3410.940	2275.101	2217.115	2179.594	2142.074	2108.654
59	3499.542	2334.105	2274.702	2236.207	2197.712	2159.217
60	3588.880	2393.780	2332.778	2293.300	2253.822	2214.344
61	3678.934	2453.882	2391.340	2350.871	2310.402	2269.993
62	3769.825	2514.473	2450.380	2408.918	2367.450	2325.982
63	3861.303	2575.540	2509.905	2467.430	2424.955	2382.470
64	3953.004	2637.114	2569.901	2526.410	2482.920	2439.420
65	4046.720	2699.102	2630.308	2585.854	2541.340	2496.820
66	4140.405	2761.600	2691.302	2645.757	2600.212	2554.067
67	4234.022	2824.603	2752.600	2706.115	2659.531	2612.047
68	4330.080	2888.107	2814.650	2766.925	2719.294	2671.063
69	4425.954	2952.111	2876.870	2828.185	2779.400	2730.814
70	4522.510	3016.518	2939.065	2889.888	2840.140	2790.302
71	4619.774	3081.380	3002.853	2952.030	2901.218	2850.401
72	4717.718	3146.718	3066.618	3014.022	3062.727	2910.832

See pp. 111 to 127.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depths $\frac{1}{4}$ inch to 10 inches. Coefficients .000 to .518.

LESSER COEFFICIENTS.

The Formula at the heads of the Columns give the Value of the Discharge, D , in Cubic feet per minute, when l , the length of the Weir, is taken in feet, and the head, h , in inches. For $l\sqrt{h^3}$ we may substitute $l h\sqrt{h}$ retaining the same standards.

Heads in inches.	Coefficient .000 $D = 4.081 l\sqrt{h^3}$.	Coefficient .005 $D = 4.50 l\sqrt{h^3}$.	Coefficient .010 $D = 4.51 l\sqrt{h^3}$.	Coefficient .015 $D = 4.81 l\sqrt{h^3}$.	Coefficient .020 $D = 4.17 l\sqrt{h^3}$.	Coefficient .018 $D = 4 l\sqrt{h^3}$.
.25	.585	.574	.564	.542	.521	.500
.5	1.054	1.024	1.004	1.534	1.474	1.414
.75	3.030	2.985	2.920	2.810	2.708	2.598
1.	4.680	4.595	4.510	4.340	4.170	4.000
1.25	6.540	6.421	6.303	6.005	5.828	5.590
1.5	8.507	8.441	8.284	7.973	7.660	7.348
1.75	10.833	10.637	10.440	10.047	9.653	9.200
2.	13.236	12.990	12.750	12.275	11.795	11.314
2.25	15.704	15.507	15.220	14.647	14.073	13.500
2.5	18.408	18.102	17.826	17.155	16.483	15.811
2.75	21.340	20.953	20.566	19.791	19.016	18.241
3.	24.316	23.874	23.433	22.550	21.668	20.785
3.25	27.418	26.920	26.422	25.427	24.432	23.430
3.5	30.641	30.085	29.529	28.416	27.304	26.192
3.75	33.982	33.366	32.749	31.515	30.281	29.048
4.	37.437	36.757	36.078	34.719	33.360	32.000
4.25	41.001	40.256	39.512	38.024	36.535	35.047
4.5	44.671	43.860	43.049	41.427	39.806	38.184
4.75	48.445	47.565	46.680	44.927	43.160	41.410
5.	52.319	51.369	50.420	48.520	46.621	44.722
5.25	56.292	55.270	54.248	52.205	50.161	48.117
5.5	60.360	59.264	58.169	55.977	53.786	51.595
5.75	64.522	63.351	62.180	59.837	57.495	55.153
6.	68.776	67.527	66.279	63.782	61.285	58.788
6.25	73.118	71.791	70.464	67.809	65.155	62.500
6.5	77.549	76.142	74.734	71.919	69.103	66.288
6.75	82.066	80.576	79.086	76.107	73.128	70.149
7.	86.667	85.094	83.521	80.374	77.228	74.082
7.25	91.351	89.693	88.034	84.718	81.402	78.085
7.5	96.116	94.372	92.627	89.138	85.648	82.159
7.75	100.962	99.129	97.297	93.631	89.966	86.301
8.	105.887	103.965	102.043	98.199	94.355	90.511
8.25	110.889	108.876	106.863	102.837	98.812	94.786
8.5	115.967	113.862	111.757	107.547	103.337	99.127
8.75	121.121	118.922	116.723	112.326	107.929	103.532
9.	126.349	124.055	121.762	117.175	112.588	108.001
9.25	131.649	129.259	126.870	122.090	117.311	112.532
9.5	137.023	134.535	132.048	127.074	122.100	117.125
9.75	142.467	139.881	137.294	132.122	126.950	121.778
10.	147.991	145.295	142.600	137.237	131.864	126.492

See pp. 111 to 127.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depths 10.25 inches to 32 inches. Coefficients .600 to .618.

MASON COEFFICIENTS.

The Formulae at the heads of the Columns give the Value of the Discharge, D , in Cubic feet per minute, when L , the length of the Weir, is taken in feet, and the head, h , in inches. For $L\sqrt{h^3}$ we may substitute $Lh\sqrt{h}$, retaining the same standards.

Heads in inches.	Coefficient .600, $D = 4.081 L\sqrt{h^3}$.	Coefficient .605, $D = 4.501 L\sqrt{h^3}$.	Coefficient .611, $D = 4.611 L\sqrt{h^3}$.	Coefficient .616, $D = 4.811 L\sqrt{h^3}$.	Coefficient .618, $D = 4.971 L\sqrt{h^3}$.	Coefficient .618, $D = 4.11\sqrt{h^3}$.
10.25	153.605	150.777	147.000	142.416	130.840	131.265
10.5	150.217	146.327	142.437	137.057	126.820	126.000
10.75	146.037	142.043	138.040	132.061	121.074	120.086
11.	142.024	137.925	133.920	128.028	116.130	115.033
11.25	138.077	133.972	129.967	124.050	112.130	111.036
11.5	134.090	129.983	125.970	120.045	108.120	107.026
11.75	130.070	125.959	121.936	116.024	104.092	103.000
12.	126.020	121.905	117.870	112.002	100.040	98.948
12.5	120.810	116.650	112.602	106.704	94.780	93.688
13.	115.342	111.300	107.250	101.315	89.453	88.361
13.5	109.617	105.703	101.650	95.763	83.837	82.745
14.	103.631	100.082	96.029	90.233	78.261	77.169
14.5	97.370	94.380	90.000	84.710	72.730	71.638
15.	90.838	87.924	83.580	79.110	67.250	66.158
15.5	84.561	81.631	77.287	73.530	61.763	60.671
16.	78.042	75.150	70.800	67.747	56.275	55.183
16.5	71.840	68.947	64.590	61.868	50.781	49.689
17.	65.004	62.100	57.750	56.180	45.284	44.192
17.5	58.681	55.782	51.434	50.707	39.787	38.695
18.	52.707	50.380	46.034	45.230	34.290	33.198
18.5	47.232	44.904	40.585	40.751	28.793	27.701
19.	41.258	38.929	35.137	35.274	23.296	22.204
19.5	35.783	33.452	29.687	29.800	17.799	16.707
20.	30.308	28.000	24.240	24.300	12.302	11.210
20.5	24.833	22.558	18.794	18.800	6.805	5.713
21.	19.358	17.283	13.347	13.350	1.308	0.216
21.5	13.883	12.158	8.900	8.900	0.811	0.169
22.	8.408	7.483	3.453	3.453	0.314	0.072
22.5	2.933	2.608	0.000	0.000	0.017	0.000
23.	0.458	0.408	0.000	0.000	0.000	0.000
23.5	0.000	0.000	0.000	0.000	0.000	0.000
24.	0.000	0.000	0.000	0.000	0.000	0.000
25.	0.000	0.000	0.000	0.000	0.000	0.000
26.	0.000	0.000	0.000	0.000	0.000	0.000
27.	0.000	0.000	0.000	0.000	0.000	0.000
28.	0.000	0.000	0.000	0.000	0.000	0.000
29.	0.000	0.000	0.000	0.000	0.000	0.000
30.	0.000	0.000	0.000	0.000	0.000	0.000
31.	0.000	0.000	0.000	0.000	0.000	0.000
32.	0.000	0.000	0.000	0.000	0.000	0.000

See pp. 111 to 127.

TABLE VI.—The Discharge over Weirs or Notches of one foot in length, in Cubic feet per minute.

Depth 33 inches to 72 inches. Coefficients .606 to .518.

LESSER COEFFICIENTS.

The Formule at the heads of the Columns give the Value of the Discharge, D , in Cubic feet per minute, whence l , the length of the Weir, is taken in feet, and the head, h , in inches. For $l\sqrt{h^3}$ we may substitute $l h\sqrt{h}$, retaining the same standards.

Heads in Inches.	Coefficient .606. $D = 4.081 l \sqrt{h^3}$.	Coefficient .595. $D = 4.591 l \sqrt{h^3}$.	Coefficient .584. $D = 4.511 l \sqrt{h^3}$.	Coefficient .562. $D = 4.341 l \sqrt{h^3}$.	Coefficient .540. $D = 4.171 l \sqrt{h^3}$.	Coefficient .518. $D = 4.1 l \sqrt{h^3}$.
33.	887.108	871.000	854.003	822.608	790.403	758.287
34.	927.736	910.800	894.050	860.375	826.095	793.015
35.	968.904	951.370	933.787	898.610	863.434	828.257
36.	1010.786	992.430	974.091	937.396	900.701	864.005
37.	1053.193	1034.070	1014.959	976.724	938.489	900.254
38.	1096.178	1076.281	1056.383	1016.588	976.793	936.997
39.	1139.732	1119.044	1098.356	1056.979	1015.003	974.226
40.	1183.848	1162.360	1140.870	1097.892	1054.014	1011.936
41.	1228.518	1206.219	1183.919	1139.319	1094.719	1050.120
42.	1273.737	1250.616	1227.406	1181.254	1135.013	1088.772
43.	1319.497	1295.545	1271.594	1223.601	1175.780	1127.886
44.	1365.792	1341.001	1316.200	1266.026	1217.043	1167.460
45.	1412.618	1386.976	1361.334	1310.051	1258.768	1207.485
46.	1459.965	1433.464	1406.963	1353.961	1300.959	1247.957
47.	1507.831	1480.461	1453.001	1398.352	1343.612	1288.872
48.	1556.208	1527.960	1499.712	1443.216	1386.720	1330.224
49.	1605.092	1575.956	1546.821	1488.550	1430.280	1372.000
50.	1654.477	1624.445	1594.413	1534.350	1474.286	1414.223
51.	1704.359	1673.422	1642.485	1580.610	1518.736	1456.862
52.	1754.732	1722.880	1691.020	1627.326	1563.622	1499.919
53.	1805.592	1772.817	1740.043	1674.493	1608.944	1543.394
54.	1856.937	1823.231	1789.524	1722.110	1654.607	1587.283
55.	1908.761	1874.104	1839.457	1770.162	1700.868	1631.573
56.	1961.046	1925.450	1889.853	1818.660	1747.467	1676.274
57.	2013.800	1977.255	1940.700	1867.592	1794.483	1721.375
58.	2067.033	2029.513	1991.992	1916.952	1841.911	1766.870
59.	2120.722	2082.227	2043.733	1966.743	1889.753	1812.763
60.	2174.867	2135.380	2095.911	2016.956	1938.000	1859.045
61.	2229.464	2188.995	2148.527	2067.580	1986.651	1905.714
62.	2284.514	2243.046	2201.578	2118.642	2035.706	1952.769
63.	2340.004	2297.529	2255.054	2170.103	2085.152	2000.202
64.	2395.939	2352.448	2308.957	2221.970	2134.995	2048.013
65.	2452.312	2407.798	2363.284	2274.257	2185.220	2096.201
66.	2509.122	2463.577	2418.032	2326.941	2235.851	2144.761
67.	2566.363	2519.770	2473.194	2380.020	2286.858	2193.690
68.	2624.032	2576.401	2528.770	2433.508	2338.240	2242.985
69.	2682.128	2633.443	2584.757	2487.386	2390.015	2292.644
70.	2740.645	2690.897	2641.149	2541.654	2442.159	2342.603
71.	2799.583	2748.766	2697.948	2596.313	2494.678	2393.043
72.	2858.937	2807.042	2755.147	2651.358	2547.568	2443.778

See pp. 111 to 127.

TABLE VII.—For finding the Mean Velocity from the Maximum Velocity at the Surface, in Mill Races, Streams, and Rivers with uniform Channels; and the Maximum Velocity from the Mean Velocity. (See p. 184.)

For the Velocity in feet per minute, multiply by 5.

Maximum velocity at the surface in large channels in inches per second.	Mean velocity in large channels in inches per second.	Mean velocity in smaller channels in inches per second.	Maximum velocity at the surface in large channels in inches per second.	Mean velocity in large channels in inches per second.	Mean velocity in smaller channels in inches per second.	Maximum velocity at the surface in large channels in inches per second.	Mean velocity in large channels in inches per second.	Mean velocity in smaller channels in inches per second.
1	.84	.75	41	34.24	33.37	81	67.04	68.80
2	1.67	1.51	42	35.07	34.23	82	68.47	69.77
3	2.51	2.27	43	35.91	35.00	83	69.91	70.68
4	3.34	3.04	44	36.74	35.95	84	70.14	71.59
5	4.18	3.81	45	37.58	36.82	85	70.98	72.50
6	5.01	4.58	46	38.41	37.69	86	71.81	73.42
7	5.85	5.36	47	39.25	38.56	87	72.65	74.33
8	6.68	6.14	48	40.08	39.43	88	73.48	75.24
9	7.52	6.92	49	40.92	40.30	89	74.32	76.16
10	8.35	7.71	50	41.75	41.17	90	75.15	77.08
11	9.19	8.50	51	42.59	42.05	91	75.99	77.99
12	10.02	9.29	52	43.42	42.92	92	76.82	78.91
13	10.86	10.09	53	44.26	43.80	93	77.66	79.83
14	11.69	10.88	54	45.09	44.68	94	78.49	80.75
15	12.53	11.69	55	45.93	45.56	95	79.33	81.67
16	13.36	12.49	56	46.76	46.44	96	80.16	82.59
17	14.20	13.30	57	47.60	47.32	97	81.00	83.51
18	15.03	14.11	58	48.43	48.21	98	81.83	84.43
19	15.87	14.92	59	49.27	49.09	99	82.67	85.36
20	16.70	15.73	60	50.10	49.98	100	83.50	86.28
21	17.54	16.55	61	50.94	50.87	101	84.34	87.20
22	18.37	17.37	62	51.77	51.76	102	85.17	88.13
23	19.21	18.19	63	52.61	52.65	103	86.01	89.06
24	20.04	19.02	64	53.44	53.54	104	86.84	89.98
25	20.88	19.85	65	54.28	54.43	105	87.68	90.91
26	21.71	20.68	66	55.11	55.33	106	88.51	91.84
27	22.55	21.51	67	55.95	56.22	107	89.35	92.77
28	23.38	22.34	68	56.78	57.12	108	90.18	93.69
29	24.22	23.18	69	57.62	58.02	109	91.02	94.62
30	25.05	24.02	70	58.45	58.91	110	91.85	95.55
31	25.89	24.86	71	59.29	59.81	111	92.69	96.48
32	26.72	25.70	72	60.12	60.71	112	93.52	97.42
33	27.56	26.54	73	60.96	61.61	113	94.36	98.35
34	28.39	27.38	74	61.79	62.52	114	95.19	99.28
35	29.23	28.24	75	62.63	63.42	115	96.03	100.21
36	30.06	29.09	76	63.46	64.32	116	96.86	101.15
37	30.90	29.94	77	64.30	65.23	117	97.70	102.08
38	31.73	30.79	78	65.13	66.13	118	98.53	103.02
39	32.57	31.65	79	65.97	67.04	119	99.37	103.95
40	33.40	32.51	80	66.80	67.95	120	100.20	104.89

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes $\frac{1}{8}$ inch to 2 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclination.			"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclination one in		$\frac{1}{8}$ inch.	$\frac{1}{4}$ inch.	$\frac{3}{8}$ inch.	$\frac{1}{2}$ inch.	$\frac{3}{4}$ inch.
0 1	09300		.14	.24	.38	.49	.57
0 2	91080		.22	.37	.50	.70	.90
0 3	21120		.28	.48	.75	.97	1.15
0 4	16840		.34	.57	.80	1.15	1.30
0 5	12072		.38	.65	1.02	1.30	1.55
0 6	10500		.42	.72	1.13	1.45	1.72
0 7	9051		.46	.78	1.24	1.58	1.88
0 8	7920		.50	.85	1.33	1.71	2.02
0 9	7040		.53	.90	1.43	1.83	2.16
0 10	6336		.57	.96	1.51	1.94	2.30
0 11	5700		.60	1.01	1.60	1.96	2.42
1 0	5280		.63	1.06	1.68	2.15	2.54
1 3	4224		.71	1.20	1.90	2.43	2.88
1 6	3520		.79	1.33	2.10	2.69	3.10
1 9	3017		.87	1.45	2.29	2.94	3.48
2 0	2640		.93	1.56	2.47	3.16	3.75
2 3	Interpolated.		.99	1.67	2.63	3.37	3.99
2 6	2112		1.05	1.77	2.79	3.58	4.24
2 9	Interpolated.		1.11	1.87	2.94	3.77	4.47
3 0	1760		1.16	1.96	3.09	3.96	4.69
3 3	Interpolated.		1.21	2.05	3.23	4.14	4.91
3 6	1508		1.26	2.14	3.37	4.32	5.12
3 9	Interpolated.		1.31	2.22	3.50	4.48	5.31
4 0	1320		1.36	2.30	3.63	4.65	5.51
4 6	Interpolated.		1.45	2.45	3.87	4.96	5.88
5 0	1056		1.54	2.61	4.11	5.27	6.24
5 6	Interpolated.		1.62	2.75	4.33	5.55	6.58
6 0	880		1.71	2.89	4.55	5.83	6.91
6 6	Interpolated.		1.78	3.02	4.76	6.10	7.22
7 0	754		1.86	3.15	4.97	6.36	7.54
7 6	Interpolated.		1.93	3.27	5.16	6.61	7.83
8 0	660		2.01	3.39	5.35	6.86	8.12
8 6	Interpolated.		2.07	3.51	5.53	7.09	8.40
9 0	587		2.14	3.62	5.72	7.32	8.68
9 6	Interpolated.		2.20	3.74	5.89	7.55	8.94
10 0	528		2.28	3.85	6.07	7.77	9.21
10 6	Interpolated.		2.33	3.95	6.24	7.99	9.47
11 0	480		2.40	4.06	6.40	8.20	9.72
11 6	Interpolated.		2.46	4.16	6.57	8.41	9.97
12 0	440		2.52	4.27	6.73	8.62	10.21

See p. 105.

TABLE VIII. For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes $\frac{1}{4}$ inch to 2 inches. Falls per mile 13 feet to 5280 feet.

Falls per mile in feet, and the hydraulic inclination.		"Hydraulic mean depth," or "mean radii," and velocities in inches per second.				
Falls.	Twelfths of an inch.	$\frac{1}{16}$ inch.	$\frac{1}{8}$ inch.	$\frac{1}{4}$ inch.	$\frac{3}{8}$ inch.	$\frac{1}{2}$ inch.
13.2	400	2.08	4.50	7.10	9.10	10.78
13.6	Interpolated.	2.71	4.50	7.24	9.27	10.98
14.1	375	2.76	4.67	7.37	9.44	11.18
14.6	Interpolated.	2.82	4.70	7.52	9.63	11.41
15.1	350	2.87	4.85	7.66	9.82	11.63
15.6	Interpolated.	2.94	4.90	7.83	10.03	11.88
16.2	325	3.00	5.07	7.99	10.24	12.13
17.0	300	3.14	5.30	8.37	10.72	12.70
18.2	275	3.30	5.58	8.80	11.27	13.35
21.1	250	3.48	5.89	9.30	11.90	14.10
23.5	225	3.70	6.26	9.87	12.65	14.99
26.4	200	3.96	6.70	10.57	13.54	16.04
30.2	175	4.28	7.24	11.42	14.63	17.33
35.2	150	4.68	7.92	12.40	16.00	18.90
37.7	140	4.88	8.24	13.00	16.60	19.74
42.2	125	5.21	8.81	13.90	17.80	21.00
48	110	5.62	9.50	14.98	19.19	22.74
52.8	100	5.94	10.03	15.85	20.30	24.06
58.7	90	6.33	10.60	16.87	21.61	25.60
65	80	6.78	11.47	18.10	23.17	27.40
72.4	70	7.35	12.42	19.59	25.00	29.73
80	60	8.05	13.61	21.48	27.51	32.60
105.6	50	8.99	15.10	23.96	30.60	36.37
117.3	45	9.57	16.18	25.53	32.70	38.75
132.0	40	10.28	17.37	27.41	35.11	41.60
150.8	35	11.14	18.81	29.71	38.06	45.10
170	30	12.23	20.68	32.62	41.78	49.51
212.2	25	13.60	23.09	36.43	46.67	55.30
264	20	15.64	26.44	41.71	53.43	63.30
352	15	18.61	31.50	49.63	63.57	75.33
528	10	23.73	40.11	63.28	81.06	96.05
580.7	9	25.20	42.70	67.37	86.20	102.25
650	8	27.08	46.78	72.22	92.51	109.61
754.3	7	29.20	49.51	78.10	100.04	118.54
880.0	6	32.05	54.15	85.43	109.43	129.66
1056	5	35.98	60.15	94.80	121.54	144.02
1320	4	40.40	68.20	107.73	137.00	163.51
1760	3	47.48	83.25	126.61	162.17	192.16
2640	2	59.47	100.53	158.59	203.14	240.70
5280	1	88.13	148.07	235.02	301.04	356.70

See p. 195.

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes $2\frac{1}{2}$ inches to 5 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.			"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations one in		$\frac{5}{8}$ inch.	$\frac{3}{4}$ inch.	$\frac{7}{8}$ inch.	1 inch.	$1\frac{1}{4}$ in. interpolated.
v. 1.							
0 1	63360		·06	·73	·70	·85	·96
0 2	31680		1·02	1·13	1·23	1·33	1·49
0 3	21120		1·30	1·45	1·58	1·70	1·91
0 4	15840		1·54	1·71	1·87	2·01	2·26
0 5	12672		1·76	1·95	2·13	2·29	2·58
0 6	10560		1·95	2·17	2·36	2·55	2·86
0 7	9051		2·13	2·37	2·58	2·78	3·13
0 8	7920		2·30	2·55	2·78	3·00	3·37
0 9	7040		2·46	2·73	2·98	3·21	3·61
0 10	6336		2·61	2·90	3·16	3·40	3·83
0 11	5760		2·76	3·06	3·33	3·59	4·04
1 0	5280		2·89	3·21	3·50	3·77	4·24
1 3	4224		3·28	3·64	3·97	4·27	4·81
1 6	3520		3·63	4·03	4·39	4·73	5·32
1 9	3017		3·96	4·39	4·79	5·16	5·80
2 0	2640		4·26	4·73	5·16	5·55	6·25
2 3	Interpolated.		4·55	5·04	5·50	5·92	6·66
2 6	2112		4·83	5·35	5·84	6·29	7·07
2 9	Interpolated.		5·09	5·64	6·15	6·62	7·46
3 0	1760		5·34	5·92	6·46	6·96	7·83
3 3	Interpolated.		5·58	6·19	6·75	7·27	8·18
3 6	1568		5·82	6·46	7·04	7·59	8·53
3 9	Interpolated.		6·06	6·71	7·31	7·88	8·86
4 0	1320		6·27	6·95	7·58	8·17	9·19
4 6	Interpolated.		6·60	7·42	8·09	8·71	9·80
5 0	1056		7·10	7·88	8·59	9·25	10·41
5 6	Interpolated.		7·48	8·30	9·05	9·76	10·97
6 0	880		7·80	8·72	9·51	10·25	11·53
6 6	Interpolated.		8·22	9·12	9·94	10·71	12·05
7 0	754		8·57	9·51	10·37	11·17	12·57
7 6	Interpolated.		8·92	9·89	10·78	11·62	13·06
8 0	660		9·24	10·25	11·18	12·04	13·54
8 6	Interpolated.		9·55	10·60	11·56	12·45	14·01
9 0	587		9·87	10·95	11·94	12·86	14·47
9 6	Interpolated.		10·18	11·28	12·31	13·26	14·91
10 0	528		10·48	11·62	12·67	13·65	15·36
10 6	Interpolated.		10·77	11·95	13·03	14·03	15·78
11 0	480		11·06	12·27	13·38	14·41	16·21
11 6	Interpolated.		11·34	12·58	13·72	14·82	16·64
12 0	440		11·62	12·89	14·05	15·22	17·07

TABLE VIII.—*For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.*

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes $2\frac{1}{2}$ inches to 6 inches. Falls per mile 13 feet to 5280 feet.

Falls per mile in feet, and the hydraulic inclinations.		"Hydraulic mean depth," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations one in	$\frac{1}{4}$ inch.	$\frac{1}{2}$ inch.	$\frac{3}{4}$ inch.	1 inch.	$1\frac{1}{4}$ in. interpolated.
13.2	400	12.20	13.40	14.83	15.98	17.08
13.6	Interpolated.	12.40	13.60	15.11	16.28	17.31
14.1	375	12.72	14.11	15.39	16.58	17.65
14.6	Interpolated.	12.98	14.39	15.70	16.91	17.92
15.1	350	13.29	14.68	16.00	17.24	18.40
15.6	Interpolated.	13.52	14.99	16.35	17.62	18.81
16.2	325	13.86	15.31	16.70	17.99	19.23
17.0	300	14.45	16.02	17.48	18.83	20.18
18.2	275	15.19	16.85	18.37	19.70	21.26
21.1	250	16.04	17.80	19.40	20.91	23.52
23.5	225	17.05	18.91	20.62	22.21	24.99
26.4	200	18.25	20.24	22.07	23.78	26.75
30.2	175	19.71	21.87	23.85	25.60	28.90
35.2	150	21.57	23.92	26.00	28.11	31.02
37.7	140	22.45	24.91	27.16	29.21	32.02
42.2	125	23.99	26.69	29.03	31.27	35.18
48.1	110	25.67	28.69	31.29	33.71	37.92
52.8	100	27.30	30.35	33.10	35.66	40.11
58.7	90	29.12	32.31	35.23	37.93	42.69
66.1	80	31.23	34.64	37.78	40.70	45.79
75.4	70	33.62	37.41	40.91	44.07	49.58
86.0	60	37.08	41.13	44.80	48.33	54.86
105.6	50	41.37	45.78	50.04	53.01	60.65
117.3	45	44.98	48.89	53.32	57.44	64.62
132.1	40	47.82	52.30	57.25	61.67	69.37
150.8	35	51.30	56.90	62.04	66.86	75.20
170.1	30	56.39	62.47	68.13	73.40	82.56
211.2	25	62.90	69.77	76.69	81.97	92.21
264.1	20	72.01	79.87	87.11	93.84	105.50
352.1	15	85.98	95.05	103.60	111.67	125.61
528.1	10	109.23	121.10	132.17	142.30	160.17
586.7	9	116.91	129.01	140.70	151.58	170.50
660.1	8	124.63	136.30	150.83	162.40	182.78
754.3	7	133.84	146.57	163.12	175.73	197.07
880.1	6	147.69	163.60	178.42	192.22	216.22
1056.1	5	163.82	181.71	198.17	213.50	240.15
1320.1	4	185.99	209.31	225.00	242.30	272.66
1760.1	3	218.58	242.40	264.42	284.86	320.43
2640.1	2	273.79	303.79	331.22	356.82	401.37
5280.1	1	403.74	459.07	499.84	528.79	604.82

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

OPEN DRAINS AND PIPES.

Diameters of pipes 6 inches to 12 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations,		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	1½ inch.	1¾ in. interpolated.	2 inches.	2½ inches.	3 inches.
0 1	63300	1.07	1.16	1.24	1.40	1.56
0 2	31680	1.06	1.80	1.94	2.10	2.41
0 3	21120	2.12	2.30	2.48	2.80	3.08
0 4	15840	2.52	2.73	2.94	3.34	3.65
0 5	12672	2.86	3.11	3.36	3.77	4.16
0 6	10560	3.18	3.46	3.72	4.19	4.62
0 7	9051	3.47	3.77	4.06	4.58	5.04
0 8	7920	3.75	4.06	4.38	4.94	5.44
0 9	7040	4.01	4.34	4.68	5.28	5.81
0 10	6336	4.26	4.61	4.97	5.60	6.17
0 11	5760	4.49	4.86	5.24	5.91	6.51
1 0	5280	4.71	5.11	5.51	6.21	6.84
1 3	4224	5.34	5.79	6.24	7.03	7.76
1 6	3520	5.91	6.41	6.91	7.79	8.58
1 9	3017	6.44	6.99	7.53	8.49	9.36
2 0	2640	6.94	7.53	8.11	9.14	10.07
2 3	Interpolated.	7.40	8.03	8.65	9.74	10.74
2 6	2112	7.86	8.52	9.18	10.36	11.40
2 9	Interpolated.	8.28	8.98	9.67	10.90	12.01
3 0	1760	8.70	9.43	10.16	11.46	12.62
3 3	Interpolated.	9.09	9.86	10.62	11.97	13.19
3 6	1508	9.48	10.28	11.08	12.48	13.76
3 9	Interpolated.	9.84	10.67	11.50	12.96	14.29
4 0	1320	10.21	11.07	11.93	13.44	14.81
4 3	Interpolated.	10.56	11.46	12.32	13.94	15.30
5 0	1056	11.53	12.54	13.51	15.23	16.78
5 3	Interpolated.	12.18	13.21	14.24	16.04	17.68
6 0	880	12.80	13.88	14.96	16.86	18.58
6 3	Interpolated.	13.38	14.51	15.64	17.62	19.42
7 0	754	13.96	15.14	16.32	18.39	20.26
7 3	Interpolated.	14.51	15.73	16.95	19.10	21.06
8 0	600	15.05	16.32	17.58	19.82	21.84
8 3	Interpolated.	15.56	16.87	18.18	20.49	22.58
9 0	587	16.07	17.43	18.78	21.17	23.32
9 3	Interpolated.	16.57	17.97	19.36	21.82	24.04
10 0	528	17.06	18.50	19.94	22.47	24.76
10 3	Interpolated.	17.54	19.01	20.49	23.09	25.45
11 0	480	18.01	19.53	21.04	23.72	26.13
11 3	Interpolated.	18.47	20.02	21.57	24.32	26.79
12 0	440	18.92	20.51	22.11	24.91	27.45

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes 6 inches to 14 inches. Falls per mile 18 feet to 5280 feet.

Falls per mile in feet, and the hydraulic inclination.		"Hydraulic mean depth," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	1½ inch.	2 inches.	2½ inches.	3 inches.	3½ inches.
18.2	400	19.07	29.34	20.30	28.08	31.44
19.6		20.34	29.77	20.70	29.52	32.03
14.1	375	20.72	24.21	27.28	30.06	32.62
14.6		21.13	24.60	27.83	30.67	33.27
15.1	350	21.55	25.18	28.08	31.27	33.93
15.6		22.01	25.72	28.00	31.94	34.66
16.2	325	22.48	26.27	29.00	32.62	35.39
17.6	300	23.53	27.50	30.00	34.15	37.05
19.2	275	24.74	28.90	32.57	35.80	38.64
21.1	250	26.13	30.53	34.41	37.91	41.14
23.5	225	27.70	32.44	36.56	40.28	43.71
26.4	200	29.72	34.72	39.13	43.12	46.70
30.2	175	32.11	37.52	42.28	46.50	50.55
35.2	150	35.13	41.04	46.20	50.07	55.30
37.7	140	36.57	42.73	48.16	53.07	57.68
42.2	125	39.08	45.63	51.46	56.71	61.53
48.1	110	42.13	49.23	55.48	61.13	66.33
52.8	100	44.67	52.07	58.60	64.07	70.17
58.7	90	47.43	55.42	62.40	68.83	74.68
66.1	80	50.87	59.44	66.00	73.81	80.00
75.4	70	55.08	64.30	72.50	79.02	86.72
88.1	60	60.30	70.57	79.53	87.63	95.00
105.6	50	67.33	78.73	88.73	97.77	106.08
117.3	45	71.70	83.88	94.54	104.17	113.03
132.1	40	77.07	90.00	101.50	113.84	121.35
150.8	35	83.55	97.63	110.03	121.24	131.55
170.1	30	91.72	107.18	120.70	133.10	144.41
211.2	25	102.44	119.70	134.00	148.65	161.20
264.1	20	117.28	137.03	154.44	170.18	184.65
352.1	15	139.50	169.00	183.78	202.50	219.72
528.1	10	177.05	207.02	234.33	258.21	280.16
580.7	9	189.43	221.34	249.45	274.87	298.24
660.1	8	203.07	237.28	267.42	294.67	319.72
754.3	7	219.61	256.61	289.20	318.67	345.77
880.1	6	240.22	281.30	316.33	348.57	378.20
1056.1	5	266.81	311.75	351.35	387.15	429.07
1320.1	4	302.62	353.05	398.91	439.55	479.93
1760.1	3	356.00	415.90	468.80	516.57	569.49
2040.1	2	445.03	521.94	587.22	647.06	702.08
5280.1	1	660.84	772.16	870.23	958.91	1040.44

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes 14 inches to 22 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	3½ inches.	4 inches.	4½ inches.	5 inches.	5½ inches.
0 1	63360	1.68	1.80	1.91	2.02	2.13
0 2	31680	2.61	2.81	2.98	3.15	3.32
0 3	21120	3.34	3.59	3.82	4.03	4.24
0 4	15840	3.96	4.25	4.52	4.78	5.02
0 5	12672	4.51	4.84	5.15	5.44	5.72
0 6	10560	5.01	5.37	5.72	6.04	6.35
0 7	9051	5.47	5.87	6.24	6.60	6.94
0 8	7920	5.90	6.33	6.74	7.12	7.48
0 9	7040	6.31	6.77	7.20	7.61	8.00
0 10	6336	6.70	7.18	7.64	8.08	8.49
0 11	5760	7.06	7.58	8.06	8.52	8.96
1 0	5280	7.42	7.96	8.47	8.95	9.41
1 3	4224	8.41	9.02	9.60	10.14	10.66
1 6	3520	9.31	9.99	10.63	11.23	11.80
1 9	3017	10.15	10.89	11.58	12.24	12.86
2 0	2640	10.93	11.73	12.47	13.18	13.86
2 3	Interpolated.	11.65	12.50	13.30	14.05	14.77
2 6	2112	12.37	13.28	14.12	14.93	15.69
2 9	Interpolated.	13.03	13.98	14.88	15.72	16.53
3 0	1760	13.69	14.69	15.63	16.52	17.36
3 3	Interpolated.	14.31	15.35	16.33	17.26	18.14
3 6	1508	14.92	16.01	17.03	18.00	18.92
3 9	Interpolated.	15.50	16.63	17.69	18.70	19.65
4 0	1320	16.07	17.25	18.35	19.39	20.38
4 6	Interpolated.	17.14	18.39	19.56	20.68	21.73
5 0	1056	18.21	19.53	20.78	21.96	23.08
5 6	Interpolated.	19.18	20.58	21.90	23.14	24.32
6 0	880	20.16	21.63	23.01	24.32	25.56
6 6	Interpolated.	21.07	22.61	24.05	25.42	26.72
7 0	754	21.98	23.59	25.09	26.52	27.87
7 6	Interpolated.	22.84	24.50	26.07	27.55	28.90
8 0	660	23.69	25.42	27.04	28.58	30.04
8 6	Interpolated.	24.50	26.29	27.97	29.55	31.06
9 0	587	25.31	27.54	28.89	30.53	32.09
9 6	Interpolated.	26.09	27.99	29.78	31.47	33.08
10 0	528	26.87	28.83	30.67	32.41	34.06
10 6	Interpolated.	27.61	29.62	31.52	33.31	35.01
11 0	480	28.35	30.42	32.37	34.20	35.95
11 6	Interpolated.	29.07	31.19	33.18	35.07	36.86
12 0	440	29.79	31.96	34.00	35.93	37.77

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Ditches, Streams, and Rivers.

For a full cylindrical pipe, divide the diameter by 4 to find the hydraulic mean depth.

Diameters of pipes 18 inches to 2 feet. Falls per mile 13 feet to 6280 feet.

Falls per mile in feet and the hydraulic inclinations.		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in.	1 inches.	1½ inches.	2 inches.	2½ inches.	3 inches.
13.2	400	33.74	35.89	37.93	39.87	41.72
13.6	Interpolated.	34.37	36.50	38.61	40.61	42.50
14.1	375	35.00	37.23	39.35	41.30	43.28
14.6	Interpolated.	35.70	37.98	39.14	42.10	44.15
15.1	350	36.40	38.73	40.92	43.02	45.01
15.6	Interpolated.	37.10	39.58	41.81	43.94	45.90
16.2	325	37.97	40.40	42.69	44.87	46.80
17.0	300	38.75	42.20	44.69	46.97	49.16
19.2	275	41.78	44.45	46.97	49.38	51.67
21.1	250	44.14	46.95	49.62	52.13	54.58
23.6	225	46.90	49.90	52.72	55.42	58.09
26.1	200	50.20	53.41	56.44	59.32	62.08
30.2	175	54.24	57.71	60.98	64.10	67.07
35.2	150	59.34	63.13	66.71	70.12	73.37
37.7	140	61.78	65.72	69.45	73.00	76.30
42.2	125	66.02	70.23	74.22	78.01	81.64
48.0	110	71.17	75.72	80.01	84.10	88.00
52.8	100	75.20	80.00	84.04	88.07	92.10
68.7	90	80.13	85.25	90.03	94.60	99.00
80.0	80	85.03	91.42	96.01	101.51	106.26
76.4	70	93.04	98.98	104.60	109.95	115.05
88.0	60	102.02	108.51	114.70	120.60	126.16
105.0	50	113.62	121.09	127.00	134.30	140.74
117.3	45	124.27	133.01	139.34	146.30	150.96
132.0	40	136.20	146.61	149.38	156.80	161.00
160.8	35	144.14	159.16	158.63	166.70	174.53
170.0	30	154.05	163.84	174.20	183.10	191.61
211.2	25	173.05	184.10	194.56	204.50	214.00
260.0	20	198.12	210.77	222.73	234.11	244.98
352.0	15	235.75	250.80	265.01	278.58	291.52
528.0	10	300.00	310.80	337.05	355.22	371.71
580.7	9	320.00	340.33	359.70	378.14	395.70
600.0	8	343.04	359.05	386.07	405.37	424.20
754.3	7	370.00	394.08	417.08	438.39	458.76
800.0	6	405.70	431.70	450.21	470.52	501.79
1050.0	5	450.71	470.40	500.71	522.60	557.34
1320.0	4	511.72	544.30	575.30	601.00	632.78
1700.0	3	601.38	638.78	676.10	710.34	743.65
2610.0	2	753.20	801.30	840.80	880.10	931.50
5280.0	1	1116.35	1187.62	1255.04	1319.17	1380.44

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

The hydraulic mean depth is found for all channels, by dividing the wetted perimeter into the area.

Hydraulic mean depths 6 inches to 10 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	6 inches.	7 inches.	8 inches.	9 inches.	10 inches.
0 1	63360	2.23	2.41	2.58	2.75	2.90
0 2	31680	3.47	3.76	4.03	4.28	4.52
0 3	21120	4.43	4.80	5.15	5.47	5.78
0 4	15840	5.26	5.69	6.10	6.49	6.85
0 5	12672	5.98	6.48	6.95	7.39	7.80
0 6	10560	6.65	7.20	7.72	8.20	8.66
0 7	9051	7.26	7.86	8.43	8.96	9.46
0 8	7920	7.83	8.48	9.09	9.67	10.21
0 9	7040	8.37	9.07	9.72	10.33	10.91
0 10	6336	8.88	9.63	10.32	10.97	11.58
0 11	5760	9.37	10.16	10.89	11.57	12.22
1 0	5280	9.84	10.67	11.43	12.15	12.83
1 3	4224	11.16	12.09	12.95	13.77	14.54
1 6	3520	12.35	13.38	14.34	15.25	16.10
1 9	3017	13.46	14.58	15.63	16.61	17.54
2 0	2640	14.50	15.71	16.84	17.90	18.90
2 3	Interpolated.	15.45	16.75	18.24	19.08	20.15
2 6	2112	16.42	17.79	19.64	20.26	21.40
2 9	Interpolated.	17.29	18.74	20.37	21.34	22.54
3 0	1760	18.17	19.69	21.10	22.42	23.68
3 3	Interpolated.	18.99	20.57	22.05	23.43	24.76
3 6	1508	19.80	21.46	23.00	24.44	25.81
3 9	Interpolated.	20.56	22.28	23.88	25.36	26.80
4 0	1320	21.33	23.11	24.77	26.32	27.80
4 6	Interpolated.	22.74	24.64	26.41	28.07	29.64
5 0	1056	24.16	26.17	28.05	29.81	31.48
5 6	Interpolated.	25.45	27.58	29.58	31.42	33.17
6 0	880	26.75	28.98	31.06	33.02	34.86
6 6	Interpolated.	27.96	30.29	32.47	34.51	36.44
7 0	754	29.17	31.60	33.87	36.00	38.02
7 6	Interpolated.	30.30	32.83	35.19	37.40	39.50
8 0	660	31.43	34.06	36.50	38.80	40.97
8 6	Interpolated.	32.51	35.22	37.75	40.12	42.37
9 0	587	33.58	36.39	38.99	41.45	43.77
9 6	Interpolated.	34.61	37.50	40.20	42.72	45.11
10 0	528	35.65	38.63	41.40	44.00	46.46
10 6	Interpolated.	36.63	39.69	42.54	45.22	47.75
11 0	480	37.62	40.76	43.69	46.44	49.03
11 6	Interpolated.	38.57	41.79	44.79	47.61	50.27
12 0	440	39.52	42.82	45.90	48.78	51.51

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

The hydraulic mean depth is found for all channels by dividing the wetted perimeter into the area.

Hydraulic mean depths 11 inches to 21 inches. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.			"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in		11 inches.	12 inches.	15 inches.	18 inches.	21 inches.
0 1	00000		3.05	3.10	3.57	3.92	4.25
0 2	01080		4.75	4.87	5.57	6.12	6.62
0 3	02120		6.07	6.25	7.12	7.82	8.46
0 4	03240		7.13	7.33	8.44	9.27	10.03
0 5	04072		8.10	8.37	9.61	10.55	11.42
0 6	05000		9.10	9.42	10.87	11.92	12.68
0 7	06051		10.04	10.40	11.96	12.80	13.86
0 8	07020		10.72	11.21	12.67	13.81	14.94
0 9	08041		11.40	11.93	13.44	14.70	15.97
0 10	09030		12.10	12.72	14.27	15.60	16.95
0 11	07000		12.83	13.42	15.05	16.53	17.88
1 0	02800		13.48	14.00	15.81	17.30	18.78
1 3	0224		15.27	15.87	17.91	19.67	21.28
1 6	0620		16.01	17.08	19.83	21.78	23.50
1 9	0017		18.23	19.27	21.92	23.73	25.68
2 0	2040		19.85	20.70	23.28	25.03	27.00
2 3	Interpolated.		21.10	22.13	24.82	27.23	29.40
2 6	2112		22.48	23.51	26.56	28.65	31.32
2 9	Interpolated.		23.68	24.70	27.77	30.30	32.90
3 0	1740		24.88	26.02	29.18	32.04	34.67
3 3	Interpolated.		25.93	27.18	30.47	33.48	36.22
3 6	1308		27.11	28.36	31.77	34.92	37.78
3 9	Interpolated.		28.16	29.45	33.01	36.20	39.28
4 0	1020		29.20	30.54	34.25	37.60	40.00
4 6	Interpolated.		31.13	32.50	36.52	40.10	43.39
5 0	1050		33.07	34.50	38.70	42.50	46.69
5 6	Interpolated.		34.85	36.44	40.87	44.88	48.50
6 0	880		36.62	38.30	42.95	47.14	51.03
6 6	Interpolated.		38.28	40.03	44.90	49.30	53.34
7 0	764		39.93	41.76	46.84	51.43	55.65
7 6	Interpolated.		41.48	43.30	48.90	53.43	57.81
8 0	660		43.04	44.91	50.48	55.42	59.97
8 6	Interpolated.		44.50	46.54	52.20	57.32	62.02
9 0	587		45.97	48.08	53.92	59.21	64.06
9 6	Interpolated.		47.30	49.50	55.58	61.03	66.04
10 0	628		48.80	51.04	57.24	62.85	68.01
10 6	Interpolated.		50.15	52.46	58.83	64.50	69.80
11 0	480		51.51	53.87	60.41	66.03	71.78
11 6	Interpolated.		52.81	55.23	61.94	68.01	73.59
12 0	440		54.11	56.50	63.47	69.08	75.40

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

The hydraulic mean depth is found for all channels by dividing the wetted perimeter into the area.

Hydraulic mean depths 24 inches to 4 feet. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations,		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	24 inches.	30 inches.	36 inches.	42 inches.	48 inches.
F. I.						
0 1	68360	4.54	5.09	5.59	6.04	6.47
0 2	31680	7.09	7.94	8.71	9.42	10.08
0 3	21120	9.06	10.15	11.14	12.04	12.89
0 4	15840	10.78	12.03	13.20	14.27	15.27
0 5	12672	12.22	13.69	15.03	16.25	17.39
0 6	10560	13.57	15.21	16.69	18.05	19.31
0 7	9051	14.83	16.61	18.23	19.71	21.09
0 8	7920	15.99	17.92	19.66	21.27	22.76
0 9	7041	17.10	19.16	21.02	22.73	24.33
0 10	6336	18.15	20.33	22.31	24.13	25.82
0 11	5760	19.15	21.45	23.54	25.46	27.24
1 0	5280	20.11	22.53	24.72	26.73	28.61
1 3	4224	22.78	25.53	28.01	30.29	32.42
1 6	3520	25.23	28.27	31.02	33.54	35.90
1 9	3017	27.49	30.81	33.80	36.55	39.12
2 0	2640	29.62	33.18	36.41	39.38	42.14
2 3	Interpolated.	31.57	35.38	38.82	41.98	44.92
2 6	2112	33.53	37.57	41.22	44.58	47.71
2 9	Interpolated.	35.32	39.58	43.43	46.96	50.26
3 0	1760	37.11	41.58	45.68	49.34	52.81
3 3	Interpolated.	38.78	43.45	47.68	51.56	55.18
3 6	1508	40.45	45.32	49.73	53.78	57.55
3 9	Interpolated.	42.00	47.07	51.64	55.85	59.77
4 0	1320	43.53	48.81	53.56	57.92	61.98
4 6	Interpolated.	46.45	52.05	57.11	61.76	66.09
5 0	1056	49.34	55.28	60.66	65.60	70.20
5 6	Interpolated.	51.99	58.25	63.91	69.12	73.97
6 0	880	54.63	61.22	67.17	72.64	77.74*
6 6	Interpolated.	57.11	63.99	70.21	75.93	81.25
7 0	754	59.58	66.76	73.25	79.21	84.77
7 6	Interpolated.	61.89	69.35	76.09*	82.20	88.06
8 0	660	64.21	71.94	78.94	85.37	91.35
8 6	Interpolated.	66.40	74.40	81.63	88.26	94.47
9 0	587	68.59	76.85*	84.32	91.19	97.59
9 6	Interpolated.	70.60	79.22	86.92	94.00	100.59
10 0	528	72.81	81.58	89.52	96.81	103.60
10 6	Interpolated.	74.83	83.84	91.99	99.40	106.47
11 0	480	76.84*	86.10	94.47	102.17	109.33
11 6	Interpolated.	78.78	88.28	96.86	104.75	112.10
12 0	440	80.72	90.45	99.25	107.33	114.86

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

The hydraulic mean depth is found for all channels by dividing the wetted perimeter into the area.

Hydraulic mean depths 4 feet 6 inches to 7 feet. Falls per mile 1 inch to 13 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	54 inches.	60 inches.	66 inches.	72 inches.	84 inches.
0 1	033333	6.80	7.24	7.60	7.94	8.58
0 2	31680	10.70	11.20	11.65	12.08	13.00
0 3	21120	13.08	14.02	14.94	15.83	17.11
0 4	15840	16.21	17.10	17.95	18.76	20.28
0 5	12672	18.40	19.47	20.43	21.35	23.13
0 6	10560	20.50	21.63	22.70	23.72	25.64
0 7	9051	22.30	23.62	24.70	25.60	28.00
0 8	7920	24.10	25.48	26.74	27.65	30.21
0 9	7041	25.83	27.24	28.50	29.38	32.30
0 10	6336	27.41	28.91	30.34	31.71	34.28
0 11	5760	28.92	30.51	32.01	33.46	36.17
1 0	5280	30.37	32.03	33.62	35.13	37.98
1 3	4224	34.41	36.30	38.10	39.81	43.04
1 6	3620	38.10	40.10	42.18	44.08	47.05
1 9	3017	41.52	43.80	45.97	48.04	51.03
2 0	2640	44.73	47.18	49.52	51.75	55.04
2 3	Interpolated.	47.60	50.30	52.70	55.17	59.54
2 6	2112	50.65	53.42	56.07	58.50	63.34
2 9	Interpolated.	53.65	56.28	59.00	61.72	66.72
3 0	1760	56.68	59.18	62.05	64.85	70.10
3 3	Interpolated.	58.67	61.70	64.84	67.70	73.25
3 6	1508	61.60	64.44	67.63	70.67	76.40*
3 9	Interpolated.	63.44	66.62	70.23	73.30	79.35
4 0	1320	65.80	69.41	72.84	76.11*	82.20
4 6	Interpolated.	70.40	74.01	77.97*	81.16	87.74
5 0	1056	74.52	78.61*	82.50	86.21	93.20
5 6	Interpolated.	78.52*	82.80	86.92	90.84	98.20
6 0	880	82.52	87.05	91.35	95.40	103.20
6 6	Interpolated.	86.25	90.98	95.58	99.78	107.87
7 0	754	89.90	94.92	99.62	104.10	112.54
7 6	Interpolated.	93.48	98.61	103.48	108.14	116.91
8 0	660	96.98	102.30	107.35	112.10	121.28
8 6	Interpolated.	100.20	105.70	111.02	116.01	125.42
9 0	587	103.50	109.27	114.98	119.84	129.50
9 6	Interpolated.	106.78	112.64	118.21	123.53	133.55
10 0	528	109.97	116.01	121.74	127.22	137.54
10 6	Interpolated.	113.02	119.22	125.11	130.74	141.34
11 0	480	116.00	122.43	128.48	134.27	145.15
11 6	Interpolated.	119.00	125.52	131.73	137.60	148.82
12 0	440	121.93	128.61	134.97	141.05	152.40

TABLE VIII.—For finding the Mean Velocities of Water flowing in Pipes, Drains, Streams, and Rivers.

The hydraulic mean depth is found for all channels by dividing the wetted perimeter into the area.

Hydraulic mean depths 8 feet to 12 feet. Falls per mile 1 inch to 12 feet.

Falls per mile in feet and inches, and the hydraulic inclinations.		"Hydraulic mean depths," or "mean radii," and velocities in inches per second.				
Falls.	Inclinations, one in	96 inches.	108 inches.	120 inches.	132 inches.	144 inches.
0 1	63360	9.18	9.75	10.28	10.79	11.27
0 2	31680	14.32	15.20	16.03	16.82	17.57
0 3	21120	18.30	19.43	20.49	21.50	22.46
0 4	15840	21.69	23.02	24.28	25.47	26.62
0 5	12672	24.70	26.21	27.64	29.00	30.31
0 6	10560	27.43	29.11	30.70	32.21	33.66
0 7	9051	29.96	31.80	33.53	35.18	36.76
0 8	7920	32.32	34.30	36.18	37.96	39.66
0 9	7041	34.55	36.67	38.67	40.58	42.40
0 10	6336	36.67	38.92	41.04	43.07	45.00
0 11	5760	38.69	41.06	43.31	45.44	47.48
1 0	5280	40.63	43.12	45.48	47.72	49.86
1 3	4224	46.04	48.87	51.54	54.07	56.50
1 6	3520	50.98	54.11	57.00	59.87	62.56
1 9	3017	55.60	58.96	62.18	65.25	68.17
2 0	2640	59.85	63.52	66.98	70.28	73.44*
2 3	Interpolated.	63.80	67.72	71.41	74.93*	78.29
2 6	2112	67.76	71.91	75.84*	79.58	83.15
2 9	Interpolated.	71.38	75.75*	79.89	83.83	87.59
3 0	1760	75.00*	79.59	83.94	88.08	92.03
3 3	Interpolated.	78.37	83.17	87.71	92.03	96.16
3 6	1508	81.74	86.75	91.48	95.99	100.30
3 9	Interpolated.	84.88	90.09	95.01	99.69	104.16
4 0	1320	88.03	93.43	98.53	103.33	108.02
4 6	Interpolated.	93.87	99.02	105.06	110.24	115.18
5 0	1056	99.70	105.82	111.59	117.09	122.84
5 6	Interpolated.	105.06	111.49	117.58	123.38	128.91
6 0	880	110.41	117.17	123.57	129.66	135.48
6 6	Interpolated.	115.40	122.47	129.16	135.53	141.61
7 0	754	120.40	127.76	134.75	141.39	147.73
7 6	Interpolated.	125.07	132.74	139.99	146.88	153.47
8 0	660	129.75	137.70	145.22	152.38	159.21
8 6	Interpolated.	134.18	142.40	150.18	157.57	164.64
9 0	587	138.00	147.10	155.13	162.77	170.07
9 6	Interpolated.	142.87	151.63	159.91	167.78	175.31
10 0	528	147.14	156.16	164.68	172.80	180.55
10 6	Interpolated.	151.21	160.48	169.24	177.58	185.55
11 0	480	155.29	164.80	173.80	182.36	190.54
11 6	Interpolated.	159.21	168.97	178.19	186.97	195.36
12 0	440	163.13	173.13	182.59	191.58	200.17

TABLE IX.—For finding the Discharge in Cubic Feet per Minute, when the Diameter of a Pipe, or Orifice, and the Velocity of Discharge are known; and vice versa.

Diameter of Pipe or Orifice in Inches.	Discharge in cubic feet per minute, for different velocities.				
	Velocity of 100 inches per second.	Velocity of 200 inches per second.	Velocity of 300 inches per second.	Velocity of 400 inches per second.	Velocity of 500 inches per second.
1	170.142	340.3	511.3	681.8	852.2
1 1/4	181.77	363.5	545.3	723.7	904.6
1 1/2	193.908	387.8	580.7	765.9	957.4
1 3/4	207.277	413.6	618.2	809.3	1011.6
2	221.108	440.2	657.8	854.0	1067.4
2 1/4	235.501	467.7	698.6	900.0	1124.6
2 1/2	249.107	496.2	740.7	947.3	1183.2
2 3/4	263.081	525.7	784.1	995.8	1243.2
3	277.433	556.2	828.8	1045.5	1304.6
3 1/4	292.107	587.7	874.8	1096.4	1367.4
3 1/2	307.108	619.2	922.1	1148.5	1431.6
3 3/4	322.433	651.7	970.7	1201.8	1497.4
4	338.107	685.2	1020.6	1256.3	1564.6
4 1/4	354.108	719.7	1071.8	1312.0	1633.2
4 1/2	370.433	755.2	1124.3	1368.9	1703.4
4 3/4	387.107	791.7	1178.1	1427.0	1775.2
5	404.108	829.2	1233.2	1486.3	1848.6
5 1/4	421.433	867.7	1289.6	1546.8	1923.6
5 1/2	439.107	906.2	1347.3	1608.5	1999.2
5 3/4	457.108	945.7	1406.3	1671.4	2076.4
6	475.433	986.2	1466.6	1735.5	2155.2
6 1/4	494.107	1027.7	1528.2	1800.8	2235.6
6 1/2	513.108	1070.2	1591.1	1867.3	2317.6
6 3/4	532.433	1113.7	1655.3	1935.0	2399.2
7	552.107	1158.2	1720.8	2003.9	2482.4
7 1/4	572.108	1203.7	1787.6	2074.0	2566.2
7 1/2	592.433	1250.2	1855.7	2145.3	2651.6
7 3/4	613.107	1297.7	1925.1	2217.8	2738.6
8	634.108	1346.2	1995.8	2291.5	2826.2
8 1/4	655.433	1395.7	2067.8	2366.4	2915.4
8 1/2	677.107	1446.2	2140.1	2442.5	3006.2
8 3/4	699.108	1497.7	2213.6	2519.8	3098.6
9	721.433	1550.2	2288.3	2598.3	3192.6
9 1/4	744.107	1603.7	2364.2	2678.0	3288.2
9 1/2	767.108	1658.2	2441.3	2758.9	3385.4
9 3/4	790.433	1713.7	2519.6	2841.0	3484.2
10	814.107	1770.2	2599.1	2924.3	3584.6
10 1/4	838.108	1827.7	2680.8	3008.8	3686.6
10 1/2	862.433	1886.2	2763.7	3094.5	3790.2
10 3/4	887.107	1945.7	2847.8	3181.4	3895.4
11	912.108	2006.2	2933.1	3269.5	4002.2
11 1/4	937.433	2067.7	3020.6	3358.8	4110.6
11 1/2	963.107	2130.2	3109.3	3449.3	4220.6
11 3/4	989.108	2193.7	3199.2	3541.0	4332.2
12	1015.433	2258.2	3290.3	3633.9	4445.4

TABLE IX.—For finding the Discharge in Cubic Feet, per Minute, when the Diameter of a Pipe, or Orifice, and the Velocity of discharge are known; and vice versa.

Discharge in cubic feet per minute, for different velocities.					Diameters of pipes in inches.
Velocity of 600 inches per second.	Velocity of 700 inches per second.	Velocity of 800 inches per second.	Velocity of 900 inches per second.	Velocity of 1000 inches per second.	
1.0227	1.1031	1.3035	1.5340	1.7044	$\frac{1}{4}$
4.0000	4.7724	5.4542	6.1350	6.8177	$\frac{1}{2}$
9.2030	10.7370	12.2718	13.8058	15.3398	$\frac{3}{4}$
16.3025	19.0805	21.8100	24.5437	27.2708	1
25.5064	29.8274	34.0885	38.3495	42.6100	$1\frac{1}{4}$
36.8155	42.0516	49.0874	55.2234	61.3503	$1\frac{1}{2}$
50.1100	58.4017	66.8134	75.1050	83.5167	$1\frac{3}{4}$
65.4400	76.3582	87.2005	98.1748	109.0831	2
82.8350	96.0408	110.4400	124.2525	138.0583	$2\frac{1}{4}$
102.2054	119.3000	136.3538	153.3081	170.4423	$2\frac{1}{2}$
123.7411	144.3046	164.0882	185.0117	206.2352	$2\frac{3}{4}$
147.2621	171.8050	196.3406	220.8033	245.4360	3
172.8285	201.0333	230.4380	259.2428	288.0475	$3\frac{1}{4}$
200.4401	233.8408	267.2535	300.0002	334.0000	$3\frac{1}{2}$
230.0071	268.4407	306.7002	345.1457	383.4052	$3\frac{3}{4}$
261.7004	305.4320	349.0050	392.6001	436.3323	4
295.5470	344.8048	394.0020	443.0205	492.5783	$4\frac{1}{4}$
331.3300	386.5632	441.7805	497.0008	552.2331	$4\frac{1}{2}$
369.0781	430.7077	492.2374	553.7071	615.2068	$4\frac{3}{4}$
409.0016	477.2384	545.4154	613.5023	681.7002	5
450.0004	526.1554	601.3205	676.4855	751.0500	$5\frac{1}{4}$
494.0045	577.4580	659.0520	742.4407	824.0408	$5\frac{1}{2}$
540.0830	631.1470	721.3110	811.4750	901.0300	$5\frac{3}{4}$
589.0480	687.2235	786.3082	883.5730	981.7478	6
639.1587	745.0852	853.2110	958.7331	1065.2045	$6\frac{1}{4}$
691.3141	805.6330	921.7520	1036.0710	1152.1900	$6\frac{1}{2}$
745.5147	869.7072	994.0100	1118.2721	1242.5245	$6\frac{3}{4}$
801.7008	936.3870	1069.0144	1202.0412	1336.2080	7
860.0510	1005.3030	1146.7350	1290.0770	1433.4100	$7\frac{1}{4}$
920.3885	1077.7800	1227.1847	1380.5828	1533.9800	$7\frac{1}{2}$
982.7704	1146.5655	1310.3605	1474.1550	1637.0507	$7\frac{3}{4}$
1047.1070	1221.7305	1396.2034	1570.7004	1745.3203	8
1112.1870	1300.2102	1476.2500	1670.2810	1857.3132	$8\frac{1}{2}$
1175.3595	1380.2528	1560.1400	1773.2810	1970.3132	9
1235.7120	1460.8810	1647.4007	1879.0084	2084.1871	$9\frac{1}{2}$
1293.2403	1540.0540	1737.0017	1987.4304	2200.0771	10
1350.0015	1619.0218	1826.2820	2097.0423	2318.0025	$10\frac{1}{2}$
1405.8580	1697.8343	1916.8100	2208.7870	2438.7633	11
1460.0357	1775.5017	2008.2470	2321.0030	2561.5595	$11\frac{1}{4}$
1512.0140	1852.0307	2101.5028	2434.2910	2686.0910	12

TABLE X.—For finding the depths of Weirs of different lengths,
the quantity discharged over each being supposed constant.
See pages 270 and 271.

Ratio of lengths.	Coeff. vents.	Ratio of lengths.	Coeff. vents.	Ratio of lengths.	Coeff. vents.	Ratio of lengths.	Coeff. vents.
.01	.0101	.105	.5174	.805	.7153	.805	.8051
.02	.0202	.110	.5510	.810	.7193	.810	.8080
.03	.0303	.115	.5801	.815	.7233	.815	.8125
.04	.0404	.120	.6093	.820	.7274	.820	.8161
.05	.0505	.125	.6353	.825	.7310	.825	.8196
.06	.0606	.130	.6607	.830	.7349	.830	.8232
.07	.0707	.135	.6841	.835	.7389	.835	.8267
.08	.0808	.140	.7065	.840	.7427	.840	.8303
.09	.0909	.145	.7280	.845	.7465	.845	.8338
.10	.0951	.150	.7482	.850	.7501	.850	.8373
.11	.0993	.155	.7673	.855	.7539	.855	.8409
.12	.0993	.160	.7850	.860	.7580	.860	.8443
.13	.0993	.165	.8003	.865	.7619	.865	.8478
.14	.0993	.170	.8146	.870	.7657	.870	.8513
.15	.0993	.175	.8283	.875	.7695	.875	.8548
.16	.0993	.180	.8413	.880	.7733	.880	.8583
.17	.0993	.185	.8537	.885	.7771	.885	.8618
.18	.0993	.190	.8655	.890	.7809	.890	.8653
.19	.0993	.195	.8769	.895	.7846	.895	.8687
.20	.0993	.200	.8880	.900	.7881	.900	.8722
.21	.0993	.205	.8983	.905	.7916	.905	.8756
.22	.0993	.210	.9083	.910	.7950	.910	.8791
.23	.0993	.215	.9175	.915	.7984	.915	.8825
.24	.0993	.220	.9268	.920	.8018	.920	.8859
.25	.0993	.225	.9359	.925	.8050	.925	.8893
.26	.0993	.230	.9449	.930	.8081	.930	.8927
.27	.0993	.235	.9537	.935	.8111	.935	.8960
.28	.0993	.240	.9621	.940	.8141	.940	.8993
.29	.0993	.245	.9699	.945	.8170	.945	.9026
.30	.0993	.250	.9773	.950	.8200	.950	.9059
.31	.0993	.255	.9841	.955	.8229	.955	.9092
.32	.0993	.260	.9901	.960	.8257	.960	.9125
.33	.0993	.265	.9953	.965	.8285	.965	.9157
.34	.0993	.270	.9997	.970	.8311	.970	.9189
.35	.0993	.275	1.0033	.975	.8337	.975	.9221
.36	.0993	.280	1.0063	.980	.8362	.980	.9253
.37	.0993	.285	1.0087	.985	.8386	.985	.9285
.38	.0993	.290	1.0109	.990	.8409	.990	.9316
.39	.0993	.295	1.0129	.995	.8431	.995	.9347
.40	.0993	.300	1.0147	1.000	.8453	1.000	.9378

TABLE XI.—Mean relative Dimensions of equally Discharging Trapezoidal Channels, with Side Slopes varying from 0 to 1, up to 2 to 1.

Half sum of the top and bottom is the mean width. The ratio of the slope, multiplied by the depth, subtracted from the mean width, will give the bottom; and if added, will give the top.

TABLE XII. gives the discharge in cubic feet per minute from the primary channel, 70 wide, and the corresponding depths taken in feet. For lesser or greater channels and discharges, see Rules, pp. 227, 229, 232, 249, and 252.

The mean widths are given in the top horizontal line, and the corresponding depths in the other horizontal lines. They may be taken in inches, feet, yards, fathoms, or any other measure whatever.									
70	60	50	40	35	30	25	20	15	10
.125	.13	.15	.17	.20	.23	.26	.29	.35	.48
.25	.27	.30	.35	.40	.45	.52	.58	.71	.98
.375	.41	.46	.51	.60	.67	.76	.88	1.09	1.51
.5	.55	.62	.73	.80	.89	1.02	1.19	1.48	2.04
.625	.68	.78	.91	1.00	1.12	1.29	1.50	1.88	2.62
.75	.82	.94	1.10	1.20	1.35	1.56	1.82	2.28	3.22
.875	.96	1.10	1.29	1.41	1.58	1.83	2.14	2.69	3.86
1.	1.10	1.26	1.48	1.62	1.81	2.10	2.46	3.11	4.50
1.125	1.24	1.42	1.67	1.89	2.04	2.37	2.70	3.54	5.19*
1.25	1.39	1.58	1.86	2.04	2.28	2.65	3.12	3.98	5.89
1.375	1.53	1.74	2.05	2.25	2.51	2.92	3.46	4.48	6.60
1.5	1.67	1.90	2.24	2.46	2.75	3.20	3.80	4.88	7.31
1.625	1.81	2.06	2.43	2.67	2.99	3.47	4.15	5.31	8.08
1.75	1.95	2.22	2.62	2.88	3.23	3.75	4.50	5.80	8.86
1.875	2.09	2.38	2.81	3.09	3.47	4.03	4.86	6.29	9.68
2.	2.23	2.51	3.00	3.31	3.72	4.32	5.22	6.78	10.60
2.125	2.37	2.70	3.19	3.52	3.96	4.61	5.58	7.29	11.37
2.25	2.51	2.86	3.38	3.73	4.21	4.91	5.95	7.81*	12.25
2.375	2.65	3.02	3.57	3.91	4.45	5.20	6.31	8.32	13.12
2.5	2.79	3.18	3.76	4.16	4.70	5.50	6.68	8.84	14.00
2.625	2.93	3.34	3.95	4.38	4.95	5.79	7.06	9.38	14.92
2.75	3.07	3.51	4.15	4.60	5.21	6.09	7.45	9.93	15.84
2.875	3.21	3.67	4.34	4.82	5.46	6.39	7.83	10.48	16.76
3.	3.35	3.84	4.54	5.04	5.72	6.69	8.22	11.03	17.68
3.125	3.49	4.03	4.73	5.26	5.97	7.00	8.62	11.60	18.68
3.25	3.63	4.17	4.93	5.49	6.23	7.31	9.02	12.17	19.68
3.375	3.77	4.33	5.13	5.72	6.49	7.62	9.42	12.74	20.68
3.5	3.91	4.50	5.33	5.95	6.75	7.93	9.82	13.32	21.68
3.625	4.05	4.66	5.53	6.17	7.01	8.25	10.23*	13.92	22.76
3.75	4.19	4.82	5.73	6.40	7.28	8.57	10.65	14.53	23.84
3.875	4.33	4.98	5.93	6.62	7.54	8.89	11.06	15.14	24.92
4.	4.48	5.14	6.13	6.85	7.81	9.21	11.48	15.75	26.00
4.125	4.63	5.30	6.34	7.09	8.08	9.55	11.93	16.38	27.18
4.25	4.77	5.46	6.54	7.35	8.36	9.90	12.39	17.02	28.36
4.375	4.91	5.62	6.75	7.60	8.65	10.26	12.86	17.68	29.68
4.5	5.05	5.79	6.95	7.85	8.95	10.63	13.34	18.34	31.00
4.625	5.19	5.95	7.15	8.10	9.26	11.01	13.83	19.02	32.40
4.75	5.33	6.12	7.35	8.36	9.58	11.40	14.33	19.71	33.84
4.875	5.48	6.29	7.56	8.63	9.91	11.80	14.84	20.42	35.36
5.	5.62	6.46	7.76	8.90	10.25	12.21	15.36	21.14	36.96
5.125	5.77	6.63	7.97	9.17	10.60	12.63	15.89	21.88	38.64
5.25	5.91	6.80	8.18	9.45	10.96	13.06	16.43	22.64	40.40
5.375	6.05	6.97	8.39	9.73	11.33	13.44	16.98	23.42	42.24
5.5	6.19	7.14	8.60	10.01	11.71	13.83	17.54	24.22	44.16
5.625	6.33	7.31	8.81	10.29	12.10	14.23	18.11	25.04	46.16
5.75	6.48	7.48	9.02	10.58	12.50	14.64	18.69	25.88	48.24
5.875	6.62	7.65	9.23	10.87	12.91	15.06	19.28	26.74	50.40
6.	6.77	7.82	9.44	11.16	13.33	15.49	19.88	27.62	52.64
6.125	6.91	8.00	9.65	11.45	13.75	15.93	20.49	28.52	54.96
6.25	7.05	8.17	9.86	11.74	14.18	16.38	21.11	29.44	57.36
6.375	7.19	8.34	10.07	12.03	14.61	16.84	21.74	30.38	59.84
6.5	7.33	8.51	10.28	12.32	15.05	17.31	22.38	31.34	62.40
6.625	7.48	8.68	10.49	12.61	15.48	17.79	23.03	32.32	65.04
6.75	7.62	8.85	10.70	12.90	15.92	18.28	23.69	33.32	67.76
6.875	7.77	9.02	10.91	13.19	16.36	18.78	24.36	34.34	70.56
7.	7.91	9.19	11.12	13.48	16.81	19.29	25.04	35.38	73.44
7.125	8.05	9.36	11.33	13.77	17.26	19.81	25.73	36.44	76.40
7.25	8.19	9.53	11.54	14.06	17.69	20.34	26.43	37.52	79.44
7.375	8.33	9.70	11.75	14.35	18.13	20.87	27.14	38.62	82.56
7.5	8.48	9.87	11.96	14.64	18.58	21.41	27.86	39.74	85.76
7.625	8.62	10.04	12.17	14.93	19.03	21.95	28.58	40.88	89.04
7.75	8.77	10.21	12.38	15.22	19.49	22.50	29.31	42.04	92.40
7.875	8.91	10.38	12.59	15.51	19.95	23.06	30.05	43.22	95.84
8.	9.05	10.55	12.80	15.80	20.42	23.62	30.80	44.42	99.36
8.125	9.19	10.72	13.01	16.09	20.89	24.19	31.56	45.64	102.96
8.25	9.33	10.89	13.22	16.38	21.37	24.77	32.33	46.88	106.64
8.375	9.48	11.06	13.43	16.67	21.85	25.36	33.11	48.14	110.40
8.5	9.62	11.23	13.64	16.96	22.34	25.96	33.90	49.42	114.24
8.625	9.77	11.40	13.85	17.25	22.83	26.57	34.70	50.72	118.16
8.75	9.91	11.57	14.06	17.54	23.33	27.19	35.51	52.04	122.16
8.875	10.05	11.74	14.27	17.83	23.83	27.81	36.33	53.38	126.24
9.	10.19	11.91	14.48	18.12	24.34	28.44	37.16	54.74	130.40
9.125	10.33	12.08	14.69	18.41	24.85	29.08	37.99	56.12	134.64
9.25	10.48	12.25	14.90	18.70	25.37	29.73	38.83	57.52	138.96
9.375	10.62	12.42	15.11	18.99	25.89	30.39	39.68	58.94	143.36
9.5	10.77	12.59	15.32	19.28	26.42	31.06	40.54	60.38	147.84
9.625	10.91	12.76	15.53	19.57	26.95	31.74	41.41	61.84	152.40
9.75	11.05	12.93	15.74	19.86	27.49	32.43	42.29	63.32	157.04
9.875	11.19	13.10	15.95	20.15	28.03	33.13	43.18	64.82	161.76
10.	11.33	13.27	16.16	20.44	28.58	33.84	44.08	66.34	166.56

TABLE XII.—Discharges from the Primary Channel in the first column of Table XI.

If the dimensions of the primary channel be in inches, divide the discharges in this table by 500; if in yards, multiply by 1.66; if in quarters, multiply by 32; and if in fathoms, by 88.2, &c.: see pp. 233, 234. The final figures in the discharges may be rejected when they do not exceed one-half per cent., or 0.5 in 100. See pages 226 to 234.

Depth of a channel whose mean width is 70 ft.—in feet.	Falls, in feet, and discharges in cubic feet per minute. Interpolate for intermediate falls; divide greater falls by 4, and double the corresponding discharges.						
	1 inch per mile, 1 in 63360.	2 inches per mile, 1 in 31680.	3 inches per mile, 1 in 21120.	4 inches per mile, 1 in 15840.	5 inches per mile, 1 in 12672.	6 inches per mile, 1 in 10560.	7 inches per mile, 1 in 8820.
1.25	47	72	93	139	176	206	239
2.5	136	210	268	403	506	596	676
3.75	249	389	498	746	940	1106	1252
5.0	387	603	770	1155	1454	1709	1935
6.25	541	849	1078	1617	2036	2396	2714
7.5	714	1113	1420	2128	2681	3153	3573
8.75	906	1401	1791	2685	3382	3978	4507
10	1100	1714	2190	3293	4134	4862	5507
11.25	1310	2012	2614	3909	4927	5793	6577
12.5	1534	2334	3053	4581	5766	6780	7690
13.75	1767	2757	3621	5279	6661	7823	8863
15.0	2013	3143	4008	6010	7588	8916	10099
16.25	2268	3540	4525	6781	8541	10014	11381
17.5	2531	3950	5053	7570	9537	11210	12703
18.75	2812	4384	5599	8386	10570	12429	14088
20	3090	4821	6161	9230	11628	13676	15513
21.25	3377	5279	6733	10092	12718	14956	16943
22.5	3674	5730	7331	10981	13833	16281	18435
23.75	3977	6210	7937	11889	14981	17646	19980
25.0	4293	6699	8563	12829	16161	19046	21531
26.25	4616	7203	9201	13800	17380	20484	23135
27.5	4947	7716	9865	14783	18624	21980	24800
28.75	5280	8233	10556	15773	19887	23560	26473
30	5621	8762	11264	16788	21166	24833	28176
31.25	5973	9310	11990	17830	22454	26410	29925
32.5	6329	9862	12614	18897	23780	27994	31714
33.75	6689	10420	13220	19963	25145	29570	33507
35.0	7049	10996	13818	21052	26509	31262	35329
36.25	7418	11574	14486	22153	27906	32860	37186
37.5	7794	12163	15136	23284	29331	34479	39080
38.75	8178	12753	15783	24440	30766	36170	41013
40	8566	13354	16470	25622	32226	37898	42954
41.25	8956	13962	17143	26830	33719	39663	44916
42.5	10173	15849	20267	30306	38254	44989	50973
43.75	11001	17140	21903	32818	41356	48630	55102
45	11833	18454	23595	35355	44546	52378	59346
46.25	12690	19802	25362	37939	47796	56203	63688
47.5	13576	21172	27218	40564	51097	60079	68097
48.75	14478	22580	29160	43253	54478	64058	72591
50	15393	23995	31122	46000	57807	68082	77154

TABLE XII.—Discharges from the Primary Channel in the first column of Table XI.

If the dimensions of the primary channel be in inches, divide the discharges in this table by 500; if in yards, multiply by 15.6, if in quarters, multiply by 32, and if in fathoms, by 88.2 etc.: see pp. 233 and 234. The final figures in the discharges may be rejected when they do not exceed one-half per cent., or 0.5 in 100. See pages 226 to 234.

Falls, inclinations, and discharges in cubic feet per minute. Interpolate for intermediate falls; divide greater falls by 4, and double the corresponding discharges.							Depth of a channel whose mean width is 70 ft.—in feet.
18 inches per mille, 1 in 3520.	21 inches per mille, 1 in 3017.	24 inches per mille, 1 in 2510.	27 inches per mille, 1 in 2347.	30 inches per mille, 1 in 2112.	33 inches per mille, 1 in 1920.	36 inches per mille, 1 in 1760.	
258	281	303	323	343	362	380	1.25
748	816	877	930	983	1040	1100	2.25
1387	1511	1627	1736	1843	1952	2037	3.75
2145	2330	2515	2684	2852	3023	3155	5.50
3004	3274	3527	3753	4021	4207	4414	6.25
3957	4311	4645	4966	5287	5553	5817	7.75
4991	5422	5850	6274	6650	6992	7342	8.75
6007	6523	7150	7631	8107	8540	8974	1.
7200	7920	8531	9124	9660	10200	10693	1.125
8514	9284	9995	10658	11318	11923	12520	1.25
9810	10607	11530	12307	13045	13741	14479	1.375
11182	12185	13152	14007	14832	15650	16448	1.50
12601	13730	14821	15780	16750	17657	18552	1.625
14060	15331	16526	17610	18700	19698	20696	1.75
15563	16907	18200	19417	20728	21840	22944	1.875
17157	18607	20141	21460	22803	24017	25242	2.
18760	20440	22030	23480	24938	26260	27601	2.125
20410	22247	23905	25547	27120	28578	30027	2.25
22104	24087	25947	27662	29395	30934	32512	2.375
23848	25988	27992	29841	31701	33381	35096	2.50
25600	27953	30100	32000	34086	35910	37725	2.625
27470	29933	32247	34384	36512	38471	40415	2.75
29318	31947	34408	36607	38658	41056	43135	2.875
31206	34002	36624	39050	41404	43680	45806	3.
33141	36112	38807	41482	44048	46308	48747	3.125
35120	38200	41223	43954	46072	49174	51664	3.25
37100	40438	43550	46438	49330	51951	54580	3.375
39140	42631	45925	48963	51993	54775	57550	3.50
41184	44872	48343	51537	54728	57659	60580	3.625
43273	47158	50807	54102	57514	60585	63650	3.75
45407	49468	53300	56840	60341	63560	66784	3.875
47551	51818	55832	59514	63200	66570	69951	4.
49711	54580	60073	64074	66013	72694	76383	4.25
50448	61508	66176	70023	75017	79017	82994	4.50
61014	66500	71025	76108	81097	85420	89707	4.75
65713	71028	77140	82250	87351	92015	96653	5.
70500	76803	82770	88200	93731	98720	103745	5.25
75383	82150	88434	94344	100200	105550	110905	5.50
80370	87500	94348	100610	106823	112540	118254	5.75
85407	93003	100275	106911	113505	119616	125664	6.

TABLE XIII.—The Square Roots of the Fifth powers of numbers for finding the Diameter of a Pipe, or dimensions of a Channel from the Discharge, or the Reverse; showing the relative Discharging Powers of pipes of different Diameters, and of any similar Channels whatever, closed or open. See pages 31, 230, 233, etc.

If d be the diameter of a pipe, in feet, and n the discharge in cubic feet per minute, then for long straight pipes we shall have for velocity of nearly 3 feet per second, $n = 2400 (d^5 s)^{\frac{1}{5}}$, and $d = 0.44 \left(\frac{n^5}{s} \right)^{\frac{1}{5}}$; or if n be the discharge per second, $n = 40 (d^5 s)^{\frac{1}{5}}$, and $d = 0.28 \left(\frac{n^5}{s} \right)^{\frac{1}{5}}$. See pages 100 to 234, and pages 42 and 43.

Relative dimen- sions or diameters of pipes.	Relative discharg- ing powers.	Relative dimen- sions or diameters of pipes.	Relative discharg- ing powers.	Relative dimen- sions or diameters of pipes.	Relative discharg- ing powers.	Relative dimen- sions or diameters of pipes.	Relative discharg- ing powers.
0.25	0.031	10.5	357.9	30.5	5138	61	20002
0.5	0.157	11	401.2	31	5351	62	20208
0.75	0.185	11.5	449.5	31.5	5500	63	21603
1	1	12	498.0	32	5733	64	22768
1.25	1.717	12.5	552.4	32.5	5923	65	24063
1.5	2.756	13	609.3	33	6150	66	25388
1.75	4.051	13.5	669.6	33.5	6400	67	26744
2	6.657	14	733.4	34	6671	68	28131
2.25	7.591	14.5	800.6	34.5	6961	69	29548
2.5	9.882	15	871.4	35	7277	70	30996
2.75	12.541	15.5	945.0	35.5	7609	71	32476
3	15.588	16	1024	36	7956	72	33988
3.25	19.042	16.5	1105.9	36.5	8319	73	35531
3.5	22.916	17	1191.0	37	8697	74	37108
3.75	27.222	17.5	1281.1	37.5	9091	75	38714
4	32	18	1374.6	38	9501	76	40351
4.25	37.21	18.5	1472.1	38.5	9927	77	42027
4.5	42.96	19	1573.6	39	10369	78	43732
4.75	49.17	19.5	1679.1	39.5	10826	79	45471
5	55.90	20	1788.9	40	11119	80	47243
5.25	63.16	20.5	1902.8	41	11761	81	49049
5.5	70.94	21	2020.9	42	12432	82	50888
5.75	79.28	21.5	2143.1	43	13135	83	52762
6	88.18	22	2270.2	44	13862	84	54669
6.25	97.66	22.5	2401.4	45	14614	85	56611
6.5	107.72	23	2537	46	15391	86	58688
6.75	118.38	23.5	2677.1	47	16194	87	60899
7	129.64	24	2821.8	48	17023	88	63145
7.25	141.58	24.5	2971.1	49	17877	89	65427
7.5	154.05	25	3125	50	18758	90	67849
7.75	167.31	25.5	3283.9	51	19665	91	70306
8	181.02	26	3448.9	52	20600	92	72801
8.25	195.50	26.5	3617.1	53	21560	93	75338
8.5	210.61	27	3788	54	22549	94	77918
8.75	226.48	27.5	3962.9	55	23564	95	80535
9	243	28	4140.6	56	24608	96	83198
9.25	260.29	28.5	4320.2	57	25679	97	85908
9.5	278.17	29	4503.0	58	26780	98	88676
9.75	296.83	29.5	4688.7	59	27910	99	91510
10	316.23	30	4879.6	60	29069	100	100000

TABLE XIV.—Weights and Measures, English and French, with their relative values.

MEASURES OF LENGTH.

12 inches	1 foot.
7.92 inches	1 link.
3 feet	1 yard.
$5\frac{1}{2}$ yards = $10\frac{1}{2}$ feet	1 pole or perch.
100 links = 22 yards...	1 chain = 4 perches.
40 perches = 220 yards...	1 furlong.
8 furlongs = 1760 yards...	1 mile.
6 feet	1 fathom.
120 fathoms	1 cable's-length.
1 Nautical mile	6082.7 feet.
69.12 miles	1 Geographical deg.
3 miles	1 league.

The Irish perch is 21 feet, or seven yards. Three inches make a palm; 4 inches a hand; 5 feet a pace. In cloth measure $2\frac{1}{2}$ inches = 1 nail; 4 nails = 1 quarter; 4 quarters 1 yard. 11 Irish miles = 14 English.

MEASURES OF SURFACE.

144 square inches	1 square foot.
62.7264 „	1 square link.
9 square feet	1 square yard.
$80\frac{1}{2}$ square yards = $272\frac{1}{2}$ square feet	1 square perch.
10,000 square links = 4,850 „	1 square chain.
10 square chains = 160 square perches	1 acre.
1 rood = 210 square yards	40 perches.
4 roods = 4,840 „	1 acre.
640 acres = 3,007,600 „	1 square mile.

The Irish perch is 40 square yards, or 441 square feet; 1 Irish acre = 1a. 2r. 19.17p. statute; and 1 statute acre = 0a. 2r. 18.77p. Irish. The Irish acre is to the English acre as 100 is to 121. 100 square feet, is a square of roofing, slating, or flooring. The Cunningham acre is = 1a. 1r. 0.01p. English; and 1 English acre is = 0a. 3r. 3.004p. Cunningham measure.

CUBIC MEASURES, AND MEASURES OF CAPACITY AND WEIGHT.

1728 cubic inches	1 cubic foot.
27 cubic feet	1 cubic yard.
$10\frac{1}{2} \times 1\frac{1}{2} \times 1$	$\approx 24\cdot75$ cubic feet	1 perch of masonry
$10\frac{1}{2} \times 10\frac{1}{2} \times 1\frac{1}{2}$	≈ 300 cubic feet	1 rod of brickwork,
$21 \times 1\frac{1}{2} \times 1$	$\approx 30\frac{1}{2}$ cubic feet	1 Irish perch of masonry

The standard gallon, imperial measure, contains 10 lbs. avoirdupois, of distilled water at 62° Fahrenheit, the barometer standing at 30 inches.

6·232 gallons	1 cubic foot.
8·006 cubic inches	1 gill.
4 gills 34·060 cubic inches	1 pint.
2 pints 68·318 cubic inches	1 quart.
2 quarts 136·637 cubic inches	1 pottle.
2 pottles 273·274 cubic inches	1 gallon.
2 gallons 546·548 cubic inches	1 peck.
4 pecks 1093·096 cubic inches	1 bushel.

The old Irish gallon contained 217·6 cubic inches, nearly, and 1 Irish gallon is therefore ≈ 7850 imperial gallons. The Irish barrel of lime still measures 40 Irish gallons, or 31·321 imperial gallons, or 4 bushels, very nearly. It is measured by a cylindrical measure 12 inches high, and about $21\frac{1}{2}$ inches in diameter, containing half the Irish barrel. In the old English liquid measures for ale and beer, 36 gallons \approx 1 barrel \approx 36 gallons, $9\frac{1}{2}$ quarts, imperial measure, nearly.

For old dry measures, 32 bushels \approx 1 chaldron \approx 31 bushels, 1 pint, imperial measure, nearly.

And 36 bushels of coal \approx 1 chaldron of coal \approx 34 bushels 3 pecks, and 1 gallon, imperial measure. The Irish barrel of wheat is 20 stone; barley 16 stone; oats 14 stone.

TROY WEIGHT.

24 grains	1 pennyweight.
20 pennyweights	1 ounce
12 ounces	1 pound.

One pound Troy \approx 29·816 cubic inches of distilled water, barometer 30 inches; thermometer 62°.

APOTHECARY'S WEIGHT.

20 Troy grains...	1 scruple.
3 scruples	1 drachm.
8 drachms	1 ounce.
12 ounces	1 pound.

The ounce weighs 480 grains, and the pound 5760 grains, both in Troy and Apothecary's weight.

AVOIRDUPOIS OR COMMERCIAL WEIGHT.

One pound Avoirdupois = 27.7274 cubic inches, when the barometer stands at 30 inches, and Fahrenheit's thermometer at 62°.

16 drachms =	437.5 Troy grains...	...	1 ounce
16 ounces =	7,000 Troy grains...	...	1 pound
14 pounds =	98,000 Troy grains...	...	1 stone
8 stone =	112 pounds	...	1 cwt.
20 cwt. =	2,240 pounds	...	1 ton

One pound Troy = .82286 pounds Avoirdupois, and one pound Avoirdupois, is equal to 1.2158 pounds Troy. One ton of water contains about 36 cubic feet, equal to 224 imperial gallons, nearly. Ten pounds of distilled water is equal to one gallon, the Barometer and Thermometer being as above stated.

FRENCH MEASURES AND WEIGHTS COMPARED WITH ENGLISH.

MEASURES OF LENGTH.

1 mètre	3.2808902 feet	1 foot English ..	.3047945 mètre
1 décimètre3280800 "	1 inch0253995 "
1 centimètre0328000 "	1 yard9143835 "
1 millimètre0032800 "	1 perch, 5½ yds.	5.0201002 "
1 kilomètre (or 1000 mètres)	.621383 mile	1 mile	1.60932 kilomètre

1000 mètres = 100 décamètres = 10 hectomètres = 1 kilomètre = 3280.849 feet. The mètre is the 10,000,000th part of a quadrant arc of the meridian or 39.3708 inches English.

MEASURES OF SURFACE

1 centiare (one square mètre) ..	107.6433 sq. ft.	119.6003 sq. yds. ..	1 are
1 deciare ..	10.76433 "	11.96003 "	1 deciare
1 are ..	1076.433 "	1.1960 "	{ 1 centiare or sq. mètre.

100 ares = 10 deciares; and 1 hectare = 24711.43 English acres, and 17 hectares are nearly equal to 42 English acres.

The old Paris foot is equal 1.06578 English feet; the French inch = 1.06578 English inches; the French line = .08832 of an English inch; the toise is equal to 6 French feet = 5.45533 English inches = 6.39448 feet. The perche is 18 French feet; and the perche royal 22 French feet. The French square foot or inch = 1.13581 English square feet or inches, and the cubic foot or inch = 1.21081 English.

MEASURES OF SOLIDITY AND CAPACITY.

1 millistere ..	.0853100 cubic ft.	1 millilitre ..	.0010279 { Eng. cub. inches.
1 centistere ..	.853100 "	1 centilitre ..	.010279 "
1 decistere ..	8.53100 "	1 decilitre ..	.10279 "
1 stère (one cubic mètre) }	35.2100 "	1 litre ..	.010279 "
1 decastère ..	3.52100 "	1 decalitre ..	.10279 "
1 hectostère ..	35.2100 "	1 hectolitre ..	1.0279 "
1 kilostère ..	352.100 "	1 kilolitre ..	10.279 "

The stère and kilolitre are each a cubic mètre, and the litre is a cubic decimètre; 50 litres are nearly 11 English gallons, and 1 hectolitre 2.751207 English bushels.

MEASURES OF WEIGHT.

.0048 grammes = 1 grain, and 7000 grammes = 1 lb. Avoirdupois.

1 milligramme ..	.015432 grains	1 gramme ..	15.432 grains
1 centigramme ..	.15432 "	1 decigramme ..	.02205 lb. avoird.
1 decigramme ..	1.5432 "	1 hectogramme ..	.2205 "
1 gramme ..	15.432 "	1 kilogramme ..	2.2046 "

1.01605 tonnes = 1 ton; and 1 tonne = .984206 ton.

A gramme is the weight of a cubic centimetre of water and its maxim. density in vacuo, 1 kilogramme = 2.2046 lbs. Troy = 2.2046 lbs. Avoirdupois. 1 metrical quintal 220.46 lbs. Avoirdupois, and 10 quintals is equal to the weight of a cubic mètre of water.

TABLE XV. Showing the Weight, Specific Gravity, strength and elasticity of various materials employed by the Physicist and Engineer. When used by the Engineer only about one-fourth of the ultimate strengths here given should be calculated from.

MATERIALS.	Modull of Rupture.	Modull of Elasticity.	Crushing force per sq. inch, in lbs.	Tensilities per sq. in. in lbs.	Weights of a cubic foot in lbs.	specific gravi- ties.
Acacia, English Growth ..	11,200	1,150,000	..	16,000	44.8	.71
Ash	12,000	1,000,000	9,000	17,000	48.0	.77
Brass, Cast	8,000,000	10,500	18,000	525.0	8.40
Beech	9,300	1,850,000	8,500	16,000	48.0	.77
Brick, Red	800	280	135.5	2.20
Brickwork	112.5	1.80
Do. Pale Red	550	900	130.9	2.08
Cedar, American, Fresh	490,000	5,600	11,400	56.8	0.91
Do. do. Seasoned	4,000	..	47.0	0.75
Copper, Cast	10,000	538.0	8.61
Do. Sheet	30,000	549.0	8.80
Do. Wire-drawn	60,000	560.0	8.88
Deal, Christiana	9,000	1,070,000	..	12,400	49.6	0.70
Do. Memel	10,400	1,530,000	37.0	0.60
Do. Norway Spruce	17,000	21.2	0.34
Elm, Seasoned	6,100	700,000	10,300	13,500	36.8	0.59
Fir, New England	6,000	2,190,000	..	10,000	34.5	0.55
Do. Riga	7,600	1,100,000	6,100	12,000	47.0	0.75
Glass	8,000,000	38,000	2,400	159.3	2.45
Iron, Wrought, English	57,000	481.2	7.70
Do. in Bars	57,000	487.0	7.80
Do. rolled in Sheets and Riveted	31,000	487	7.8
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