THE PRECISE CALCULATION OF PIPE DRAIN AND SEWER DIMENSIONS

FOR USE IN

WATER SUPPLY, DRAINAGE, &c.

C. E. HOUSDEN

ECONOMIC WATER SUPPLY AND DRAINAGE

THE PRECISE
AND THEREFORE ECONOMIC
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OF
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PREFACE

THIS small work aims at providing and explaining the use of a short series of Hydraulic Tables (based on a careful comparison of all available coefficients) and some good drain and sewer designs to which the Tables apply, wherefrom engineers, contractors, and others interested in the 'supply of water' or the 'drainage of land' can, adopting any desired coefficient, rapidly, confidently and accurately ascertain the *safe minimum* dimensions, and therefore *the lowest reliable cost* of the pipes, drains and sewers required for such purposes.

C. E. H.

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CHAPTER I

THE TABLES

The preparation and scope of the Tables.—Some of the accompanying Tables have been framed and are applied on the same principles as the author's 'Practical Hydraulic Tables and Diagrams' (Longmans, Green & Co., 1907), to which they are a self-contained independent supplement, adding to, revising and simplifying the more useful original Tables in the light of the knowledge and experience gained in their practical application and use. The remainder extend their scope.

The dimensions of pipes of several useful types of masonry or concrete drains and sewers and of drains in earth can, it will be found, be easily, neatly and accurately ascertained from the complete series, utilising to the full all available fall, and adopting at will a fair selection of generally accepted coefficients used by Kutter, Unwin, Fanning, Bazin, &c.

The improved Tables are of special use in the precise determination of drain and sewer dimensions, the calculation of which is by no means a simple matter when, as is usually the case, only the required discharge and the available slope in the water surface are known.

A table of squares and square roots will be found of much assistance in their application. (See Appendix F.)

2. Hydraulic formulæ applicable to pipes and channels.—The general formulæ for ascertaining the flow of water in pipes and channels, on which the Tables are mainly based, are :

(a) $$F = A v \quad . \quad . \quad . \quad . \quad \text{(i)}$$

where

(i) F is the discharge required, or under supply, in a pipe or channel in *cubic feet per second* (cusecs);

(ii) A is the water area of a pipe or channel in *square feet*; and

(iii) v is the mean velocity of flow *in feet per second.*

(b) $$v = C\sqrt{R}\sqrt{S} \quad . \quad . \quad . \quad \text{(ii)}$$

where

(i) R is 'the hydraulic mean radius' of a pipe or the 'hydraulic mean depth' of a channel,

i.e. $$R = \frac{\text{the water area in square feet}}{\text{the wetted perimeter in feet}} = \frac{A}{P} \quad . \quad \text{(iii)}$$

(ii) S (the 'hydraulic gradient' or 'virtual slope' of a pipe or channel) is the sine of the inclination, or fall per unit of length, of the water surface, practically :

$$S = \frac{\text{available head in feet}}{\text{the length of the pipe or channel in feet}} = \frac{H}{L} \quad . \quad \text{(iv)}$$

(iii) C is a coefficient derived from experiment, and depending mainly on the roughness of the interior surface, but also to some extent on R and S.

3. **Development of the formulæ.**—Squaring formula (i) we have—

$$F^2 = A^2 v^2$$

or as $v = C\sqrt{R}\sqrt{S}$ (formula (ii))

$$\therefore F^2 = A^2 C^2 R S = A^2 C^2 R \frac{H}{L}$$

whence $$\frac{L}{H} F^2 = A^2 C^2 R \quad . \quad . \quad . \quad . \quad \text{(v)}$$

Therefore for pipes, for which $A = \frac{\pi D^2}{4}$ and $R = \frac{D}{4}$ (D being the diameter of the pipe *in feet*)—

$$\frac{L}{H} F^2 = \frac{\pi^2 D^4}{16} \times C^2 \times \frac{D}{4}$$

$$= C^2 \times \frac{\pi^2}{64} \times D^5 \quad . \quad . \quad . \quad . \quad . \quad \text{(vi)}$$

or with the diameter of the pipe expressed *in inches* (*d*)—

$$\frac{L}{H} F^2 = C^2 \times 0{\cdot}155 \times \left(\frac{d}{12}\right)^5 \quad . \quad . \quad . \quad \text{(vii)}$$

(whence

$$\frac{4L}{H} F^2 = C^2 \times 0{\cdot}155 \times \left(\frac{d}{12}\right)^5 \text{ for half pipes} \quad . \quad \text{(vii}_A\text{))}$$

As 1 cusec is equal to $6{\cdot}25 \times 60 = 375$ gallons per minute (*galmins* as an abbreviation = say G)—

$$\frac{L}{H} G^2 = C^2 \times 0{\cdot}155 \times 140625 \times \left(\frac{d}{12}\right)^5$$

$$= \frac{C^2 d^5}{11{\cdot}5} \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(viii)}$$

(whence

$$\frac{4L}{H} G^2 = \frac{C^2 d^5}{11{\cdot}5} \text{ for half pipes} \quad . \quad . \quad . \quad \text{(viii}_A\text{))}$$

4. **Values of C, how ascertained.**—Some generally accepted values of C can be obtained from—

(a) Kutter's formula—

$$C = \frac{41{\cdot}6 + \frac{1{\cdot}811}{n} + \frac{0{\cdot}00281}{S}}{1 + \left(41{\cdot}6 + \frac{0{\cdot}00281}{S}\right)\frac{n}{\sqrt{R}}} \quad . \quad . \quad \text{(ix)}$$

in which

$n = 0{\cdot}010$ for pure cement plaster, coated clean pipes.

$n = 0{\cdot}011$ for mixed cement plaster, clean pipes in best order.

$n = 0{\cdot}013$ for ashlar concrete and brickwork, pipes in ordinary condition.

$n = 0{\cdot}015$ for rough brickwork, incrusted iron.

$n = 0{\cdot}025$ for rivers and canals in good order.

(b) Values of ζ for clean coated and rusted iron pipes. formulated in Tables in Professor Unwin's 'A Treatise on Hydraulics' (A. & C. Black, London) and used in the formula:

$$C = \sqrt{\frac{2g}{\zeta}} \text{ (in which } 2g = 64{\cdot}4) \quad . \quad . \quad \text{(x)}$$

(c) Values of C for clean pipes and for channels in 'A Treatise on Water Supply and Hydraulic Engineering' (J. T. Fanning: D. Van Nostrand, New York).

In all the above cases the values of C depend on the velocity in and consequently on the 'hydraulic gradient' or 'virtual slope' of a particular pipe or channel as well as on its hydraulic mean radius or depth.

The values of C and consequently of $\frac{L}{H}F^2$ therefore vary to some extent *with the slope*. (See Table I.)

A fixed value of $C = 91{\cdot}6$ for clean pipes can be deduced from Box's formula $\frac{L}{3} = \frac{(3d)^5 H}{G^2}$, whence $\frac{L}{H}G^2 = 3\,(3d)^5 = 729d^5$ ('Practical Hydraulics,' Thomas Box: E. & F. N. Spon, London), and a fixed value of $C = 76{\cdot}2$ for rusted iron pipes from the formula $H = \frac{LF^2}{900D^5}$, whence $\frac{L}{H}F^2 = 900\,D^5$, used by A. E. Silk in the preparation of his 'Tables for calculating the Discharge of Water in Pipes' (E. & F. N. Spon, London).

(d) Bazin's values of C (Unwin) for—

(i) Canals in earth newly dressed, which are:—

If $R = 1$, $C = 62{\cdot}1$. If $R = 2$, $C = 75{\cdot}5$.
If $R = 3$, $C = 83{\cdot}6$. If $R = 4$, $C = 89{\cdot}1$.

(ii) Ordinary earth canals, which are :—

If R = 1, C = 47. If R = 2, C = 59·1.

If R = 3, C = 66·8. If R = 4, C = 72·3.

5. **The Tables shortly described.**—*Tables I and II*, for use in obtaining the dimensions of pipes and of masonry or concrete drains and sewers, have both been framed from Kutter's values of C with $n = 0·013$ and $n = 0·011$ (very nearly) after a careful comparison of the values of C calculated by all the above-mentioned methods.

From the comparisons so made it has been ascertained that for pipes over 6 in. in diameter (dealt with in Table I) the values of $\frac{L}{H}F^2$ (formula (vii)) are, using Unwin's values of C for clean pipes, for all practical purposes the same as those obtained from Kutter's coefficients with $n = 0·013$ [equivalent closely to Bazin's and Fanning's values of C for planks, ashlar, concrete, and brick] for a 1 in. larger diameter (Appendix A), and that the values of $\frac{L}{H}F^2$ with $n = 0·011$ (Kutter) are further very nearly the same as those obtained from Unwin's values of C for asphalted pipes (which differ but little from his values of $\frac{L}{H}F^2$ for clean pipes) and from Fanning's values of C for clean pipes (Appendix B), also that the additions to be made to clean pipe diameters to allow for eventual incrustation when needed (ascertained as in Appendix C) are those shown in col. III of Table I.

These additions are found to increase uniformly with increase in diameter and are clearly due to eddies, which considerably retard velocity, produced by a roughened interior, and not to such actual extensive reductions in pipe diameters.

For pipes under 6 in. in diameter, the comparison of the values of $\frac{L}{H}G^2$ (formula (viii)) with Kutter's $n = 0·013$ and Fanning's and Box's values of C (roughly equivalent to

Kutter's $n = 0·011$) are shown in Table II. The dimensions for incrusted pipes being ascertained from col. I of the Table.

[A consideration of cols. 5 and 6 of Appendix B shows :—

(i) That the differences in the coefficients used therein do not practically, for pipes under 12 in. in diameter, affect the dimensions of pipe diameters ascertained from the values of $\frac{L}{H} F^2$ prepared from the said coefficients.

(ii) That the diameters for larger pipes obtainable from Box's coefficients are clearly too great.

(iii) That diameters calculated from Kutter's $n = 0·011$ and Fanning's coefficients for clean pipes can be brought into accord with those obtainable from col. II of Table I by *deducting* from the latter 1 in. for pipes from 24 in. to 42 in. in diameter and 2 in. for pipes from 43 in. to 60 in. in diameter.]

Table III applicable to drains in earth on 'the most economical section' (type II, Chap. II) has been prepared from the values of $\frac{L}{H} F^2$ for depths increasing from 1 ft. by tenths of a foot (easily laid out with a levelling staff) to 6 ft. ascertained from—

(a) The mean values of $C\sqrt{R}$, which for any value of R are practically the same for all slopes, deduced from Kutter's formula with $n = 0·025$ (equivalent about to Fanning's values of C for 'Smooth loam and some vegetation').

(b) Bazin's values of C given above for—

(i) 'Canals in earth newly dressed' (Fanning's 'Smooth sandy soil').

(ii) 'Ordinary earth canals' (Fanning's 'Regular soil, some vegetation').

Table IV facilitates the calculation of the values of $\frac{L}{H} F^2$ and $\frac{L}{H} G^2$.

Table V gives the areas, values of R, $C\sqrt{R}$, &c., for drains in earth on type II from 6 ft. to 8 ft. deep with side slopes of 1 to 1, and other slopes if needed.

Table VI gives the areas, bedwidths, perimeters, &c., of similar (type II) drains from 1 ft. to 6 ft. deep with side slopes varying from 0 to 1 to 3 to 1.

Table VII is a Table of the fifth powers of numbers for use in ascertaining the dimensions of very large pipes and drains.

Table VIII facilitates the calculation of Kutter's values of C with $n = 0{\cdot}013$.

The application of all the above Tables is illustrated in Chapter IV.

Table IX (Appendix D) gives the end areas of drains on type II for various depths and side slopes.

Its application is explained in the Appendix.

6. **Discharges to be allowed for.**—The provision to be made for 'Water supply' and 'Drainage' respectively will depend to a considerable extent on local conditions and requirements, the following should however in most cases suffice :—

(a) *For Water Supply* an allowance of *one cusec* for 10,000 persons equivalent to a maximum (24-hour) flow of 54 gallons per head or a daily (12-hour) allowance of 27 gallons per person.

(b) *For drainage* a run off of *one cusec* from each 100,000 sq. ft. of area drained (equivalent very nearly to an intensity of run off of ½ in. per hour) in localities where the average annual rainfall is 80 in., and a proportionate increase or decrease for a greater or lesser rainfall.

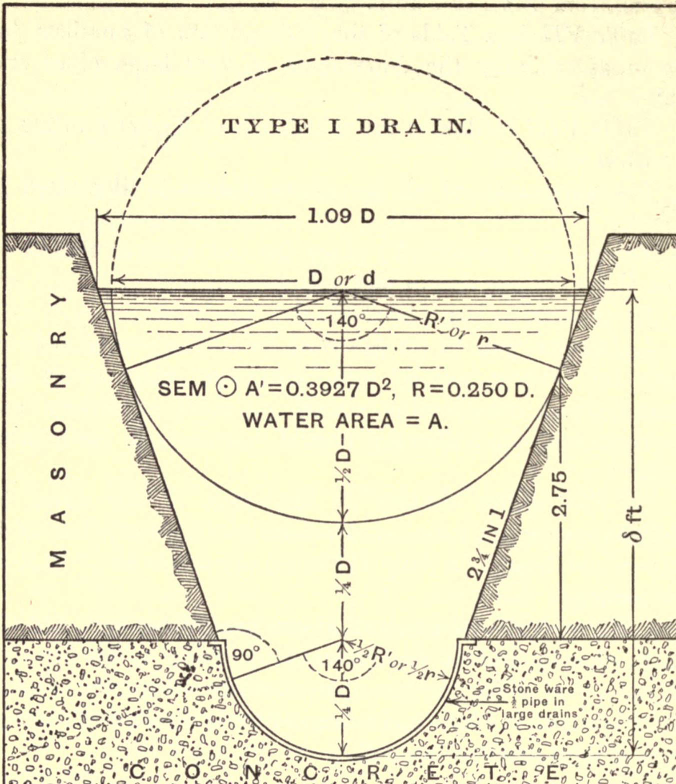
CHAPTER II

DRAINS AND SEWERS

Good drain designs.—Four good designs for drains are illustrated in Plates I, II, and III.

The design in Plate I (hereafter referred to as a type I drain) is specially suitable for masonry or concrete drains as are also, to a minor extent, the types illustrated in Plate III.

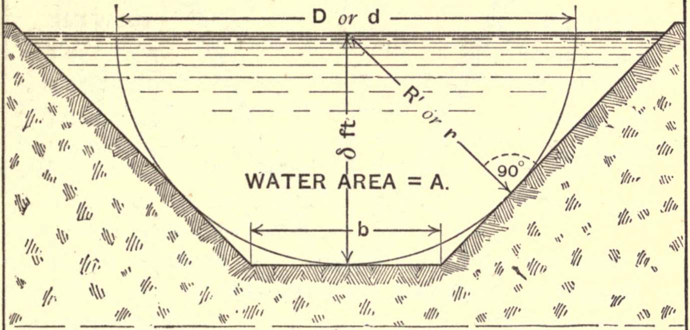
PLATE I.



DRAIN	SEWER
$A = 0.700\ D^2$, $P = 2.375\ D$,	$A = 1.089\ D^2$, $P = 3.946\ D$,
$R = 0.294\ D$, $r' = 0.500$.	$R = 0.276\ D$, $r' = 0.680$.

PLATE II.

TYPE II DRAIN.



SIDE SLOPES

0 TO 1	$A=2\delta^2$	$b=2\delta$	$r'=0.786$
½ " 1	" $=1.74\delta^2$	" $=1.24\delta$	" $=0.902$
1 " 1	" $=1.83\delta^2$	" $=0.83\delta$	" $=0.859$
1½ " 1	" $=2.11\delta^2$	" $=0.61\delta$	" $=0.745$
2 " 1	" $=2.47\delta^2$	" $=0.47\delta$	" $=0.636$
2½ " 1	" $=2.89\delta^2$	" $=0.39\delta$	" $=0.543$
3 " 1	" $=3.33\delta^2$	" $=0.33\delta$	" $=0.472$

THE HYDRAULIC MEAN DEPTH

(R) always $= \frac{\delta}{2} = \frac{R'}{2}$

PLATE III.

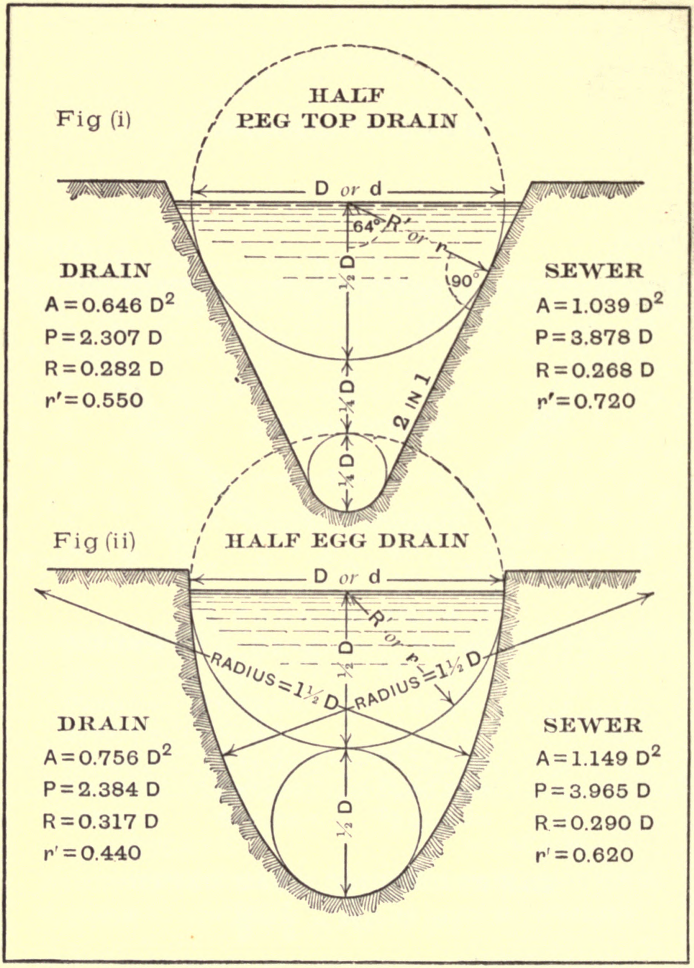
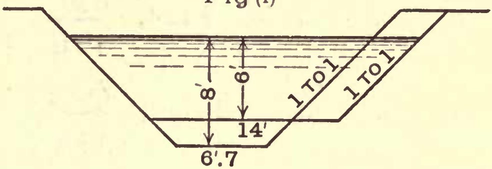


Fig (i)
HALF
PEG TOP DRAIN
D or d
64°
R' or r'
90°
½ D
¼ D
¼ D
2 IN 1
DRAIN
A=0.646 D²
P=2.307 D
R=0.282 D
r'=0.550
SEWER
A=1.039 D²
P=3.878 D
R=0.268 D
r'=0.720
Fig (ii)
HALF EGG DRAIN
D or d
R' or r'
RADIUS=1½ D
RADIUS=1½ D
½ D
½ D
DRAIN
A=0.756 D²
P=2.384 D
R=0.317 D
r'=0.440
SEWER
A=1.149 D²
P=3.965 D
R=0.290 D
r'=0.620

The design illustrated in Plate II (hereafter referred to as a type II drain) is well suited for drains in earth, being 'the most economical section' (Unwin). (See Fig. (i).)

Fig (i)



In this section the water areas and hydraulic mean depths of the two drains, and consequently their discharging capacities are practically the same (Chap. IV, p. 23).

The saving in earth work is self-evident but not great, being about 9 c. ft. per ft. run for depths of 10 ft. and 8 ft.

It is greater in deep cutting and in ground with a cross slope. (See Appendix D.)

2. **Drain designs all usefully based on an inscribed semicircle.**—As all the designs illustrated are based on inscribed semicircles, it follows that, if the velocity in a particular drain can be ascertained, the relative portion of the entire discharge flowing through the area covered by the inscribed semicircle can be calculated, and the dimensions of the semicircle (and therefore of the entire drain) ascertained as in the case of semicircular drains or half pipes.

When, as in the case of a type II drain, the hydraulic mean depth of the drain is the same as the hydraulic mean radius of the inscribed semicircle, i.e. $\frac{D}{4}$ or $\frac{R'}{2}$ (R' being the radius *in feet*), the velocity in the entire drain will be the same as that in a semicircular drain of the same type and of the dimensions of the inscribed semicircle, and the amount of flow through the area covered by the inscribed semicircle can be ascertained from a consideration of the proportion which the respective areas bear one to another.

In a type II drain these are :—

(i) when the side slopes are 0 to 1, $\dfrac{0{\cdot}3927D^2}{2\left(\dfrac{D}{2}\right)^2} = 0{\cdot}786$

(ii) ,, ,, ,, ½ to 1, $\dfrac{0{\cdot}3927D^2}{1{\cdot}74\left(\dfrac{D}{2}\right)^2} = 0{\cdot}902$

(iii) ,, ,, ,, 1 to 1, $\dfrac{0{\cdot}3927D^2}{1{\cdot}83\left(\dfrac{D}{2}\right)^2} = 0{\cdot}859$

(iv) ,, ,, ,, 1½ to 1, $\dfrac{0{\cdot}3927D^2}{2{\cdot}11\left(\dfrac{D}{2}\right)^2} = 0{\cdot}745$

(v) ,, ,, ,, 2 to 1, $\dfrac{0{\cdot}3927D^2}{2{\cdot}47\left(\dfrac{D}{2}\right)^2} = 0{\cdot}636$

(vi) ,, ,, ,, 2½ to 1, $\dfrac{0{\cdot}3927D^2}{2{\cdot}89\left(\dfrac{D}{2}\right)^2} = 0{\cdot}543$

(vii) ,, ,, ,, 3 to 1, $\dfrac{0{\cdot}3927D^2}{3{\cdot}33\left(\dfrac{D}{2}\right)^2} = 0{\cdot}472$

The figures in the last column may be designated '*reduction coefficients*' and denoted by r' (see Plates).

When however the hydraulic mean depth of a drain is greater than the hydraulic mean radius of the inscribed semicircle, as in the case of drains illustrated in Plates I and III, the velocity will be increased, and such increase in velocity has to be taken into account.

From actual calculation it has been ascertained that the average relative velocities would for ordinary diameters and slopes be as under :—

	drain		*semicircular drain*
In a type I drain as	112	:	100.
In a half peg top drain as	109	:	100.
In a half egg drain as	118	:	100.

and r' in these cases would be

(i) $$r' = \frac{0\cdot3927D^2}{0\cdot7000D^2} \times \frac{100}{112} = 0\cdot50$$

(ii) $$r' = \frac{0\cdot3027D^2}{0\cdot6460D^2} \times \frac{100}{109} = 0\cdot55$$

(iii) $$r' = \frac{0\cdot3927D^2}{0\cdot7560D^2} \times \frac{100}{118} = 0\cdot44$$

3. **Conversion of drains into sewers.**—Drains of the types illustrated in Plates I and III can be converted into useful types of masonry or concrete sewers by arching them over as shown in dotted lines on the Plates, the required dimensions being ascertained from the circles on which the designs are based.

A more economical design for sewers is to cover the drains over with stone or reinforced concrete slabs.

The relative increase in velocities is, in the case of arched sewers, found to be as follows :—

	sewer		*circular drain*
In a type I sewer as	107	:	100
In a peg top sewer as	105	:	100
In an egg shaped sewer as	112	:	100

The values of r' being

(i) $$r' = \frac{0\cdot7854D^2}{1\cdot089D^2} \times \frac{100}{107} = 0\cdot68$$

(ii) $$r' = \frac{0\cdot7854D^2}{1\cdot039D^2} \times \frac{100}{105} = 0\cdot72$$

(iii) $$r' = \frac{0\cdot7854D^2}{1\cdot149D^2} \times \frac{100}{112} = 0\cdot62$$

['*Reduction Coefficients*' can be similarly ascertained for any drain or sewer in which a semicircle or circle can be inscribed.]

4. **Special advantages of a drain on type I.**—A drain on type I has the following special advantages :—

(i) As for it the value of

$$\frac{4L}{H} \times \left(\frac{F}{2}\right)^2 = \frac{L}{H} F^2$$

the value of D $\left(\frac{d}{12}\right)$, which is the same as the depth of the drain (δ), can be ascertained from the calculated values of $\frac{L}{H} F^2$ as in the case of a pipe.

(ii) The areas, perimeters and values of R, for the drain, when running partially full, can be easily calculated.

For drains running full, $\frac{3}{4}$ full, $\frac{1}{2}$ full and $\frac{1}{4}$ full, they are:

Area = 0·700D²	P = 2·375D	R = 0·294D
„ = 0·454D²	P = 1·845D	R = 0·246D
„ = 0·252D²	P = 1·315D	R = 0·192D
„ = 0·197D²	P = 0·786D	R = 0·251D

Whence the proportionate discharging capacities will be 100, 58, 26, 25 or more roughly 1, $\frac{3}{5}$, $\frac{1}{4}$, $\frac{1}{4}$: the relative velocities being as 100 : 87 : 70 : 87, a good cleansing velocity being thus secured.

(iii) A drain on this type is easy to construct and keep clean.

(iv) A portion (lower) of the drain need only be constructed to begin with, the upper portion being in earth, until funds allow of the full masonry or concrete section being carried to completion.

CHAPTER III

PERMISSIBLE VELOCITIES

Velocities in a system of pipes.—The velocities in the pipes in a distribution system cannot well be accurately regulated, as it is self-evident, that not only will the head available at the source of supply, and therefore the head, in other words the pressure, in the system generally affect the pipe velocities, but that the velocity in a particular pipe will also vary at times, according to the consumption of water and the consequent draw off from other pipes in the system. When all the taps are open at the same time the velocity will be lower in any given pipe, than it will be when this pipe is alone being drawn on.

At the same time there is a permissible limit to the velocity in a pipe.

If the velocities are great, it will be difficult to obtain sufficient pressure in the distant parts of the area under supply in hours of large consumption, and the risk to the mains from sudden variations of flow, causing what is known as hydraulic shock, will be great; the question therefore needs consideration.

2. **The actual velocity in a pipe, how ascertained.**—The velocity in a pipe can be accurately calculated from the already referred to general formula (ii):

$$v = C\sqrt{R}\sqrt{S}, \text{ in which } S = \frac{H}{L}$$

The velocity in any pipe or half pipe can be also very closely calculated from the following formulæ once the discharge and diameter are known:—

(a) For pipes $v = \frac{G}{2d^2}$ . . . (xiA)

(b) For half pipes $v = \frac{G}{d^2}$ . . . (xiB)

when the discharge (G) is in galmins; and

(a) For pipes $v = \frac{375F}{2d^2}$ . . . (xiiA)

(b) For half pipes $v = \frac{375F}{d^2}$ . . . (xiiB)

when the discharge (F) is in cusecs.

The velocities so ascertained will always be a trifle, $\frac{1}{49}$th, above the true velocities as actually, for pipes,

$$v = 0{\cdot}49\frac{G}{d^2} \quad \text{or} \quad = 0{\cdot}49\frac{375F}{d^2}$$

3. **Permissible velocity in a pipe.**—Professor Unwin in 'A Treatise on Hydraulics' gives a rough rule for ascertaining the maximum safe velocity: his formula is (v' being the permissible velocity)—

$$v' = 1{\cdot}45\text{D} + 2 \quad . \quad . \quad \text{(xiii)}$$

(D being the diameter of the pipe *in feet*).

With the diameter expressed *in inches* (d) the formula becomes—

$$v' = 0{\cdot}12d + 2 \quad . \quad . \quad . \quad \text{(xiv)}$$

4. **Permissible velocities in drains and sewers.**—The velocity in a masonry or concrete drain or sewer should not as a rule exceed 5 ft. per sec. and in a drain in earth 3 ft. per sec. (see Chap. IV, 'Working Examples').

CHAPTER IV

WORKING EXAMPLES

Small discharges.—Assume to begin with, that it is desired to ascertain the diameter of a clean pipe ($n = 0{\cdot}011$, very nearly) to discharge 4 galmins, the length of the pipe being 1000 ft. and the head available 10 ft.

The 'hydraulic gradient' or 'virtual slope' of the pipe $\left(\frac{\text{H}}{\text{L}}\right)$ will then be $\frac{10}{1000}$, or 1 in 100, and therefore $\frac{\text{L}}{\text{H}}\text{G}^2 = 100 \times 4^2 = 1600$ and the diameter of the required pipe (d) would from Table II, col. II be $1\frac{1}{4}$ in.

For an incrusted pipe it would from Table II, col. 1 be $1\frac{3}{4}$ in.

For a clean coated pipe ($n = 0{\cdot}010$) its diameter could be safely taken at 1 in.

For a stoneware pipe ($n = 0{\cdot}013$) d would $= 1\frac{3}{4}$ in.

For a semicircular stoneware half-pipe ($n = 0{\cdot}013$) $\frac{4\text{L}}{\text{H}}\text{G}^2$ would $= 6400$, whence, from col. 1, Table II, $d = 2$ in. and δ therefore $= \frac{1}{12}$ ft.

2. **Large discharges. — Pipes.** — Suppose that the 'hydraulic gradient' of a single pipe or the 'average hydraulic gradient' of a series of connected pipes is found to be 1 in 1764 (square of 42), and that the required discharge in the single pipe or in a pipe in the series is 21 cusecs, then from Table IV, from the horizontal column opposite a required discharge of 21 cusecs—

1000	times	441	for 1 =	441,000
100	,,	3087	,, 7 =	308,700
10	,,	2646	,, 6 =	26,460
1	,,	1764	,, 4 =	1,764

and the value of $\frac{L}{H}F^2 = 1764 \times 21^2 = 777,924$

A check on the calculation is thus secured.

Using now Table I, cols. II and IX, the required diameter of a clean pipe ($n = 0{\cdot}011$ very nearly) would be 39 in. (Unwin).

For a clean coated pipe for Kutter's $n = 0{\cdot}010$ it could be taken at 38 in. or even 37 in., from Unwin's coefficients it would however be safer to keep it at 39 in. (para. 5, Chap. I).

For $n = 0{\cdot}011$ exactly or for Fanning's coefficients $d = 39 - 1 = 38$ in. (para. 5, Chap. I).

For an incrusted pipe (cols. II and III) $d = 39 + 6 = 45$ in. (It is the same from Appendix C.)

3. **Large discharges.—Masonry or concrete drains and sewers.**—For a semicircular unlined masonry or concrete drain ($n = 0{\cdot}013$) the value of

$\frac{4L}{H}F^2$, formula (viiA), would be $4 \times 777,924 = 3,111,696$

whence, from cols. I and IX, Table I,

$$d = 52 \text{ in., and } \delta = \frac{D}{2} = 2 \text{ ft. } 2 \text{ in.}$$

For a mixed cement lined drain ($n = 0{\cdot}011$ very nearly) d would = 51 in. For a pure cement lined drain it could safely be taken at 50 in.

The value of d for $n = 0{\cdot}013$ can however in the above case be more accurately ascertained from Table I in the following manner:—

The difference between the values of $\frac{L}{H}F^2$ for a 52 in. diameter and a 51 in. diameter is $3{,}274{,}000 - 2{,}924{,}000 = 350{,}000$ or say 35,000 for each tenth of an inch and between 3,111,696 and 2,924,000 it is 187,696; therefore $\frac{187696}{35000} =$ say 6 and the exact diameter of an unlined masonry or concrete semicircular drain would thus at its large end be 51·6 in., a proportionate *discharge*, not proportionate *area*, being adopted for the central or other section, as the drain is an open one with a steady flow into it along its whole length. For a discharge of 1 cusec—

$$\frac{4L}{H}F^2 = 4 \times 1764 \times 1 = 7056$$

whence

$d = 17$ in. and $A = 0·3927 \times 17^2 = 113·5$ sq. in. $= 0·8$ sq. ft.

The drain area with $d = 51·6$ in. would be $0·3927 \times 51·6^2 = 1035·8$ sq. in. or 7·2 sq. ft. or about only nine times the area needed for a discharge of 1 cusec, whereas the discharge capacity (21 cusecs) is over twenty times as much.

The velocity in a semicircular drain with $d = 51·6$ in. and $F = 21$ cusecs would from formula (xiiB) be—

$$v = \frac{375 \times 21}{51·6^2} = 3·0 \text{ ft. per sec.}$$

whence $F = 7·2 \times 3 = 21·6$ cusecs against a required discharge of 21 cusecs.

For a type I drain the value of $\frac{4L}{H}F^2$ would be the same as the value of $\frac{L}{H}F^2$ for a circular drain, i.e. 777,924, and d, which is also the *depth* of the drain, therefore (with $n = 0·013$) $= 40$ in., whence $\delta = 3$ ft. 4 in.

For a half peg top drain $21 \times 0·55 = 11·6$ and $4 \times 1764 \times$

$11{\cdot}6^2 = 7056 \times 135 = 952{,}560$, whence, with $n = 0{\cdot}013$, $d =$ 42 in.

For a half egg drain $21 \times 0{\cdot}44 = 9{\cdot}2$ and $4 \times 1764 \times 9{\cdot}2^2 = 7056 \times 84{\cdot}7 = 597{,}643$, whence (for $n = 0{\cdot}013$) $d = 38$ in.

The area of a type I drain 40 in. in depth would be $0{\cdot}700 \times 40^2 = 1120$ sq. in. $= 7{\cdot}8$ sq. ft.

The velocity in a type I drain would be from formula (xiiB) allowing for increased velocity $= \frac{375 \times 10{\cdot}5}{40^2} \times \frac{112}{100} = 2{\cdot}8$ ft. per sec., whence $F = 7{\cdot}8 \times 2{\cdot}8 = 21{\cdot}84$ cusecs against a required discharge of 21 cusecs.

For a type I sewer $21 \times 0{\cdot}68 = 14{\cdot}3$ and $\frac{L}{H} F^2 = 1764 \times 14{\cdot}3^2 = 360{,}800$ and therefore $d = 35$ in.

Also, as $A = 9{\cdot}2$ sq. ft. and $v = \frac{375 \times 14{\cdot}3}{2 \times 35^2} \times \frac{107}{100} = 2{\cdot}35$ ft. per sec., $F = 9{\cdot}2 \times 2{\cdot}35 = 22{\cdot}8$ cusecs.

The safe values for the above reasons being—

		in.
1.	Clean pipe	$d = 39$
2.	Clean coated pipe	$d = 38$
3.	Incrusted pipe	$d = 45$
4.	Semicircular unlined masonry drain	$d = 52$
5.	,, ,, more exactly	$d = 51{\cdot}6$
6.	Semicircular drain lined mixed cement	$d = 51$
7.	,, ,, ,, pure cement	$d = 50$
8.	Type I drain unlined	$d = 40$
9.	Half peg top drain unlined	$d = 42$
10.	Half egg drain unlined	$d = 38$
11.	Type I sewer unlined	$d = 35$

If by slightly raising ($0{\cdot}13$ ft. in 1764 ft. or $0{\cdot}075$ per 1000) the water level at the source of supply, or if, by assuming that the outlet level is lowered by an equal amount, the hydraulic gradient is steepened from 1 in 1764 to 1 in $\frac{689000}{21^2} = 1$ in 1562 (689,000 being the exact value of $\frac{L}{H} F^2$ for a 39 in. pipe with $n = 0{\cdot}013$ for a virtual slope of 1 in 1000 to 2000), the values of d above given could be reduced by 1 in. in each case (No. 5 to 50·4 in. exactly).

4. **Large discharges.—Drains in earth.**—For drains in earth on type II with side slopes of 1 to 1 the 'reduction co-efficient' (r') would be say 0·86, and therefore $21{\cdot}0 \times 0{\cdot}86 = 18$.

The value of $\frac{4L}{H}F^2$ for a hydraulic gradient of 1 in 1764 would then be $7056 \times 324 = 2{,}286{,}144$ and the values of δ, the depths of the required drains, would from Table III be—

For Kutter's $n = 0{\cdot}025$	2·7 ft.
„ Bazin's (i)	2·6 ft.
„ Bazin's (ii)	2·8 ft.

In the first case $C\sqrt{R} = 69{\cdot}9$ (Table III) and $A = 13{\cdot}34$ sq. ft. (Table VI), therefore the velocity in the drain will be $\frac{69{\cdot}9}{42} = 1{\cdot}67$ ft. per sec., and $F = 13{\cdot}34 \times 1{\cdot}67 = 22{\cdot}3$ cusecs, against a required discharge of 21 cusecs.

The approximate velocity can be more easily ascertained from $v = \frac{F}{A} = \frac{21}{13{\cdot}34} =$ say 1·6 ft. per sec.

This shows that the velocity in the drain is a safe one, i.e. well under 3 ft. per sec.

For Bazin (i) $F = 12{\cdot}37 \times 1{\cdot}79 = 22{\cdot}14$ cusecs.
For Bazin (ii) $F = 14{\cdot}35 \times 1{\cdot}46 = 20{\cdot}95$ cusecs.

For a drain with side slopes of 3 to 1, $21 \times 0{\cdot}47 =$ say 10, and $7056 \times 100 = 705{,}600$, whence the values of δ are (Table III)—

Kutter ($n = 0{\cdot}025$)	2·2 ft. and	$F = 16{\cdot}12 \times 1{\cdot}43 = 23$	cusecs
Bazin (i)	2·1 ft. „	$F = 14{\cdot}69 \times 1{\cdot}52 = 22{\cdot}33$	„
Bazin (ii)	2·3 ft. „	$F = 17{\cdot}62 \times 1{\cdot}24 = 21{\cdot}85$	„

5. **Very large discharges.—General.**—The dimensions of pipes and drains and sewers to suit very large discharges can be ascertained from the following approximate formulæ:

(a) *For pipes and masonry or concrete drains or sewers* ($n = 0{\cdot}013$):

$$D^5 = \frac{\frac{L}{H} F^2}{2600} \text{ and } \frac{\frac{4L}{H} F^2}{2600} \text{ respectively} \quad . \quad (xv)$$

(b) *For drains in earth on type II:*

(i) For Kutter's coefficients with $n = 0{\cdot}025$—

$$D^5 = \frac{\frac{4L}{H} F^2}{900} \quad . \quad . \quad . \quad (xvi)$$

(ii) For Bazin's (i) coefficients—

$$D^5 = \frac{\frac{4L}{H} F^2}{1000} \quad . \quad . \quad . \quad (xvii)$$

(iii) For Bazin's (ii) coefficients—

$$D^5 = \frac{\frac{4L}{H} F^2}{800} \quad . \quad . \quad . \quad (xviii)$$

These formulæ are applied as follows:

Suppose that for a pipe ($n = 0{\cdot}013$) $\frac{L}{H} = 2000$ and $F = 200$,

then $$D^5 = \left(\frac{d}{12}\right)^5 = \frac{2000 \times 200^2}{2600} = \text{say } 38{,}000$$

and from Table VII the value of D is somewhere between 7 ft. and 8 ft.

But as $d^5 = 30{,}800 \times 12^5 = 7{,}664{,}025{,}600$, therefore $d =$ 95 in. more exactly.

This would also be the *depth* of an unlined masonry or concrete drain on type I.

With $n = 0{\cdot}011$ very nearly, $d = 94$ in.

Taking the diameter of the pipe at 8 ft., we have $R = \frac{8}{4} = 2$ and $\sqrt{R} = 1{\cdot}41$, whence (using Table VIII) from Kutter's formula, the value of $C\sqrt{R}$ (with $n = 0{\cdot}013$), $= 183{\cdot}9$, and as $\sqrt{2000} = 44{\cdot}7$ the velocity would be $\frac{183{\cdot}9}{44{\cdot}7} = 4{\cdot}12$ ft. per sec.

The area of an 8 ft. diameter pipe $= 0{\cdot}7854 \times 8^2 = 50{\cdot}27$ sq. ft.

$$\therefore\ F = 50{\cdot}27 \times 4{\cdot}12 = 207 \text{ cusecs.}$$

This shows that the ascertained diameter is very approximately correct.

The allowance for incrustation in this case would from analogy be $\frac{96 - 6}{6} = 15$ in. Large single pipes or circular or arched sewers are therefore better avoided as far as possible —two, each to carry half the required discharge, being used instead, if found cheaper.

For a drain in earth on type II with side slopes of 1 to 1 and $n = 0{\cdot}025$, if $\frac{L}{H} = 4000$ and $F = 200$, D^5 (formula xvi) will equal $\frac{4 \times 4000 \times (200 \times 0{\cdot}86)^2}{900} = 526{,}000$, and the value of D is from Table VII somewhere between 13 ft. and 14 ft.

But as $537{,}824 - 371{,}293 = 166{,}531$, the difference for each tenth of a foot will be, say, 16,650; also as $526{,}000 - 371{,}000 = 155{,}000$ and as $\frac{155{,}000}{16{,}650} = 0{\cdot}9$, the exact value of D will be $13{\cdot}9$ ft., whence $\delta = \frac{D}{2} = 6{\cdot}95$ ft. = say 7 ft.

The area then = $89{\cdot}7$ sq. ft. (Table V), and $v = \frac{138}{\sqrt{4000}} = 2{\cdot}2$ ft. per sec., whence $F = 89{\cdot}7 \times 2{\cdot}2 = 197{\cdot}34$ cusecs, the required discharge being 200 cusecs.

It will therefore be safer to adopt a drain 7·1 ft. deep.

When, however, the depth of a drain in earth on type II exceeds 6 ft. it will often be advisable to change the type of drain and find a new value for the bedwidth (b) for a depth of 6 ft., or any other desired depth given in Table VI.

Suppose that the required depth to water level for a drain on the 'most economical section' with side slopes of 1 to 1 is found to be 8 ft., then R will $=$ 4 ft. and $C\sqrt{R}$ (Kutter) $= 151$ (Table V), also $A = 117{\cdot}12$ sq. ft. As the required side slopes are 1 to 1, the end areas will for a 6 ft. depth equal $6 \times 6 =$ 36 sq. ft. whence the central area $= 117 - 36 = 81$ sq. ft. and therefore $b = \frac{81}{6} =$ say 14 ft., as increase in perimeter will necessitate an increase in area if the hydraulic mean depth is to be approximately the same. The total area of the new drain would thus be $(14 + 6)\,6 = 120$ sq. ft., and the new perimeter $=$ (from Table VI) $14 + (P - b) = 14 + 16{\cdot}98 =$ say 31, whence $R = \frac{120}{31} = 3{\cdot}9$.

The velocity for any slope will then be very nearly the same as in the 8 ft. deep drain on type II ($R = 4$), and the discharge also practically the same. For a slope of 1 in 4000 v (with $R = 4$) $= \frac{151}{63{\cdot}3} = 2{\cdot}40$ ft. per sec., and $F = 117 \times 2{\cdot}4 =$ 280 cusecs; also v (with $R = 3{\cdot}9$) $= \frac{148{\cdot}4}{63{\cdot}3} = 2{\cdot}35$ ft. per sec., and $F = 120 \times 2{\cdot}35 = 282$ cusecs.

This, however, might not, in another instance, have been the case; a further calculation to ascertain a suitable area and velocity to give the required discharge would then be necessary.

With $b = 15$, A would $= 120 + 6 = 126$ sq. ft., and $P =$ $31 + 1 = 32$, whence $R = 4$ and $C\sqrt{R}$ (Kutter) $= 151$, the velocity for a slope of 1 in 4000 being $\frac{151}{63{\cdot}3} = 2{\cdot}40$ ft. per sec., whence $F = 126 \times 2{\cdot}4 = 302$ cusecs.

6. **Permissible velocities.**—Taking for pipes the examples worked out in paras. 1 and 2 above, the velocity in a clean

pipe $1\frac{1}{4}$ in. in diameter with a discharge of 4 galmins would be from formula (xiA) $= \frac{4}{2 \times 1\frac{1}{4} \times 1\frac{1}{4}} = 1{\cdot}28$ ft. per sec. and the permissible velocity from formula (xiv)—

$$v' = 0{\cdot}12 \times 1\tfrac{1}{4} + 2 = 2{\cdot}15 \text{ ft. per sec.}$$

For a 39 in. pipe discharging 21 cusecs the actual velocity would be from formula (xiiA) $\frac{375 \times 21}{2 \times 39^2} = 2{\cdot}55$ ft. per sec. and the permissible velocity from formula xiv $= 0{\cdot}12 \times 39 + 2 = 6{\cdot}68$ ft. per sec.

No increase in diameter is therefore in either case necessary.

When the velocity in a masonry or concrete drain is found to exceed 5 ft. per sec., and in a drain in earth 3 ft. per sec., it will generally be necessary to ascertain the slope in the water surface needed to keep the velocity down to the desired maximum, by providing falls at suitable intervals.

This slope can be calculated from formula (ii)—

$$v = C\sqrt{R}\sqrt{S}$$

In an earthen drain, maximum permissible velocity 3 ft. per sec., with $C\sqrt{R} = 151{\cdot}0$ (Kutter $R = 4$) $\sqrt{S} = \frac{3}{151} =$ say $\frac{1}{50}$, and the required slope in the water surface is 1 in 50^2 or 1 in 2500.

For a velocity of 5 ft. per sec., the safe slope in the water surface would be 1 in 900, for $C\sqrt{R} = 151$.

7. The general application of the Tables.—Tables in the form of the present ones can be used for the solution of most hydraulic problems—see several examples of the practical application of similar Tables in Vol. IV, 'Building Construction,' Rivington's Series (the Tables in which depend, however, on Darcy's coefficients alone); also the author's work mentioned in para. 1, Chap. I,

8. **The full utility of the Tables.**—As correct methods have been formulated for (with a choice of coefficients) accurately calculating, for any available fall, the dimensions of pipes and of masonry or concrete drains and sewers to the tenths of an inch, and of drains in earth to the nearest tenth of a foot, it follows that if the dimensions so ascertained can be adopted there will in each case be a, even if only small, saving in quantities and consequently in cost, which will in large schemes generally make an appreciable difference in the total expenditure (see Appendix E).

There should be no practical difficulty in constructing masonry or concrete drains or drains in earth to the exact calculated sections.

With pipes ordinary market or available sizes will generally have to be used.

The actual heads needed for given discharges can however in such cases be ascertained from the Tables, and the total head at disposal in a long line of pipes or in a system of pipes utilised to the best advantage, any surplus head found available being used, if so desired, to steepen the hydraulic gradients, and thus reduce the size or sizes and therefore cost of the most expensive pipe or pipes.

Suppose that we have to deal with a line of four pipes each 1000 ft. long, and that the total head available is 10 ft., the 'average hydraulic gradient' of the line of pipes, so long as no one pipe rises above this gradient, will then be 1 in 400.

If the required discharges are 10, 8, 6, and 4 cusecs respectively, we have from Table I for incrusted pipes :—

$400 \times 10^2 = 40{,}000$ and d (market size) $= 22 + 3 = 25$ in.

$400 \times 8^2 = 25{,}600$ „ „ „ $= 21 + 3 = 24$ „

$400 \times 6^2 = 14{,}400$ „ „ „ $= 19 + 3 = 22$ „

$400 \times 4^2 = 6{,}400$ „ „ „ $= 16 + 2 = 18$ „

The ultimate heads required would be—

In the 16 in. pipe from $\frac{1000 \times 4^2}{H} = 7,800$, $H = 2\cdot05$ ft.

" " 19 in. " " $\frac{1000 \times 6^2}{H} = 18,900$, $H = 1\cdot91$ ft.

" " 21 in. " " $\frac{1000 \times 8^2}{H} = 31,800$, $H = 2\cdot01$ ft.

Or a total head of $5\cdot97$ ft.

This would leave a head of say $10 - 6 = 4$ ft. available for the 22 in. pipe, and therefore for it $\frac{L}{H} F^2 = \frac{1000}{4} \times 10^2 = 25,000$, and a 21 in. pipe can be substituted for the 22 in. one. In fact a 20 in. pipe could well be used, and the required diameters fixed at 24 in., 23 in., 22 in., 18 in.

9. **'Average hydraulic gradients,' when most useful.**— By adopting the system of 'average hydraulic gradients,' (see the Author's 'Practical Hydraulic Tables') the required sizes of pipes in a complicated system of Water Supply can be quickly and accurately ascertained, the first gradient used being that from the source of supply to the *highest* point in the system at which water, however small the amount may be, is required—the surplus head always available (see above) being used, if so desired, as a reserve to overcome friction in bends, elbows, &c., which so far has not been taken into account, and is in large projects generally speaking a negligible quantity, as all taps are never likely to be open at the same time, and the surplus head can be utilised in reducing the size of the usually long and expensive supply main or that of any other large pipe in the system.

TABLE I—giving the values of $\frac{L}{H}F^2$ with $n = 0·013$ and $n = 0·011$ very nearly for diameters from 1 in. to 60 in. for 'virtual slopes' of 1 in 1 to 1 in 4000.

Pipe diameters		Addition for Incrustation	Values of $\frac{L}{H}F^2$ for virtual slopes between 1 in. :—						
$n = 0·013$	$n = 0·011$ very nearly*		1 to 100	100 to 200	200 to 300	300 to 500	500 to 1000	1000 to 2000	2000 to 4000
I	II	III	IV	V	VI	VII	VIII	IX	X
in.	in.								
1	—		0·001	0·001	—	—	—	—	—
2	1		0·048	0·048	0·048	—	—	—	—
3	2		0·49	0·48	0·48	0·47	—	—	—
4	3		2·5	2·5	2·5	2·4	2·4	—	—
5	4		8·9	8·9	8·8	8·6	8·4	8·0	—
6	5		25	25	24	24	23	22	20
7	6		58	58	57	57	56	55	51
8	7		122	122	121	118	116	112	108
9	8		236	235	234	232	227	217	197
10	9	1 in.	423	421	419	415	410	400	350
11	10		719	715	712	703	691	666	619
12	11		1,159	1,154	1,150	1,140	1,120	1,080	1,000
13	12		1,800	1,796	1,790	1,780	1,750	1,690	1,570
14	13		2,700	2,695	2,690	2,670	2,650	2,530	2,350
15	14		3,970	3,950	3,940	3,900	3,850	3,700	3,500
16	15		5,650	5,630	5,600	5,570	5,500	5,300	5,000
17	16	2 in.	7,900	7,880	7,860	7,800	7,700	7,500	7,000
18	17		10,800	10,750	10,700	10,600	10,500	10,200	9,600
19	18		14,500	14,480	14,440	14,300	14,200	13,800	13,000
20	19	3 in.	19,200	19,100	19,000	18,900	18,600	18,200	17,200

* Or adopting Unwin's coefficients for clean pipes.

TABLE I—*continued*

Pipe diameters		Addition for incrustation	Values of $\frac{L}{H}F^2$ for virtual slopes between 1 in. :—						
$n = 0{\cdot}013$	$n = 0{\cdot}011$ very nearly*		1 to 100	100 to 200	200 to 300	300 to 500	500 to 1000	1000 to 2000	2000 to 4000
I	II	III	IV	V	VI	VII	VIII	IX	X
in.	in.								
21	20	3 in.	25,000	24,900	24,800	24,700	24,400	23,700	22,000
22	21		32,200	32,100	32,000	31,800	31,400	30,500	29,000
23	22		41,000	40,900	40,800	40,600	40,000	39,000	36,000
24	23		51,700	51,500	51,300	51,100	50,500	49,300	47,000
25	24		65,800	65,600	75,400	65,000	64,000	63,000	60,000
26	25	4 in.	80,000	79,700	79,400	79,000	78,000	76,000	73,000
27	26		100,000	97,700	97,400	97,000	96,000	94,000	90,000
28	27		123,000	122,700	122,400	122,000	120,000	118,000	113,000
29	28		148,000	147,700	147,400	147,000	145,000	142,000	136,000
30	29		174,000	173,000	172,000	171,000	169,000	166,000	160,000
31	30		212,000	211,700	211,200	210,000	208,000	204,000	195,000
32	31	5 in.	251,000	250,500	250,000	249,000	246,000	241,000	230,000
33	32		290,000	289,000	288,000	287,000	284,000	279,000	267,000
34	33		348,000	347,000	346,000	345,000	341,000	335,000	322,000
35	34		406,000	405,000	404,000	402,000	398,000	390,000	376,000
36	35		464,000	463,000	462,000	460,000	455,000	447,000	430,000
37	36		547,000	546,000	545,000	542,000	537,000	527,000	508,000
38	37	6 in.	630,000	629,000	628,000	625,000	620,000	608,000	586,000
39	38		714,000	712,000	710,000	708,000	701,000	689,000	665,000
40	39		830,000	828,000	826,000	823,000	815,000	801,000	774,000

* Or adopting Unwin's coefficients for clean pipes.

TABLE I—*continued*

Pipe diameters		Addition for Incrustation	Values of $\frac{L}{H}F^2$ for virtual slopes between 1 in.:—						
$n = 0{\cdot}013$ I	* $n = 0{\cdot}011$ very nearly II	III	1 to 100 IV	100 to 200 V	200 to 300 VI	300 to 500 VII	500 to 1000 VIII	1000 to 2000 IX	2000 to 4000 X
41	40		946,000	944,000	942,000	939,000	935,000	915,000	884,000
42	41	6 in.	1,062,000	1,060,000	1,058,000	1,054,000	1,045,000	1,027,000	994,000
43	42		1,221,000	1,219,000	1,217,000	1,212,000	1,202,000	1,182,000	1,144,000
44	43		1,380,000	1,377,000	1,375,000	1,370,000	1,358,000	1,336,000	1,290,000
45	44		1,538,000	1,535,000	1,532,000	1,527,000	1,515,000	1,490,000	1,440,000
46	45	7 in.	1,750,000	1,747,000	1,744,000	1,738,000	1,724,000	1,697,000	1,645,000
47	46		1,963,000	1,959,000	1,955,000	1,948,000	1,933,000	1,903,000	1,846,000
48	47		2,176,000	2,171,000	2,167,000	2,157,000	2,142,000	2,110,000	2,048,000
49	48		2,452,000	2,448,000	2,444,000	2,435,000	2,416,000	2,381,000	2,313,000
50	49		2,730,000	2,725,000	2,720,000	2,710,000	2,690,000	2,650,000	2,578,000
51	50		3,006,000	3,001,000	2,996,000	2,987,000	2,965,000	2,924,000	2,843,000
52	51	8 in.	3,364,000	3,358,000	3,353,000	3,343,000	3,320,000	3,274,000	3,185,000
53	52		3,721,000	3,716,000	3,710,000	3,700,000	3,674,000	3,624,000	3,527,000
54	53		4,078,000	4,072,000	4,067,000	4,055,000	4,028,000	3,974,000	3,869,000
55	54		4,535,000	4,528,000	4,521,000	4,506,000	4,477,000	4,419,000	4,306,000
56	55		4,992,000	4,983,000	4,974,000	4,957,000	4,926,000	4,865,000	4,743,000
57	56		5,449,000	5,438,000	5,428,000	5,408,000	5,375,000	5,310,000	5,180,000
58	57	9 in.	6,016,000	6,006,000	5,997,000	5,977,000	5,941,000	5,870,000	5,728,000
59	58		6,583,000	6,574,000	6,565,000	6,547,000	6,507,000	6,429,000	6,276,000
60	59		7,150,000	7,142,000	7,133,000	7,116,000	7,073,000	6,989,000	6,824,000

* Or adopting Unwin's coefficients for clean pipes.

TABLE II—giving for all slopes, the values of $\frac{L}{H}G^2$ for small pipes from $\frac{3}{8}$ in. to 8 in. in diameter

Values of $\frac{L}{H}G^2$			Diameter of pipe in inches	Values of $\frac{L}{H}G^2$			Diameter of pipe in inches
Kutter with $n = 0.013$	Box (about = Kutter	Fanning * with $n = 0.011$)		Kutter with $n = 0.013$	Box (about = Kutter	Fanning * with $n = 0.011$)	
I	II	II	III	I	II	II	III
			in.				in.
0·5	5·6	5·5	$\frac{3}{8}$	24,800	69,200	83,500	$2\frac{1}{2}$
2·3	23	21	$\frac{1}{2}$	69,000	177,000	215,000	3
23	173	190	$\frac{3}{4}$	164,000	367,000	454,000	$3\frac{1}{2}$
127	729	820	1	354,000	746,000	926,000	4
476	2,225	2,460	$1\frac{1}{4}$	1,250,000	2,278,000	2,900,000	5
1,296	5,536	6,350	$1\frac{1}{2}$	3,480,000	5,770,000	7,350,000	6
3,125	11,950	13,650	$1\frac{3}{4}$	8,157,000	12,250,000	16,400,000	7
6,750	23,330	27,830	2	17,160,000	23,900,000	32,600,000	8

* Velocity 5 ft. per sec. Unwin's coefficients for small pipes are not available.

TABLE III—giving the values of $\frac{L}{H}F^2$, $C\sqrt{R}$, &c., for drains in earth on type II, for depths increasing by the tenths of a foot from 1 ft. to 6 ft.

$\delta = \frac{D}{2}$ ft.	R	$\sqrt{R}$	Kutter $n = 0·025$		Bazin (i)		Bazin (ii)	
			$C\sqrt{R}$	$\frac{L}{H}F^2$	$C\sqrt{R}$	$\frac{L}{H}F^2$	$C\sqrt{R}$	$\frac{L}{H}F^2$
1·0	0·50	0·70	31·9	10,200	38·2	14,700	28·4	8,100
1·1	0·55	0·74	34·9	17,500	40·9	24,200	30·5	13,400
1·2	0·60	0·77	37·5	26,200	43·2	38,800	32·2	21,500
1·3	0·65	0·80	40·0	46,000	45·5	59,500	34·0	33,200
1·4	0·70	0·83	42·4	70,500	47·8	89,500	35·8	49,300
1·5	0·75	0·86	44·7	102,700	50·1	127,800	37·7	73,500
1·6	0·80	0·89	47·0	145,000	52·6	181,600	39·2	102,500
1·7	0·85	0·92	49·2	200,000	55·0	252,000	41·5	142,000
1·8	0·90	0·94	50·9	274,000	57·0	344,000	43·0	195,000
1·9	0·95	0·97	53·6	372,000	59·5	458,000	45·0	266,000
2·0	1·00	1·00	55·8	490,000	62·1	611,000	47·0	351,000
2·1	1·05	1·02	57·9	648,000	64·0	793,000	48·5	458,000
2·2	1·10	1·05	59·9	832,000	66·6	1,000,000	50·6	590,000
2·3	1·15	1·07	62·1	1,000,000	68·6	1,300,000	52·1	766,000
2·4	1·20	1·09	64·1	1,400,000	70·6	1,700,000	53·8	978,000
2·5	1·25	1·11	66·1	1,700,000	72·6	2,000,000	55·5	1,200,000
2·6	1·30	1·14	68·0	2,100,000	75·2	2,500,000	57·7	1,500,000
2·7	1·35	1·16	69·9	2,600,000	77·4	3,200,000	59·6	1,900,000
2·8	1·40	1·18	71·8	3,200,000	79·5	3,900,000	61·1	2,300,000
2·9	1·45	1·20	73·7	3,900,000	81·7	4,700,000	63·0	2,800,000
3·0	1·50	1·22	75·6	4,600,000	83·9	5,600,000	64·6	3,400,000
3·1	1·55	1·24	77·4	5,500,000	86·2	6,800,000	66·4	4,000,000
3·2	1·60	1·26	79·2	6,600,000	88·3	8,100,000	68·3	4,800,000

TABLE III—*continued*

$\delta = \frac{D}{2}$ ft.	R	$\sqrt{R}$	Kutter n = 0·025		Bazin (i)		Bazin (ii)	
			$C\sqrt{R}$	$\frac{L}{H}F^2$	$C\sqrt{R}$	$\frac{L}{H}F^2$	$C\sqrt{R}$	$\frac{L}{H}F^2$
3·3	1·65	1·28	81·0	7,700,000	90·6	9,700,000	70·1	5,800,000
3·4	1·70	1·30	82·8	9,100,000	92·8	11,500,000	72·0	6,900,000
3·5	1·75	1·32	84·5	10,700,000	95·2	13,600,000	73·9	8,200,000
3·6	1·80	1·34	86·2	12,400,000	97·4	15,800,000	75·8	9,600,000
3·7	1·85	1·36	87·9	14,400,000	99·8	18,500,000	77·8	11,200,000
3·8	1·90	1·37	89·6	16,700,000	101·4	21,500,000	79·5	13,100,000
3·9	1·95	1·39	91·3	19,300,000	103·8	24,900,000	81·1	15,300,000
4·0	2·00	1·41	93·0	22,200,000	106·5	28,900,000	82·7	17,700,000
4·1	2·05	1·43	94·7	25,500,000	108·5	33,100,000	84·3	20,500,000
4·2	2·10	1·45	96·3	29,000,000	110·6	37,700,000	86·0	23,200,000
4·3	2·15	1·46	97·9	32,700,000	111·9	42,800,000	87·5	26,500,000
4·4	2·20	1·48	99·5	37,000,000	114·0	48,600,000	89·2	30,100,000
4·5	2·25	1·50	101·1	41,500,000	116·2	54,900,000	90·8	34,100,000
4·6	2·30	1·51	102·7	47,000,000	117·6	61,900,000	92·5	38,500,000
4·7	2·35	1·53	104·3	52,900,000	119·8	69,700,000	94·1	43,600,000
4·8	2·40	1·54	105·9	59,000,000	121·2	78,400,000	95·8	49,000,000
4·9	2·45	1·56	107·5	66,500,000	123·2	87,600,000	97·3	55,000,000
5·0	2·50	1·58	109·0	71,500,000	125·4	107,900,000	99·0	61,500,000
5·1	2·55	1·60	110·5	82,100,000	127·8	109,500,000	100·6	68,700,000
5·2	2·60	1·61	112·0	90,800,000	129·3	121,200.000	102·3	76,500,000
5·3	2·65	1·62	113·5	101,300,000	130·7	135,500,000	103·8	85,700,000
5·4	2·70	1·64	115·0	109,700,000	133·0	147,300,000	105·5	93,500,000
5·5	2·75	1·66	116·5	124,500,000	134·3	167,300,000	107·1	106,500,000
5·6	2·80	1·68	118·0	137,000,000	137·6	184,500,000	108·8	117,600,000
5·7	2·85	1·69	119·5	149,300,000	139·1	201,800,000	110·4	129,000,000
5·8	2·90	1·70	121·0	164,800,000	140·6	222,900,000	112·0	142,800,000
5·9	2·95	1·72	122·5	180,500,000	142·9	245,100,000	113·8	157,000,000
6·0	3·00	1·73	124·0	198,000,000	144·6	257,000,000	115·6	164,200,000

TABLE IV—for ascertaining the values of $\frac{L}{H}F^2$ and $\frac{L}{H}G^2$ as explained on page 17. For a required discharge of say 88·5 cusecs a mean may be taken between values ascertained for 88 cusecs and 89 cusecs, $\frac{3}{10}$ the difference for 88·3 being added to ascertained value for 88.

Required Discharge	Square of Discharge Multiplied by								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1	1	2	3	4	5	6	7	8	9
2	4	8	12	16	20	24	28	32	36
3	9	18	27	36	45	54	63	72	81
4	16	32	48	64	80	96	112	128	144
5	25	50	75	100	125	150	175	200	225
6	36	72	108	144	180	216	252	288	324
7	49	98	147	196	245	294	343	392	441
8	64	128	192	256	320	384	448	512	576
9	81	162	243	324	405	486	567	648	729
10	100	200	300	400	500	600	700	800	900
11	121	242	363	484	605	726	847	968	1089
12	144	288	432	576	720	864	1008	1152	1296
13	169	338	507	676	845	1014	1183	1352	1521
14	196	392	588	784	980	1176	1372	1568	1764
15	225	450	675	900	1125	1350	1575	1800	2025
16	256	512	768	1024	1280	1536	1792	2048	2304
17	289	578	867	1156	1445	1734	2023	2312	2601
18	324	648	972	1296	1620	1944	2268	2592	2916
19	361	722	1083	1444	1805	2166	2527	2888	3249
20	400	800	1200	1600	2000	2400	2800	3200	3600
21	441	882	1323	1764	2205	2646	3087	3528	3969
22	484	968	1452	1936	2420	2904	3388	3872	4356
23	529	1058	1587	2116	2645	3174	3703	4232	4761
24	576	1152	1728	2304	2880	3456	4032	4608	5184
25	625	1250	1875	2500	3125	3750	4375	5000	5625
Galmins or Cusecs	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**

TABLE IV—*continued*

Required Discharge	Square of Discharge Multiplied by								
	1	2	3	4	5	6	7	8	9
26	676	1352	2028	2704	3380	4056	4732	5408	6084
27	729	1458	2187	2916	3645	4374	5103	5832	6561
28	784	1568	2352	3136	3920	4704	5488	6272	7056
29	841	1682	2523	3364	4205	5046	5887	6728	7569
30	900	1800	2700	3600	4500	5400	6300	7200	8100
31	961	1922	2883	3844	4805	5766	6727	7688	8649
32	1024	2048	3072	4096	5120	6144	7168	8192	9216
33	1089	2178	3267	4356	5445	6534	7623	8712	9801
34	1156	2312	3468	4624	5780	6936	8092	9248	10404
35	1225	2450	3675	4900	6125	7350	8575	9800	11025
36	1296	2592	3888	5184	6480	7776	9072	10368	11664
37	1369	2738	4107	5476	6845	8214	9583	10952	12321
38	1444	2888	4332	5776	7220	8664	10108	11552	12996
39	1521	3042	4563	6084	7605	9126	10647	12168	13689
40	1600	3200	4800	6400	8000	9600	11200	12800	14400
41	1681	3362	5043	6724	8405	10086	11767	13448	15129
42	1764	3528	5292	7056	8820	10584	12348	14112	15876
43	1849	3698	5547	7396	9245	11094	12943	14792	16641
44	1936	3872	5808	7744	9680	11616	13552	15488	17424
45	2025	4050	6075	8100	10125	12150	14175	16900	18225
46	2116	4232	6348	8464	10580	12696	14812	16928	19044
47	2209	4418	6627	8836	11045	13254	15463	17672	19881
48	2304	4608	6912	9216	11520	13824	16128	18432	20736
49	2401	4802	7203	9604	12005	14406	16807	19208	21609
50	2500	5000	7500	10000	12500	15000	17500	20000	22500
Galmins or Cusecs	1	2	3	4	5	6	7	8	9

TABLE IV—*continued*

Required Discharge	Square of Discharge Multiplied by								
	1	2	3	4	5	6	7	8	9
51	2601	5202	7803	10404	13005	15606	18207	20808	23409
52	2704	5408	8112	10816	13520	16224	18928	21632	24336
53	2809	5618	8427	11236	14045	16854	19663	22472	25281
54	2916	5832	8748	11664	14580	17496	20412	23328	26244
55	3025	6050	9075	12100	15125	18150	21175	24200	27225
56	3136	6272	9408	12544	15680	18816	21952	25088	28224
57	3249	6498	9747	12996	16245	19494	22743	25992	29241
58	3364	6728	10092	13456	16820	20184	23548	26912	30276
59	3481	6962	10443	13924	17405	20886	24367	27848	31329
60	3600	7200	10800	14400	18000	21600	25200	28800	32400
61	3721	7442	11163	14884	18605	22326	26047	29768	33489
62	3844	7688	11532	15376	19220	23064	26908	30752	34596
63	3969	7938	11907	15876	19845	23814	27783	31752	35721
64	4096	8192	12288	16384	20480	24576	28672	32768	36864
65	4225	8450	12675	16900	21125	25350	29575	33800	38025
66	4356	8712	13068	17424	21780	22136	30492	34848	39204
67	4489	8978	13467	17956	22445	26934	31423	35912	40401
68	4624	9248	13872	18496	23120	27744	32368	36992	41616
69	4761	9522	14283	19044	23805	28566	33327	38088	42849
70	4900	9800	14700	19600	24500	29400	34300	39200	44100
71	5041	10082	15123	20164	25205	30246	35287	40328	45369
72	5184	10368	15552	20736	25920	31104	36288	41472	46656
73	5329	10658	15987	21316	26645	31974	37303	42632	47961
74	5476	10952	16428	21904	27380	32856	38332	43808	49284
75	5625	11250	16875	22500	28125	33750	39375	45000	50625
Galmins or Cusecs	1	2	3	4	5	6	7	8	9

TABLE IV—*continued*

Required Discharge	Square of Discharge Multiplied by								
	1	2	3	4	5	6	7	8	
76	5776	11552	17328	23104	28880	34656	40432	46208	51984
77	5929	11858	17787	23716	29645	35574	41503	47432	53361
78	6084	12168	18252	24336	30420	36504	42588	48672	54756
79	6241	12482	18723	24964	31205	37446	43687	49928	56169
80	6400	12800	19200	25600	32000	38400	44800	51200	57600
81	6561	13122	19683	26244	32805	39366	45927	52488	59049
82	6724	13448	20172	26896	33620	40344	47068	53792	60516
83	6889	13778	20667	27556	34445	41334	48223	55112	62001
84	7056	14112	21168	28224	35280	42336	49392	56448	63504
85	7225	14450	21675	28900	36125	43350	50575	57800	65025
86	7396	14792	22188	29584	36980	44376	51772	59168	66564
87	7569	15138	22707	30276	37845	45414	52983	60552	68121
88	7744	15488	23232	30976	38720	46464	54208	61952	69696
89	7921	15842	23763	31684	39605	47526	55447	63368	71289
90	8100	16200	24300	32400	40500	48600	56700	64800	72900
91	8281	16562	24843	33124	41405	49686	57967	66248	74529
92	8464	16928	25392	33856	42320	50784	59248	67712	76176
93	8649	17298	25947	34596	43245	51894	60543	69192	77841
94	8836	17672	26508	35344	44180	53016	61852	70688	79524
95	9025	18050	27075	36100	45125	54150	63175	72200	81225
96	9216	18432	27648	36864	46080	55296	64512	73728	82944
97	9409	18818	28227	37636	47045	56464	65863	75272	84681
98	9604	19208	28812	38416	48020	57624	67228	76832	86436
99	9801	19602	29403	39204	49005	58806	68607	78408	88209
100	10000	20000	30000	40000	50000	60000	70000	80000	90000
Galmins or Cusecs	1	2	3	4	5	6	7	8	9

TABLE V—giving the areas and values of R, $C\sqrt{R}$, &c., for drains on Type II from 6 ft. to 8 ft. deep.

$\delta = \frac{D}{2}$	R	$\sqrt{R}$	Area, with side slopes 1 to 1	Other slopes	Values of $C\sqrt{R}$		
					Kutter $n = 0·025$	Bazin (i)	Bazin (ii)
6·0	3·0	1·73	65·88		124·0	144·6	115·6
6·1	3·05	1·75	68·09		125·4	146·8	117·4
6·2	3·10	1·76	70·35		126·8	148·5	118·9
6·3	3·15	1·78	72·63		128·2	150·2	120·3
6·4	3·20	1·79	74·96		129·6	151·8	121·7
6·5	3·25	1·80	77·32	Multiply 1 to	131·0	153·4	123·2
6·6	3·30	1·82	79·71	1 areas by	132·4	155·1	124·6
6·7	3·35	1·83	81·15	1·09 for 0 to	133·8	156·8	126·0
6·8	3·40	1·84	84·62	1; by 0·95	135·2	158·4	127·4
6·9	3·45	1·86	87·13	for ½ to 1;	136·6	160·0	128·9
7·0	3·50	1·87	89·67	1·15 for 1½	138·0	161·7	130·3
7·1	3·55	1·88	92·25	to 1; 1·35	139·3	163·4	131·7
7·2	3·60	1·90	94·87	for 2 to 1;	140·6	165·0	133·2
7·3	3·65	1·91	97·52	1·58 for 2½	141·9	166·7	134·6
7·4	3·70	1·93	100·21	to 1; 1·82	143·2	168·3	136·0
7·5	3·75	1·94	102·94	for 3 to 1.	144·5	170·0	137·5
7·6	3·80	1·95	105·70		145·8	171·6	138·9
7·7	3·85	1·96	108·50		147·1	173·3	140·3
7·8	3·90	1·97	111·34		148·4	174·9	141·7
7·9	3·95	1·98	114·21		149·7	176·6	143·2
8·0	4·00	2·00	117·12		151·0	178·2	144·6

TABLE VI—giving the areas (A), bedwidths
slopes of 0 to 1 to 3

Side slopes	0 to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	2·00	2·00	4·00	2·00
1·1	2·42	2·20	4·40	2·20
1·2	2·88	2·40	4·80	2·40
1·3	3·38	2·60	5·20	2·60
1·4	3·92	2·80	5·60	2·80
1·5	4·50	3·00	6·00	3·00
1·6	5·12	3·20	6·40	3·20
1·7	5·78	3·40	6·80	3·40
1·8	6·48	3·60	7·20	3·60
1·9	7·22	3·80	7·60	3·80

Side slopes	½ to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	1·74	1·24	3·48	2·24
1·1	2·10	1·36	3·83	2·47
1·2	2·50	1·49	4·17	2·68
1·3	2·94	1·61	4·52	2·91
1·4	3·41	1·74	4·87	3·13
1·5	3·92	1·86	5·22	3·36
1·6	4·45	1·98	5·57	3·59
1·7	5·03	2·11	5·92	3·81
1·8	5·64	2·23	6·26	4·03
1·9	6·28	2·36	6·61	4·25

Side slopes	1 to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	1·83	0·83	3·66	2·83
1·1	2·21	0·91	4·03	3·12
1·2	2·63	0·99	4·39	3·40
1·3	3·09	1·08	4·76	3·68
1·4	3·58	1·16	5·12	3·96
1·5	4·12	1·24	5·49	4·25
1·6	4·68	1·32	5·86	4·54
1·7	5·28	1·41	6·22	4·81
1·8	5·93	1·49	6·59	5·10
1·9	6·61	1·58	6·95	5·37

Side slopes	1½ to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	2·11	0·61	4·22	3·61
1·1	2·55	0·67	4·64	3·97
1·2	3·04	0·73	5·06	4·33
1·3	3·57	0·79	5·49	4·70
1·4	4·14	0·85	5·91	5·06
1·5	4·74	0·92	6·33	5·41
1·6	5·40	0·98	6·75	5·77
1·7	6·10	1·04	7·17	6·13
1·8	6·83	1·10	7·60	6·50
1·9	7·62	1·17	8·02	6·85

Side slopes	2 to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	2·47	0·47	4·94	4·47
1·1	2·98	0·52	5·43	4·91
1·2	3·55	0·56	5·93	5·37
1·3	4·17	0·61	6·42	5·81
1·4	4·84	0·66	6·92	6·25
1·5	5·56	0·71	7·41	6·70
1·6	6·32	0·75	7·90	7·15
1·7	7·14	0·80	8·40	7·60
1·8	8·00	0·85	8·88	8·03
1·9	8·92	0·89	9·39	8·50

Side slopes	2 to 1½			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	2·89	0·39	5·78	5·39
1·1	3·50	0·43	6·36	5·93
1·2	4·16	0·47	6·94	6·47
1·3	4·88	0·51	7·51	7·00
1·4	5·66	0·55	8·09	7·54
1·5	6·50	0·59	8·67	8·08
1·6	7·40	0·62	9·25	8·63
1·7	8·35	0·66	9·83	9·17
1·8	9·36	0·70	10·40	9·70
1·9	10·43	0·74	10·98	10·24

Side slopes	3 to 1			
$\delta = \frac{D}{2}$	A	b	P	P−b
1·0	3·33	0·33	6·66	6·33
1·1	4·03	0·36	7·33	6·97
1·2	4·80	0·39	8·00	7·61
1·3	5·63	0·43	8·66	8·23
1·4	6·53	0·46	9·33	8·87
1·5	7·49	0·49	10·00	9·51
1·6	8·53	0·53	10·66	10·13
1·7	9·62	0·56	11·32	10·76
1·8	10·99	0·59	11·99	11·40
1·9	12·02	0·63	12·65	12·02

(*b*), perimeters (P), &c., of drains on Type II, with side to 1. (See Plate II.)

2·9	2·8	2·7	2·6	2·5	2·4	2·3	2·2	2·1	2.0	$\delta = \frac{D}{2}$	Side slopes
16·82	15·68	14·58	13·52	12·50	11·52	10·58	9·68	8·82	8·00	A	0 to 1
5·80	5·60	5·40	5·20	5·00	4·80	4·60	4·44	4·20	4·00	*b*	
11·60	11·20	10·80	10·40	10·00	9·60	9·20	8·80	8·40	8·00	P	
5·80	5·60	5·40	5·20	5·00	4·80	4·60	4·40	4·20	4·00	P−*b*	
14·63	13·64	12·68	11·76	10·88	10·02	9·20	8·42	7·67	6·96	A	½ to 1
3·60	3·47	3·34	3·22	3·10	2·97	2·85	2·73	2·60	2·48	*b*	
10·09	9·74	9·40	9·05	8·70	8·35	8·00	7·66	7·31	6·96	P	
6·49	6·27	6·06	5·83	5·60	5·38	5·15	4·93	4·71	4·48	P−*b*	
15·39	14·35	13·34	12·37	11·43	10·54	9·68	8·85	8·07	7·32	A	1 to 1
2·41	2·32	2·24	2·16	2·07	1·99	1·91	1·83	1·74	1·66	*b*	
10·61	10·25	9·88	9·52	9·15	8·78	8·42	8·05	7·69	7·32	P	
8·20	7·93	7·64	7·36	7·07	6·79	6·51	6·22	5·95	5·66	P−*b*	
17·75	16·54	15·58	14·26	13·10	12·15	11·16	10·21	9·30	8·44	A	1½ to 1
1·77	1·71	1·64	1·58	1·52	1·46	1·40	1·34	1·28	1·22	*b*	
12·24	11·82	11·39	10·97	10·55	10·13	9·70	9·28	8·86	8·44	P	
10·47	10·11	9·75	9·39	9·03	8·64	8·30	7·94	7·58	7·22	P−*b*	
20·77	19·35	18·00	16·70	15·44	14·23	13·07	11·95	10·89	9·88	A	2 to 1
1·36	1·32	1·27	1·22	1·18	1·13	1·08	1·03	0·99	0·94	*b*	
14·33	13·83	13·34	12·84	12·35	11·86	11·36	10·87	10·37	9·88	P	
12·97	12·51	12·07	11·62	11·17	10·73	10·28	9·84	9·38	8·94	P−*b*	
24·30	22·66	21·07	19·54	18·06	16·65	15·29	13·99	12·75	11·56	A	2½ to 1
1·13	1·09	1·05	1·01	0·98	0·94	0·90	0·86	0·82	0·78	*b*	
16·76	16·18	15·61	15·03	14·45	13·87	13·29	12·72	12·14	11·56	P	
15·63	15·09	14·56	14·02	13·47	12·93	12·39	11·86	11·32	10·78	P−*b*	
28·00	26·11	24·28	22·51	20·81	19·18	17·62	16·12	14·69	13·32	A	3 to 1
0·96	0·92	0·89	0·86	0·83	0·79	0·76	0·73	0·69	0·66	*b*	
19·31	18·65	17·98	17·32	16·65	15·98	15·32	14·65	13·99	13·32	P	
18·35	17·73	17·09	16·49	15·82	15·19	14·56	13·92	13·30	12·66	P−*b*	

TABLE VI—*continued*

3·9	3·8	3·7	3·6	3·5	3·4	3·3	3·2	3·1	3·0	$\delta = \frac{D}{2}$	Side slopes.
30·42	28·88	27·38	25·92	24·50	23·12	21·78	20·48	19·22	18·00	A	0 to 1
7·80	7·60	7·40	7·20	7·00	6·80	6·60	6·40	6·20	6·00	*b*	
15·60	15·20	14·80	14·40	14·00	13·60	13·20	12·80	12·40	12·00	P	
7·60	7·60	7·40	7·20	7·00	6·80	6·60	6·40	6·20	6·00	P−*b*	
26·46	25·12	23·82	22·55	21·32	20·11	18·95	17·82	16·72	15·66	A	½ to 1
4·83	4·71	4·59	4·46	4·34	4·21	4·09	3·97	3·84	3·72	*b*	
13·57	13·22	12·88	12·53	12·18	11·83	11·48	11·14	10·79	10·44	P	
8·74	8·51	8·29	8·01	7·84	7·62	7·39	7·17	6·95	6·72	P−*b*	
27·83	26·43	25·04	23·72	22·41	21·15	19·92	18·74	17·58	16·47	A	1 to 1
3·23	3·15	3·07	2·99	2·91	2·82	2·73	2·65	2·57	2·49	*b*	
14·27	13·90	13·54	13·18	12·81	12·44	12·08	11·71	11·35	10·98	P	
11·04	10·75	10·47	10·19	9·90	9·62	9·35	9·06	8·78	8·49	P−*b*	
32·09	30·47	28·89	27·34	25·85	24·39	22·93	21·61	20·27	18·99	A	1½ to 1
2·38	2·32	2·26	2·19	2·13	2·07	2·01	1·95	1·89	1·83	*b*	
16·46	16·04	15·61	15·19	14·77	14·35	13·93	13·50	13·08	12·66	P	
14·08	13·72	13·35	13·00	12·64	12·28	11·92	11·55	11·19	10·83	P−*b*	
37·57	35·67	33·81	32·01	30·26	28·55	26·85	25·29	23·74	22·23	A	2 to 1
1·83	1·79	1·74	1·69	1·65	1·60	1·55	1·50	1·46	1·41	*b*	
19·25	18·77	18·28	17·78	17·29	16·80	16·30	15·80	15·31	14·82	P	
17·42	16·98	16·54	16·09	15·64	15·20	14·75	14·30	13·85	13·41	P−*b*	
43·96	41·73	39·56	37·45	35·40	33·41	31·47	29·60	27·77	26·01	A	2½ to 1
1·52	1·48	1·44	1·41	1·37	1·33	1·29	1·25	1·21	1·17	*b*	
22·54	21·96	21·39	20·81	20·23	19·65	19·01	18·50	17·92	17·34	P	
21·02	20·48	19·95	19·40	18·86	18·32	17·78	17·25	16·71	16·17	P−*b*	
50·65	48·09	45·59	43·16	40·79	38·49	36·26	34·10	31·00	29·97	A	3 to 1
1·29	1·25	1·22	1·19	1·16	1·12	1·09	1·06	1·02	0·99	*b*	
25·97	25·31	24·64	23·98	23·31	22·64	21·98	21·31	20·65	19·98	P	
24·68	24·06	23·42	22·79	22·15	21·52	20·89	20·25	19·63	18·99	P−*b*	

TABLE VI—*continued*

4·9	4·8	4·7	4·6	4·5	4·4	4·3	4·2	4·1	4·0	$\delta = \frac{D}{2}$	Side slopes.
48·02	46·08	44·18	42·32	40·50	38·72	36·98	35·28	33·62	32·00	A	0 to 1
9·80	9·60	9·40	9·20	9·00	8·80	8·60	8·40	8·20	8·00	*b*	
19·60	19·20	18·80	18·40	18·00	17·60	17·20	16·80	16·40	16·00	P	
9·80	9·60	9·40	9·20	9·00	8·80	8·60	8·40	8·20	8·00	P−*b*	
41·77	40·09	38·44	36·82	35·23	33·69	31·17	30·69	29·25	27·84	A	½ to 1
6·07	5·95	5·82	5·70	5·58	5·4	5·33	5·20	5·08	4·96	*b*	
17·05	16·70	16·36	16·00	15·66	15·31	14·96	14·62	14·27	13·92	P	
10·98	10·75	10·54	10·30	10·08	9·85	9·63	9·42	9·19	8·96	P−*b*	
43·93	42·16	40·42	38·72	37·06	35·42	33·83	32·28	30·76	29·28	A	1 to 1
4·07	3·98	3·90	3·82	3·73	3·65	3·57	3·48	3·41	3·32	*b*	
17·93	17·57	17·20	16·84	16·47	16·10	15·74	15·37	15·00	14·64	P	
13·86	13·59	13·30	13·02	12·74	12·45	12·17	11·89	11·59	11·32	P−*b*	
50·66	48·61	46·91	44·65	42·73	40·85	39·01	37·22	35·47	33·76	A	1½ to 1
2·99	2·93	2·87	2·81	2·74	2·68	2·62	2·56	2·50	2·44	*b*	
20·68	20·26	19·83	19·41	18·99	18·57	18·15	17·72	17·30	16·88	P	
17·69	17·33	16·96	16·60	16·25	15·89	15·53	15·12	14·80	14·44	P−*b*	
59·30	56·91	54·56	52·27	50·02	47·82	45·65	43·57	41·52	39·52	A	2 to 1
2·30	2·25	2·21	2·16	2·11	2·07	2·02	1·97	1·93	1·88	*b*	
24·21	23·71	23·22	22·72	22·23	21·74	21·24	20·75	20·25	19·76	P	
21·91	21·46	21·01	20·56	20·12	19·67	19·22	18·78	18·32	17·88	P−*b*	
69·39	66·59	63·84	61·15	58·52	55·95	53·44	50·98	48·48	46·24	A	2½ to 1
1·91	1·87	1·83	1·79	1·76	1·72	1·68	1·64	1·60	1·56	*b*	
28·32	27·74	27·17	26·59	26·01	25·43	24·85	24·28	23·70	23·12	P	
26·41	25·87	25·34	24·80	24·25	23·71	23·17	22·64	22·10	21·56	P−*b*	
79·96	76·72	73·56	70·46	67·43	63·47	61·57	58·74	56·28	53·28	A	3 to 1
1·62	1·58	1·55	1·52	1·49	1·45	1·42	1·39	1·35	1·32	*b*	
32·63	31·97	31·30	30·64	29·97	29·30	28·64	27·97	27·31	26·64	P	
31·01	30·39	29·75	29·12	28·48	27·85	27·22	26·58	25·96	25·32	P−*b*	

TABLE VI—*continued*

6·0	5·9	5·8	5·7	5·6	5·5	5·4	5·3	5·2	5·1	5·0	$\delta = \frac{D}{2}$	Side slopes
72·00	69·62	67·28	64·98	62·72	60·50	58·32	56·18	54·08	52·02	50·00	A	0 to 1
12·00	11·80	11·60	11·40	11·20	11·00	10·80	10·60	10·40	10·20	10·00	*b*	
24·00	23·60	23·20	22·80	22·40	22·00	21·60	21·20	20·80	20·40	20·00	P	
12·00	11·80	11·60	11·40	11·20	11·00	10·80	10·60	10·40	10·20	10·00	P−*b*	
62·64	60·57	58·53	56·53	54·57	52·64	50·74	48·88	47·05	45·26	43·50	A	½ to 1
7·44	7·32	7·19	7·07	6·94	6·82	6·70	6·57	6·45	6·32	6·20	*b*	
20·88	20·53	20·18	19·84	19·49	19·14	18·79	18·44	18·11	17·75	17·40	P	
13·44	13·21	12·99	12·77	12·55	12·32	12·09	11·87	11·66	11·43	11·20	P−*b*	
65·88	63·70	61·56	59·46	57·88	55·36	53·36	51·40	49·48	47·62	45·75	A	1 to 1
4·98	4·90	4·81	4·73	4·65	4·57	4·48	4·40	4·32	4·23	4·15	*b*	
21·96	21·59	21·23	20·86	20·50	20·13	19·76	19·40	19·03	18·67	18·30	P	
16·98	16·69	16·42	16·13	15·85	15·56	15·28	15·00	14·71	14·44	14·15	P−*b*	
75·96	73·45	70·98	68·55	66·27	63·83	61·53	59·27	57·05	54·88	52·75	A	1½ to 1
3·66	3·60	3·54	3·48	3·42	3·36	3·29	3·23	3·17	3·11	3·05	*b*	
25·32	24·90	24·48	24·05	23·63	23·21	22·79	22·37	21·94	21·52	21·10	P	
21·66	21·30	20·94	20·57	20·21	19·85	19·50	19·14	18·77	18·41	18·05	P−*b*	
88·92	85·98	83 09	80·25	77·46	74·72	72·03	69·38	66·79	64·24	61·75	A	2 to 1
2·82	2·77	2·73	2·68	2·63	2·59	2·54	2·49	2·44	2·40	2·35	*b*	
29·64	29·15	28·65	28·16	27·66	27·17	26·68	26·18	25·69	25·19	24·70	P	
26·82	26·38	25·92	25·48	25·03	24·58	24·14	23·69	23·25	22·79	22·35	P−*b*	
104·04	100·60	97·22	93·90	90·63	87·42	84·27	81·18	78·16	75·17	72·25	A	2½ to 1
2·34	2·30	2·26	2·22	2·18	2·15	2·11	2·07	2·03	1·99	1·95	*b*	
34·68	34·10	33·52	32·95	32·37	31·79	31·21	30·63	30·06	29·48	28·90	P	
32·34	31·80	31·26	30·73	30·19	29·64	29·10	28·56	28·03	27·49	26·65	P−*b*	
119·88	115·92	112·02	108·19	104·43	100·73	97·10	93·54	90·04	86·61	83·25	A	3 to 1
1·98	1·95	1·91	1·88	1·85	1·82	1·78	1·75	1·72	1·68	1·65	*b*	
39·96	39·29	38·63	37·96	37·30	36·63	35·96	35·30	34·63	33·97	33·30	P	
37·98	37·34	36·72	36·08	35·45	34·81	34·18	33·55	32·91	32·29	31·65	P−*b*	

TABLE VII—for use in ascertaining the dimensions of very large pipes and drains. (See pages 21 and 22.)

Table of Fifth Powers of Numbers

No.	Fifth Power	No.	Fifth Power	No.	Fifth Power
1	1	35	52,521,875	68	1,453,933,568
2	32	36	60,466,176	69	1,564,031,349
3	243	37	69,343,957	70	1,684,700,000
4	1,024	38	79,235,168	71	1,804,229,351
5	3,125	39	90,224,199	72	1,934,917,632
6	7,776	40	102,400,000	73	2,073,071,593
7	16,807	41	115,856,201	74	2,219,006,624
8	32,768	42	130,691,232	75	2,373,046,875
9	59,049	43	147,008,443	76	2,535,525,376
10	100,000	44	164,916,224	77	2,706,784,157
11	161,051	45	184,528,125	78	2,887,174,368
12	248,832	46	205,962,976	79	3,077,056,399
13	371,293	47	229,345,007	80	3,276,800,000
14	537,824	48	254,803,968	81	3,486,784,401
15	759,375	49	282,475,249	82	3,707,398,432
16	1,048,576	50	312,500,000	83	3,939,040,643
17	1,419,857	51	345,025,251	84	4,182,119,424
18	1,889,568	52	380,204,032	85	4,437,053,125
19	2,476,099	53	418,195,493	86	4,704,270,176
20	3,200,000	54	459,165,024	87	4,984,209,207
21	4,084,101	55	503,284,375	88	5,277,319,168
22	5,153,632	56	550,731,776	89	5,584,059,449
23	6,436,343	57	601,692,057	90	5,904,900,000
24	7,962,624	58	656,356,768	91	6,240,321,451
25	9,765,625	59	714,924,299	92	6,590,815,232
26	11,881,376	60	777,600,000	93	6,956,883,693
27	14,348,907	61	844,596,301	94	7,339,040,224
28	17,210,368	62	916,132,832	95	7,737,809,375
29	20,511,149	63	992,436,543	96	8,153,726,976
30	24,300,000	64	1,073,741,824	97	8,587,340,257
31	28,629,151	65	1,160,290,625	98	9,039,207,968
32	33,554,432	66	1,252,332,576	99	9,509,900,499
33	39,135,393	67	1,350,125,107	100	10,000,000,000
34	45,435,424				

TABLE VIII—giving the values of x and y for ascertaining Kutter's values of C (with $n = 0{\cdot}013$) from the formula

$$C = \frac{x\sqrt{R}}{\sqrt{R} + y}$$

I Virtual Slope 1 in	II x	III y	I Virtual Slope 1 in	II x	III y
100	181·250	0·545230	1300	184·622	0·589066
200	181·531	0·548883	1400	184·903	0·592719
300	181·812	0·552536	1500	185·184	0·596372
400	182·093	0·556189	1600	185·465	0·600025
500	182·374	0·559842	1700	185·746	0·603678
600	182·655	0·563495	1800	186·027	0·607331
700	182·936	0·567148	1900	186·308	0·610984
800	183·217	0·570801	2000	186·589	0·614637
900	183·498	0·574454	2100	186·870	0·618290
1000	183·779	0·578107	2200	187·151	0·621943
1100	184·060	0·581760	2300	187·432	0·625596
1200	184·341	0·585413	2400	187·713	0·629249
Add for each extra 100	0·281	0·003653	Add for each extra 100	0·281	0·003653

APPENDIX A—comparing the values of $\frac{L}{H}F^2$ for some pipes from 6 in. to 48 in. in diameter for differences of one inch, the velocities in the pipes being the same and ranging from 1 ft. to 5 ft. per sec., the values of $\frac{L}{H}F^2$ being calculated from Kutter's coefficients with $n = 0·013$ and Unwin's coefficients for clean pipes equivalent very nearly to Kutter's coefficients with $n = 0·011$. See Appendix B.

Velocities in feet per second															
1 to 2				2 to 3				3 to 4				4 to 5			
Kutter $n = 0·013$		Unwin clean pipe		Kutter $n = 0·013$		Unwin clean pipe		Kutter $n = 0·013$		Unwin clean pipe		Kutter $n = 0·013$		Unwin clean pipe	
d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$	d	$\frac{L}{H}F^2$
in.		in.		in.		in.		in.		in.		in.		in.	
7	57	6	54	7	57	6	56	7	58	6	57	7	58	6	58
10	415	9	423	10	419	9	430	10	421	9	446	10	423	9	455
13	1,780	12	1,880	13	1,780	12	1,920	13	1,800	12	1,960	13	1,800	12	1,990
19	13,800	18	15,000	19	14,200	18	15,400	19	14,400	18	15,760	19	14,500	18	16,000
25	60,000	24	67,900	25	63,000	24	69,400	25	65,000	24	70,900	25	65,400	24	72,400
37	508,000	36	538,000	37	527,000	36	563,700	37	537,000	36	577,400	37	542,000	36	577,400
49	2,313,000	48	2,380,000	49	2,381,000	48	2,432,800	49	2,398,000	48	2,491,000	49	2,416,000	48	2,491,000

APPENDIX B—comparing the values of $\frac{L}{H}F^2$ for some pipes from 6 in. to 48 in. in diameter with velocities of from 1 ft. to 5 ft. per second deduced from Kutter's Coefficients (with $n = 0{\cdot}011$ exactly) from Unwin's Coefficients for asphalted iron pipes, and from Fanning's values of C for clean pipes; also, for all velocities, from Box's fixed value of C (91·6) for clean pipes.

Velocities in feet per second	1 to 2			2 to 3			3 to 4			4 to 5			All
	Values of $\frac{L}{H}F^2$												
Pipe diameters in inches	Kutter $n = 0{\cdot}011$	Unwin asphalted	Fanning clean	Kutter $n = 0{\cdot}011$	Unwin asphalted	Fanning Clean	Kutter $n = 0{\cdot}011$	Unwin asphalted	Fanning clean	Kutter $n = 0{\cdot}011$	Unwin asphalted	Fanning clean	Box clean
										Diameters from Table I Col. II., in parenthesis			
in. 6	36	49	45	36	53	49	38	56	50	(6″) 38	(6″) 57	(6″) 52	(6″) 41
9	375	377	382	376	410	396	377	432	410	(9″) 378	(9″) 445	(9″) 420	(9″) 307
12	1,700	1,640	1,670	1,730	1,790	1,766	1,760	1,880	1,820	(12″) 1,780	(12″) 1,950	(12″) 1,850	(12″) 1,290
18	16,100	13,000	4,100	16,200	14,000	14,600	16,300	14,900	15,000	(18″) 16,400	(18″) 15,000	(18″) 15,500	(17″) 9,830
24	73,700	58,000	66,000	75,800	62,700	68,000	77,500	66,700	69,600	(25″) 79,000	(24″) 69,000	(25″) 71,000	(22″) 41,400
36	658,000	459,000	576,000	668,000	499,000	585,000	676,000	529,000	593,000	(37″) 682,000	(36″) 549,000	(37″) 608,000	(33″) 314,400
48	2,900,000	1,970,000	2,750,000	2,980,000	2,140,000	2,770,000	3,070,000	2,275,000	2,800,000	(50″) 3,160,000	(48″) 2,380,000	(50″) 2,850,000	(43″) 1,325,000

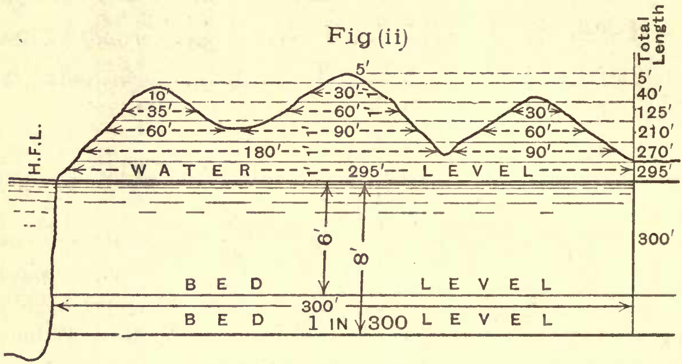
APPENDIX C—comparing the values of $\frac{L}{H}F^2$ for incrusted pipes from 6 in. to 48 in. in diameter derived for all velocities from Unwin's and Silk's values of C for incrusted pipes with some high velocity (4 ft. to 5 ft. per second) values of $\frac{L}{H}F^2$ for pipes calculated from Unwin's coefficients for clean pipes (Appendix A).

Pipe diameters	Values of $\frac{L}{H}F^2$ Unwin Clean	Unwin Incrusted	Silk Incrusted	Pipe diameters	Values of $\frac{L}{H}F^2$ Unwin Clean	Unwin Incrusted	Silk Incrusted
in.				in.			
6	1″ 58	26	28	29		83,600	73,400
7		56	59	30	229,400	101,700	87,800
8		112	116	31		122,600	101,000
9		205	212	32		145,200	116,000
10		346	355	33	5″	170,000	135,000
11		623	594	34		197,000	158,000
12	1,900	908	900	35		226,000	185,000
13	2″	1,340	1,300	36	577,000	260,600	218,700
14		1,960	1,900	37		297,000	248,000
15		2,800	2,700	38		338,600	282,000
16		3,500	3,400	39	6″	382,700	324,000
17		5,400	4,800	40		433,000	366,000
18	16,000	7,300	6,800	41		488,000	413,000
19		9,800	8,800	42	1,276,000	569,000	472,500
20	3″	13,000	11,300	43		664,000	525,000
21		16,000	14,400	44		740,000	585,000
22		20,700	18,600	45	7″	825,000	660,000
23		26,000	22,400	46		916,000	733,000
24	72 ,400	32,400	28,800	47		1,020,000	817,000
25		39,500	35,300	48		1,128,000	922,000
26	4″	47,800	42,600	49		1,228,000	
27		57,800	52,200				
28		69,400	62,100				

APPENDIX D

THE quantities of earthwork in the two drain sections if carried for a length of 300 ft. in deep cutting through ground having the longitudinal section illustrated in Fig. (ii) would, calculated from Table IX [which has been prepared from the author's 'Practical

Fig (ii)



HORIZONTAL SCALE 100′ = 1″
VERTICAL SCALE 10′ = 1″

Earthwork Tables' (Longmans, Green, & Co., 1907) and is used in the manner therein advocated], be as follows, the cross slope in the ground, if not too great, being entirely neglected as practically it makes, for any given section, but little difference—see Fig. (iii) below.

(a) *In the* 8 *ft. deep drain, bedwidth* 6·7 *ft., side slopes* 1 *to* 1.

Depth of drain	Central area			End areas (Table IX)	Total area	Length	Contents
ft.	ft.	ft.	sq. ft.	sq. ft.	sq. ft.	ft.	c. ft.
1 to 8	6·7 ×	8	= 53·6	+ 64	= 117·6	× 300 =	35,280
9	6·7 ×	1	= 6·7	+ 17	= 23·7	× 295 =	6,992
10	6·7 ×	1	= 6·7	+ 19	= 25·7	× 270 =	6,939
11	6·7 ×	1	= 6·7	+ 21	= 27·7	× 210 =	5,817
12	6·7 ×	1	= 6·7	+ 23	= 29·7	× 125 =	3,712
13	6·7 ×	1	= 6·7	+ 25	= 31·7	× 40 =	1,268
13½	6·7 ×	½	= 3·3	+ 13·25	= 16·55	× 5 =	83
					Total	. .	60,091

(b) *In the* 6 *ft. deep drain, bedwidth* 14 *ft., side slopes* 1 *to* 1.

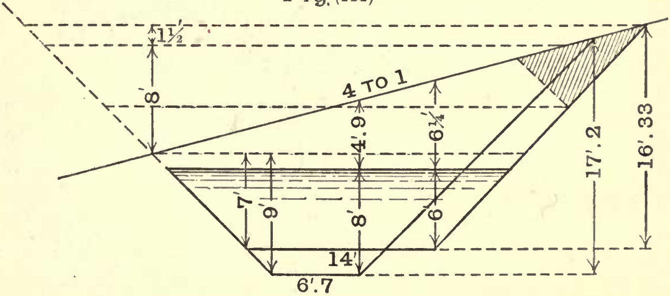
Depth of drain	Central area		End areas (Table IX)	Total area	Length	Contents
ft.	ft.	ft. sq. ft.	sq. ft.	sq. ft.	ft.	c. ft.
1 to 6	14 × 6	= 84 +	36	= 120	× 300 =	36,000
7	14 × 1	= 14 +	13	= 27	× 295 =	7,965
8	14 × 1	= 14 +	15	= 29	× 270 =	7,830
9	14 × 1	= 14 +	17	= 31	× 210 =	6,510
10	14 × 1	= 14 +	19	= 33	× 125 =	4,125
11	14 × 1	= 14 +	21	= 35	× 40 =	1,400
$11\frac{1}{2}$	14 × $\frac{1}{2}$	= 7 +	11·25	= 18·25	× 5 =	91
				Total	. .	63,921

or an increase of about 3,900 c. ft. or $\frac{3900}{300} = 13$ c. ft. per foot run.

2. The quantities of earthwork in a cutting or bank *however long* carried through or over ground with a longitudinal contour *however varied* can be similarly ascertained from Table IX, or from the fuller Tables given in the Author's 'Practical Earthwork Tables' above referred to, the preparation of a large number of cross sections being entirely avoided as well as the subsequent calculations therefrom.

3. When the ground through which a drain has to be carried has a considerable cross slope which cannot well be neglected, the cross sections of the two drains would be as shown in Fig. (iii) which has been prepared for a cross slope of 4 to 1.

Fig. (iii)



The maximum depths ascertained by calculation being—

(i) For the 8 ft. drain $\frac{12{\cdot}9 \times 4}{3} = 17{\cdot}2 = \text{say } 17 \text{ ft.}$

(ii) For the 6 ft. drain $\frac{12{\cdot}25 \times 4}{3} = 16{\cdot}33 = \text{say } 16{\cdot}5 \text{ ft.}$

The respective areas ascertained from Table IX are:

(a) *In the* 8 *ft. drain, maximum depth* 17 *ft.*

$17 \times 6{\cdot}7 + 289 = 403$ less 160 (the end areas for a depth of 8 ft. with $S + S' = 1 + 4 = 5$) $= 243$ sq. ft.

(b) *In the* 6 *ft. drain, maximum depth* 16½ *ft.*

$16{\cdot}5 \times 14 + 272{\cdot}25 = 503{\cdot}25$ less $225{\cdot}625$ (the end areas for a depth of 9½ ft. with $S + S' = 5$) $= 277{\cdot}625$ sq. ft., say 278 sq. ft.

An increase of 35 c. ft. per foot run.

TABLE IX—giving the end areas of drains on Type II for various depths and side slopes.

Depth of drain in feet	Side Slopes													
	½ to 1 (S+S′=1)		1 to 1 (S+S′=2)		1½ to 1 (S+S′=3)		2 to 1 (S+S′=4)		2½ to 1 (S+S′=5)		3 to 1 (S+S′=6)		4 to 1 (S+S′=8)	
	Areas in square feet													
	One foot layer	Total	One foot layer	Total	One foot layer	Total	One foot layer	Total	One foot layer	Total	One foot layer	Total	One foot layer	Total
1	0·5	0·5	1	1	1·5	1·5	2	2	2·5	2·5	3	3	4	4
2	1·5	2·0	3	4	4·5	6·0	6	8	7·5	10·0	9	12	12	16
3	2·5	4·5	5	9	7·5	13·5	10	18	12·5	22·5	15	27	20	36
4	3·5	8·0	7	16	10·5	24·0	14	32	17·5	40·0	21	48	28	64
5	4·5	12·5	9	25	13·5	37·5	18	50	22·5	62·5	27	75	36	100
6	5·5	18·0	11	36	16·5	54·0	22	72	27·5	90·0	33	108	44	144
7	6·5	24·5	13	49	19·5	73·5	26	98	32·5	122·5	39	147	52	196
8	7·5	32·0	15	64	22·5	96·0	30	128	37·5	160·0	45	192	60	256
9	8·5	40·5	17	81	25·5	121·5	34	162	42·5	202·5	51	243	68	324
10	9·5	50·0	19	100	28·5	150·0	38	200	47·5	250·0	57	300	76	400
11	10·5	60·5	21	121	31·5	181·5	42	242	52·5	302·5	63	363	84	484
12	11·5	72·0	23	144	34·5	216·0	46	288	57·5	360·0	69	432	92	576
13	12·5	84·5	25	169	37·5	253·5	50	338	62·5	422·5	75	507	100	676
14	13·5	98·0	27	196	40·5	294·0	54	392	67·5	490·0	81	588	108	784
15	14·5	112·5	29	225	43·5	337·5	58	450	72·5	562·5	87	675	116	900
16	15·5	128·0	31	256	46·5	384·0	62	512	77·5	640·0	93	768	124	1024
17	16·5	144·5	33	289	49·5	433·5	66	578	82·5	722·5	99	867	132	1156
18	17·5	162·0	35	324	52·5	486·0	70	648	87·5	810·0	105	972	140	1296
19	18·5	180·5	37	361	55·5	541·5	74	722	92·5	902·5	111	1083	148	1444
20	19·5	200·0	39	400	58·5	600·0	78	800	97·5	1000·0	117	1200	156	1600
Additions for half feet	½ mean of 1′ layers + 0·125		½ mean of 1′ layers + 0·25		½ mean of 1′ layers + 0·375		½ mean of 1′ layers + 0·50		½ mean of 1′ layers + 0·625		½ mean of 1′ layers + 0·75		½ mean of 1′ layers + 1·0	

APPENDIX E

THAT the calculation of the exact dimensions of masonry or concrete drains makes an appreciable saving in cost is easily demonstrated.

Suppose that the ascertained 'hydraulic gradient' is 1 in 1000 and the required discharge 0·2 cusecs. Then for a semicircular concrete drain ($n = 0{\cdot}013$) $\frac{4L}{H}F^2 = 4 \times 1000 \times (0{\cdot}2)^2 = 160$, and from Table I $d = 9$ in., but as $(227 - 116)\frac{1}{10} = 11$ and as $160 - 116 = 44$, d more exactly $= 8{\cdot}4$ in. Taking it at 8·5 in. and assuming that the top widths of the sides of the concrete drains (backs vertical) are each 4 in. and that the depth of concrete below the drain bottom is 4 in. also, the concrete areas in the two drains will respectively be:

$$\text{With } d = 9 \text{ in. } A = 17 \text{ in.} \times 8\tfrac{1}{2} \text{ in.} - 0{\cdot}4 \times 9^2$$
$$= 144{\cdot}5 - 32{\cdot}4 = 112 \text{ sq. in.}$$

$$\text{With } d = 8\tfrac{1}{2} \text{ in. } A = 16\tfrac{1}{2} \text{ in.} \times 8\tfrac{1}{4} \text{ in.} - 0{\cdot}4 \times 8{\cdot}5^2$$
$$= 136{\cdot}1 - 28{\cdot}9 = 107 \text{ sq. in.}$$

or a saving of 5 sq. in. in 112 sq. in. or 4·5 per cent.

A concrete drain on Type I with $d = 9$ in. would, under similar circumstances, discharge 0·4 cusecs, and the concrete areas, would for a similarly dimensioned drain, be:

$$\text{With } d = 9 \text{ in. } A = 17 \text{ in.} \times 13 \text{ in.} - 0{\cdot}7 \times 9^2$$
$$= 221 - 56{\cdot}7 = 164{\cdot}3 \text{ sq. in.}$$

$$\text{With } d = 8{\cdot}5 \text{ in. } A = 16\tfrac{1}{2} \text{ in.} \times 12\tfrac{1}{2} \text{ in.} - 0{\cdot}7 \times 8{\cdot}5^2$$
$$= 206{\cdot}3 - 50{\cdot}6 = 155{\cdot}7 \text{ sq. in.}$$

or a saving of 8·6 sq. in. in 164·3 sq. in. or 5·2 per cent.

In larger drains on Type I the saving would be somewhat less; with $d = 31$ in. and 30·5 in. respectively (the top widths at sides and depth of foundation being 6 in. instead of 4 in.) the saving in area would be 18 sq. in. in 918 sq. in. or 2 per cent.

Small drains with d under 9 in. should, as a rule, be semicircular ones, as they are easier to construct and keep clean than small drains on Type I.

APPENDIX F

The squares and square roots of numbers can, with a little trouble, be ascertained from Cols. I. and II. of Table IV. in the following manner very approximately :—

Required the square of 44·72.

The square of 45 is	2025
" " " 44 is	1936
A difference of	89

and as $89 \times 0{\cdot}72 = 64$, the square of 44·72 is $1936 + 64 = 2000$.

Per contra, the square root of 2000 is $44 + \frac{64}{89} = 44 + 0{\cdot}72 = 44{\cdot}72$.

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