has passed away. Section 3 of the main channel from junction X to Y is, therefore, in its entire course subject to a maximum discharge of 101.83 cubic feet per second.

Branch Channel 4.

This branch channel has a length of 2,310 lineal feet, and the rainwater entering at the head thereof at point F will have covered, on completion of the period of rainfall, with a velocity of 4 lineal feet per second, a distance of $720 \times 4 = 2{,}880$ lineal feet. As this length exceeds the total length of branch channel 4, the profile of same above point Y must be capable of discharging the whole of the run-off of sub-area S.A. 4 = 56.72 cubic feet per second.

Main Channel, Section 5.

Point Y being the junction of Section 3 of the main channel, the Branch Channel 4 and the former being governed by the waters above the points C, and D reaching there simultaneously, we must first determine the time elapsing in their doing so in order to find what quantity of run-off from sub-area 4 will have to be taken into consideration in Section 5 of the main channel.

The length of Section 3 of the main channel is 2,310 lineal feet, and the time elapsing for the water to pass through this length at a velocity of 4 feet per second would be $\frac{2310}{4} = 577$ seconds which, with the period of time elapsing for the waters to reach point X from C and D, viz., 204 seconds, makes a total of 781 seconds for the waters to reach point Y from points C and D. During this interval Branch Channel 4 will have discharged all the run-off of sub-area 4, as, with a velocity of 4 lineal feet per second, the last water reaching the head of the former at F will have traversed a distance of $781 \times 4 = 3{,}124$ lineal feet, and, therefore, passed through junction Y before the waters from C and D reach there. Since no other waters enter Section 5 of the main channel between points Y and Z, the entire length is governed by the maximum quantity of water delivered into same by Section 3, viz., 101.83 cubic feet per second.

Branch Channel 6.

The length of Branch Channel 6 is 5,610 feet and the velocity attained therein is 3 lineal feet per second. During the period of rainfall, equalling 720 seconds, Branch Channel 6 will have discharged the waters for a length of $720 \times 3 = 2.160$ lineal feet up to point G, leaving a balance of $47.5 - \frac{2830 \times 330 + 660 \times 495}{43560} = 26.14$ acres of sub-drainage, area 6 still contributing with a run-off of $26.14 \times 3.052 = 79.07$ cubic feet per second, which forms the maximum passing through Branch Channel 6 below point G.

Main Channel, Section 7.

Point Z being the junction of Section 5 of the main channel and Branch Channel 6, and the former being governed by the water above points C and D reaching there simultaneously, we must first determine the time thus occupied in order to find what quantity of run-off from sub-area 6 will have to be taken into consideration in Section 7 of the main channel. We know that it takes 781 seconds for the waters to reach point Y from points C and D, and, as the distance between Y and Z with a velocity of 4 lineal feet per second would take $\frac{660}{4} = 165$ seconds, the total time occupied for these waters to reach point Z would be $781 + 165 = 946$ seconds. During this interval Branch Channel 6, with a velocity of 3 feet per second, will have discharged the waters for a length of $946 \times 3 = 2{,}838$ lineal feet up to point H.

The duration of the rain being 720 seconds, the rain will have ceased for a period of $946 - 720 = 226$ seconds, during which time Branch Channel 6 will have emptied itself below its head to point J, a distance of $226 \times 3 = 678$ lineal feet. The balance of sub-drainage area S.A. 6, still contributing to Section 7 of the main channel will, therefore, be $47.5 - \frac{660 \times 495 + 330 \times 2508 + 330 \times 678}{43560} = 15.87$ acres, giving a maximum run-off of $15.87 \times 3.025 = 48.00$ cubic feet per second. The total quantity of water reaching junction Z simultaneously by Section 5 of the main channel and Branch Channel 6 is, therefore, $101.83 + 48.00 = 149.82$ cubic feet per second.

Examination Under Condition 2.

Let the drainage area be subject to a rainfall at the rate of 2 inches per hour extending over 2 hours equal to 2.016 cubic feet per second per acre.

As this duration of rainfall is longer than that of the passage of the run-off through the conduit for any portion of the drainage scheme, the whole of the run-off must be taken into consideration throughout.

The run-off of the sub-areas for a rainfall of 2 inches per hour for a period of 2 hours amounts to—

S.A. 1 = 15	× 2·016	× 0·8 =	24·2	cubic feet per second.		
S.A. 2 = 18·75	× 2·016	× 0·8 =	30·24	,,	,,	,, ,,
S.A. 3 = 16·25	× 2·016	× 0·8 =	26·21	,,	,,	,, ,,
S.A. 4 = 18·75	× 2·016	× 0·6 =	22·68	,,	,,	,, ,,
S.A. 6 = 47·5	× 2·016	× 0·6 =	57·46	,,	,,	,, ,,
			160·79	,,	,,	,, ,,

Having thus the discharge requirements for two widely differing intensities and durations to which this drainage system may be subject at our disposal, it behoves us to make a comparison of the respective values in order to determine the critical one for which each portion must be designed.

Main Channel	Branch Channel	Discharge Capacity in cubic feet per second required for a rainfall of: 5 ins. per hour for a duration of 12 minutes	2 ins. per hour for a duration of 2 hours	Discharge Capacity for which Conduit must be designed.
		cubic feet per second	cubic feet per second	cubic feet per second.
1		60·5	24·2	60·5
	2	60·05	30·24	60·05
3		101·83	80·65	101·83
	4	56·72	22·68	56·72
5		101·83	103·33	103·33
	6	79·07	57·46	79·07
7		149·82	160·79	160·79

For the greater portion of the drainage scheme, the 5-inch rainfall for a duration of 12 minutes proves to be the critical factor for the discharge requirements, with the exception of Sections 5 and 7 of the main channel, where the 2-inch rainfall for a duration of 2 hours becomes such.

The heavy rainfall with short duration is, therefore, the critical one for the branch channels of the system, while the lesser rainfall with a long duration governs the main channel in its lower stretches.

From the above it becomes apparent of what vital consequence it is to determine correctly the critical rainfall and consequent maximum discharge requirements for each portion of the drainage scheme, and for that purpose it is necessary to calculate and tabulate the discharge requirements for a large number of rainfalls of varying intensities and durations.

Having this information to hand, the calculation of the size of the channels may be undertaken with due regard that profile and grade shall produce the assumed velocities for the various channels and sections thereof, and, in case of the calculated velocity deviating from that assumed, the calculation for those portions of the system further down stream must be so adjusted as to suit the altered conditions.

Grade of Pipe Conduits.

The grade of pipe conduits is limited on one hand by the necessity of providing a velocity sufficient to carry off any matter getting into same, and on the other hand of restricting the velocity to prevent attrition of the inner surface.

Pipe conduits conveying sewage are less liable to attrition than those conveying stormwater, as the latter carry a considerable amount of grit washed into them from the streets.

It is impossible to lay down a hard and fast limit regarding the extremes of the grade, as the velocity is also affected by the profile of the pipe conduit; but, as a guidance, we advise for stormwater channels no flatter grade than will produce a minimum velocity of 2.5 feet per second, nor a steeper one than will produce a maximum velocity of 15 feet per second; and for sewers, we advise no flatter grade than will produce a minimum velocity of 3 feet per second for reticulation and 2 feet per second for main channels.

Calculation of Cross Section of Conduit.

Of the many formulae in use for the calculation of the cross section of a pipe conduit, that of Kutter is the most frequently applied.

$$1. \quad V = C \sqrt{r.s}$$

$$2. \quad Q = V \times A$$

where V = velocity of flow in feet per second.
C = a variable co-efficient.
r = hydraulic mean depth or radius in feet = area ÷ wetted perimeter.

$$s = \text{sine of slope} = \frac{\text{difference of levels between 2 points}}{\text{distance between same}}$$

A = discharge area of pipe.
Q = quantity of discharge in cubic feet per second.

The process of calculation is as follows:—

After having settled on the quantity of water Q in cubic feet per second to be carried off at a given point and the fall of the Pipe Conduit below this point—

1. Choose a profile.
2. Ascertain for this profile, or portion of same, the discharge area A in square feet and the wetted perimeter P in feet.
3. Out of A and P resolve $r = \frac{A}{P}$,
4. Determine the co-efficient C by means of the formula:—

$$C = \frac{41{\cdot}6 + \frac{0{\cdot}00281}{S} + \frac{1{\cdot}811}{n}}{1 + \frac{\left(41{\cdot}6 + \frac{0{\cdot}00281}{S}\right) n}{\sqrt{r}}}$$

wherein n represents the co-efficient of roughness of the wetted perimeter of the conduit which, in case of Monier pipes, amounts to 0.01.

5. Determine the velocity V in feet per second by the formula $V = C\sqrt{r.s}$
6. Determine finally the quantity of discharge in cubic feet per second by the formula $Q = V \times A$.

EXAMPLE.

Design Section 7 of the main channel, shown in Fig. 7, for the computed discharge of 160.8 cubic feet per second on a grade of 1 in 1,200.

$$S = \frac{1}{1{,}200} = \cdot 0008\dot{3}$$

Assume the profile as a 6ft. 0in. diameter circular Monier pipe. The co-efficient of roughness n = 0.01.

Area $= 6 \times 6 \times .7854 = 28.27$ sq. feet.
Wetted Perimeter $= 6 \times 3.1416 = 18.85$ lin. feet.
Hydraulic Radius $r = 28.27 \div 18.85 = 1.5$.

$$\text{Co-efficient } C = \frac{41{\cdot}66 + \dfrac{0{\cdot}00281}{0{\cdot}00083} + \dfrac{1{\cdot}811}{0{\cdot}01}}{1 + \dfrac{41{\cdot}66 + \dfrac{0{\cdot}00281}{0{\cdot}00083}}{\sqrt{1{\cdot}5}} \times 0{\cdot}01} = 165{\cdot}3$$

Velocity $V = C\sqrt{r.s.} = 165.3\sqrt{1.5 \times .00083} = 5.84$ lin. feet per second.

Discharge $Q = V \times A = 5.84 \times 28.27 = 165.09$ cubic feet per second.

Hence a 6ft. 0in. diameter Monier pipe line on a grade of 1 in 1,200 will provide the necessary discharge capacity.

It will invariably be found that a number of trial calculations are necessary to arrive at the desired result, and, as these calculations are tedious and take up a considerable time, Gummow, Forrest's tables of the velocities and discharges of circular and oviform Monier pipes are appended, which greatly facilitate operations.

GRAPHICAL METHOD.

The following diagrams illustrate a method used by me, and have been prepared to compare with other methods of calculation for catchments as set out in Gummow, Forrest's catalogue:—

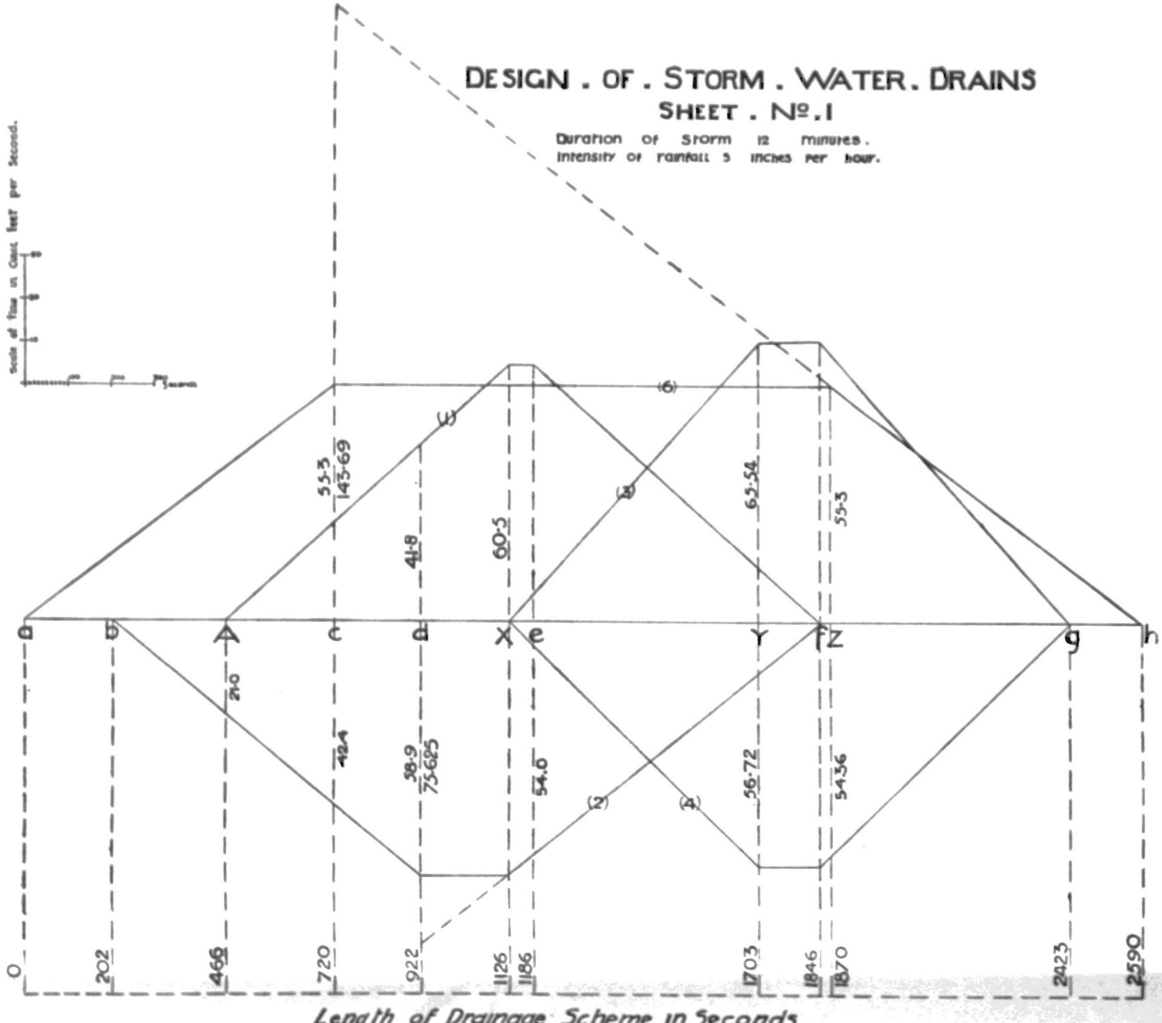
The method of constructing these diagrams is as follows:—

(1) Referring to Sheet No. 2.—Lay off horizontally, to any scale, lengths a b, b c, etc., of main and branch drains in a straight line to represent the times in seconds required for water to flow through each section. In

DESIGN . OF . STORM . WATER . DRAINS

SHEET . No. 1

Duration of Storm 12 minutes.
Intensity of rainfall 5 inches per hour.



Design . of . Storm . Water . Drains
Sheet . No 2

Duration of Storm 12 minutes.
Intensity of rainfall 5 inches per hour.

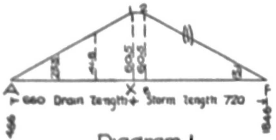


Diagram 1
Main Drain A X (1)

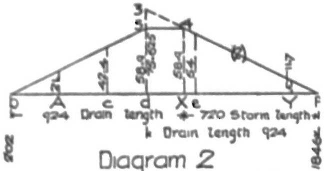


Diagram 2
Branch Drain b X (2)

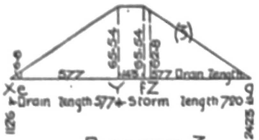


Diagram 3
Main Drain XY (3)

Diagram 3A
Combining (1) (2) & (3)
for flow in Main Drain XY (3)

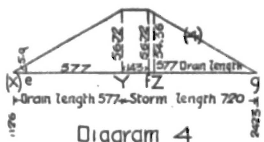


Diagram 4
Branch Drain (X) Y (4)

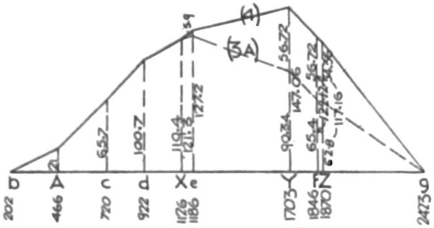


Diagram 5
Combining (3A) & (4)
for flow in Main Drain YZ (5)

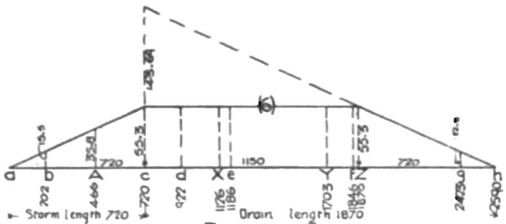


Diagram 6
Branch Drain aZ (6)

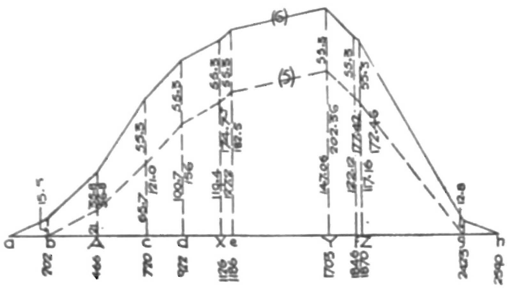


Diagram 7

Main Drain Zh.

these diagrams the line is a h, in which A X = time length of main drain (1); b X = ditto of branch drain (2); X Y = ditto of main drain (3), (X) Y = ditto of branch drain (4); main drain Y Z has no catchment and no individual diagram; a Z = ditto of branch drain (6).

(2) Determine areas of catchments contributing to each section of main drain.

(3) Then, having fixed on a maximum rate of rainfall and co-efficient of run-off, determine the volume of run-off in cubic feet per second.

(4) Determine the time length of storm; in this case 12 minutes or 720 seconds.

(5) If the main drains have catchments contributing directly throughout their length, they will have their independent diagrams as well as the branch drains, but not otherwise.

(6) Let main drain A X have its own catchment, as well as one branch drain junctioning at X.

(7) The capacity of main drain A X will be governed solely by the run-off from its own catchment. This run-off will be 60.5 cubic feet per second, as previously determined.

(8) From X lay off to the right, i.e., in the direction of flow, a length X f = duration of storm in seconds, and A X to left representing time length of first section of main drain = e f. In this case the time length of section e f is less than that of storm, hence e lies to the right of X.

(9) At e, erect to any scale ordinate e 2 = 60.5 cubic feet per second; also an equal ordinate X1, at X; join A, 1, 2, f — then the ordinates between f and A represent the flow in A X from start to finish of storm, and the maximum ordinate lies clearly between X and e, and = 60.5 cubic feet per second, for which the drain must be designed.

(10) Now a branch drain junctions at X, and the maximum run-off from its catchment = 75.625 cubic feet per second. At the start of the storm the water collected in branch near X will flow into main drain at junction at same time as lowest water from A X, and they will travel together till they arrive at f, when the storm ceases. But the branch drain has a time length d f, which is greater than the duration of the storm. At d, erect ordinate d3 = 75.625, and join 3 f; erect ordinate at X, cutting 3 f in 4. Ordinate X 4 will represent volume of flow in main X Y, due to branch at X when storm ceases. Now, as no further rain falls, the last or furthest water contained in branch drain will take a time d f to reach X. Lay off b X to left of junction, equal to time length df of branch drain. Now it is clear that if during storm of time length X f the maximum rate of discharge = 75.625, and as b d = X f, the

ordinate d5 must equal X4. Join up b 5 4 f, the ordinates to which represent the volume of flow in main drain X Y, due to water from branch drain.

Suppose main drain X Y have a catchment. Lay-off time length of storm Y g and time length of main drain f g = X Y. As in case of section A X. the time length of main drain is less than that of storm. Therefore, erect at Y and f ordinates = 65.54 maximum volume of discharge from its own catchment and complete diagram as before. When storm ceases the first water from main drain A X, and branch at X, will have arrived at f, and that from main drain X Y will have arrived at g.

Therefore, if all diagrams for drains above Y be combined, they will represent the flow in X Y from start of storm to discharge of last water, as shown.

As illustrated, the capacity must equal that of combined ordinate at c, which is seen to be the greatest = 121.3 cubic feet per second.

The other diagrams are similarly constructed.

In any system of drainage there must be the main drain, with branch drains and laterals feeding into the branch drains. For the purpose of calculation, the laterals are designed for maximum run-off from their catchments. The main and branch drains may be considered to be open channels, with storm water from their respective catchments feeding in from the surface for full length, according to which assumption the diagrams represent the volumes of flow at any point in the system, and enable the drains to be designed correctly.

It should be noticed that the assumption made postulates that the flow from the individual catchments arrives at the drain at a maximum and uniform rate during period of rainfall, i.e., does not take into consideration the gradual increase to maximum, and then the gradual reduction, as rain ceases to fall. The design will therefore be on the safe side. No matter how designed, the capacities of drains should be checked by a method frequently adopted in the law courts. That is, the system should be capable of discharging the run-off from the catchment, as calculated by one or other of the formulae previously mentioned.

But the beauty (if I may say so) of the above detail method lies in the fact that it may be applied to a storm of varying intensity. For, suppose the system to be so extensive that it is desired to see what is the effect of a storm of longer duration than 12 minutes, and of less intensity. This can be readily done. Suppose, for instance, that it is desired to ascertain the effect of a storm of one hour's duration, and of an average intensity of 2 inches per hour, during 12 minutes, of which the average intensity is 5 inches per hour. All it is necessary to do is to construct two sets of diagrams, one for the rate of 2 inches per hour and for 60 minutes' duration, and another for 5 inches — 2 inches = 3 inches per hour for 12 minutes' duration. If the latter set be traced, and the tracing inverted over the diagram for 2-inch rate, by moving the inverted tracing of any combined diagram for 3-inch rate along the base line of the corresponding diagram for 2-inch rate, the increment of volume can be ascertained, and it can be seen at what point in the long storm the greater but shorter storm produces the greatest effect on any part of system.

The time lengths of main and branch drains, and the diagrams representing their respective discharges are:—

Main Drains.	Diagrams.	Branch Drains.	Diagrams.
$AX = \frac{1980}{3} = 660$ secs.	(1)	$bX = \frac{2310}{2.5} = 924$ secs.	(2)
$XY = \frac{2310}{4} = 577$ secs.	(3)	$(X)Y = \frac{2310}{4} = 577$ secs.	(4)
$YZ = \frac{660}{4} = 167$ secs.	(5)	$aZ = \frac{5610}{3} = 1870$ secs.	(6)
$Zh =$	(7)		

The duration of storm is 12 minutes = 720 seconds. When the time length of storm exceeds the time length of drain, the latter will govern the maximum discharge; and when the time length of the storm is less than the storm length of drain, the former will govern the maximum discharge.

The individual diagrams represent the variations of flow in the several drains, commencing at zero, increasing to a maximum; and then, when rain ceases, gradually diminishing to zero again. The combined diagrams indicate the variations of flow in main drains, due to combined flows from branches and main drains contributing above point of junction.

Sheet No. 1 shows diagrams for a rainfall at the rate of 5 inches per hour continuing for 12 minutes. The storm is supposed to commence, continue and finish at this rate. The diagrams for each main drain which has a catchment of its own directly contributing, and each branch drain, are grouped, but not superimposed, along one time—length base line, so that by referring to the block-plan of drainage system it can be seen what ordinates should be combined for maximum capacity of any main drain.

Sheet No. 2 has been prepared to show a separate diagram for each main and branch drain of sheet No. 1, on its own time —length base line. The position and letter on this base line, where flow has reached when storm ceases (being the right-hand end of diagram, in direction of flow), enables one to trace the gradual increase of volume from commencement of storm.

In fact, the diagrams show accurately the fluctuation of flow from start to finish of storm in any branch or main drain. Further, by laying off the time-lengths of all main drains, and plotting on the relative time-lengths of branch drains, much difficulty and confusion is obviated in constructing diagrams and superimposing them. The combined diagrams show the relative amount contributed by main and branch drains to volume of flow in any main drain below point of junction of branch drain at any instant. If the maximum ordinates from diagrams be tabulated they can be compared with the results given in Gummow, Forrest's catalogue. From the table appended it will be seen that the method adopted in the trade publication is not accurate; but it has been given in full for comparison, because it is a method which has been widely adopted.

Sheet No. 3 gives diagrams for a rainfall at the rate of 2 inches per hour lasting for one hour. In Gummow, Forrest's catalogue it is taken as lasting for 2 hours; but, as a duration of one hour is more than sufficient to give maximum results, I have adopted the latter, so as to obtain larger diagrams. It will be seen that the results by diagrams agree with those from catalogue throughout, the reason being that the storm length exceeds the drain length in every case. The method adopted in the catalogue ignores the effect of the increasing volume at start of storm, and of decreasing volume towards end of storm, the ordinates from which, for some diagrams, overlap the maximum ordinates for others. This is the case where the storm length is less than the time-length of drain.

Sheet No. 4 gives diagrams for a rainfall at the rate of 3 inches per hour for 12 minutes. By properly combining the diagrams of sheets Nos. 3 and 4 the maximum volume of discharge for main and branch drains may be accurately determined, as for a rainfall, at the rate of 2 inches per hour, lasting for one hour, during a period of which the intensity is 5 inches per hour, lasting for 12 minutes.

The results taken from the diagrams are given in the table, and it is instructive to compare them with those obtained by other methods.

CHECK BY DIRECT FORMULA.

Frequently the maximum capacity of drains is determined by calculating the run-off by some formula, as—

$c\ r\ A^{\frac{3}{4}}\ S^{\frac{1}{4}}$ Burkli Ziegler.

$c\ r\ A^{\frac{4}{5}}\ S^{\frac{1}{5}}$ McMath.

$1{\cdot}571\ r\ A^{\frac{2}{3}}$ Vicars.

Adopting the latter for sake of example.

The maximum run-off will be that for a storm not of 12 minutes' duration, but indefinitely long, only with a period of

Design. of. Storm. Water. Drains
Sheet. No 3

Duration of Storm 1 hour.
Intensity of rainfall 2 inches per hour.

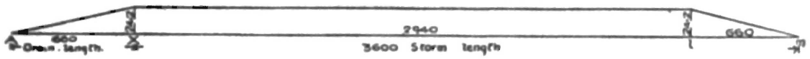


Diagram 1
Main Drain AX (1)

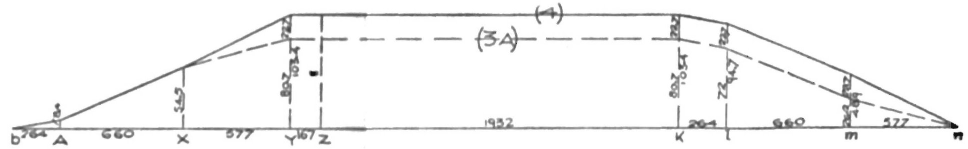


Diagram 5
Combining (3A) & (4)

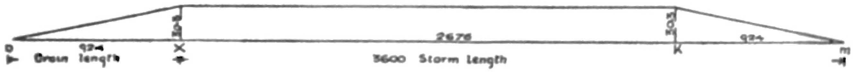


Diagram 2
Branch Drain bX (2)

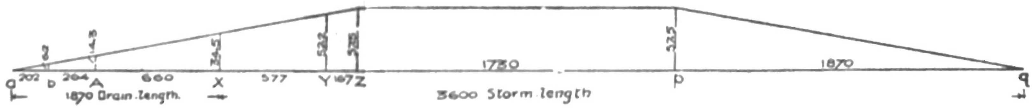


Diagram 6
Branch Drain aZ (6)

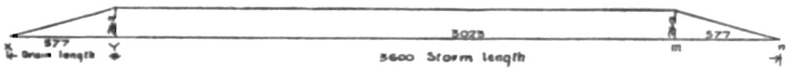


Diagram 3
Main Drain XY (3)

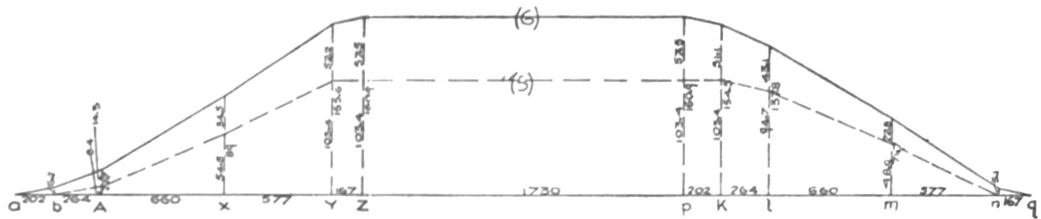


Diagram 7
Combining (5) & (6)

Main Drain Zh.

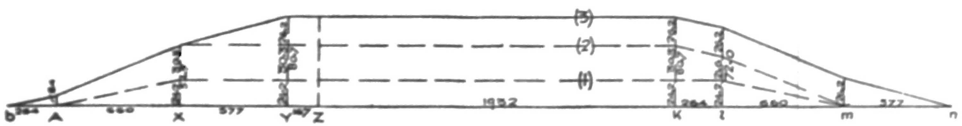
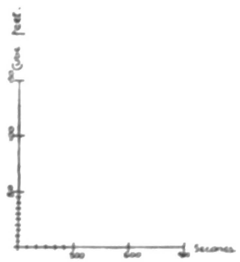


Diagram 3A
Combining (1) (2) & (3)



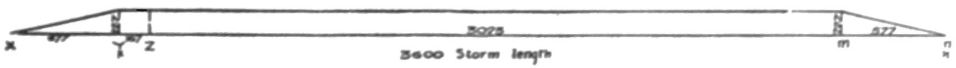


Diagram 4
Branch Drain (X)Y (4)

DESIGN . OF . STORM . WATER . DRAINS
SHEET . № 4

Duration of Storm 12 minutes.
Intensity of rainfall 3 Inches per hour.

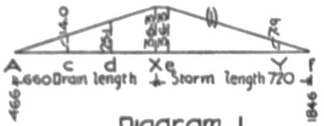


Diagram 1
Main Drain AX (1)

Diagram 4
Branch Drain (X) Y

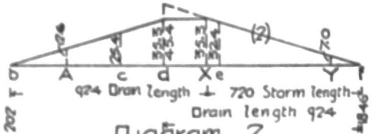


Diagram 2
Branch Drain bX (2)

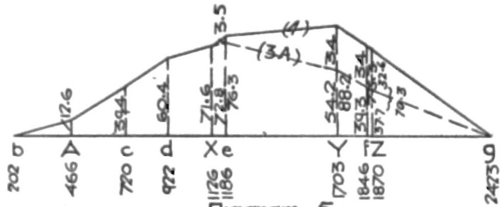


Diagram 5

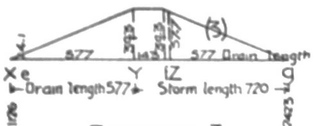


Diagram 3
Main Drain X Y (3)

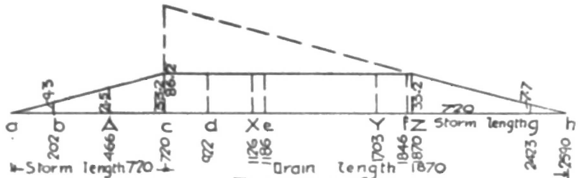
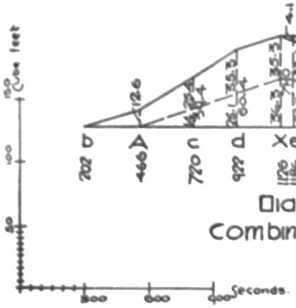


Diagram 6
Branch Drain a Z



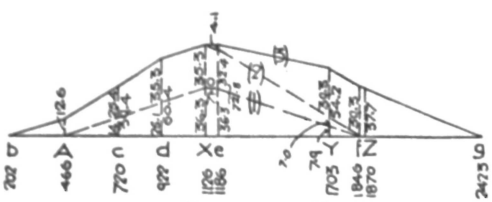


Diagram 3A
Combining (1)(2) & (3)

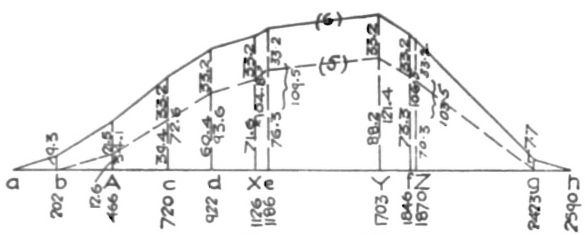


Diagram 7

maximum rate of 5 inches per hour for 12 minutes; but as the above formula is based on 10 minutes' period, it will equal $5\text{in.} \times \frac{12}{10} = 6\text{in.}$ rate for 10 minutes.

Then capacity of

(1)= $1{\cdot}571 \times 6 \times 15^{\frac{2}{3}} = 54{\cdot}72$ cubic ft. per sec.

(2)= $1{\cdot}571 \times 6 \times 18{\cdot}75^{\frac{2}{3}} = 66{\cdot}36$ " "

(3)=(1)+(2)+(3) $1{\cdot}571 \times 6 \times 50^{\frac{2}{3}} = 128{\cdot}4$ " "

(4)= $1{\cdot}571 \times 6 \times 18{\cdot}75^{\frac{2}{3}} = 66{\cdot}36$ " "

(5)=(1)+(2)+(3)+(4) $1{\cdot}571 \times 6 \times 68{\cdot}75^{\frac{2}{3}} = 158{\cdot}4$ " "

(6)= $1{\cdot}571 \times 6 \times 47{\cdot}5^{\frac{2}{3}} = 123{\cdot}6$ " "

(7)=(1)+(2)+(3)+(4)+(6) $1{\cdot}571 \times 6 \times 116{\cdot}25^{\frac{2}{3}} = 224{\cdot}4$ " "

Where the percentage of run-off is arbitrarily fixed irrespective of area, as has been done in the trade catalogue referred to, no formula can be expected to apply. It is, however, of interest to compare the results obtained by formula. The above results are given in table.

The following table sets out the capacities of the several main and branch drains, as determined by Gummow, Forrest's catalogue, by the graphic method, and by direct formula:—

Drain.	Gummow, Forrest for rainfall of			Diagram for rainfall of				Formula
	5in. per hour for 12 mins.	2in. per hour for 2 hrs.	Max.	3in. per hour for 12 mins.	2in. per hour for 2 hours*	Total.	5in. per hour for 12 mins.	$Q=1{\cdot}571rA^{\frac{2}{3}}$
(1) Main AX ...	60·5	24·2	60·5	36·3	24·2	60·5	60·5	54·72
(2) Branch bX ...	60·05	30·24	60·05	35·3	30·3	65·6	58·9	66·36
(3) Main XY ... (1)+(2)+(3) ...	...	..	...	(39·3)	(26·2)	(0)	(65·54)	...
	101·83	80·65	101·83	72·8	80·7	153·5	121·3	128·4
(4) Branch (X) Y	56·72	22·68	56·72	34·0	22·7	56·7	56·72	66·36
(5) Main YZ ... (1) + (2) + (3) + (4) + (5) ...	...	...	...	(0)	(0)	...	(0)	(0)
	101·83	103 33	103·33	88·2	103·4	191·6	147·06	158·4
(6) Branch aZ ...	79·07	57·46	79·07	33·2	57·5	90·7	55·3	123·6
(7) Main Zh ... (1)+(2)+(3)+(4) +(5)+(6) ...	...	...	...	(0)	(0)	...	(0)	(0)
	149·82	160·79	160·79	121·4	160·9	282·3	202·3	224·4

* Two hours retained because it makes no difference in discharges and avoids confusion in comparing results.

Comparison of Graphic and Analytical Methods.

In the case of rainfall for short duration it should be noted, in case of Branch Drain (6), if velocity had been taken at 6 feet per second the rate of discharge would have been doubled, according to Gummow, Forrest's method, and my diagrams, and correctly so, which indicates a preference for method of formula for design for small systems, as it gives maximum values.

The most prominent difference between the results by my diagrams and those of Gummow, Forrest is in the case of branch drain (6), which is 5,610ft. long, the velocity of flow 3ft. per second, and the time-length 1,870 seconds. According to the diagram, for a rainfall of 5 inches per hour for 12 minutes the maximum discharge is 55.3 cubic feet per second, which Gummow, Forrest gives at 79.07 cubic feet per second. The correctness of my result can be proved as follows:—When storm ceases, the volume at head of drain will be zero, increasing to a maximum in storm length = 720 seconds. Therefore, the balance of drain = 1,870 — 720 = 1,150 seconds length, will be subject to accumulative effect of full duration of rainfall. The total volume of run-off for 720 seconds = 47.5 × 3.025 × 720 = 103,464 cubic feet, and for 1,150 seconds length, the volume in drain = $\frac{1,150}{1,870} \times$ 103,464 cubic feet; and this takes 1,150 seconds to flow through, which gives $\frac{1,150 \times 103,464}{1,870 \times 1,150}$ = 55.3 cubic feet per second. Or, look at it another way: Suppose the drain were blocked at the outlet, and for full duration of storm = 720 seconds, the whole of the storm-waters would be accumulated in the drain, which is the worst possible condition, and gives a maximum result under any conditions. Then 103,464 cubic feet of water are contained in a drain 5,610 feet long. The cross-sectional area would be $\frac{103,464}{5,610}$ square feet; and as capacity is square feet × velocity, the maximum discharge to be provided for is $\frac{103,464}{5,610 \times 3}$ = 55.3 cubic feet per second, as on diagram, not 79.07, as given by Gummow, Forrest. In the case of a drain whose time-length is less than storm-length, the result, as obtained above, must be reduced in the ratio of $\frac{\text{time-length of drain.}}{\text{storm-length.}}$ For example, in branch drain (4) the time-length is 577 seconds, i.e., it would empty itself in 577 seconds. Clearly, therefore, it can only hold the run-off for that period, and this equals 18.75 acres × 3.025 cubic feet per second × 577 seconds = contents in cubic feet; and to have this capacity, if we divide by the length in feet we obtain the sectional area = $\frac{8{\cdot}75 \times 3{\cdot}025 \times 577}{2310}$ square feet, and this multiplied by the velocity gives the maximum possible discharge = $\frac{18{\cdot}75 \times 3{\cdot}025 \times 577 \times 4}{2310}$ = 18.75 × 3.025 = 56.72 cubic feet per second, as given on diagram (4), sheet No. 1, and by Gummow, Forrest.

A table of heavy rainfalls experienced in Sydney, obtained from the Commonwealth Meteorological Bureau, is appended, and it is thought will be of value to engineers.

Note.—ERRATA IN PRELIMINARY PAPER ON DESIGN OF STORM-WATER DRAINS.

(Proc. S.U. Eng. Socty. Vol. XVI., 1911).

p. 66 line 5, for "$C = 1{\cdot}0, 0{\cdot}8$, etc., read "$c = 1{\cdot}0, 0{\cdot}8$, etc."

p. 66 line 22, for "$Q = 1{\cdot}11\ C.R^{\frac{1}{2}}A^{\frac{3}{4}} = 1{\cdot}57\ crA^{\frac{2}{3}}$"

read $Q = 1{\cdot}11\ C.R^{\frac{1}{2}}A^{\frac{2}{3}} = 1{\cdot}57\ CrA^{\frac{2}{3}}$"

RECORDS OF HEAVY RAINS, SYDNEY WEATHER BUREAU.

Date.	Total Fall and Duration of Storm			Heaviest Part of Storm.			Heaviest Shower.		
	Total Fall.	Dur-ation.	Rate per Hour.	Total Fall.	Dur-ation.	Rate per Hour.	Total Fall.	Dur-ation.	Rate per Hour.
	Inches	hs. ms.	Inches.	Inches.	hs. ms.	Inches.	Inches.	m. s.	Inches
1844.									
Oct. 15 ..	20.41	..	..	20.30	20.30	0.99	5.40	120.0	2.70
1869.									
Feb. 11 ..	2.00	4.0	0.50	2.00	4.0	0.50	0.70	8.0	5.23
1873.									
Feb. 25 ..	8.90	11.30	1.03	2.20	1.0	2.20	1.50	20.0	4.50
1878.									
Feb. 5 ..	7.53	23.0	0.33	5.54	8.30	0.65	1.50	17.0	5.30
„ 6 ..	3.35	24.0	0.14	1.20	1.20	0.90	0.23	3.0	4.60
„ 8 ..	1.58	24.0	1.07	1.02	2.0	0.51	0.25	3.0	5.00
Aug. 1 ..	2.28	24.0	0.10	1.00	0.6	10.00	1.00	6.0	10.00
1882.									
April 5 ..	6.48	4.30	1.44	3.00	1.30	2.00	0.40	4.0	6.00
1884.									
April 6 ..	6.45	20.0	0.32	5.63	12.0	0.47	1.00	10.0	6.00
1887.									
April	1.76	3.30	0.50	1.76	3.30	0.50	0.15	3.30	2.46
1888.									
Dec. 16 ..	1.60	0.55	1.74	1.60	0.55	1.74	0.16	1.48	5.33
1889.									
May 25 ..	2.23	19.0	0.12	..	..	..	..	..	..
„ 26 ..	4.05	24.0	0.17	0.66	2.10	0.31	0.22	2.12	6.00
„ 27 ..	4.69	24.0	0.19	1.10	1.0	1.10	..	..	..
„ 28 ..	8.36	24.00	0.35	0.70	1.0	0.70	0.22	8.0	1.65
1890.									
Feb. 17 ..	1.51	18.30	0.08	..	..	..	..	..	..
„ 18 ..	3.21	24.0	0.13	2.53	3.0	0.84	0.22	3.45	3.52
„ 19 ..	0.36	..	..	..	..	..	..	..	..
„ 20 ..	3.25	24.0	0.14	2.60	7.15	0.36	0.275	3.0	5.50
„ 21 ..	0.85	24.0	0.04	0.44	0.30	0.88	0.20	8.0	1.50
March 4 ..	2.13	8.0	0.27	0.80	0.40	1.20	0.20	6.0	2.00
„ 25 ..	5.66	24.0	0.24	0.66	1.45	0.38	0.22	12.0	1.10
May 2 ..	2.35	9.30	0.25	1.32	3.30	0.38	0.22	5.30	2.40
June 27 ..	2.57	13.0	0.20	1.76	3.15	0.54	0.35	10.0	2.10
„ 28 ..	4.48	9.30	0.47	3.50	5.0	0.70	0.21	3.9	4.00
1891.									
Feb. 24 ..	1.50	12.0	0.13	0.40	0.9	2.67	0.14	2.24	3.50
1892.									
Jan. 11 ..	1.81	3.0	0.60	1.54	0.23	4.02	0.88	10.0	5.28
March 20 ..	4.22	15.45	0.27	0.29	0.10	1.74	0.21	5.0	2.52
Dec. 18 ..	2.13	17.0	0.13	0.80	0.22	2.18	0.20	3.0	4.00
1893.									
March 8 ..	3.30	12.0	0.27	0.80	0.43	1.12	0.17	4.0	2.55
„ 23 ..	2.17	17.0	0.13	1.00	1.0	1.00	0.22	3.0	4.40
1896.									
June 4 ..	4.88	24.0	0.20	2.35	1.50	1.28	0.18	1.51	5.84
1897.									
June 1 ..	2.66	8.15	0.32	2.00	4.45	0.42	0.20	4.22	2.75
1900.									
May 23 ..	2.91	11.50	0.24	2.00	4.46	0.42	0.20	3.20	3.60

For records for Melbourne, Brisbane, Perth and Hobart, see pp. 109, 110, 111.