HIGHWAY SURFACE DRAINAGE

DESIGN GUIDE FOR HIGHWAYS WITH A POSITIVE COLLECTION SYSTEM

Reprint Only

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DESIGN GUIDE FOR HIGHWAYS WITH

A POSITIVE COLLECTION SYSTEM

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INTRODUCTION

This design guide develops a relatively simple design method applicable to a wide range of highway drainage systems.

It would be possible to get involved in complex calculations for many facets of this problem but the overall expected and required accuracy of the solution must be kept in mind. Due to inherent uncertainty in rainfall estimation all that can be expected is a system which will handle most rainfalls reasonably efficiently while preventing danger and inconvenience to the road user.

A number of simplifying assumptions are made in this guide but comparisons with more sophisticated calculations, experimental data, and semi-empirical design methods for specific systems show the method to be well within necessary limits of accuracy.

Design principles are first set out in the main text which is followed by a summarised design procedure. Charts used in the design process are then grouped in a separate section for quick reference. A number of examples illustrating various design aspects are followed by a reference list.

The suggested use of the guide is:

a Work through the Design Procedure step by step.

b Refer back to the main text on Principles and Details for amplification where necessary.

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DESIGN PRINCIPLES AND DETAILS

1.1 RAINFALL CALCULATION

A method for estimating precipitation for a given duration and return period for any location in the country is set out in *"The Frequency of High Intensity Rainfalls in New Zealand"*, Robertson (1). Since this was not available in metric edition when this guide was being written, the relevant metric figures and tables along with the calculation method have been included in this document. In the imperial units' edition a lower limit for rainfall duration calculation of ten minutes was used. Surface drainage applications require a calculation down to five minutes' duration so the appropriate graph (chart 6) has been extended to this lower limit. It was calculated that the scale should be extended such that the difference between the 5 and 10 minute co-ordinates be the same as between the 20 and 30 minute co-ordinates.

If an updated metric edition of the publication becomes available, the graphs and figures should be used, incorporating this adjustment.

The steps involved in the rainfall calculation are as follows:

a Determine 2 year 2 hour rainfall (chart 1);

b determine 20 year 2 hour rainfall (chart 2);

c determine 2 year ½ hour rainfall (chart 3);

d determine 20 year ½ hour rainfall (chart 4);

e on a return period plotting sheet (chart 5) join the co-ordinates from (a) and (b) and read the precipitation for the specified return period;

f similarly, on the same sheet join the co-ordinates from (c) and (d) and read the preciptation for the specified return period;

g on a depth duration plotting sheet (chart 6), plot the two co-ordinates of precipitation for the specified return period (e) and (f) against duration. Join the points and read off rainfall for the required duration.

In many cases the availability of local data may be considered to provide more accurate rainfall figures than the general method. In these cases this data should certainly be used as the procedure outlined embraces the whole country and tends to produce rather coarse results.

1.2 DESIGN DISCHARGE AND TIME OF CONCENTRATION

The 'rational' method is the most widely used and probably the best method for calculating surface water discharge in the highway drainage situation. The method is based on the formula:

$$Q = \frac{C\ I\ A}{3.6 \times 10^6}$$

in which Q = runoff rate

C = co-efficient of impermeability

I = rainfall intensity (mm/hr)

A = area contributing

The *co-efficient of impermeability* may be defined as the proportion of the entire rainfall that reaches the sewer. C varies for different surfaces, and where there are a combination of different surfaces in a catchment, a weighted average (C_W) may be used.

The generally accepted values for C are as below with steeper areas tending to have higher co-efficients:

Type of Surface	Co-efficient of Impermeability
Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadways or shoulders	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed areas	0.5 - 0.7

The value of I depends on the time of concentration for the part of the system being designed. *Time of Concentration* (T) is the time for the runoff from the farthest point of the catchment to reach the point under consideration. The rainfall intensity corresponding to a duration of T generally gives the greatest discharge from a catchment. T is found by summing values of flow time over the various parts of the catchment. The time of flow over a pavement surface in seconds is given by:

$$T_S = 500\ n\ \frac{l_f{}^{2/3}}{S_f{}^{1/3}} \quad \text{(chart 7)}$$

in which n = Mannings roughness
l_f = flow path length
S_f = flow path slope

S_f will not be constant in many situations, but an average value will generally be sufficient. If desired, the flow path can be broken up into small sections each with an average slope. Each section time can be calculated then all times totalled to give T_S.

Gutter flow time is given by:

$$T_G = \frac{5\ L}{4\ \nu_1}$$

in which ν_1 = gutter velocity at the inlet

L = length of contributing gutter.

Pipe flow time is found directly from the tabulated velocity:

$$T_p = \frac{L_p}{\nu_p}$$

in which L_p = appropriate pipe length

ν_p = water velocity in the pipe

The first two equations are only approximate, but their accuracy is adequate for the purpose.

A concentration time less than 5 minutes should not be used (regardless of the value which is calculated), since water requires some time to build up above the surface asperities.

1.3 SURFACE WATER DEPTH

1.3.1 General Case

Restricting the surface water depth is necessary on highways because of the danger of vehicles aquaplaning.

The depth of surface water flow above the top of a texture is given by the formula (NAASRA (2)):

$$d = 0.046 \frac{(l_f I)^{\frac{1}{2}}}{S_f^{\frac{1}{5}}} \quad \text{(chart 9)}$$

in which d = depth of flow (mm) at the end of the flow path

l_f = length of flow path (m)

I = rainfall intensity (mm/hr)

S_f = flow path slope

On a straight section of road without super-elevation the flow path slope is given by:

$$S_f = (S_l^2 + S_c^2)^{\frac{1}{2}}$$

in which S_l = longitudinal slope (grade)

S_c = cross fall of pavement

Flow path length is given by:

$$l_f = W \frac{S_f}{S_c}$$

in which W = width of pavement contributing to flow

Surface roughness has a negligible effect in the hydraulics of rainwater flow. However, rougher surfaces are desirable because more ponding can occur before water level rises above the texture. An insignificant amount of water flows below the top of the texture.

The critical depth for aquaplaning ranges from 4 mm to 10 mm depending on tyre and pavement surface. The surface water depth therefore, should be restricted to 4 mm for all but special situations where super-elevation produces long, curved flow paths. Higher depths may be accepted over limited areas.

The nature of the problem requires the return period for surface water depth to be less than that for the longitudinal drainage system. Some risk must be accepted as conditions conducive to aquaplaning only occur for a short time of total pavement life.

A complete prevention of aquaplaning risk could involve excessive time and money as the relevant factors are very difficult to control to the extent of restricting surface water depth. Consequently a two year return period is sufficient. In addition a minimum time of concentration of 5 minutes should be used to allow flow to build up above the texture of the pavement.

Increasing pavement crossfall does not have a marked effect on flow depth, but a desirable lower limit of 2.5 per cent will minimize ponding in surface deformations.

In multilane highways the lowest lane can be made steeper. This will speed the drainage of water from the most critical area and it will also allow catchpit spacing to be increased because of higher channel capacity from the steeper crossfall. The formation of a two-way crossfall is a most effective method of decreasing flow depth as flow length can be halved on straight roads. This is effective on straight roads only. If a section of roadway contains less than 60% straights or they are less than 1 km long, then the benefits may be limited. An important consideration to be taken into account is the additional cost of providing collection facilities on both sides of the pavement where two-way crossfall is present.

A factor which is omitted from this method is the relation between rainfall intensity and visibility, hence vehicle speed. In areas of high intensity rainfall, speed will be reduced due to restricted visibility, so it can be argued that a greater depth of surface water would be required to cause aquaplaning at these lower speeds. It is difficult to make a quantitative allowance for speed reduction, but it should be kept in mind as an added safety factor in these areas of high intensity rainfall.

1.3.2 Special Cases

There are three common special situations where surface water depth may become critical. These are curved alignments, gore areas at ramps and super-elevation development.

a Curved Alignments, Twin Roadways: Drainage from the higher carriageway should be collected in the median on curved alignments.

The shoulder on the higher carriageway may add up to 3 m to the width contributing to the flow, but this may be avoided by breaking back the shoulder crossfall. This breaking back can be hazardous for the motorist and the shoulder should not be cut back within 1 metre of the roadway and should not be cut back at all if crossfall is greater than 5%. The slope of cutback shoulders should be limited so that there is not greater than a 6% difference between roadway and shoulder slopes. In addition, absolute shoulder crossfall should not exceed 5.5% except on curves when required to match the crossfall of adjacent traffic lanes.

It is essential that all runoff from batters or verges is contained in a collector drain and not allowed to pass onto the roadway.

b Gore Areas at Ramps: Gore area and the taper length of entry and exit ramps may substantially increase the width of pavement contributing to water flow. For ramps on the back of curves a crown may be introduced between the edges of the through lanes and the ramp provided that super-elevation is sufficiently small that total change is limited to not more than 6%.

The situation is worse on the low sides of curves where the full pavement width is draining across the ramp which is an area of uncertainty for the motorist due to changing speeds, direction and grades. It may be possible to extend a grated trench down the gore area to intercept runoff from the main carriageway.

c Super-elevation Development: In some circumstances the development of super-elevation by normal means can result in excessive water depths for design rainfall. This tends to be the case where a wide pavement occurs with either:

i Small path slope - this occurs on flat longitudinal grades where crossfall is zero (normally at a TP), or

ii Long flow paths - these occur on steep longitudinal grades where the flow path component due to crossfall is relatively small allowing water to tend to run along the pavement instead of transversely off it.

Two methods are illustrated which reduce surface water depth over areas of super-elevation development. These are diagonal and longitudinal crowning.

1. Diagonal Crowning: This is an excellent hydraulic solution as water is drained directly across the pavement as on a straight section of road. The diagonal crown is developed over the length normally used to develop super-elevation equal to crossfall. This gives an effective TP in the same position as for normal super-elevation development.

   The main arguments against diagonal crowning are the practical difficulties of paver laying and the questionable effect of the sudden localised change in crossfall to be negotiated by vehicles.

2. Longitudinal Crowning: Super-elevation development is longitudinally offset in two or more sections to give shorter curved flow paths. This is easier to paver lay than a diagonal crown but requires a greater pavement length for development of full width super-elevation.

   It may still give quite substantial depths of surface water.

The choice of either of the above methods will depend on individual circumstances. The longitudinal crowning alternative is generally favoured where acceptable depths can be attained.

Methods of super-elevation development are illustrated in figures 1, 2 and 3. They show contours, illustrative flow paths and surface water depths under a design rainfall. The examples each have a pavement width of 16 m, normal crossfall 2.5% design rainfall 80 mm/hr and longitudinal slopes 0.2% (Fig 1), 3.0% (Fig 2) and 10.0% (Fig 3). Normal super-elevation is developed from -2.5% to +2.5% over 72 m.

Surface water depths are calculated by first scaling off flow path lengths directly from contour plans, calculating flow path slopes (directly or from contours) and then using Chart 8. Consider the case of normal super-elevation development on 0.2% longitudinal slope for the longest flow path.

At end of flow path:

Flow path length	l_f	=	50 m
Flow path slope	S_f	=	0.018
	I	=	80 mm/hr
	$l_f I$	=	4000
Hence depth	d	=	6.4 mm (chart 8)

At minimum flow path slope:

l_f	=	25 m
S_f	=	0.002
I	=	80 mm/hr
$l_f I$	=	2000
d	=	7.2 m

Figures 1, 2 and 3 show flow characteristics of surface water and give an indication of relative merits of methods used for super-elevation development. It is to be noted that the stagger required for longitudinal crowning depends on longitudinal slope (grade). The 18 m stagger in figure 1 for the small 0.2% grade is sufficient to break up longest flow path, however this is not the case in figure 2 with 3% grade. A 72 m stagger for 3% grade (full development of first half of pavement before starting second half) is satisfactory as shown. The 10% grade would require a minimum stagger of about 157 m to 124 m with associated decreases in depth from 8.1 mm to 7.2 mm.

1.4 SURFACE COLLECTION AND INLETS

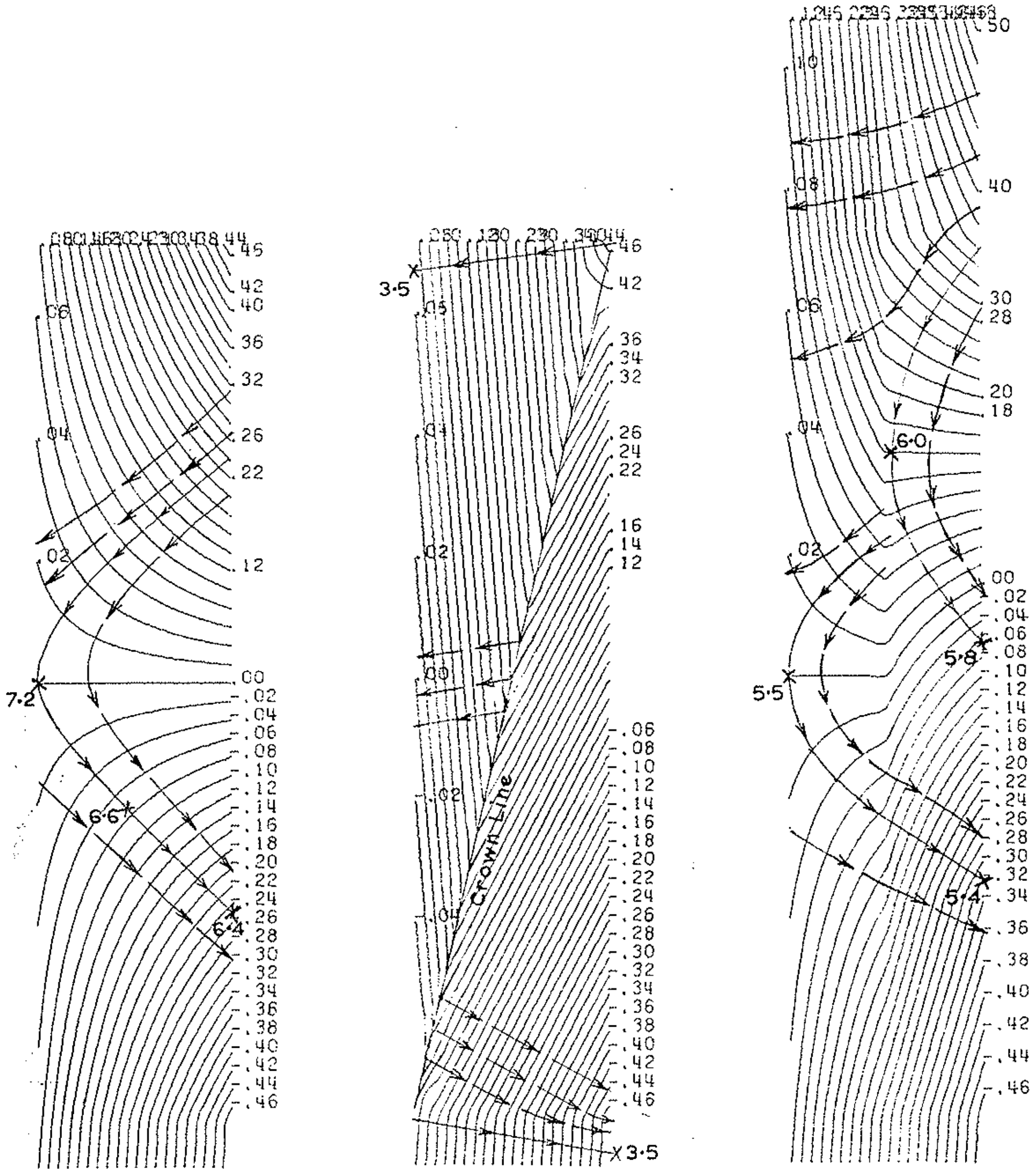
1.4.1 Definitions

A definition of a few basic terms used in this document is included at this point to avoid confusion as there is a wide range of terminology in use around the country:

An inlet is the structure which allows water to be removed from the surface, eg a grate or kerb opening.

A catchpit is the space beneath an inlet which traps debris and transfers water to an underground pipe.

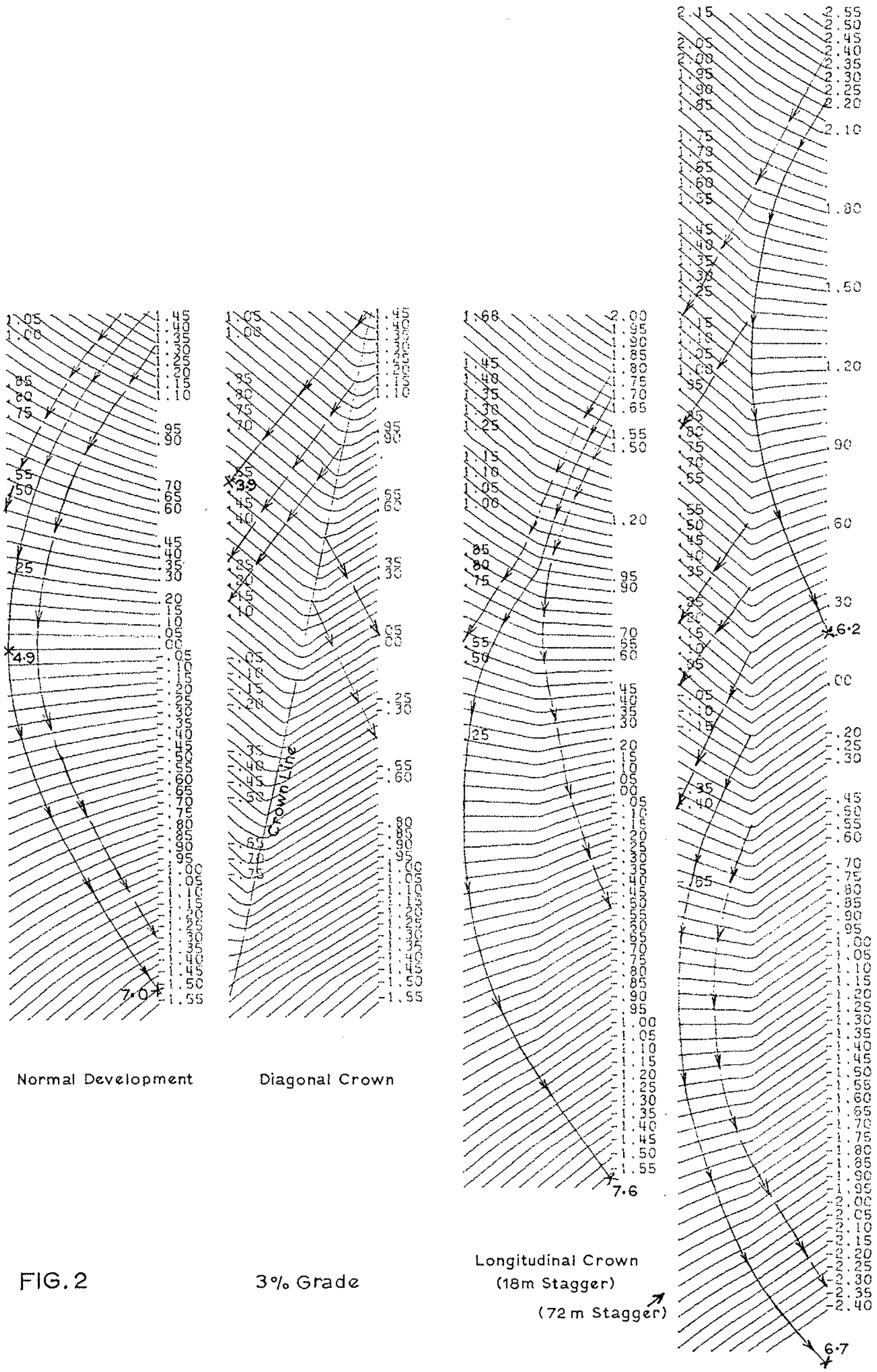
A sump is a catchpit either at the bottom of a sag or where water is static above the inlet.



Normal Development | Diagonal Crown | Longitudinal Crown (18m Stagger)

FIG. 1

0·2% Grade



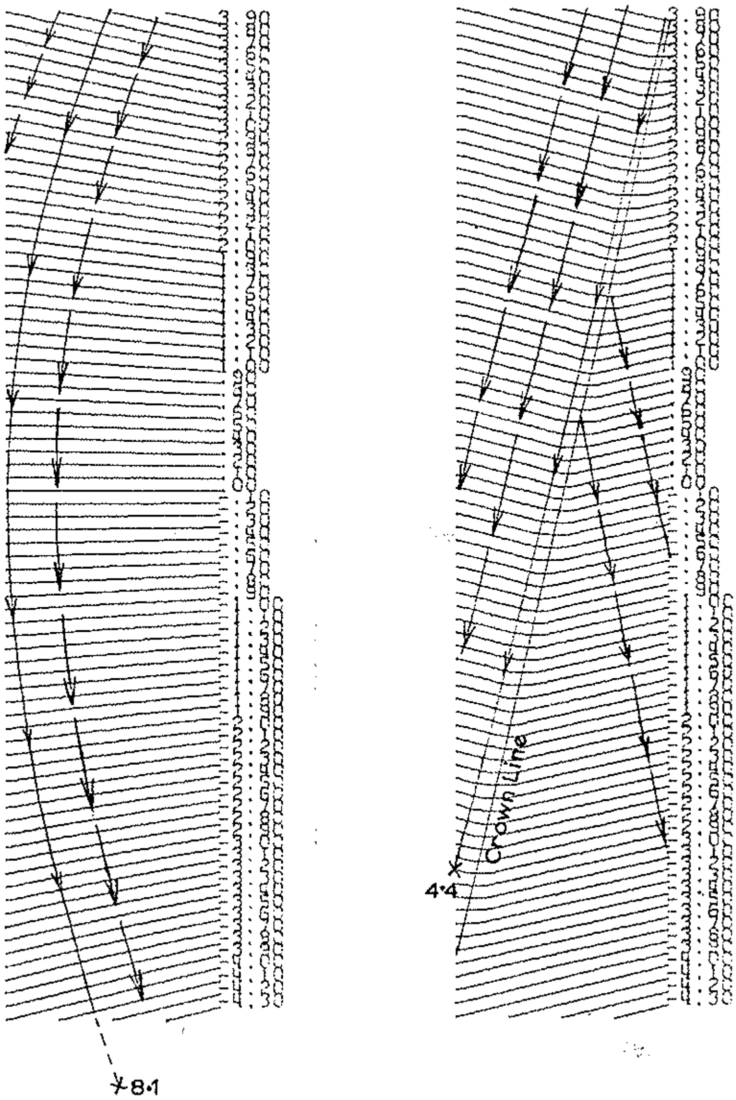
Normal Development

Diagonal Crown

Longitudinal Crown
(18m Stagger)
(72 m Stagger)

FIG. 2

3% Grade



Normal Development

Diagonal Crown

FIG. 3 10% Grade

1.4.2 Collection by Kerb and Channel

This is the most common method of surface collection because it is the cheapest and simplest in practice. Inlet structures in kerb and channelling are of three major varieties: kerb-opening, grate and combination (refs 3 and 4).

Capacity can be improved by depressing the inlet below the normal invert of the waterway. However, depressed inlets should not be used in the highway situation as they are a serious obstruction to traffic which can be expected to travel close to the kerb at high speeds.

Another means of increasing capacity is to use deflector vanes in conjunction with kerb opening inlets. If these are in the plane of the gutter they will not be an obstruction to traffic. A disadvantage of both depressed inlets and deflectors is high cost when constructed in conjunction with concrete extrusion machines, because an increased amount of handwork becomes necessary. Grate bars should be orientated parallel to the flow in all cases as cross grates are highly susceptible to clogging with debris and are much less efficient hydraulically. There can be a problem with bicycle tyres slipping through the grate if gaps are greater than 20 mm, however the effect can be minimised by placing a few crossbars underneath the grate. Provided these crossbars are well recessed they will not affect the grate efficiency.

The kerb opening of a combination inlet adds little to the capacity of the inlet, but it is a desirable feature because it accepts much of the debris that would otherwise clog the grate, and also accepts water if the grate does clog up.

The exact calculation of inlet capacity is a difficult hydraulic problem and the following simplified procedures are recommended for design purposes:

a Grate Inlet: As the velocity of flow varies considerably through the cross section, flow is considered in two segments defined by gutter, grate and road geometry.

Firstly flow directly above the grate is considered and average velocity and discharge found according to Mannings formula or nomograph (chart 9).

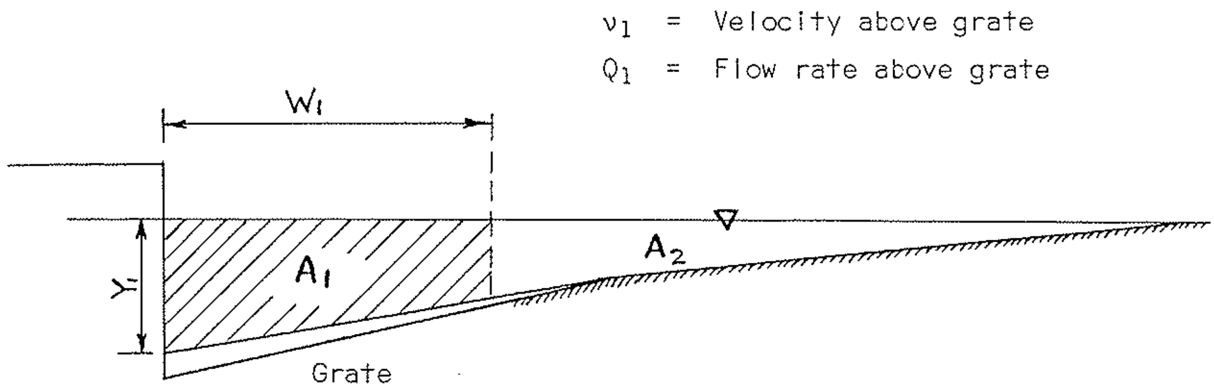


Figure 4

The length of grate (L_1) required to capture the top of this flow from area A_1 above the grate is found by modifying the equation of length of capture for flow into an open pit to allow for the gratings.

For flow into an open pit (ref 3):

$$L_1 = \nu_1 \left(\frac{2Y_1}{g}\right)^{\frac{1}{2}}$$

in which ν_1 = velocity in the gutter

Y_1 = depth of flow at kerb face

Modified, this equation gives:

$$L_1 = m\nu_1\left(\frac{Y_1}{g}\right)^{\frac{1}{2}} \text{ (chart 10)}$$

in which the value of m depends on the grate (ref 3).

For grates with longitudinal bars and gaps about the same size as bars, a value of 4 should be used. There is little information available about values for m for other bar configurations, but where 3 bars are placed transversely for structural purposes, m increases to 8 which is a good indication of the inefficient behavour of cross grates.

L_1 is now calculated and the length of grate should be made greater than this value to ensure that all flow directly above the grate is taken in. If this is not possible, then inlet flow from above the grate is decreased by the ratio $(L_G/L_1)^2$ where L_G is grate length. Bypass from above the grate is:

$$Q_B = \left(1 - \left[\frac{L_G}{L_1}\right]^2\right) Q_1$$

Flow outside the grate is now considered as a single section and velocity calculated. As the grate extends to the edge of the gutter or very close, Mannings roughness value can be taken as that for the pavement. Some of this flow will discharge into the inlet and the amount is determined from the equation:

$$Y_3 = 6.81 \left(\frac{L_G S_c}{\nu_2}\right)^2$$

in which Y_3 = vertical displacement between the edge of the grate and road position where the outer flow path is captured (fig 5)

ν_2 = velocity in this section

S_c = road crossfall

As the value of Y_3 defines the portion of flow captured by the inlet from outside the grate, the corresponding flow width (W_3) may now be found. Now the area of captured flow is found and by assuming velocity is constant across the section and discharge, Q_3 into the inlet is determined by ratio of areas.

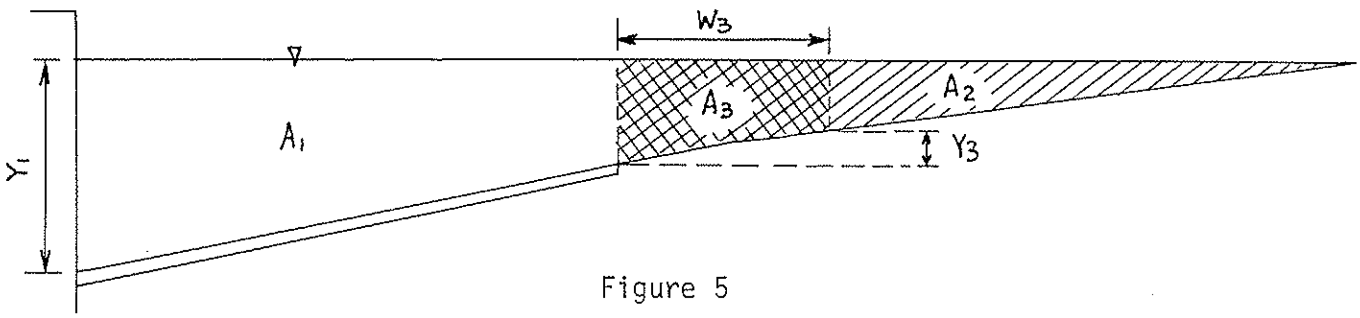


Figure 5

A_2 = area outside the grate

A_3 = captured area outside the grate

Q_2 = flow outside grate

Q_3 = captured flow outside grate

also Q_o = total gutter flow

and Q = discharge into inlet

We now have:

$$Q_3 = \left(\frac{A_3}{A_2}\right) Q_2$$

$$Q_o = Q_1 + Q_2$$

$$Q = (Q_1 - Q_B) + Q_3$$

where in most situations $Q_B = 0$

therefore $Q = Q_1 + Q_3$

Design of dished medians may be carried out by the same general procedure as for kerb and channelling.

b Kerb Inlet: Kerb inlets without a good deflector are generally inefficient and are not recommended. Those with deflectors become competitive on slopes greater than about 5%. For the design of deflectors with rectangular corrugation, the following relationship has been found experimentally, incorporating dimensionless analysis (ref 3).

$$\frac{Q}{LY_1\sqrt{gY_1}} = 0.058\left(\frac{c}{b}\right)^{\frac{1}{20}}\sqrt{\frac{S_\ell}{n}}\left(\frac{W}{L}\right)^{\frac{1}{2}}$$

in which Q = discharge into inlet

L = length of kerb opening

W = width (across channel) of deflector vane construction

g = acceleration due to gravity

b = width of deflector vane

c = clearance between deflector vanes

S_ℓ = longitudinal channel slope

n = Mannings roughness coefficient for gutter

The tests from which the equation was derived had the following limitations:

$$\frac{W}{L} > \frac{1}{3}$$

$$\frac{a}{L} \geqslant \frac{1}{50}$$

$$22 \geqslant \frac{\sqrt{S}}{n} \geqslant 12$$

$$\frac{Q}{Q_o} \geqslant 0.7$$

$$N \geqslant 6$$

and deflectors oriented at 45^0 to kerb line

in which N = number of deflectors

a = height of deflector vanes at kerb opening

Q_o = total gutter flow

Hence the equation should be used with caution outside these limits.

As can be seen from the equation, Q is fairly insensitive to $\frac{c}{b}$ so it can be written and used in most cases as:

$$Q = 0.06\, Y_1\sqrt{(gY_1LW)}\ \frac{\sqrt{S\ell}}{n} \quad \text{(chart 11)}$$

in which $\frac{c}{b}$ is assumed to be 2. This is reasonably accurate for $\frac{c}{b}$ up to about 5.

The inlet capacity is now calculated for maximum permitted flow width by using the appropriate equation, above. The value of Mannings n should be that of the gutter.

c Sump Inlet: A kerb or combination inlet should be used as grates in sump inlets are very prone to clogging with debris. Grate inlet capacity depends on the total area between bars and is independent of the orientation of the grate to flow. The design capacities are:

Grate inlet	Q_G	$= 0.6A\,(2gh)^{\frac{1}{2}}$	(chart 12)
Kerb inlet	Q_K	$= 1.66hL$	(chart 13)
Combination inlet	Q	$= Q_G + Q_K$	

in which h = depth of ponding determined by allowable encroachment

A = total area between bars

L = length of kerb opening

Sumps should be designed to take all flow plus an allowance for bypass from clogged upstream inlets. Therefore, the design flow value should be the calculated inlet flow plus the largest inlet flow upstream of the sump.

d Side Entry Sumps: Kerb inlets without a deflector are generally inefficient, but in some circumstances they can be the best overall solution. A good example of this is the 'side entry sump' type inlet where the carriageway is taken at normal crossfall right to the kerb face. Collection is by a sump type inlet recessed and depressed behind the line of the kerb.

The inlet is designed as first a kerb-opening to remove water from the carriageway, then a sump to transfer water to the underground sewer system.

An advantage of this type of inlet is that there is no interference from channels or inlets near the kerb face allowing traffic to travel close to the kerb in safety. Kerb opening does, however, need to be fairly long (about 3 m) and the sump surrounds should be sloped back (ie not vertical) to assist vehicles which may enter the kerb opening.

Design procedure is as follows:

Gutter flow is calculated (chart 9) then divided by total cross-sectional area of flow to give design velocity, V.

$$\nu = \frac{Q_o}{A_o}$$

in which Q_o = total flow

A_o = total cross-sectional area of flow

Width of flow captured is given by:

$$W = 6.81\ S_c\left(\frac{L_k}{\nu}\right)^2$$

in which S_c = crossfall

L_k = length of kerb opening

Area of captured flow (defined by W) is calculated, hence inlet flow is given by:

$$Q = \nu A$$

in which A = area of captured flow.

The flow Q is that which is removed from the carriageway to the recessed inlet. This is designed as a sump to determine required depression of inlet to present flow over the sides of sump surrounds or back out the kerb opening.

1.4.3 Special Collection Methods

Kerb and channel can be a traffic hazard and in some cases another method of surface water collection may be more desirable.

a **Slot Drains:** Slot drains are excellent because there is no encroachment of water on the road under design load. Furthermore, they do not interfere with traffic. However, they are difficult to clean and are relatively expensive to install.

Allowable slot drain discharge is calculated using Mannings equation:

$$Q = \frac{1}{n} R^{2/3} \sqrt{S_\ell}\, A$$

in which n = Mannings roughness of the slot drain

R = hydraulic radius of water flowing in the drain

S_ℓ = longitudinal slope

A = cross-sectional area of water in the drain

Where the drain reaches capacity it is cleared through a collector pipe.

b **Continuous Grate and Channel:** Discharge calculations for continuous grating over a channel can again be carried out using Mannings equation. This system causes minimum traffic disturbance, however, it is much more expensive than the alternatives.

1.4.4 Inlet Positioning

Inlets should be spaced such that gutter flow will not encroach on traffic lanes for a return period of 5 years. Determination of position for all types of inlet can be made in the following manner:

For the roadway under consideration, plot c hannel capacity against distance stating at a crest. Plot the estimated runoff against distance on the same graph. The intersection of the two curves gives the required position of the first inlet.

Subtract inlet flow from runoff at this point and set off flow against distance until it intersects the channel capacity curve again, giving the second inlet position. Continue this process to a sag where a sump inlet is designed (fig 6).

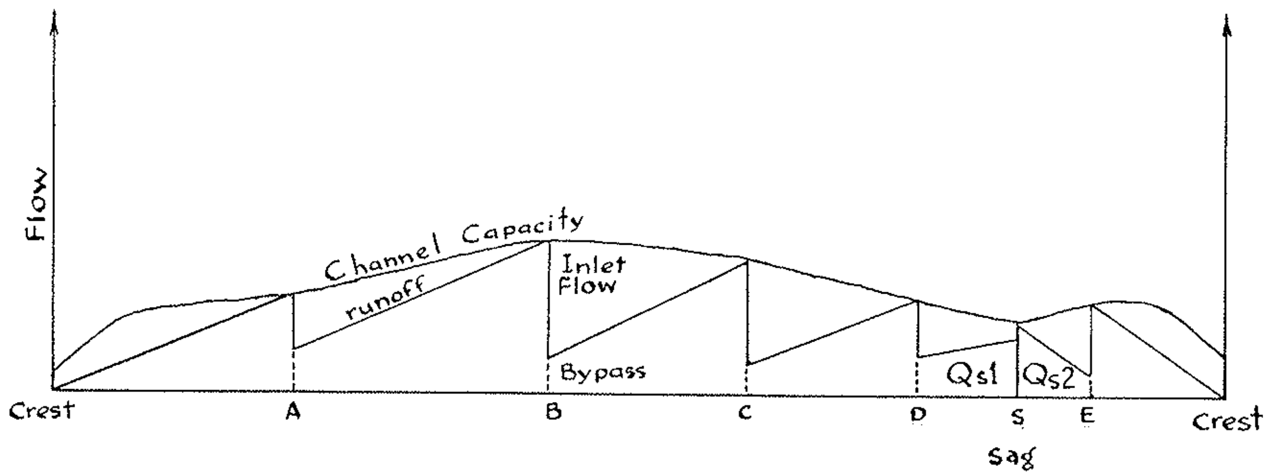


Figure 6

Inlets are at A, B, C, D and E and a sump inlet at S. The design flow for the sump is given by:

$$Q_S = Q_S1 = Q_S2 + Qmax$$

in which Q_S1 = calculated flow at the sump from one side of the sag

Q_S2 = calculated flow at the sump from the other side of the sag

$Qmax$ = the maximum inlet flow above the sump. (In fig 6 this would be the inlet flow at C)

In a number of situations a graphical construction will not be necessary as the gutter capacity will be constant, allowing inlet position to be found by direct calculation.

A special situation arises for very flat grades less than about 0.1% which occurs especially in the lower chords of a sag. As the channel grade tends to zero, the capacity according to the nomograph also tends to zero. However, this capacity is a theoretical value for uniform flow in an infinitely long gutter and in practice the presence of an inlet on a perfectly flat grade will induce a non-uniform flow in the gutter.

Because of this effect, a minimum value of longitudinal slope of 0.1% should be used in calculating the inlet spacing. Furthermore, it should never be necessary to have an inlet spacing less than 15 m even when a smaller value has been calculated. The stormwater system should be checked for the fifty year storm. In general the road should be negotiable on at least half a traffic lane which should have no more than 100 mm depth of surface water. Long term requirements will depend on the specific circumstances and should be part of the design brief.

1.5 DESIGN DETAILING

1.5.1 Catchpits, Manholes and the Underground Pipe Network

There should be sufficient depth in the catchpit below the outlet level to catch large debris which may block pipes. The depth required depends on the extent of debris in the area and frequency with which catchpits are cleaned. A nominal amount of the order of 0.5 m is usual. Other dimensions are dependent on the inlet and pipe positioning and geometry.

Pipes should be designed from any suitable graphs or tables which usually assume they are flowing just full, ie there is no surcharging.

Each manhole should have a drop between inlet and outlet to compensate for head losses through it. These drops between the respective pipe soffits should be at least (ref 5):

$.25 \dfrac{v_p^2}{2g}$ for straight sections of sewer

$.30 \dfrac{v_p^2}{2g}$ for horizontal angles up to 10^0

$.40 \; \frac{vp^2}{2g}$ for horizontal angles from $10^0 - 22\frac{1}{2}^0$

$.45 \; \frac{vp^2}{2g}$ for horizontal angles from $22\frac{1}{2} - 45^0$

$.50 \; \frac{vp^2}{2g}$ for horizontal angles from $45^0 - 90^0$

Note: vp = calculated pipe velocity.

Pipe depth is determined by structural considerations which produce not less than about 0.30 m cover to any pipe. This allows some surcharging in catchpits, resulting in discharge additional to design flow before a pipe is overloaded and water rejected from a catchpit or manhole.

It is necessary to lay pipes so that they do not silt up from deposits left by slow flowing water. For this reason pipes should be laid on a steep a grade as practicable and not less than the figures below for the listed diameters:

Diameter (mm)	Grade (%)
150	1.0
225	0.5
300	0.35
375	0.25

1.5.2 Design Procedure for Sewer System (Escritt 6)

Draw a plan of the sewer system and classify each section between inlets as shown in fig 7.

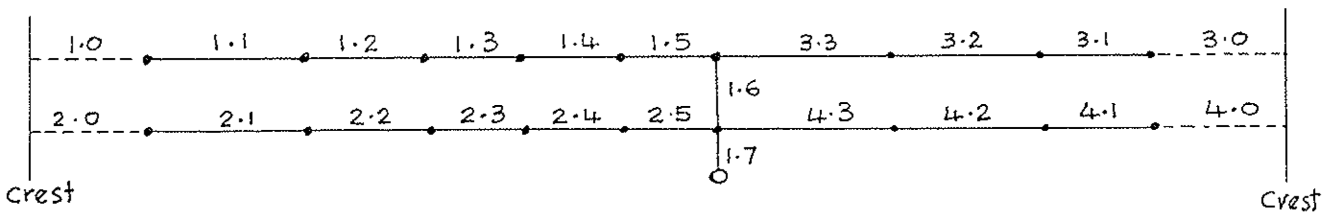


Figure 7

1	2	3	4	5	6	7	8	9	10	11	12
Length No.	Slope	Dist	Pipe Diam	Vel	Time	Time of Conc	Rainfall Intensity	Inlet Flow	Total Flow	Pipe Cap	In/Out

Figure 8

Fill in 1, 2, and 3 from initial conditions. Guess 4 and get 5 and 11 from design tables. Calculate 6, 7, 8, 9, 10 and 12. Now compare 10 and 11 (i.e. capacity and discharge) and adjust pipe diameter if necessary and start again. For subsequent lengths, it is generally best to use as a first guess the same pipe size as the preceding length, however, in many cases it will be obvious which pipe size is required because of limitations in the available sizes.

As time of concentration increases and intensity decreases down the system, the whole procedure of inlet capacity calculation should be repeated for each new intensity. However, it will be sufficiently accurate in most cases to consider inlet flow proportional to intensity for determination of design flow. This will probably make no difference in the pipe size chosen because of the limitations in pipe range.

A full calculation may be carried out in marginal situations.

DESIGN PROCEDURE

The step by step procedure is set out below and should be generally applicable to all surface drainage designs.

1. Draw a plan and longitudinal section of the road. It is convenient to draw cross-sections of the drainage channel and adjacent roadway at this stage.

2. Draw flow paths onto the plan from the necessary contours.

3. Calculate times of surface flow (T_S) using chart 7. If the times are greater than five minutes they are used as times of concentration for surface water depth calculations. Note however, that this will seldom occur.

4. Use the time of concentration (T_S or 5 minutes) to calculate 2 year design rainfall (charts 1, 2, 3, 4, 5, and 6).

5. Calculate the depths of surface water flow at critical areas (end of long flow paths and on very small slopes) from chart 8. Depths should be generally less than 4 mm but up to 7 mm can be permitted over small areas.

6. Calculate channel capacity and plot this against the length of road (chart 9).

7. Estimate the coefficient of impermeability for the catchment area.

8. Calculate the 5 year rainfall intensity using an estimated time of concentration (usually the minimum 5 minutes).

9. Plot runoff against length to find the inlet position (i.e. where runoff reaches channel capacity). Check the time of concentration (by adding gutter flow time to surface flow) and adjust if necessary. Adjustment to the nearest minute is sufficiently accurate.

10. Calculate inlet capacity to determine bypass (if any):

    a Grate Inlet

    Grate should be long enough to capture all flow directly above the grate (chart 10). Find the captured flow from outside the grate and bypass for the particular situation.

    b Kerb Inlet With Deflector

    Deflector inlet flow for prescribed geometrical restrictions is given by chart 9.

    c Kerb Inlet Without Deflector *(Note: Side Entry Sump)*

    Width of flow captured hence inlet flow and bypass are calculated.

11. Add the inlet bypass value to the runoff value to determine the position of the next inlet.

12. Repeat steps 9 to 11 until a sag is reached where a sump will be needed.

13. Design sump inlets at the bottom of sags for higher flows than actually calculated at the inlet to allow for upstream overflow (chart 12 and 13).

14. Design catchpits to have sufficient depth below outlet to trap debris.

15. Design manholes to have a drop between inlet and outlet to allow for head losses.

16. Design the sewer system with the aid of suitable pipe flow graphs or tables. This involves an iterative process in selecting the pipe size. Pipes should be laid on a grade sufficiently steep to avoid silting up.

DESIGN CHARTS

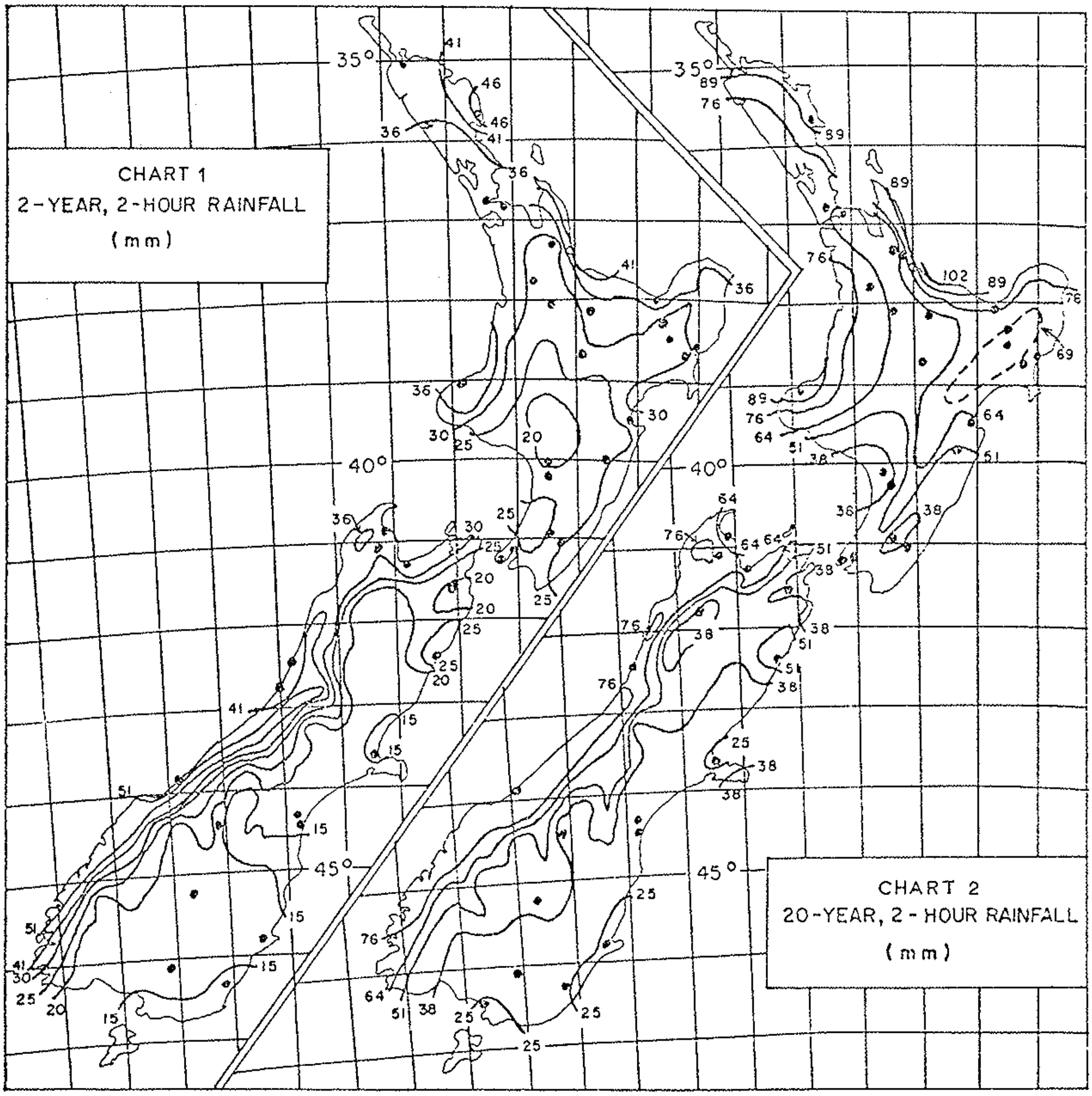


CHART 1
2-YEAR, 2-HOUR RAINFALL
(mm)
CHART 2
20-YEAR, 2-HOUR RAINFALL
(mm)
35°
40°
45°

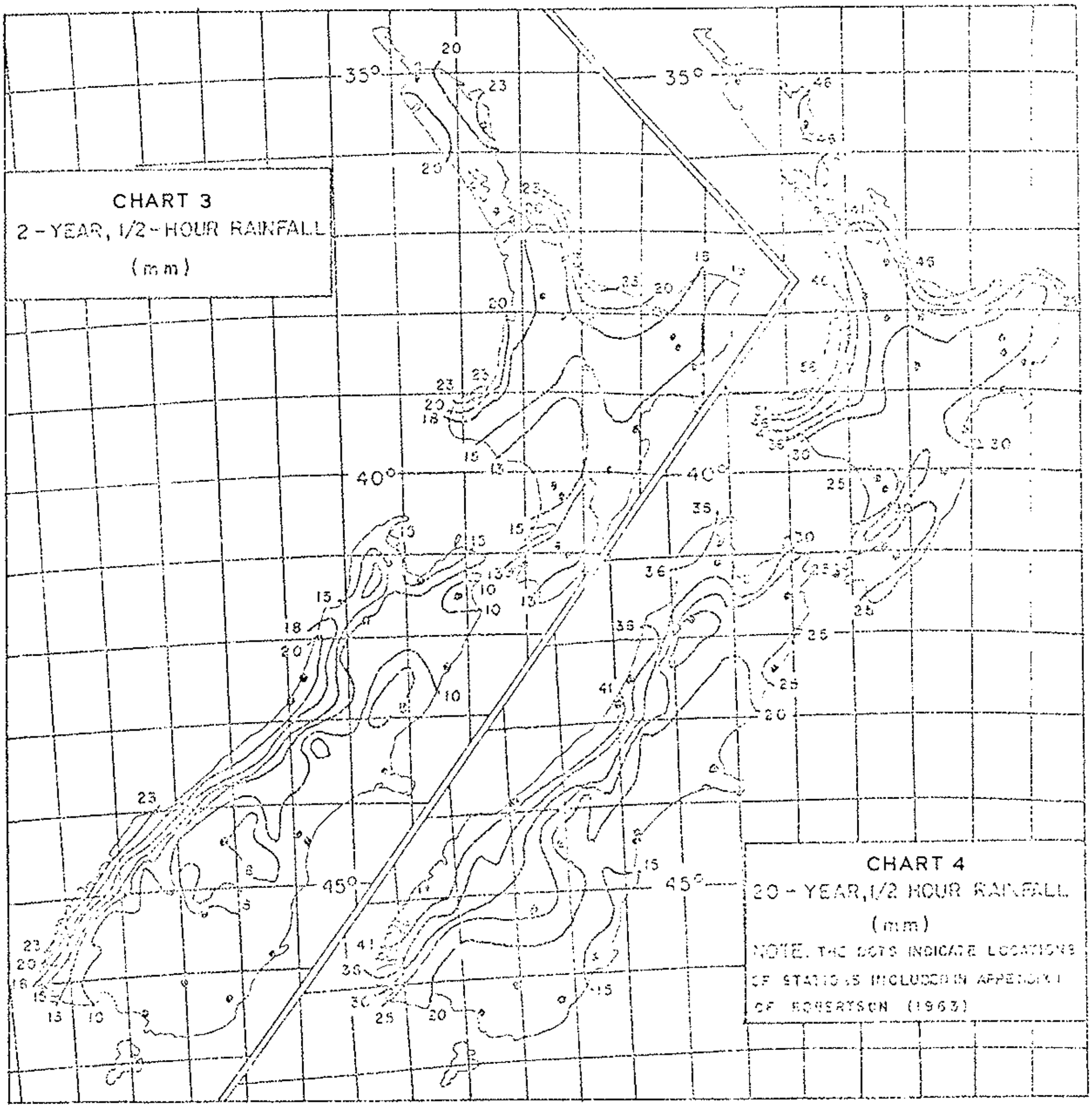


CHART 3
2 - YEAR, 1/2 - HOUR RAINFALL
(mm)
CHART 4
20 - YEAR, 1/2 HOUR RAINFALL
(mm)
NOTE. THE DOTS INDICATE LOCATIONS
OF STATIONS INCLUDED IN APPENDIX I
OF ROBERTSON (1963)
35°
40°
45°

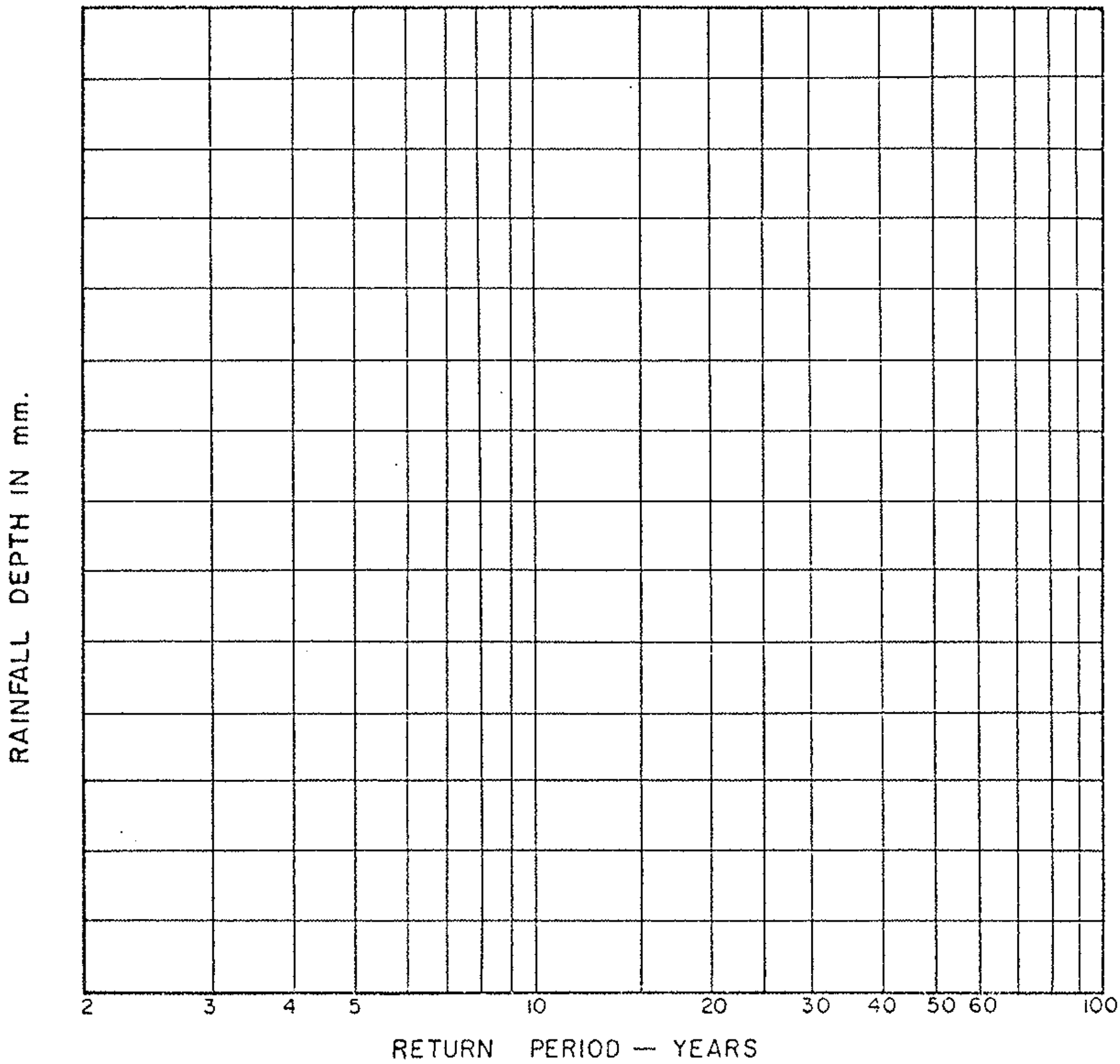
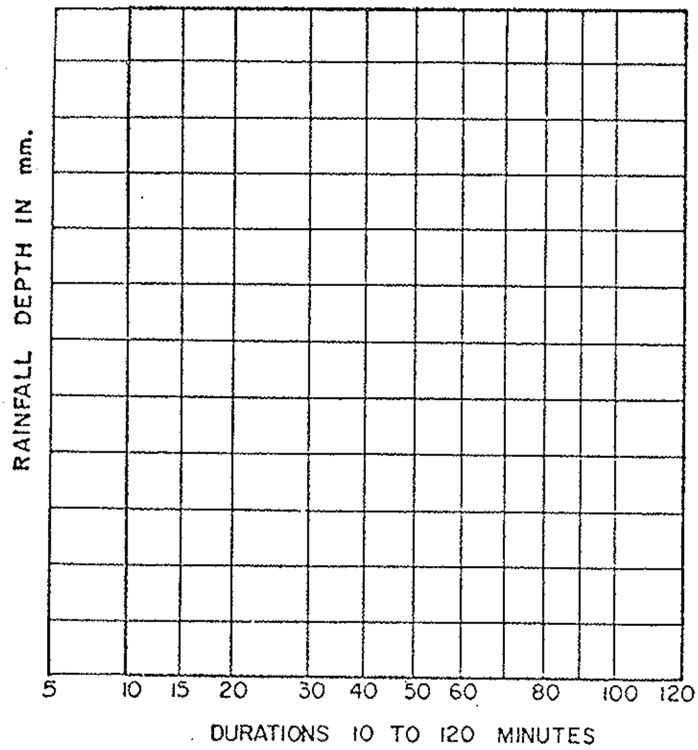


CHART 5 RETURN PERIOD PLOTTING DIAGRAM



DEPTH-DURATION PLOTTING DIAGRAM

CHART 6

CHART 7. TIME FOR FLOW OVER A PAVED SURFACE

$$T_s = 500n \times \ell_f^{2/3} / S_f^{1/3}$$

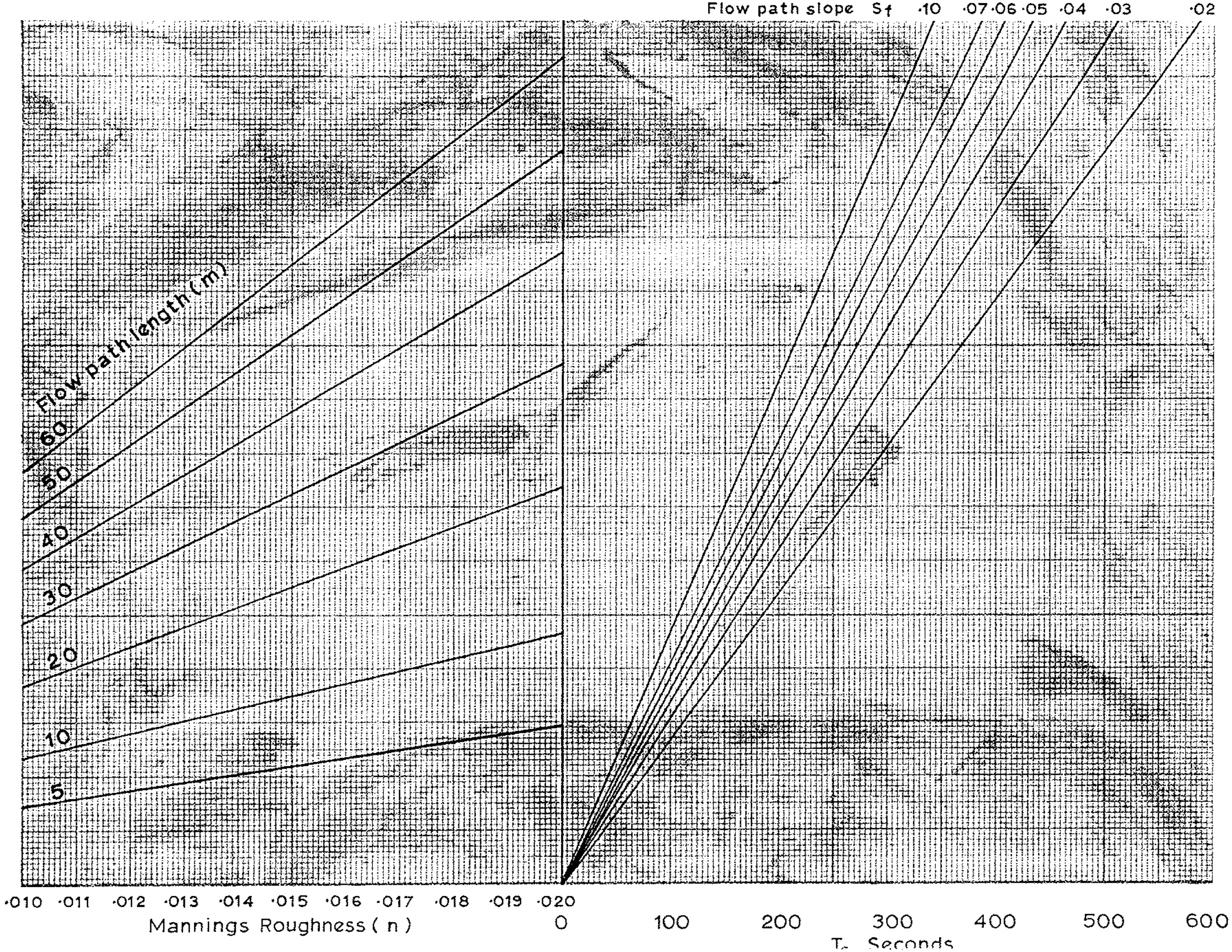


CHART 8 SURFACE WATER DEPTH

$$d = 0{\cdot}046\,(\ell_f I)^{\frac{1}{2}} / S_f^{\frac{1}{5}}$$

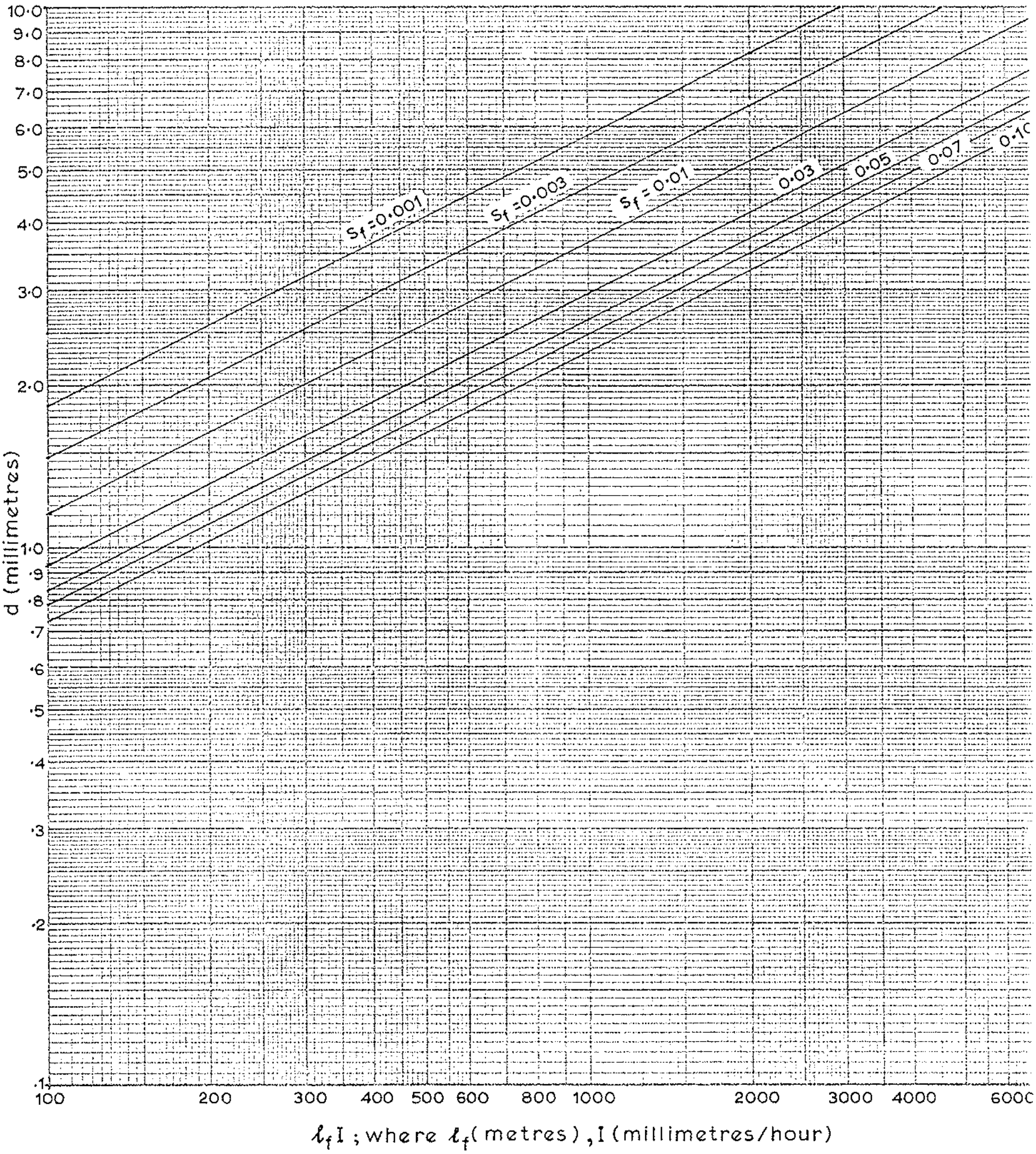
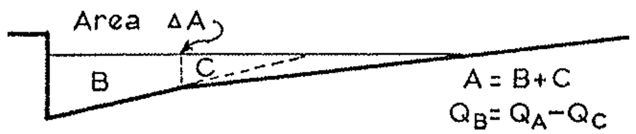


CHART 9. DISCHARGE FROM A TRIANGULAR SECTION

Discharge can be calculated for non triangular sections which are a combination of triangular sections.

$$Q = \frac{3/8\, S_L^{1/2}\, d^{8/3}}{n S_c}$$



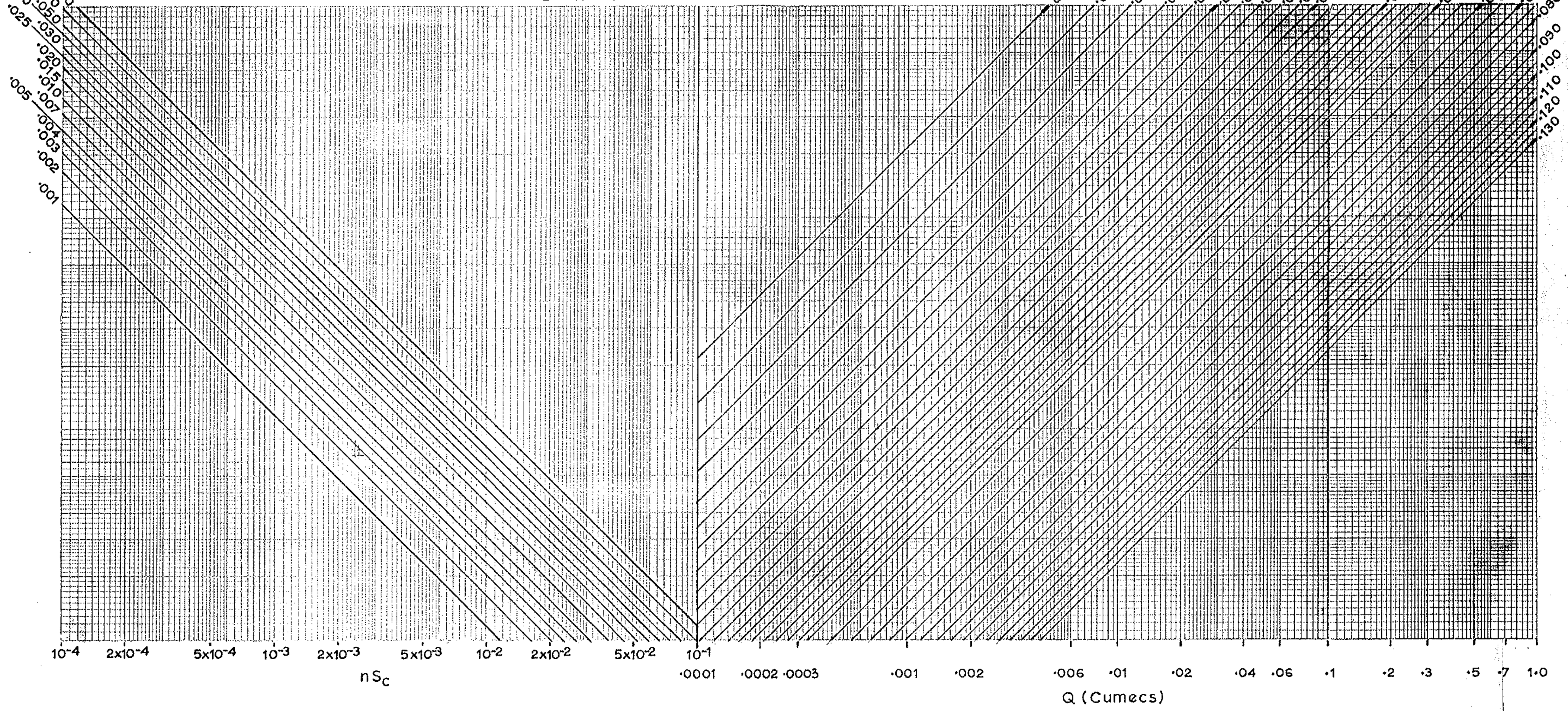


CHART 10. GRATE LENGTH FOR 100% CAPTURE OF FLOW DIRECTLY ABOVE INLET

$L_1 = mV_1\sqrt{\frac{Y_1}{g}}$ where $m = 4$, hence $L_1 = 1{\cdot}28V_1\sqrt{Y_1}$

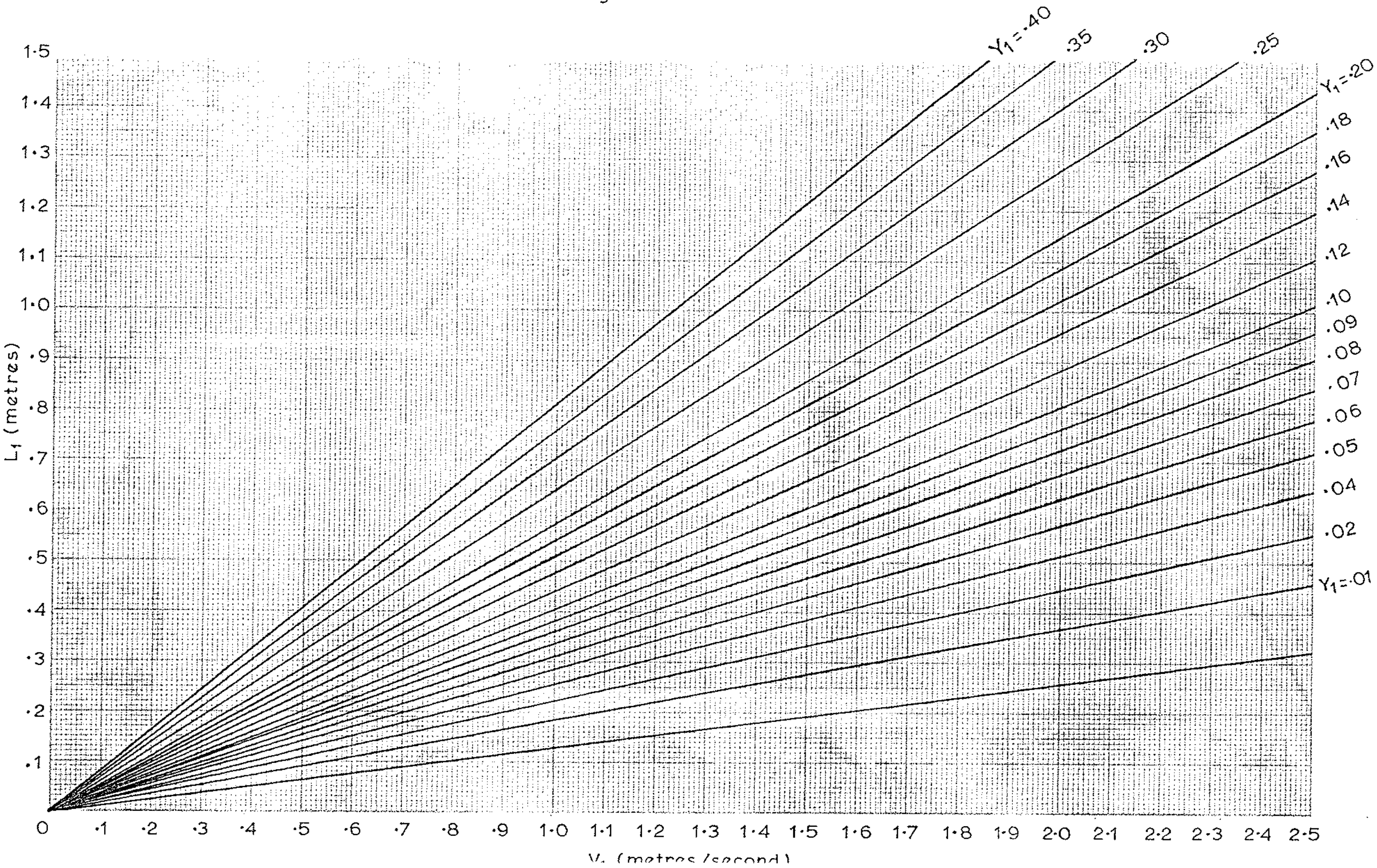


CHART 11. DEFLECTOR INLET FLOW

$$Q = 0{\cdot}188\,Y_1^{3/2}\,\frac{\sqrt{LWS}}{n}$$

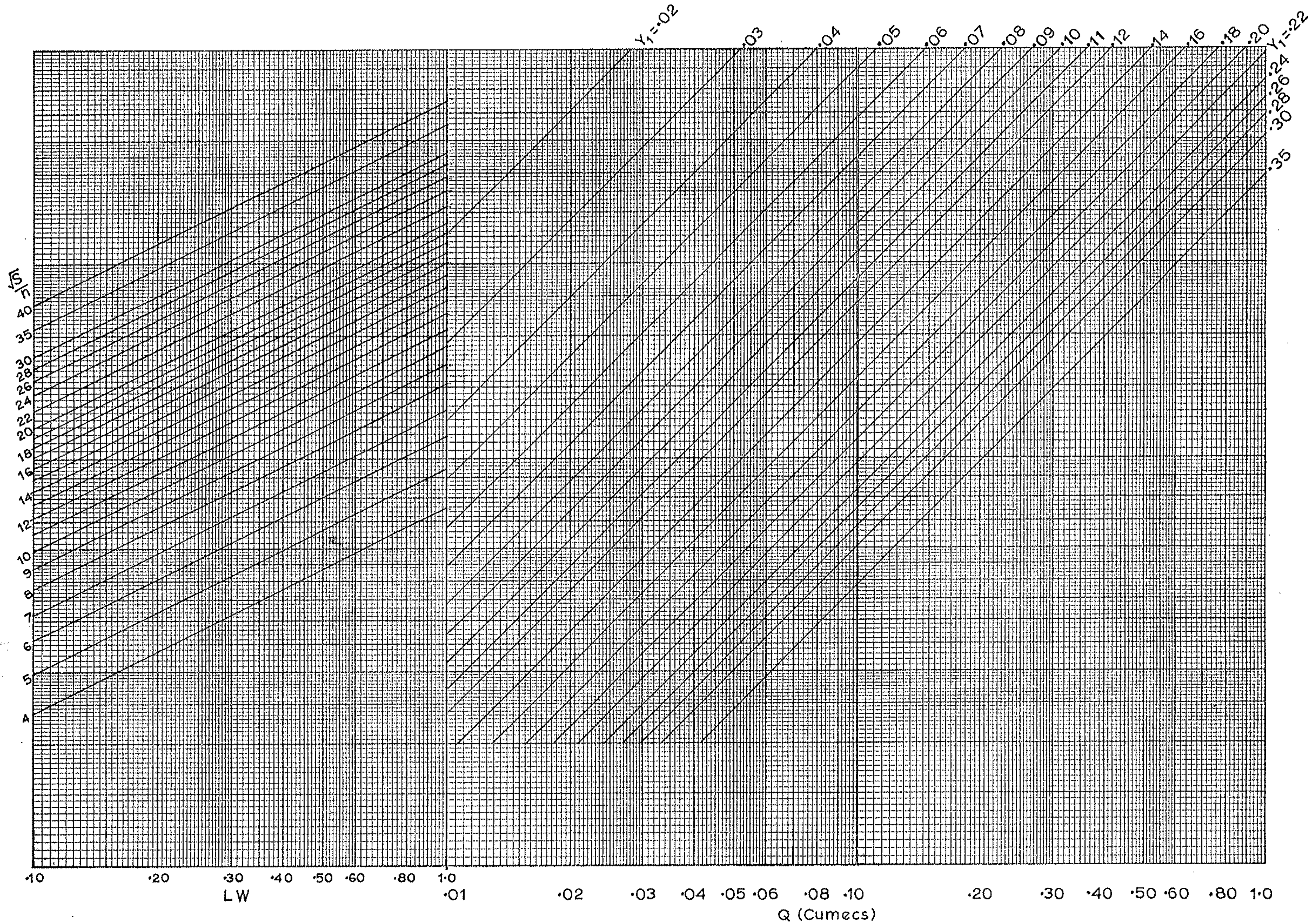


CHART 12. GRATE INLET FLOW AT A SUMP

$Q_G = 0{\cdot}6A\sqrt{2gh}$

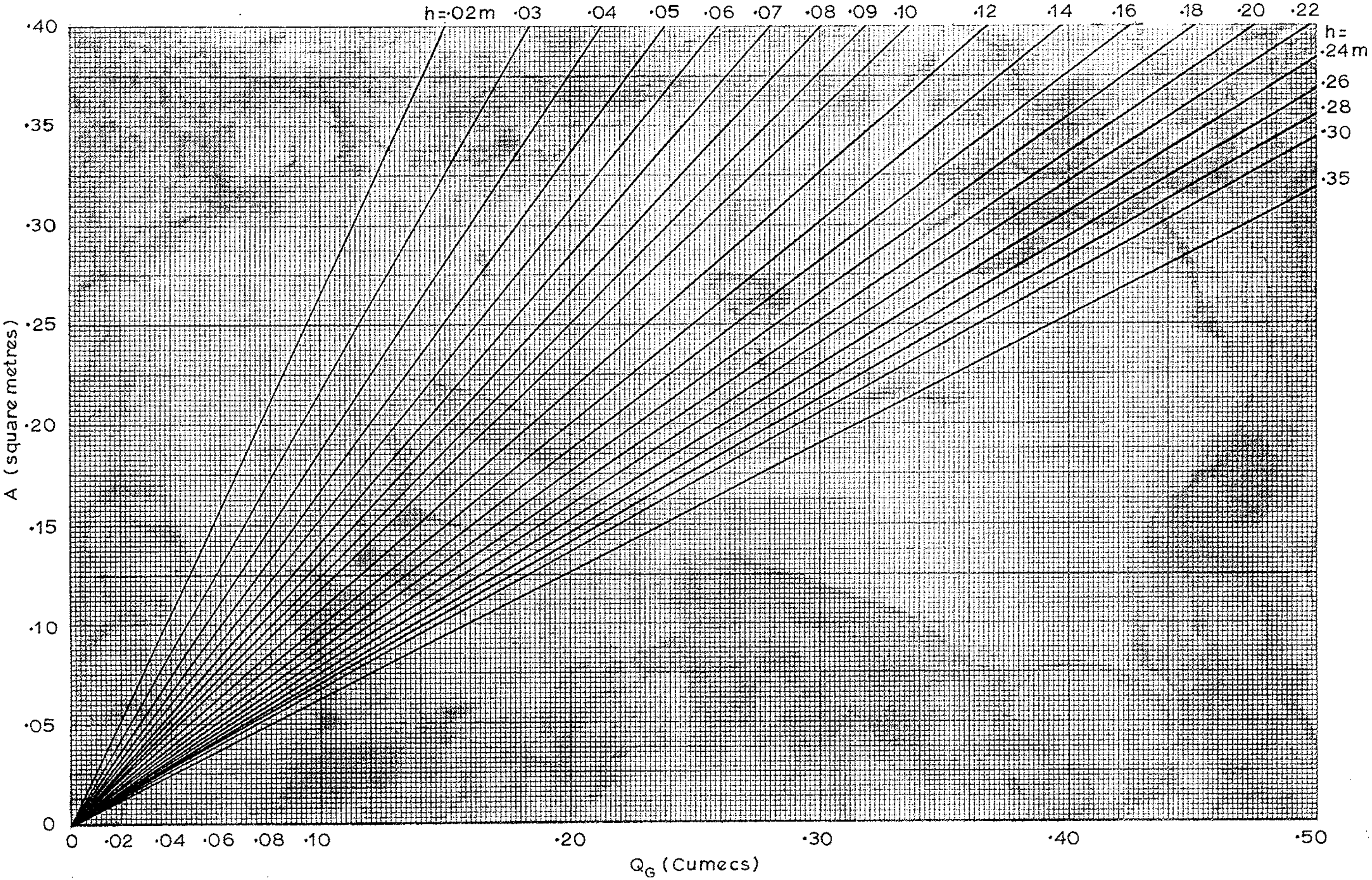
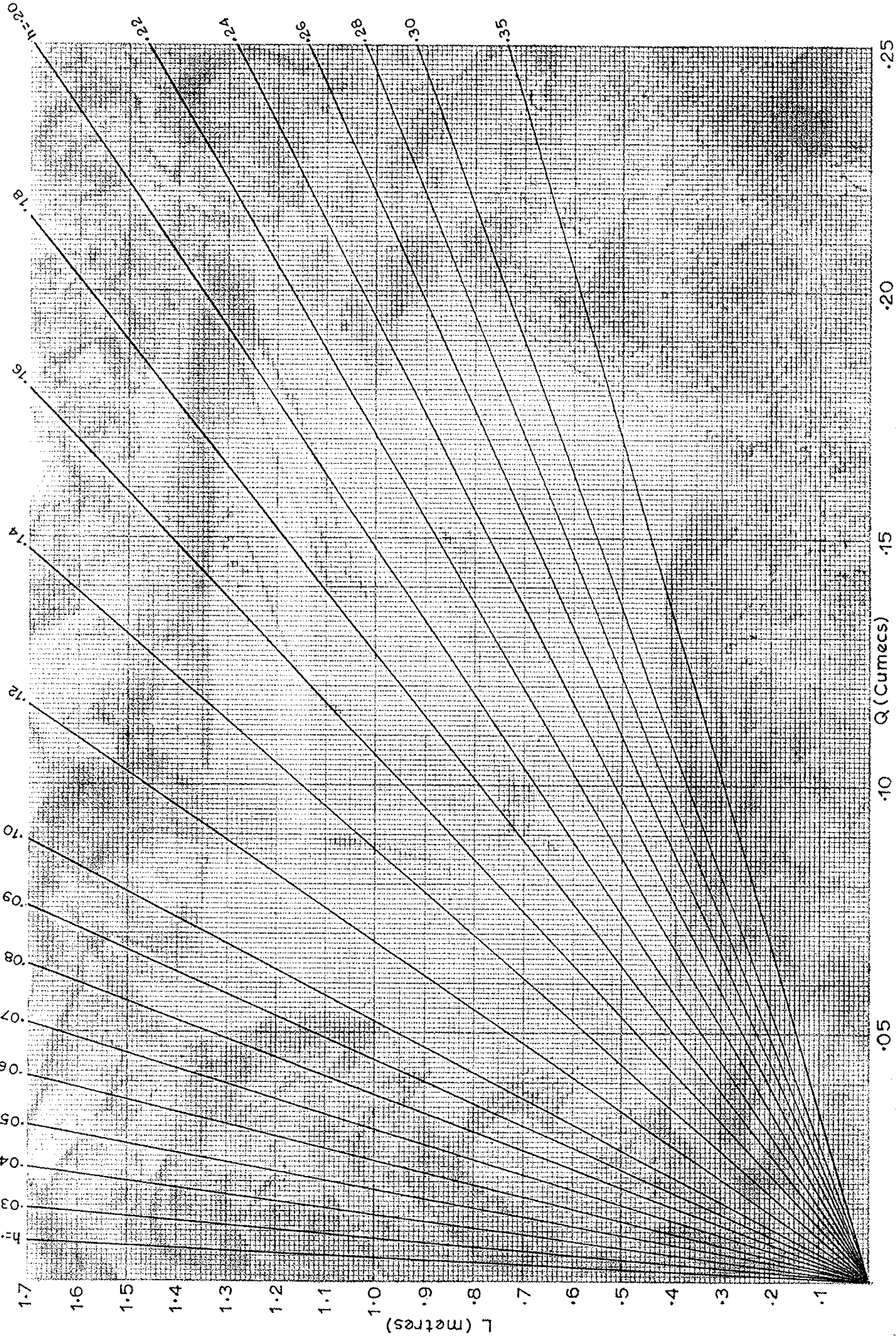


CHART 13. KERB INLET FLOW AT A SUMP

$Q_k = 1{\cdot}66\,h^{3/2}\,L$



Ministry of Works and Development

Office H/O RD/DIVN | Job No. | File 62/36/15 | Sheet No. 1 of 14

Computed G.J.O / 4/1976 | Description EXAMPLE CALCS | PROJECT HWY SURFACE DRAINAGE DESIGN GUIDE

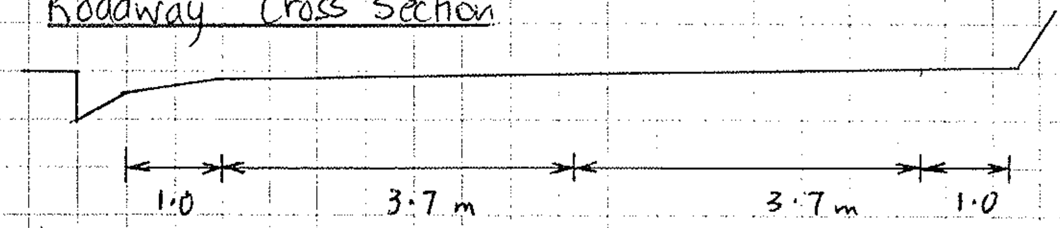
Checked B.C.T. / 9/1977

EXAMPLE 1

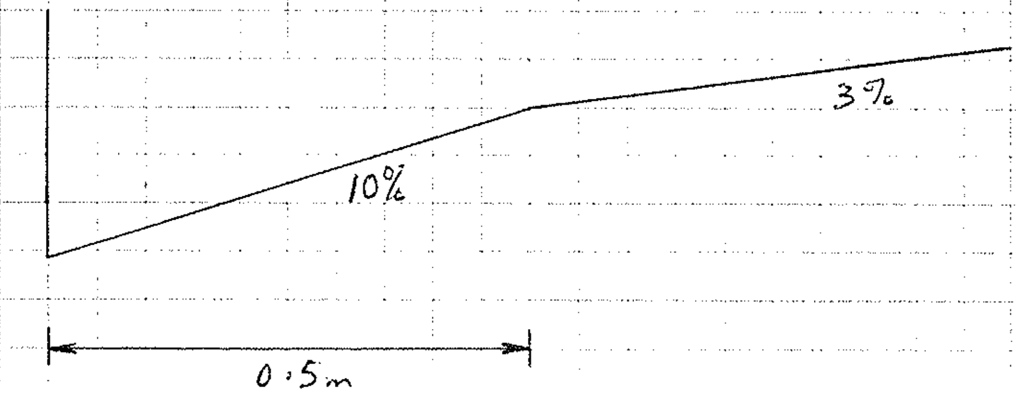
This example illustrates the increased inlet spacing where flow is permitted beyond the gutter.

Consider a two lane highway as below!

Roadway Cross Section



Gutter Cross Section



Longitudinal slope = 1%
Mannings n gutter = 0·013
pavement = 0·016

Consider two cases in the Auckland area:

(a) gutter just full
(b) encroachment of 1m on roadway

Rainfall Determination:

This will be the same for a and b

Length of Flow Path:

$$l_f = W\left(1 + \left(\frac{S_\ell}{S_c}\right)^2\right)^{\frac{1}{2}}$$

$$= 9.4\left(1 + \left(\frac{0.01}{0.03}\right)^2\right)^{\frac{1}{2}}$$

$$= 9.91\ m$$

Flow path slope:

$$S_f = \left(S_\ell^2 + S_c^2\right)^{\frac{1}{2}}$$

$$= 0.0316$$

Time of Surface Flow:

T_s = 120 seconds (Chart 7)

but since T_s is less than 5 minutes assume time of concentration

T = 5 minutes

Rainfall from maps:

(a) 2 year 2 hour = 36 mm
(b) 20 year 2 hour = 76 mm
(c) 2 year ½ hour = 19 mm
(d) 20 year ½ hour = 42 mm

5 year 5 minute rainfall = 11.5 mm
Intensity = 138 mm/hr

2 year 5 minute rainfall = 7.5 mm
Intensity = 90 mm/hr

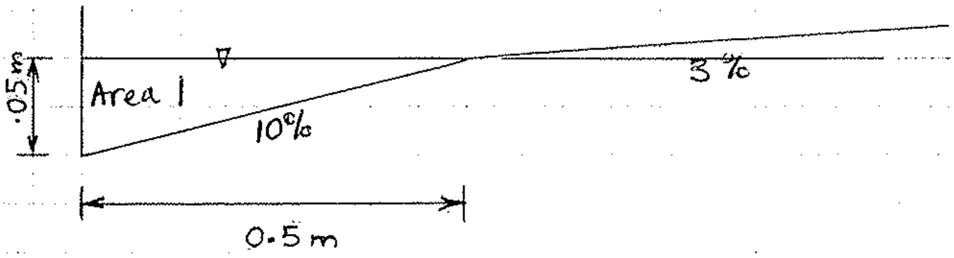
Surface Depth:

$$l_f I = 9.91 \times 90$$
$$= 892$$
$$d = 2.75\text{mm} \quad \text{(chart 8)}$$

since d is less than 4mm hence O.K.

CASE a - gutter just full

Gutter capacity



$$n = 0.013$$
$$S_c = 0.10$$
$$nS_c = 1.3 \times 10^{-3}$$
$$d = 0.05$$
$$S_l = 0.01$$
$$Q_1 = 0.0096 \text{ cumecs} \quad \text{(chart 9)}$$

Assume an inlet grate 1m long by 0.5m wide. All flow is directly above the grate.

$$v_1 = \frac{Q_1}{A_1}$$
$$= .0096 / \tfrac{1}{2} \times .5 \times .05$$
$$= 0.768 \text{ m/s}$$

$L_1 = 0.22$m (chart 10)
Since this is less than the grate length, all flow will be accepted by the grate

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$$Q_o = Q_i$$

Spacing is given by:

$$L = 3.6 \times 10^6 \; Q/CIW$$
$$= .0096 \times 3.6 \times 10^6 / .9 \times 138 \times 9.9$$
$$L = 28.1 \text{ m}$$

Check the assumption that time of concentration is 5 minutes

$$T_G = 1.25 \frac{L}{v_i}$$

$$= 1.25 \times \frac{28.1}{.768}$$

$$= 46 \text{ seconds}$$

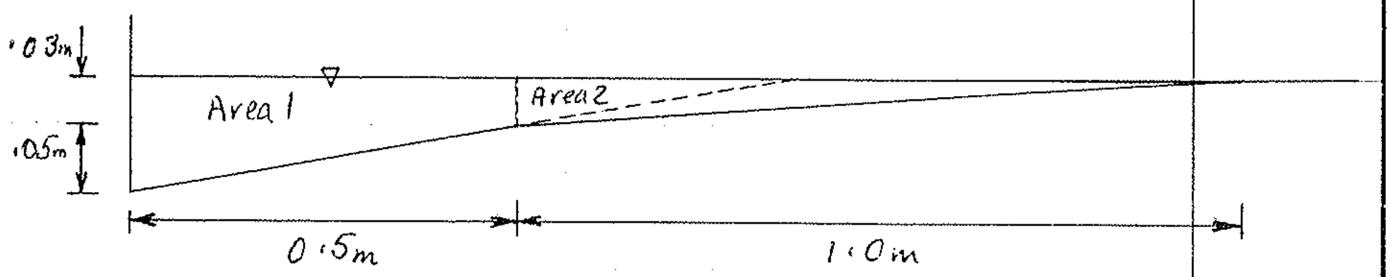
Total time of concentration

$$= 130 + 46$$
$$= 176 \text{ seconds}$$

since this value is less than 5 minutes hence O.K.

Inlet spacing for case A is 28.1 m

CASE b - 1m encroachment



Area 1

$$n S_c = 1.3 \times 10^{-3}$$
$$Q_1 = .0345 - .0026$$
$$= .0319 \text{ cumecs (chart 9)}$$

$$A_1 = \cdot 5 \times (\cdot 08 \times \cdot 8) - \cdot 5\,(\cdot 03 \times \cdot 3)$$
$$A_1 = \cdot 0275\ m^2$$

$$v_1 = Q_1 / A_1$$
$$= 1 \cdot 16\ m/s$$

<u>Area 2</u>

$$n S_c = 0 \cdot 016 \times \cdot 03$$
$$= 4 \cdot 8 \times 10^{-4}$$

$$Q_2 = 0 \cdot 0069 \text{ cumecs (chart 9)}$$

$$A_2 = \cdot 5 \times \cdot 03 \times 1$$
$$= \cdot 015\ m^2$$

$$v_2 : 0 \cdot 460\ m/s$$
$$L_1 = 0 \cdot 43\ m \text{ (chart 10)}$$

This is less than the grate length hence all of the flow directly above the grate is accepted by the grate.

$$Y_3 = 6 \cdot 81\ (L_G S_c / v_2)^2$$
$$= 6 \cdot 81\ (1 \times \cdot 03 / \cdot 460)^2$$
$$= 0 \cdot 0289\ m$$

hence $W_3 = 0 \cdot 963\ m$

$$A_3 = (\cdot 03 - \cdot 0289/2) \times \cdot 963$$
$$= 0 \cdot 0150$$

$$Q_3 = v_2 A_3$$
$$= 0 \cdot 460 \times \cdot 0150$$
$$= \cdot 0069$$

To the nearest 0.0001 cumecs there is no bypass,

hence $Q = 0.0319 + 0.0069$
$= 0.0388$ cumecs

Spacing is given by:

$$L = 3.6 \times 10^6 \, Q / CIW$$
$$= .0388 \times 3.6 \times 10^3 / .9 \times 138 \times 9.9$$
$$= 113.6$$

Check assumption that time of concentration is five minutes

$$T_G = 1.25 \frac{L}{v_1}$$
$$= 1.25 \times 113.6 / 1.16$$
$$= 122 \text{ seconds}$$

$$T = T_G + T_S$$ T_S from chart 7
$$= 122 + 120$$
$$= 242 \text{ seconds}$$

since the value is less than 5 minutes it is OK to use 5 minutes

The inlet spacing is increased from about 28 m to 114 m by allowing an encroachment of 1m on the roadway.

EXAMPLE 2

This example compares a grate inlet and deflector inlet in the same gutter.
The roadway and rainfall is the same as Example 1 but the longitudinal slope is 6%.

$$S_l = 0{\cdot}06$$
$$S_f = (.03^2 + .06^2)^{\frac{1}{2}}$$
$$= .0671$$

$$l_f = 9{\cdot}4 \left(\frac{.0671}{.03}\right)$$
$$= 21{\cdot}0\text{m}$$

T_s = 155 seconds from chart 7

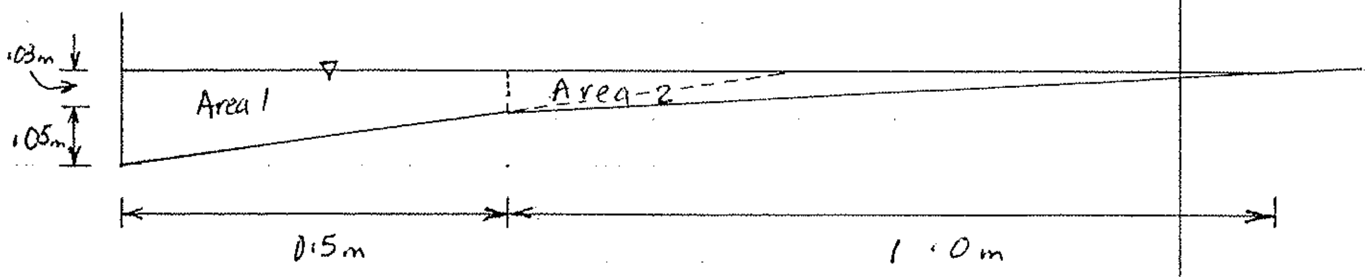
Assume 5 minute time of concentration

Surface Depth: $l_f\ I = 21{\cdot}0 \times 90$
$= 1890$

$d = 3{\cdot}5\text{mm}$

since the value is less than 4 mm it is acceptable.

Case (a) - Grate Inlet



Area 1

$$n\,S_c = 1{\cdot}3 \times 10^{-3}$$
$$Q_1 = .072 - .006$$
$$= .066 \text{ cumecs}$$

$$A_1 = .0275\ m^2$$
$$v_1 = \frac{.066}{.0275}$$
$$= 2.40\ m/s$$

Area 2

$$n\,S_c = 4.8 \times 10^{-4}$$
$$Q_2 = .016 \text{ cumecs}$$
$$A_2 = .015\ m^2$$
$$v_2 = \frac{Q_2}{A_2}$$
$$= 1.07\ m/s$$

$$Y_3 = 6.81 \left(\frac{L_G S_c}{v_2}\right)^2$$
$$= 0.00535\ m$$

hence $W_3 = 0.178m$

$$A_3 = .178\left(.03 - \left(\frac{.00535}{2}\right)\right)$$
$$= .00486$$

$$Q_3 = v_2 A_3$$
$$= 0.005 \text{ cumecs}$$

$$L_1 = 0.869\ m$$
(less than 1m so O.K)

Total flow Q_0 = 0.082 cumecs
Total inlet flow Q = 0.071 cumecs
Bypass flow = 0.011 cumecs

Spacing is given by

$$L = \frac{3.6 \times 10^6 Q}{C I W}$$

$$= \frac{3.6 \times 10^6 \times .071}{.9 \times 138 \times 9.9}$$

$$= 207.9 \text{ m}$$

say $L = 208$ m

Check time of concentration

$$T_G = 1.25 \frac{L}{V_1}$$

$$= \frac{1.25 \times 207.9}{2.40}$$

$$= 108 \text{ seconds}$$

$$T = T_S + T_G$$

$$= 155 + 108$$

$$= 263 \text{ seconds}$$

Since the value is less than 5 minutes, use 5 minutes as time of concentration

CASE (b)

Assume that a deflector inlet is used which is applicable to the equation used in chart II.

$$LW = 1 \times .5$$

$$= 0.5 \text{ m}^2$$

$$\frac{S^{\frac{1}{2}}}{n} = \frac{.06}{.013}$$

$$= 18.8$$

$$y_1 = 0.08 \text{ m}$$

$$Q = 0.056 \text{ cumecs} \quad \text{(chart II)}$$

Spacing is given by

$$L = \frac{3.6 \times 10^6 \times .056}{.9 \times 138 \times 9.9}$$
$$= 163.9$$

say $L = 164$ m

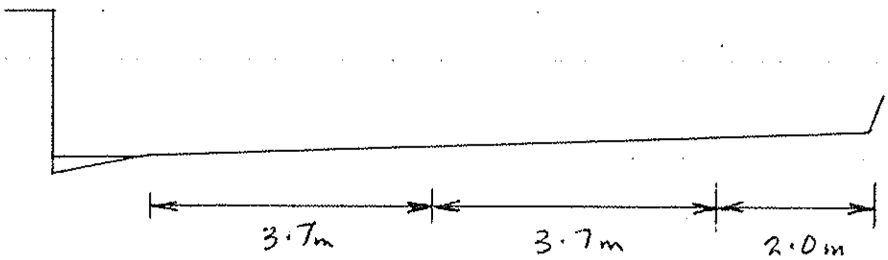
All other factors being equal, a grate inlet would be preferable as spacing required is 208 m compared with 164 m

EXAMPLE 3: Drainage System

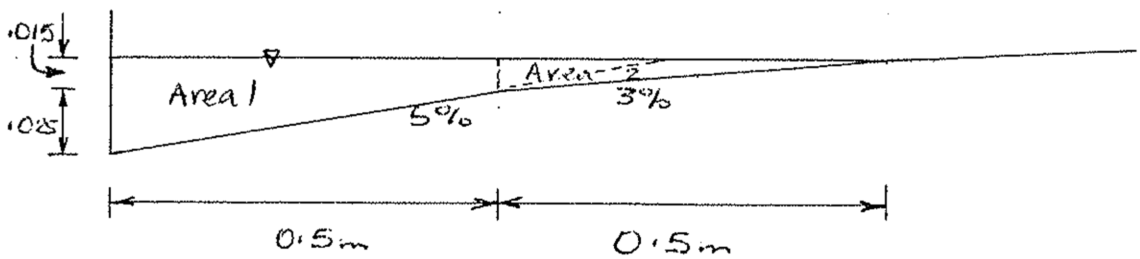
This example illustrates inlet spacing and underground pipe network in a sag

Consider a two lane highway as below:

Roadway Cross-section



Gutter Cross-section



Longitudinal slope is a transition from +0.5% to -3% to +2% as on the diagram

Rainfall as in example 1

Surface Depth

This will be worst at the end of the largest flow path which is on the constant 3% slope

$$S_f = (S_\ell^2 + S_c^2)^2$$
$$= 0.0424$$

$$l_f = W \left(\frac{S_f}{S_c}\right)$$
$$= 9.4 \times \left(\frac{.0424}{.03}\right)$$
$$= 13.3\,m$$

$$l_f I = 13.3 \times 90$$
$$= 1197$$
$$d = 3.0\,mm \quad \text{(chart 8)}$$

since the value is less than 4 mm it is acceptable and $T_s = 130$ seconds (chart 7)
therefore assume 5 minutes to be the time of concentration

Gutter capacity

Calculate roadway profile and plot gutter capacities as shown using chart 9 where:

Area 1.

$$n S_c = .013 \times .05$$
$$= 6.5 \times 10^{-4}$$

Area 2

$$n S_c = .016 \times .03$$
$$= 4.8 \times 10^{-4}$$

Assume use of a grate inlet 0.5m by 0.5m

Constant 3% slope will give the greatest bypass!

Q_1 = ·0176 cumecs
Q_2 = ·00185 cumecs
Q_c = ·0195 cumecs

$$A_1 = (\cdot 5 \times \cdot 025 \times \tfrac{1}{2}) + (\cdot 015 \times \cdot 5)$$
$$= 0\cdot 01375 \, m^2$$

$$v_1 = \frac{Q_1}{A_1}$$
$$= 1\cdot 28 \, m/s$$
$$L_1 = 0\cdot 33 \, m$$

This value is less than the grate length of 0·5m so there is no bypass directly above the grate.

$$A_2 = \tfrac{1}{2} \times \cdot 015 \times \cdot 5$$
$$= \cdot 00375 \, m^2$$

$$v_2 = \frac{Q_2}{A_2}$$
$$= 0\cdot 493 \, m/s$$

$$Y_3 = 6\cdot 81 \left(\frac{L_e S_c}{v_2}\right)^2$$
$$= 0\cdot 00638 \, m$$

$$W_3 = 0\cdot 212 \, m$$

$$A_3 = \cdot 212 \left(\cdot 015 - \left(\frac{\cdot 00638}{2}\right)\right)$$
$$= 0\cdot 0025$$

$$Q_3 = 0\cdot 493 \times 0\cdot 0025$$
$$= 0\cdot 0012 \text{ cumecs}$$

Inlet flow $Q = .0176 + .0012$
$= .0188$ cumecs

Bypass $= .00185 - .0012$
$= .0007$ cumecs

Find the limiting longitudinal slope where all flow is captured by the inlet. This will be the slope for which

$$Y_3 = .0015$$

We have
$$Y_3 = 6.81 \left(\frac{L_G S_c}{v_2}\right)^2$$

$$Y_3 = 6.81 \left(\frac{L_G S_c A_2}{Q_2}\right)^2$$

$$Q_2 = \sqrt{\frac{6.81}{Y_3}} \; L_G S_c A_2$$

$$= \sqrt{\frac{6.81}{.015}} \times .5 \times .03 \times .00375$$

$$= .0012 \text{ cumecs}$$

hence $S_L = 2.5\%$ chart 9
All flow is captured on slopes less than 2.5%

Surface runoff to each inlet is given by
$$Q = \frac{CIWL}{3.6 \times 10^6}$$

in which $C = 0.9$
$W = 10.4$ m
$I = 138$ mm/hr

hence $Q = 0.000359\,L$

Plot lines of $Q = 0.000359L$ to determine inlet positions as shown

From the diagram, the first intersection of runoff and capacity curves is at 16m from the summit. An inlet is placed here which captures all the flow. Runoff is again plotted from zero at this inlet and the next inlet is located 29m further on. This process is continued to the sump.

Where slope is greater than 2.5% there will be some bypass flow which must be calculated.

Sump design flow

$$Q_s = Q_{s1} + Q_{s2} + Q_{max}$$
$$= .0035 + .0045 + .0188$$
$$= 0.0268 \text{ cumecs}$$

Try a combination kerb and grate sump assuming that half the grate area is open.

$$A = .5 \times .5 \times .5$$
$$= 0.125 m^2$$

$$h = 0.015 + (.5 \times .025)$$
$$= 0.0275 m$$

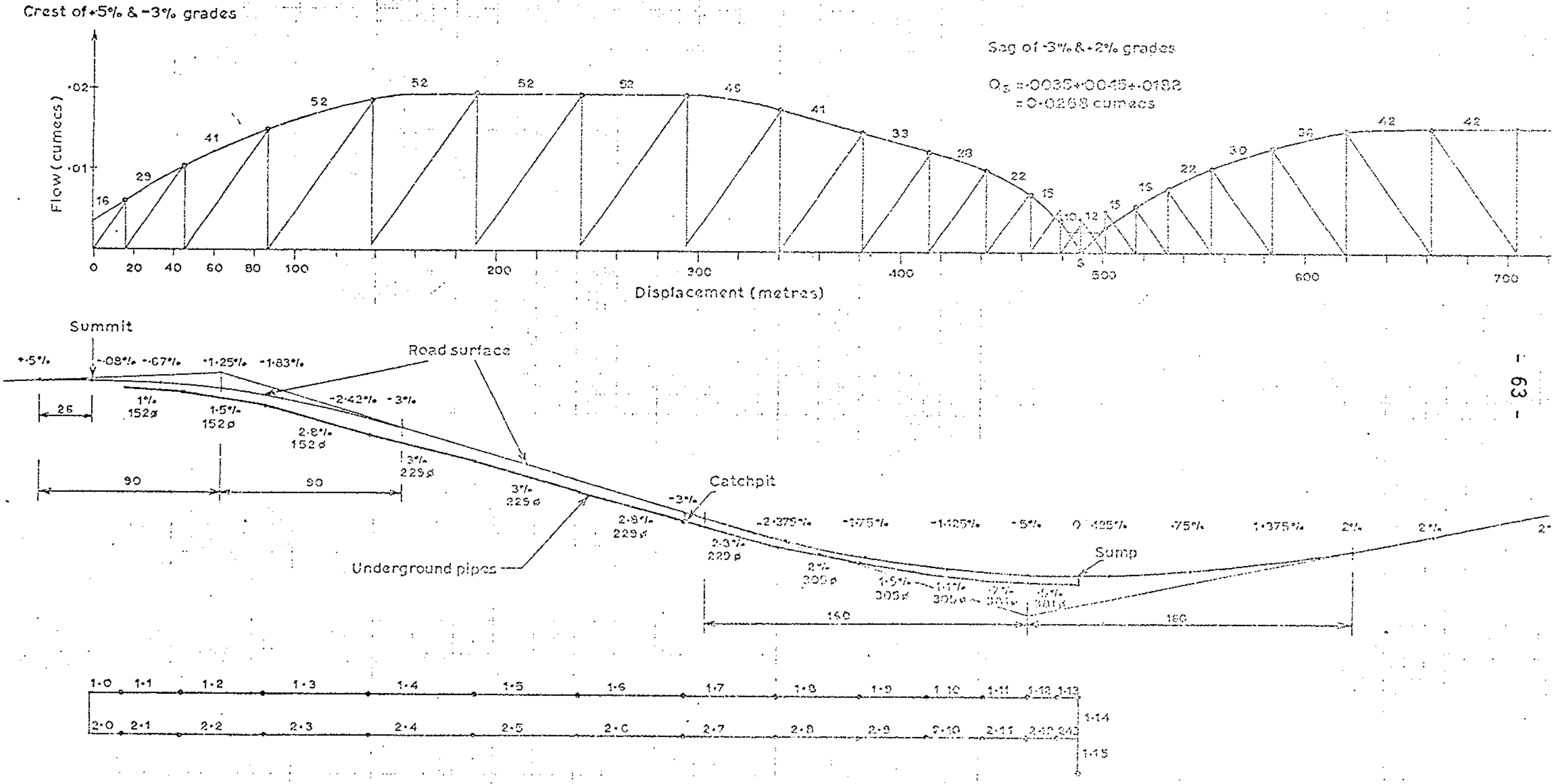
Grate capacity Q_G = 0.055 cumecs (fig 16)
Kerb inlet Q_K = 0.0066 cumecs (fig 17)

$$Q_S = Q_G + Q_K$$
$$= 0.0616 \text{ cumecs}$$

Since this value is greater than 0.0268 cumecs it is OK.

Time of concentration is much less than 5 minutes for all sections hence initial assumption of 5 minutes is ok for all sections.

DRAINAGE SYSTEM (Example 3)



EXAMPLE 3

1 LENGTH NUMBER	2 SLOPE %	3 DISTANCE m	4 PIPE DIAM	5 VELOCITY	6 TIME	7 TIME OF CONC	8 RAINFALL INTENSITY	9 INLET FLOW	10 TOTAL FLOW AT POINT	11 PIPE CAPACITY	12 DROP Inlet/Outlet	13 COMMENTS
1.0		16	mm		182		138 mm/hr	m³/hr				
1.1	1.0	29	152	1.01	29	182	138	.0060	.0060	.0185		Smallest OK
1.2	1.5	41	152	1.24	33	211	138	.0105	.0165	.0227	.013	OK
1.3	2.8	52	152	1.71	30	244	138	.0146	.0211	.0311	.020	OK
1.4	3.0	52	152	1.77	29	274	138	.0188	.0399	.0322	.037	Too Small
			229	2.30	23					.0942	.040	OK
1.5	3.0	52	229	2.30	23	297	138	.0188	.0587	.0942	.068	OK
1.6	2.8	52	229	2.22	23	320	138	.0188	.0775	.0910	.068	OK
1.7	2.8	46	229	2.22	21	343	127	.0173	.0886	.0910	.063	OK
1.8	2.0	41	305	2.25	18	364	127	.0160	.1046	.164	.063	OK
1.9	1.5	33	305	1.95	17	382	127	.0140	.1186	.142	.065	OK
1.10	1.0	28	305	1.59	18	399	115	.0108	.1172	.116	.049	Increase Slope
	1.1			1.66	17	417	115			.121	.049	OK
1.11	0.7	22	381	1.53	14	416	115	.0080	.1252	.174	.035	OK
1.12	0.5	15	381	1.29	12	430	115	.0065	.1317	.147	.030	OK
1.13	0.5	10	381	1.29	8	442	115	.0033	.1350	.147	.021	OK
1.14	2.0	21	381	2.59	8	450	115	.0223	.257	.295	.042	OK
1.15	2.0	30	457	2.91		458	107		.478	.477	.131	Near enough

NOTE: Length 1.14 takes an additional discharge of 0.100 cumecs from the other side of the hill.
Length 1.15, the outlet pipe, takes flow from both gutters on each side of the hill,
ie $0.257 \times \frac{107}{115} \times 2 = 0.478$ cumecs

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