CALLIOPE SHIRE COUNCIL

STORMWATER DRAINAGE DESIGN STANDARD

October, 1989.

CALLIOPE SHIRE COUNCIL DESIGN STANDARDS

* Road Design Standard (Second Edition) 1989
* Stormwater Drainage Design Standard October, 1989
* Water Supply Design Standard
* Sewerage Design Standard

Calliope Shire Council
Don Cameron Drive
Calliope
Queensland, 4680

Phone: (079) 757133 Fax: (079) 757106

CONTENTS

PART I - RURAL CATCHMENTS

PART I

RURAL CATCHMENTS

SECTION 1 - INTRODUCTION

1.1 This standard has been prepared for use in conjunction with Calliope Shire Council's Road Design Standard for the guidance of Consulting Engineers engaged in the preparation of engineering plans for roadworks within the Shire, which incorporate stormwater drainage.

It is anticipated that by clearly setting out Council's design criteria for such works, time and effort will be saved on the part of both designers and Council staff, by reducing the necessity for amendment of submitted designs.

1.2 This standard is intended to be a general guide, and it is appreciated that there may be circumstances where a departure from the recommendations herein can be justified.

It is not intended that issue of this standard should inhibit the submission by Consultants of designs, based on alternative methods, professional experience and established engineering practice. In such situations, alternative solutions will always be considered by Council on their merits.

1.3 The distribution of this standard does not imply limitation in any way of Council's rights to impose differing standards, nor limitation of the Shire Engineer's discretion to interpret engineering requirements in respect of a particular situation, having regard to good engineering practice.

1.4 The differences in design approach to Rural and Urban situations have been recognised by dividing this standard into two (2) parts which represent these two traditional areas as follows:-

PART I - Rural Catchments (ungauged $<$500 hectares)
PART II - Urban Catchments ($<$ 20 hectares)

1.5 Full calculations of all drainage design shall be submitted for checking together with a Catchment Plan, showing the total catchment and sub catchment areas used in the calculations.

-o0o-

SECTION 2 - DESIGN FLOWS - RURAL CATCHMENTS

2.1 GENERAL

The following is based on the Rational Method and may be used for unguaged rural catchments having a catchment area less than 500 hectares.

For catchment areas greater than 500 hectares, reference should be made to "Australian Rainfall and Runoff" (1987).

2.2 RATIONAL FORMULA

Peak discharge flows are calculated by the formula:-

$$Q = 0.0028\ C\ I_{t,y}\ A$$

where
- Q = peak discharge flow (m^3/sec)
- C = runoff co-efficient
- $I_{t,y}$ = average rainfall intensity (mm/hr)
- A = catchment area (ha)

2.3 RUNOFF CO-EFFICIENT (C)

The runoff co-efficient shall be $C = 0.7$ for all times of concentration (t_c), average recurrence intervals (A.R.I.) and rainfall intensities.

2.4 TIME OF CONCENTRATION (t_c)

The time of concentration (t_c) is derived from the mainstream length (L), (measured from the most remote point in the catchment to the inlet of the hydraulic structure) and the slope (S) of the mainstream length.

(a) Preferred Method

The preferred method of calculating the time of concentration is given by the equation:-

$$t_c = \frac{9.1 \quad L}{A^{0.1} \; S_e^{0.2}}$$

where t_c = time of concentration (mins)
L = mainstream length (km)
A = Catchment Area (h_a)
S_e = equal area slope of the mainstream runoff length (m/km) determined as follows

Equal Area Slope (S_e) is derived for variable mainstream slopes such that a line of slope (S_e) gives equal areas as per Fig.#2.1.

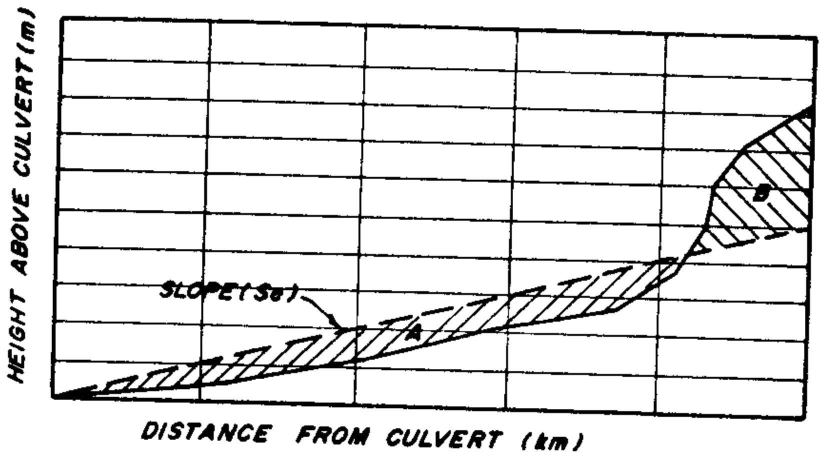


FIG. 2·1

(b) Alternative Method

Preliminary values for times of concentration may be estimated by the equation:-

$$T_c = 16.7 \ L/V$$

where t_c = time of concentration (mins)
L = mainstream length (km)
V = approx. velocity of stream (m/sec) as determined from the following Table.

SECTION 2.4 (Cont.)

Type of Country	Average Slope of Catchment Surface (%)	Approx. Velocity of Stream (m/s)
Flat	0 to 1.5	0.3
Rolling	1.5 to 4	0.7
Hilly	4 to 8	0.9
Steep	8 to 15	1.5
Very Steep) Rocky) Mountainous)	15	3.0

Stream Velocity - related to Mainstream Slope

TABLE 2.1

2.5 AVERAGE RECURRENCE INTERVALS (ARI)

Average Recurrence Intervals for the various road classifications shall be in accordance with the following table:-

Road Classification	Design A.R.I. (derives Qd)	Check A.R.I. (derives Qc)
(a) Major roads crossing major creeks (usually requiring a bridge)	1 in 50	1 in 100
(b) All roads except the following	1 in 20	1 in 50
(c) Rural roads serving less than 20 lots	1 in 10 (Subject to approval of Shire Engineer)	1 in 50
(d) Rural access roads serving less than 10 lots	1 in 5 (Subject to approval of Shire Engineer)	1 in 50
(e) Roads crossing major creeks	Causeway = 1 in 2, or 1 in 1, (subject to approval by Shire Engineer)	1 in 50

Average Recurrence Intervals (A.R.I.)

TABLE 2.2

2.6 AVERAGE RAINFALL INTENSITY ($I_{t,y}$)

The average rainfall intensity is determined for the appropriate average recurrence intervals (A.R.I.) from TABLE 2.2 using the intensity-frequency-duration (I.F.D.) curves Chart 2.1 and the time of concentration (t_c).

The curves in Chart 2.1 are derived by the Bureau of Meteorology from the formula and list of co-efficients given in Table 2.3.

-o0o-

PREPARED BY THE BUREAU OF METEOROLOGY FOR THE TANNUM/BOYNE AREA

ln(I) = a + b*(ln(T)) + c*(ln(T))**2 + d*(ln(T))**3 + e*(ln(T))**4 + f*(ln(T))**5 + g*(ln(T))**6

I = Intensity in millemetres per hour

T = Time in hours

Return Period (Years)	a	b	c	d	e	f	g
1	3.5654	-0.5999	-0.0568	0.00783	0.002912	-0.0002680	-0.0000715
2	3.8215	-0.5938	-0.0518	0.00763	0.002576	-0.0002240	-0.0000698
5	4.0690	-0.5758	-0.0391	0.00629	0.001779	-0.0000270	-0.0000778
10	4.1954	-0.5668	-0.0322	0.00591	0.001330	0.0000454	-0.0000773
20	4.3444	-0.5586	-0.0261	0.00524	0.000953	0.0001441	-0.0000826
50	4.5155	-0.5491	-0.0191	0.00451	0.000503	0.0002518	-0.0000864
100	4.6328	-0.5425	-0.0146	0.00396	0.000248	0.0003333	-0.0000925

RAINFALL INTENSITY IN MM/HR FOR VARIOUS DURATIONS AND RETURN PERIODS

DURATION (HOURS)	RETURN PERIOD 1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
0.083	110.	143.	184.	209.	243.	289.	326.
0.100	103.	134.	173.	197.	229.	273.	307.
0.167	85.2	110.	142.	161.	188.	224.	252.
0.333	63.4	81.9	104.	118.	137.	162.	182.
0.500	52.0	67.1	85.4	96.6	112.	132.	148.
1.000	35.4	45.7	58.5	66.4	77.0	91.4	103.
2.000	22.8	29.6	38.6	44.2	51.8	62.0	70.2
3.000	17.3	22.6	30.0	34.6	40.8	49.2	56.0
6.000	10.8	14.2	19.4	22.7	27.1	33.2	38.2
12.000	6.77	9.06	12.7	15.1	18.3	22.8	26.4
24.000	4.36	5.90	8.49	10.3	12.6	15.9	18.7
48.000	2.79	3.81	5.66	6.96	8.63	11.1	13.2
72.000	2.07	2.84	4.32	5.38	6.73	8.76	10.5

TABLE 2.3

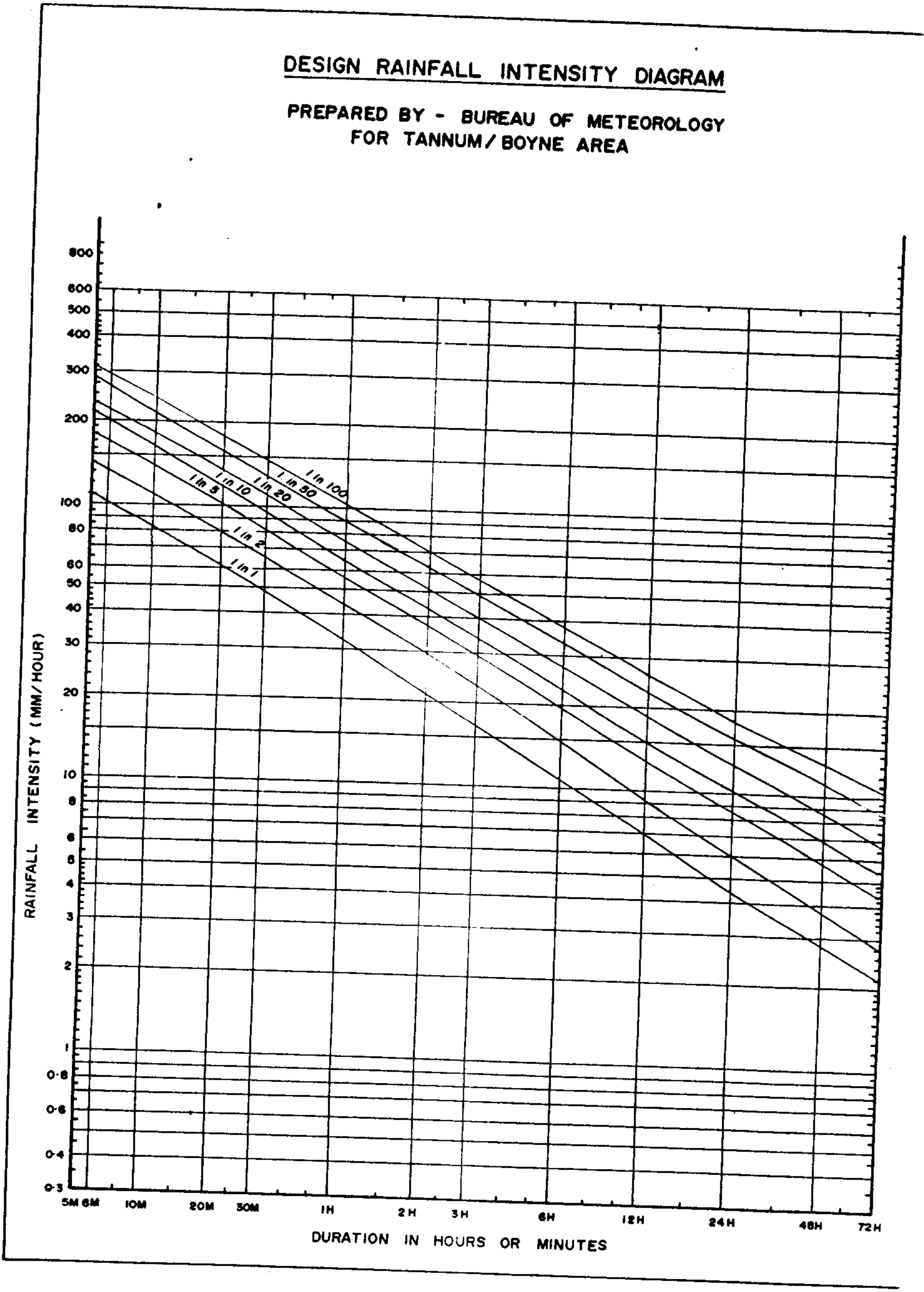

DESIGN RAINFALL INTENSITY DIAGRAM
PREPARED BY - BUREAU OF METEOROLOGY
FOR TANNUM/BOYNE AREA
RAINFALL INTENSITY (MM/HOUR)
800
600
500
400
300
200
100
80
60
50
40
30
20
10
8
6
5
4
3
2
1
0·8
0·6
0·4
0·3
1 in 100
1 in 50
1 in 20
1 in 10
1 in 5
1 in 2
1 in 1
5M 6M
10M
20M
30M
1H
2H
3H
6H
12H
24H
48H
72H
DURATION IN HOURS OR MINUTES

Chart 2.1

SECTION 3 - CULVERT DESIGN

3.1 GENERAL

Culvert design should be based on the inlet-outlet method as the preferred procudeure, which is detailed in this section.

Location and Slope of Culvert - in rural areas, the culvert should be located to fit the natural channel in line and grade, following moderate curvature and breaks in grade as far as practical.

3.2 ALLOWABLE HEADWATER

Culverts shall be designed to accommodate Design ARI (Q_D) flows (see Table 2.2) with a maximum headwater (HW) level 150mm below the shoulder level, and ponding should be contained within the banks of the natural water course.

Headwater levels for ARI $\leqslant$ 50 years (Q50) flows shall not cause damage to adjacent property and this may be achieved by either reducing the fill heights to allow overtopping or increasing the culvert capacity.

3.3 OVERTOPPING PROTECTION

Generally, embankment protection would not be required for culverts designed for ARI $\geqslant$ 20 year (Q_{20}) flows, however the Shire Engineer may require downstream protection where overtopping could cause extensive damage.

Where design flows of ARI $<$ 20 years are used, the design shall be in accordance with Section 4 - Floodway Design incorporating a culvert with a minimum design capacity as determined by the Shire Engineer.

3.4 OVERTOPPING FLOW DEPTHS

To allow a vehicle to pass through a length of shallow water, the vertical alignment (grade line) shall be such that the maximum overtopping depth is 200mm with the following flows:-

Culvert Design Flow (Q_D)	Design Flow for 200mm Maximum Overtopping Depth
Q_{20}	Q_{50}
$< Q_{20}$	Q_{20}

TABLE 3.1

3.5 INLET-OUTLET DESIGN METHOD

By assuming firstly inlet control and then outlet control, the headwater depth (HW) for each case is calculated and the higher value which denotes the type of control governing the flow is adopted.

(a) Inlet Control

For the design of most conventional culverts with inlet control, the headwater depth (HW) is obtained from Charts #3.1 and #3.4.

Design charts for other types of culverts should be obtained from the manufacturer.

(b) Outlet Control

For outlet control, the headwater depth (HW) is computed for all outlet conditions including all tailwater depths from the formula:-

$$HW = H + h_o - L\,S_o$$

where (H) and (h_o) are determined as follows:-

and L = length of the culvert (m)

S_o = Slope of the culvert barrel (m/m)

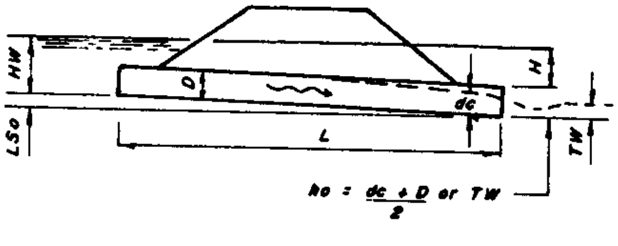


FIG. 3.1

SECTION 3.5 (Cont.)

(i) **Calculating (H)**

The head (H) is obtained from Charts #3.3 and #3.6 for the design of most conventional culverts.

Design charts for other types of culverts should be obtained from the manufacturer.

(ii) **Calculating** (h_o)

* When the tailwater depth is above the obvert of the culvert:-

  e.g. If $TW \geqslant D$ then $h_o = TW$

* When the tailwater depth is below the obvert of the culvert:-

$$\left.\begin{array}{ll} & h_o = TW \\ \text{or} & h_o = \dfrac{(d_c + D)}{2} \end{array}\right\} \text{whichever is the greater}$$

where d_c = critical depth from Charts #3.2 and #3.5
D = culvert diameter or box culvert height

(iii) **Calculating (TW)**

The tailwater depth may be calculated in accordance with Section #5.5

It should be realised that in some instances, the tailwater depth (TW) need not be calculated. The headwater depth (HW) for inlet control is usually calculated first. Substituting this HW depth in the outlet control equation -

$$h_o = HW \text{ (inlet control)} + LS_0 - H$$

(h) and (LS_o) can be calculated assuming full flow initially, therefore a (h_o) value can determine at which outlet control will govern. Experience or otherwise will indicate whether this (h_o) value is feasible - if obviously not, then the control will automatically be at the inlet and the (TW) depth need not be determined. When scouring at the outlet may be a problem, however, some indication of the (TW) level is required.

3.6 OUTLET PROTECTION LENGTH

(a) Culvert Outlet Velocity

(i) Inlet Control

The outlet velocity (V_1) may be assumed equal to the normal velocity in open channel flow in the culvert barrel as computed by Manning's formula for the rate of flow, barrel size, slope and roughness. This can be readily determined from Charts #3.7 to #3.17. Tailwater is not considered effective in reducing outlet velocities for most inlet control conditions.

(ii) Outlet Control

The outlet velocity (V_1) in this case is obtained by dividing the design flow (Q_D) by the cross-sectional area of flow at the outlet.

* If the tailwater submerges the outlet, the cross-sectional area of flow corresponds to the full area of pipe or box culvert.

* If the tailwater is below the obvert of the pipe, the cross-sectional area of flow is that which corresponds to the tailwater depth or critical depth, whichever is the greater area of flow.

(b) Flow Transition Angle

The transition angle (Θ) for the flow boundaries is given by the equation:-

SECTION 3.6 (Cont.)

$$\tan \Theta = \frac{\sqrt{gd}}{3v}$$

where v = Average of the flow velocities at the culvert outlet (V_1) and in the channel (V_2) = Maximum Permissible Channel Velocity - refer Section #6.2

$$v = \frac{(V_1 + V_2)}{2} \text{ m/sec.}$$

d = Average of the flow depths at the culvert outlet (d_1) and in the channel (d_2) = tailwater depth

$$d = \frac{(d_1 + d_2)}{2}$$

$$g = 9.8\text{m/sec}^2$$

Chart #3.19 is a nomograph from which the tangent or the flare angle in degrees may be ontained from flow velocity (v) and depth (d).

(d) Length of Protection

Length of protection can be calculated from the transition angle (Θ) above and the geometry of the culvert outlet:-

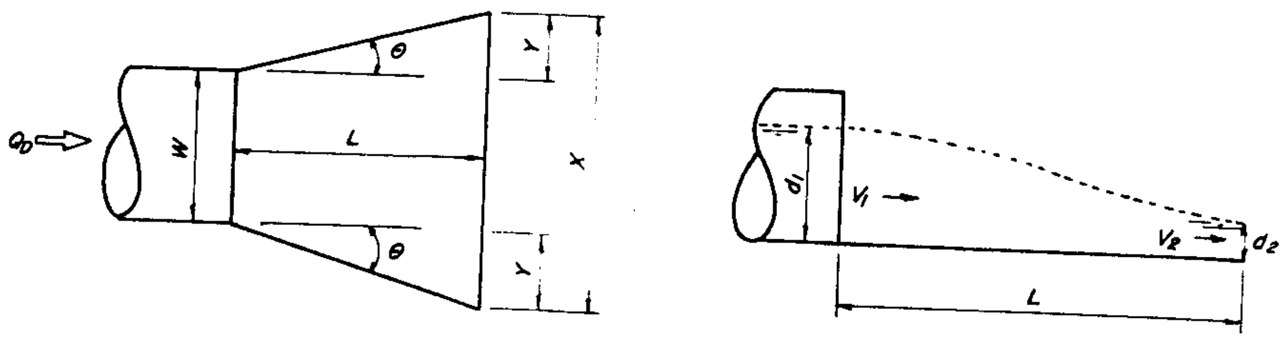


FIG. 3.2

$$L = \left(\frac{Q_D}{V_2 d_2}\right) - W \quad 2 \text{ Tan } \Theta$$

where L = Length of protection works (m)
Q = Design flow (m^3/sec)
V_2 = Max. permissible flow velocity in channel (m/sec)
d_2 = Depth of flow in the channel (tailwater depth) (m)
W = Width of the culvert outlet (m)
Θ = Flow transition angle (degrees)

Irrespective of the flow transition angle (Θ), 30° wing walls shall be used as standard unless the culvert is skewed. The minimum lenght of concrete apron protection shall be 1.5D where D is the culvert pipe diameter or height of box culverts.

(d) Protection Type

Where the length of protection works is greater than the minimum length of 1.5D, gabion protection in accordance with Section #6.6 and S.W.D.S.#1 shall be used.

3.7 EXAMPLE (see page 16)

Given

* Design Flow $Q_{D(20)}$ = 6.3m^3/sec.
* Maximum Allowance Velocity in Channel = 1.8m/sec.
* Tailwater Depth = 0.930m (Example - Section #5.5)
* Slope of Culvert = 0.5%
* Fill Height = 1.5m (height of channel bank in this case)
* Length of Culvert = 9.76m

Solution

Maximum Allowable H.W. Height = 1.5 - 0.15 = 1.350m

Maximum Culvert Size = 1.5 - 0.3(say) = 1.200m

Trial Culvert Size

assume flow through culvert = 2.0m/sec.

therefore $\frac{6.3m^3/sec}{2.0m/sec} = 3.15m^2$

Trial Number of Cells

$$\frac{3.15}{\pi\left(\frac{1.2}{2}\right)^2} = 2.79 \text{ say } 3 \text{ cells}$$

Flow per Cell $= \frac{6.3m^3/sec}{3} = 2.1m^3/sec$

SECTION 3.7 (Cont.)

Inlet Control

Chart #3.1 for D = 1.200m and Q = 2.1m^3/sec

$\frac{HW}{D} = 1.01$

$HW_{(in)} = 1.01 \times 1.2 = \underline{1.21m}$

Outlet Control $HW = H + h_o - hS_o$

(i) Find (H)

From Chart #3.3 for D = 1.2m and Q = 2.1m/sec

$\underline{H = 0.3m}$

(ii) h_o = T.W. = $\underline{0.930m}$
or $h_o = \frac{d_c + D}{2}$
From Chart #3.2 → d_c = 0.800m
therefore $h_o = \frac{0.800 + 1.200}{2} = \underline{1.000m}$

) h_o = 1.000m

(iii) Find LS_o = 9.76 x 0.5% = $\underline{0.05m}$
therefore $HW_{(out)}$ = 0.30 + 1.00 - 0.05 = $\underline{1.250m}$
Control 1.250 > 1.210 therefore Outlet Control
Maximum H.W. = 1.350 therefore O.K

Culvert Outlet Velocity and Flow Depth

TW = 0.930) therefore Outlet Velocity calculated using
d_c = 0.800) flow depth = 0.930m
therefore d_1 = T.W. = $\underline{0.930m}$

$d_1/D = \frac{0.930}{1.200} = 0.78$

Proportioned Area of Flow from Chart #3.18

For $\frac{d_1}{D} = 0.78$ therefore P = 0.84

therefore Outlet Velocity $V_1 = \frac{Q}{PA} = \frac{2.1}{0.84 \times \pi \left(\frac{1.2}{2}\right)^2} = 2.21$m/sec

Channel Velocity and Depth

d_2 = T.W. = $\underline{0.930m}$

V_2 = Maximum Allowance = $\underline{1.8m/sec}$

Outlet Protection

average depth of flow $d = \frac{d_1+d_2}{2} = \frac{0.930+0.930}{2} = 0.930m$

average velocity of flow $V = \frac{V_1+V_2}{2} = \frac{2.21+1.8}{2} = 2.01m/sec$

Flare Angle $= \tan\Theta = \frac{\sqrt{gd}}{3V} = \frac{\sqrt{9.8x0.93}}{3 \times 2.01} = \tan\Theta = 0.50$

Protection Length (L) $= \left(\frac{Q_D}{V_2 d_2} - w\right) / 2\tan\Theta$

$w = 3 \times 1.2 + 2 \times 0.6 = 4.8m$

$L = \left(\frac{6.3}{1.8 \times 0.93} - 4.8\right) / 2 \times 0.50$

$L = -1.04m$

The negative number relates to the mathematics and geometry in the derivation of length (L) (and not to actual flow conditions) and is brought about by the minimum spacing 0.600m between pipes. Only positive values of (L) are compared with the minimum length of protection (1.5D).

In this case provide minimum protection length

$1.5D = 1.5 \times 1.2 = 1.80m$

Other design options can be derived for $Q_{(D)}$ as per the example (page 16), and the most appropriate alternative chosen, usually on a cost basis.

Check Flow

The selected culvert is then checked for a Q_{50} flow to ensure upstream ponding is below the Maximum Allowance Headwater (HW).

If overtopping of the embankment takes place at Q_{50}, consideration of possible scour damage may be required.

-o0o-

PROJECT Example

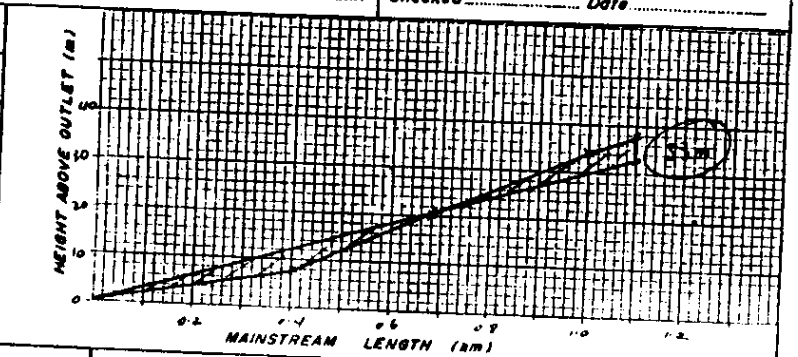
CULVERT DESIGN

Water Course Name ....
Structure Chainage ....

Designed .... Date ....
Checked .... Date ....

DESIGN FLOWS

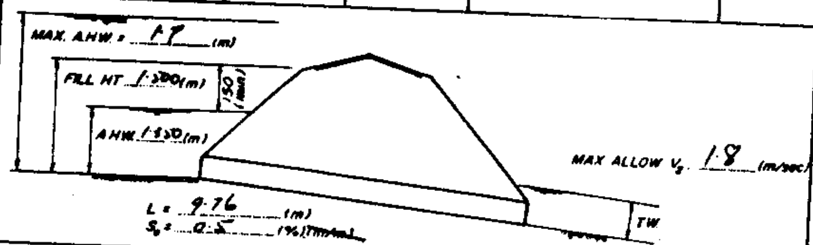
CATCHMENT AREA A	MAINSTREAM SLOPE: CATCHMENT HEAD LEVEL R_1	OUTLET LEVEL R_2	MAINSTREAM LENGTH L	$S_e = \frac{R_1 - R_2}{L}$	TIME OF CONC. $t_c = \frac{91L}{A^{0.1} S_e^{0.2}}$	A.R.I.	INTENSITY $I_{t,y}$	CO-EFF OF RUNOFF C	DESIGN FLOW $Q = 0.0028\, C I_{t,y} A$
ha	m	m	km	m/km	mins	years	mm/hr		m³/sec
32	0	33	1.1	30	36	20	100	0.7	Q(20) = 6.3
✓	✓	✓	✓	✓	✓	50	115	0.7	Q(50) = 7.2
									Q( ) =

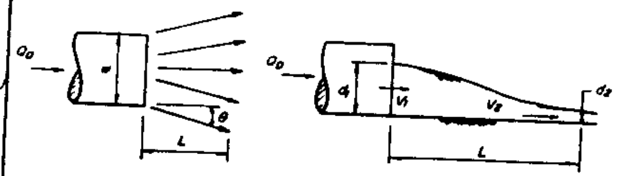


FLOW INFORMATION

$Q_o(20)$ = 6.3 (m³/sec)
TW = 0.910 (m)

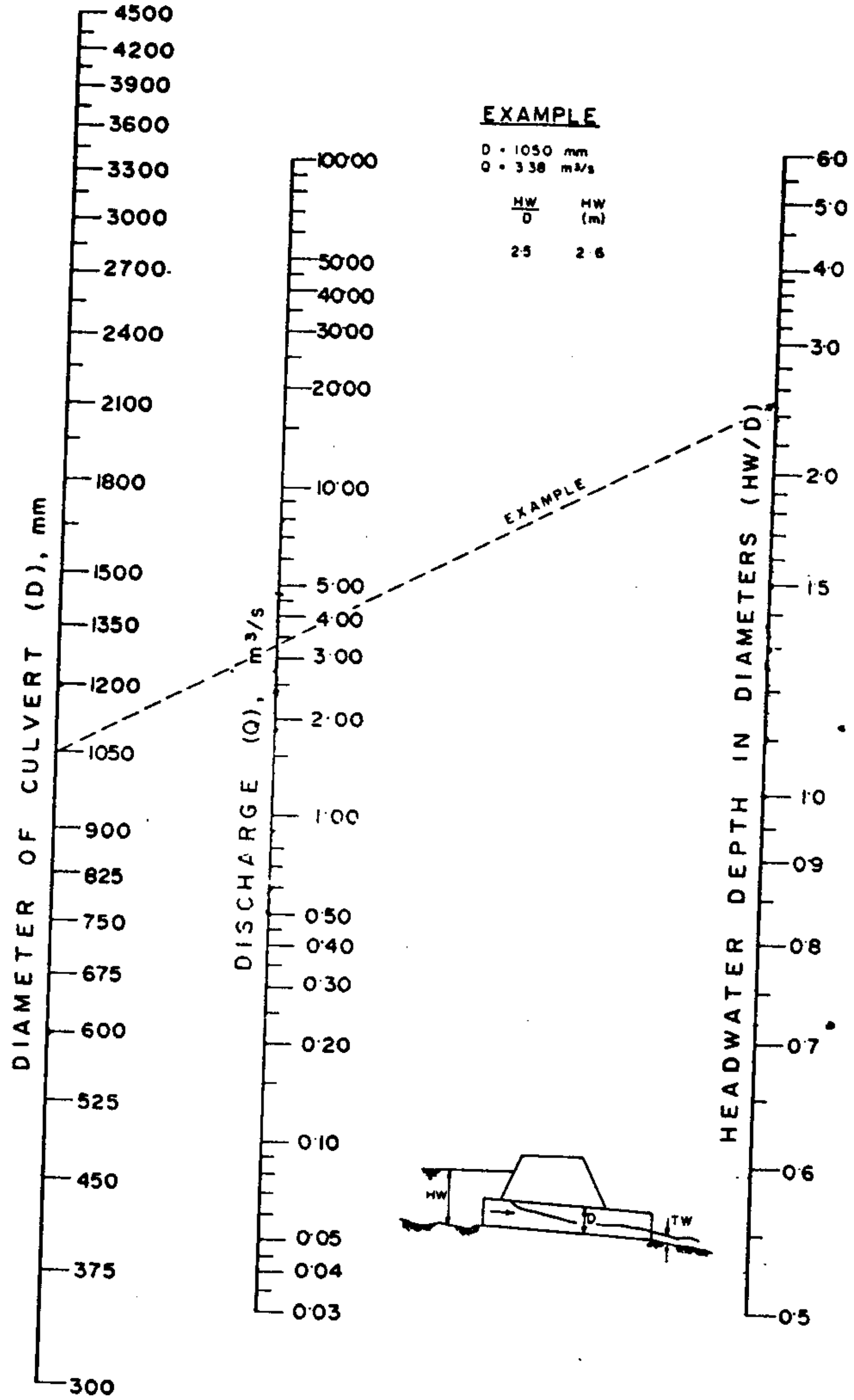
Q_d( ) = 7.2 (m³/sec)
TW = 1.010 (m)





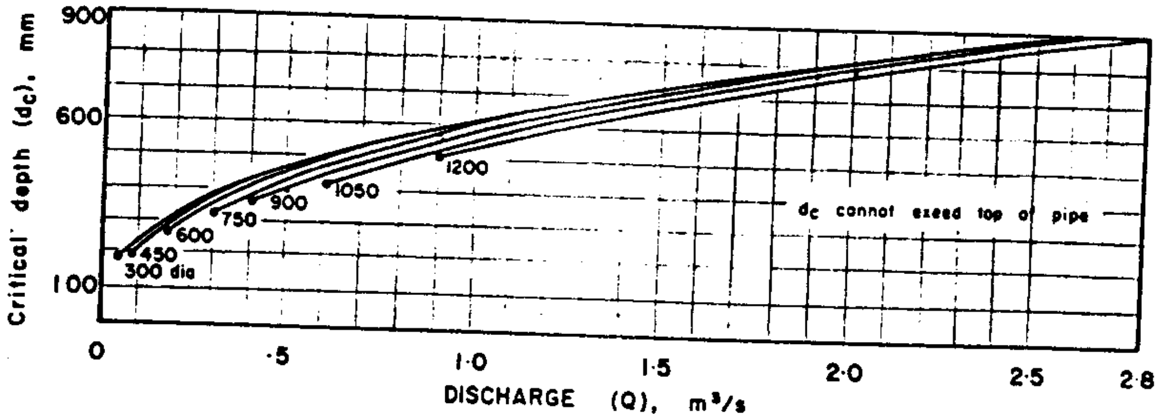
HEADWATER CALCULATION

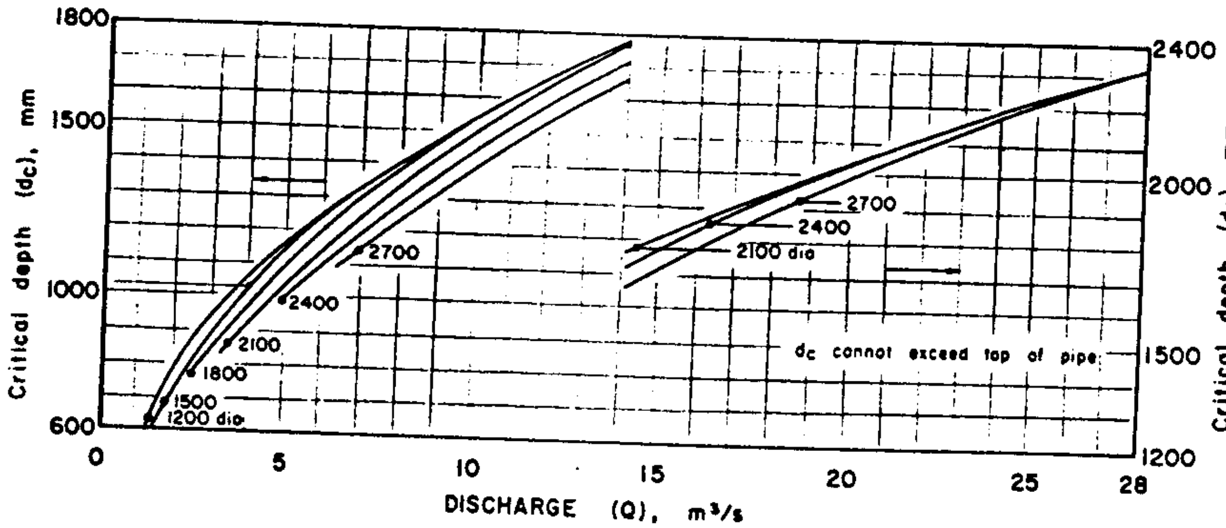
Q	PIPE (D) or B × D	Q/B	A	INLET CONTROL HW/D	INLET CONTROL HW	OUTLET CONTROL $HW = H + h_o - LS_o$: H	TW	d_c	$\frac{d_c + D}{2}$	h_o	LS_o	HW	CONTROLLING HW	CULVERT OUTLET VEL/DEPTH: d_1	d/D	Prop. Area of Flow P	$V_1 = Q/PA$	CHANNEL VEL/DEPTH: d_2	V_2	$d = (d_1 + d_2)/2$	$V = (V_1 + V_2)/2$	FLARE ANGLE $\tan\theta = \frac{\sqrt{g \cdot d}}{3V}$	W	PROTECTION LENGTH $L = \frac{Q_o}{V_2 d_2} - W/2\tan\theta$	STANDARD PROTECTION LENGTH 1.5 D	REQUIRED PROTECTION LENGTH L - 1.5 D	COMMENTS
m³/sec		m³/sec/m	m²		m	m	m	m	m	m	m	m		m			m/sec	m	m/sec	m	m/sec		m	m	m	m	
2.1	(3 φ)1200 (	/	1.13	1.01	1.210	0.50	0.910	0.900	1.000	1.000	0.05	1.210	o/c	TW = 0.910	0.79	0.84	2.21	0.930	1.9	0.930	2.01	0.50	4.8	-1.04			Provide Std. Protection
6.3	2400×1200	2.63	2.88	1.19	1.42																						HW. too high
6.3	2700×1200	2.33	5.24	1.09	1.30	0.28	0.910	0.800	1.000	1.000	0.05	1.230	i/c	0.650			3.6	0.93	1.8	0.79	2.1	0.34	2.7	1.56	1.8	STD	STD. Protection O.K. for both
											Assumed		d_1 =	0.730													
7.2	2700×1200	2.67	3.24	1.20	1.44	0.33	0.93	0.90	1.050	1.050	0.05	1.33	i/c				3.47	0.93	1.8	0.93	2.63	0.38	2.7	1.40	1.8	STD.	
																											O.K. Q50 below Max H.W.
																											Adopt 1×2700×1200

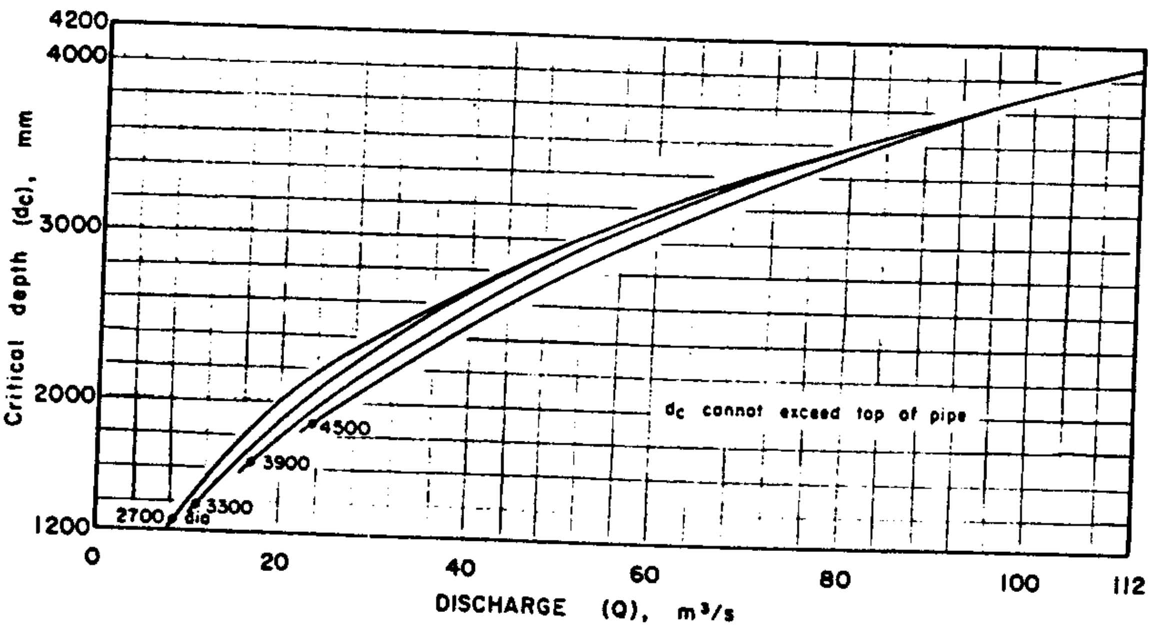


Headwater depth for concrete pipe culverts with inlet control

Chart 3.1

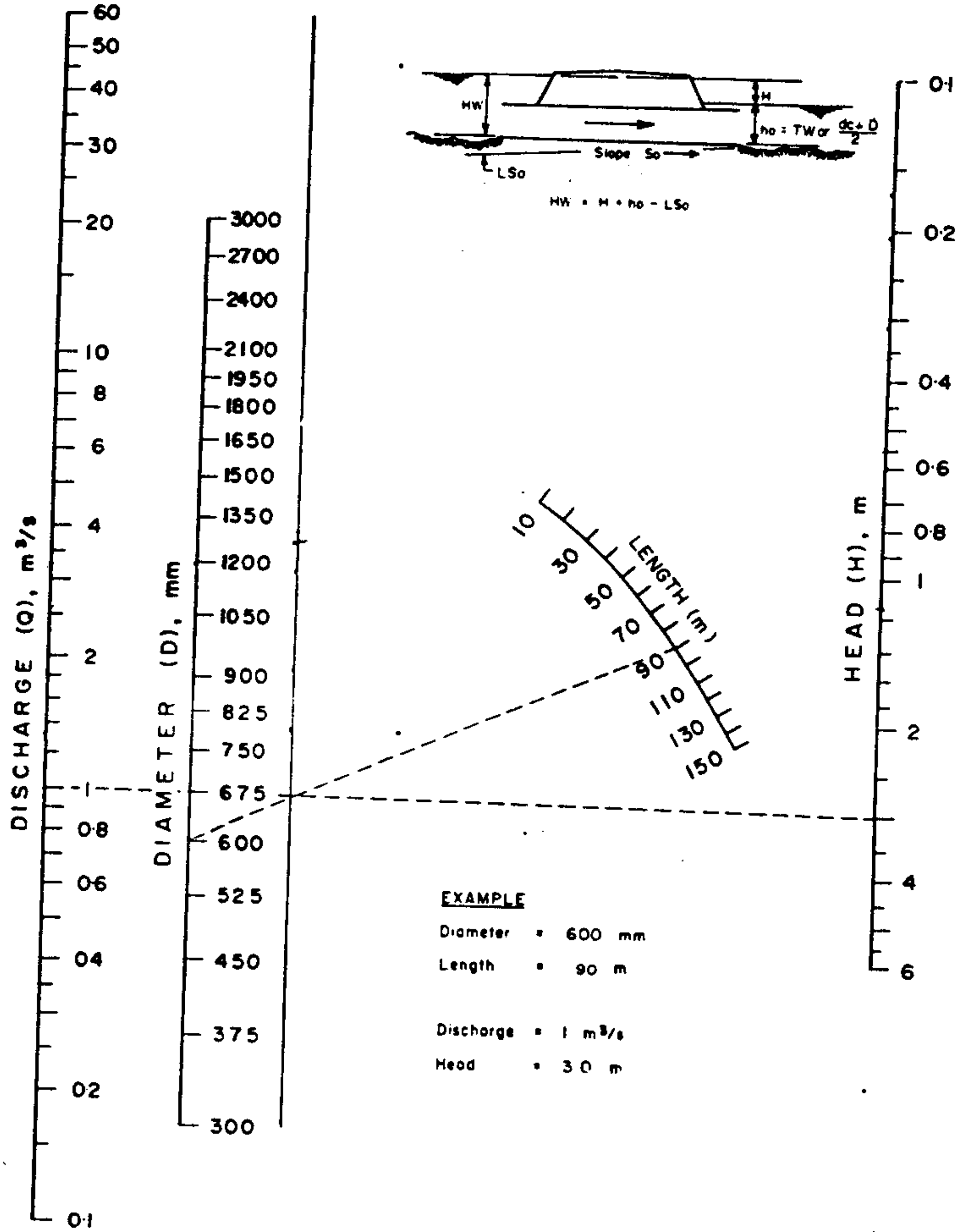






Critical depth d_c circular pipe

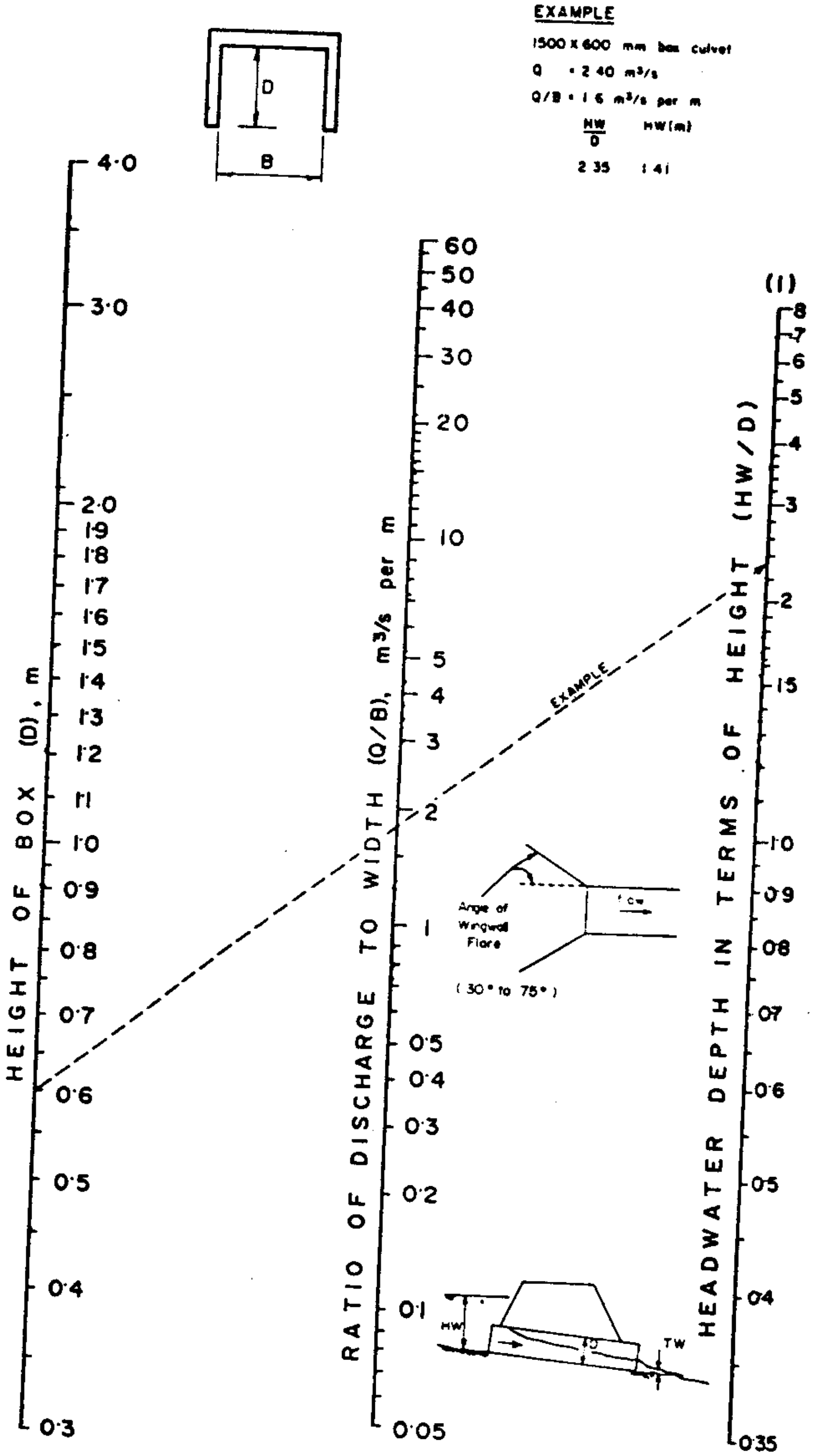
Chart 3.2



Head for concrete pipe culverts flowing full, n=0·012 with outlet control

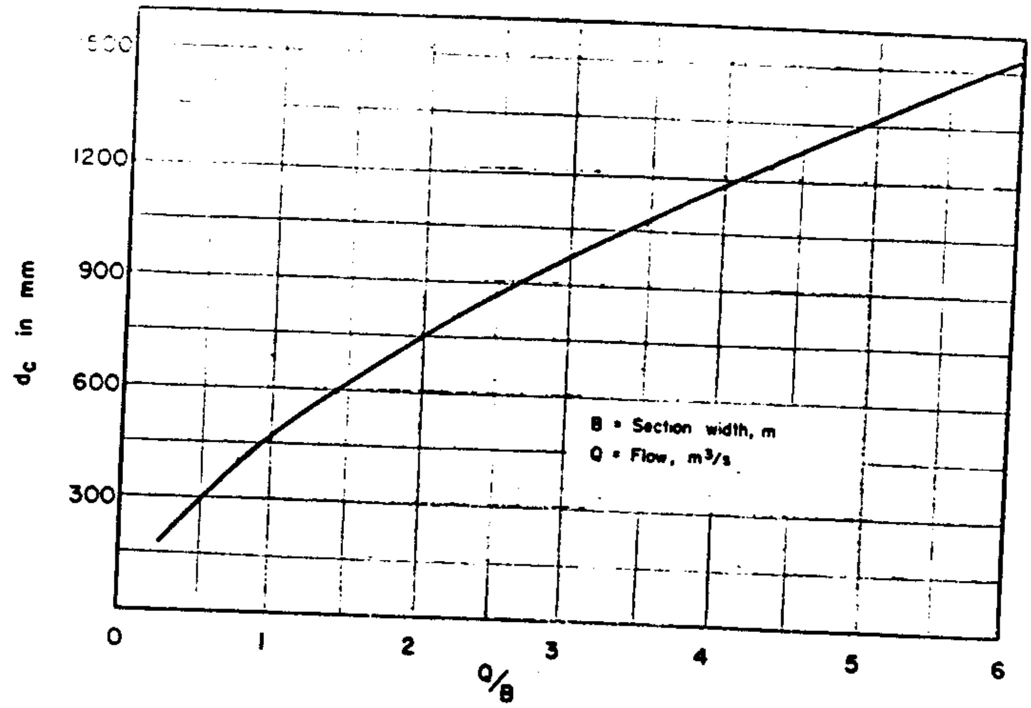
Chart 3.3

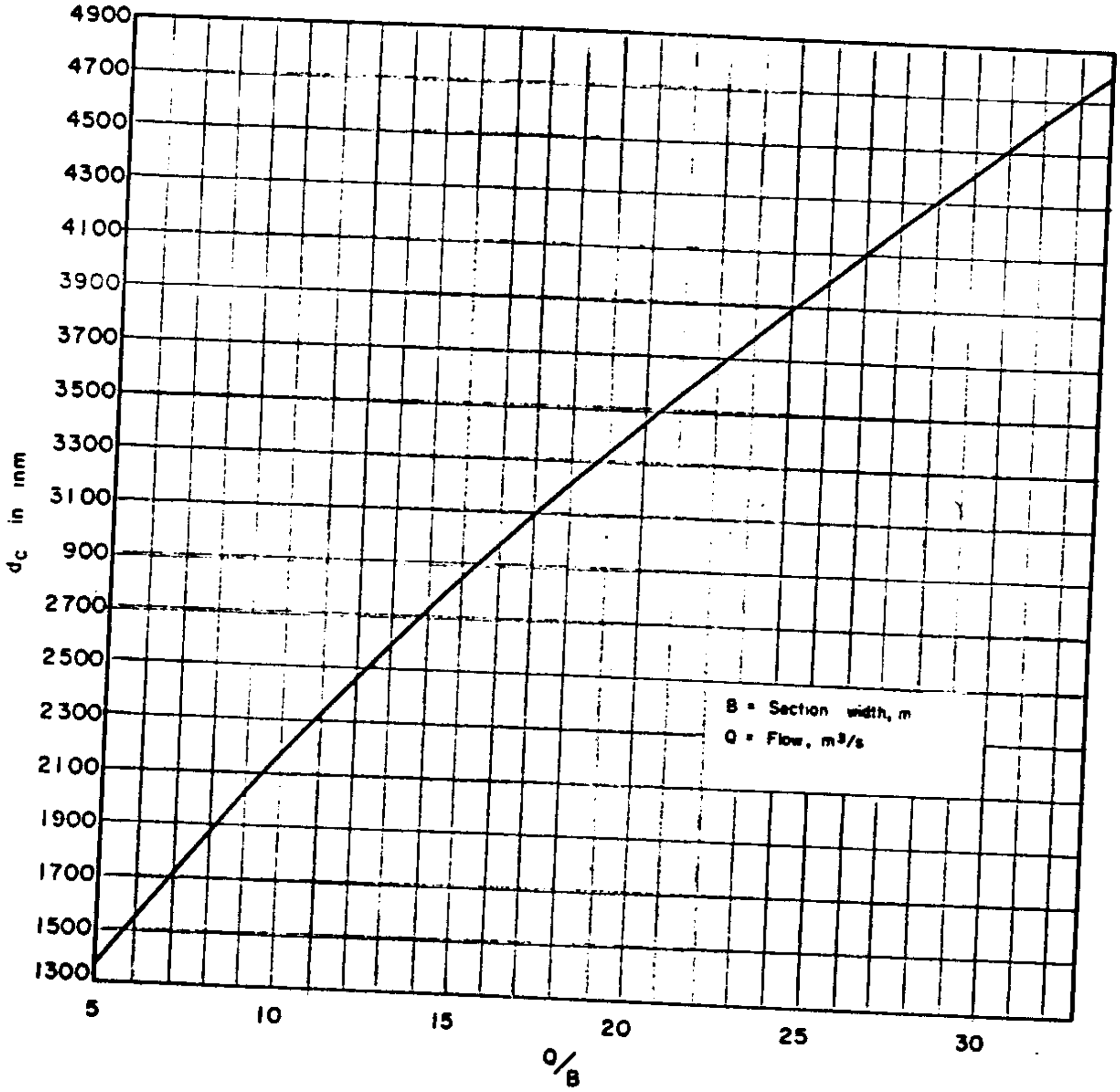
(Blank)



Headwater depth for box culverts with inlet control

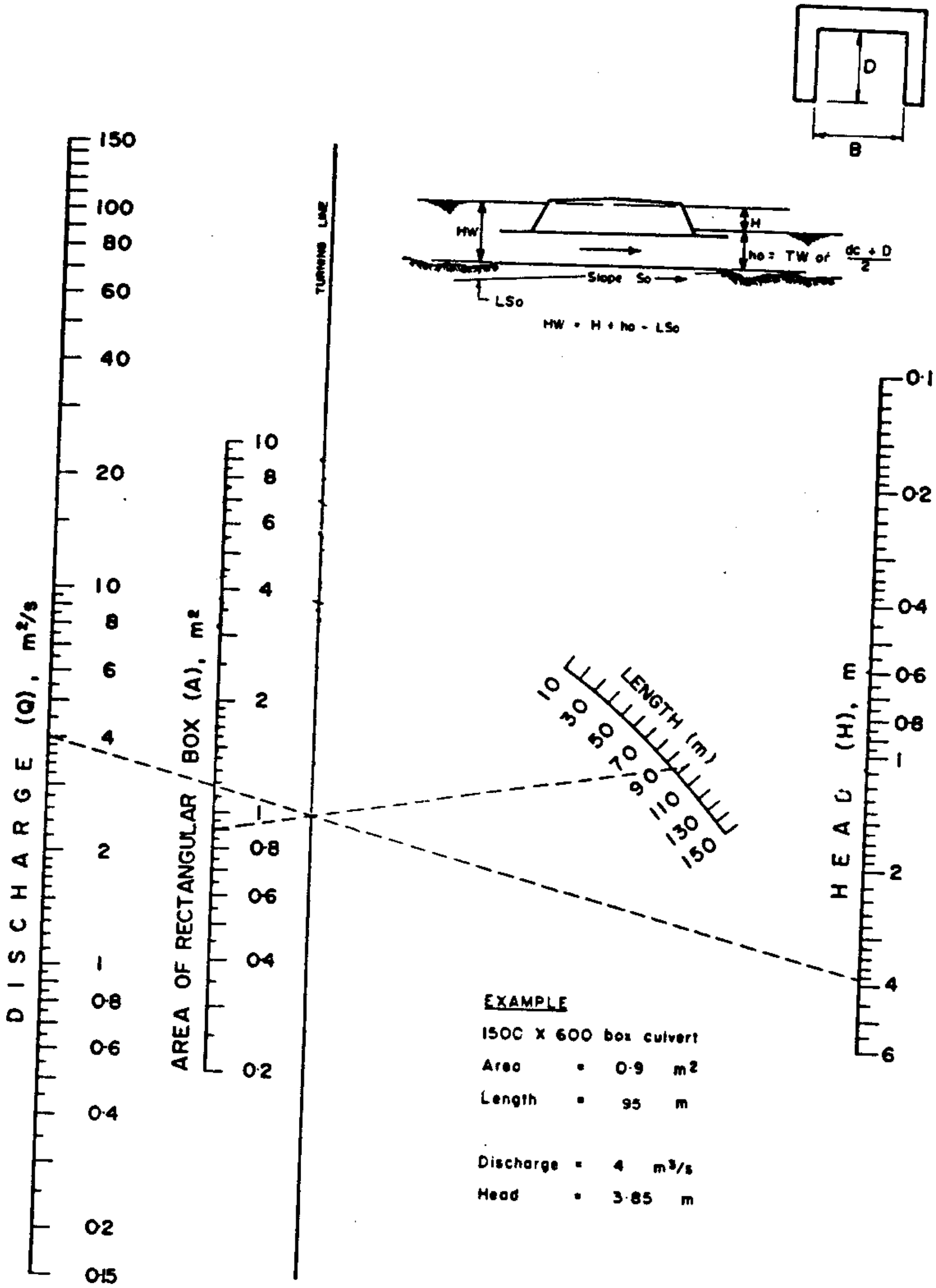
Chart 3.4





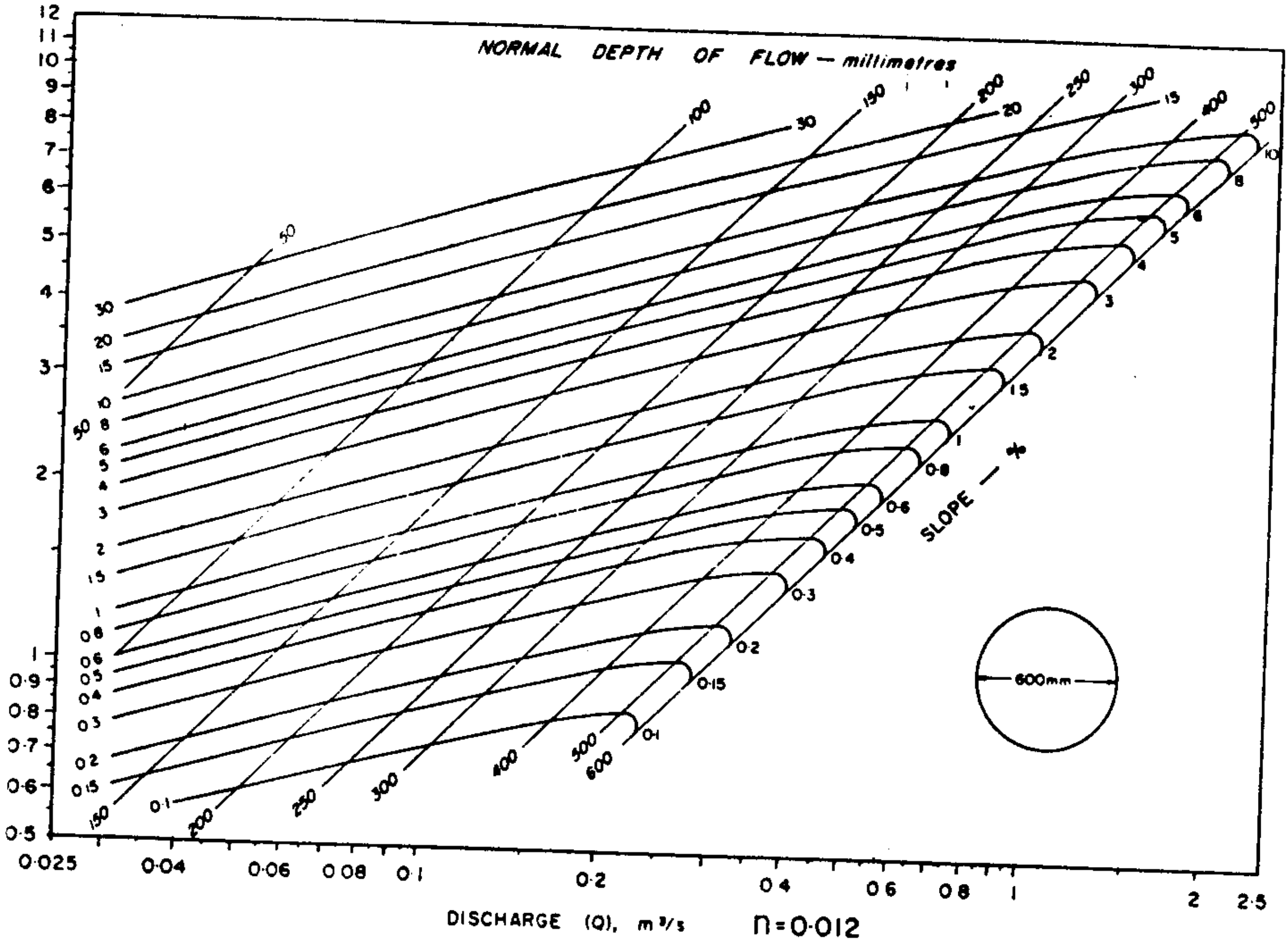
Critical depth d_c rectangular section

Chart 3.5



Head for concrete box culverts flowing full, n = 0·012 with outlet control

Chart 3.6


NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
600mm
DISCHARGE (Q), m³/s
n=0·012

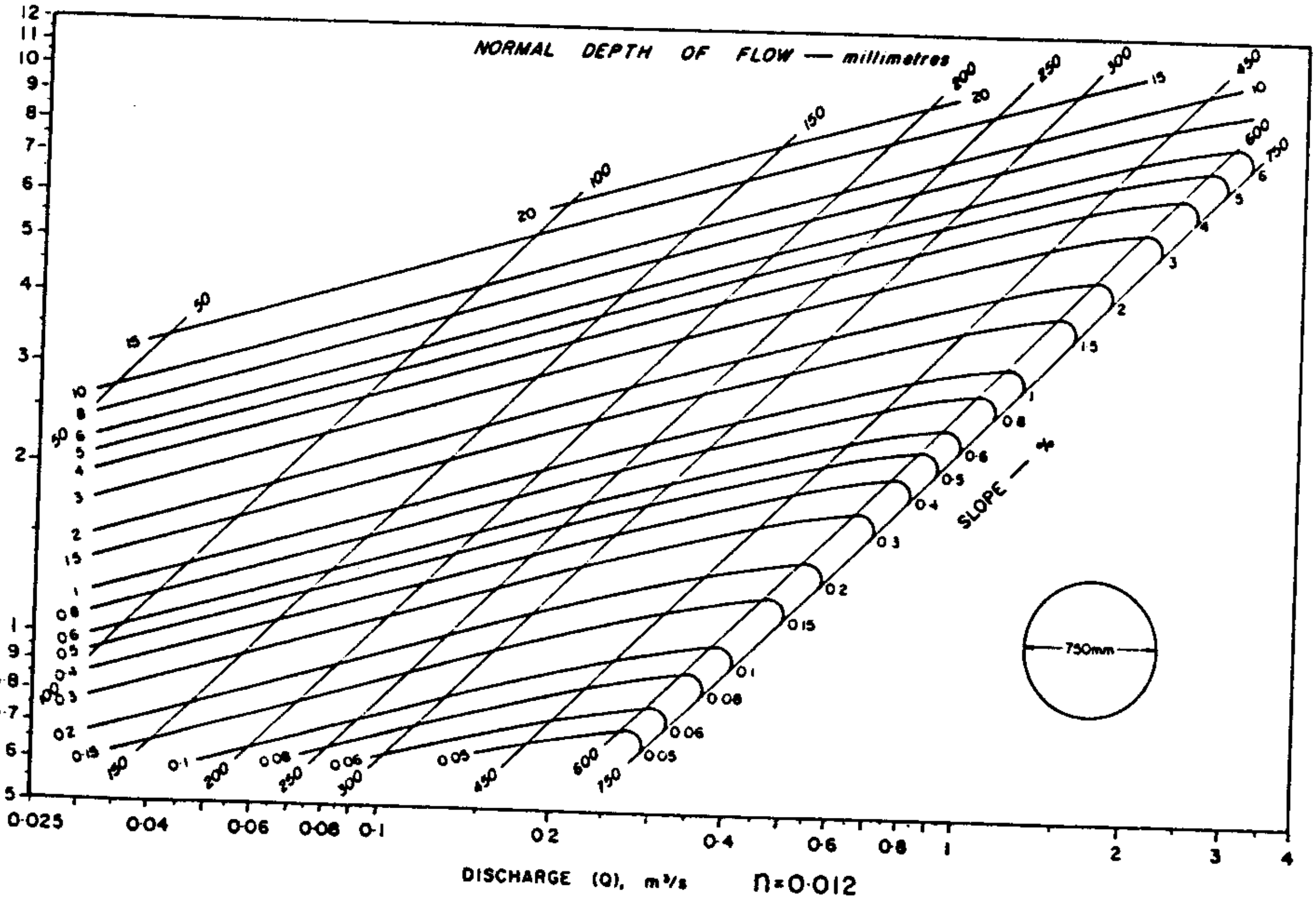
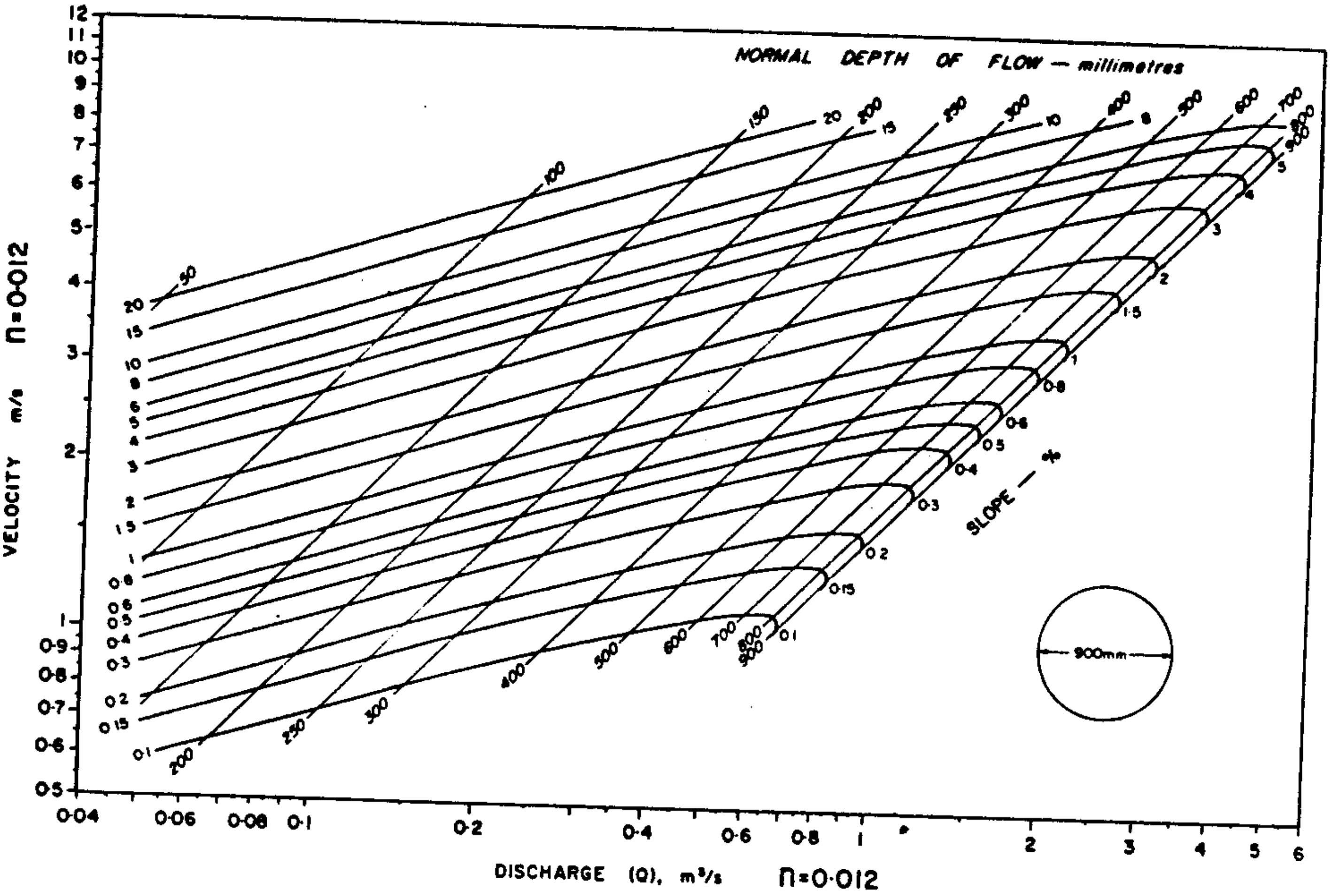

NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
750mm
DISCHARGE (Q), m³/s
n=0·012

Chart 3.7


NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
900mm
VELOCITY m/s
n=0·012
DISCHARGE (Q), m³/s
n=0·012

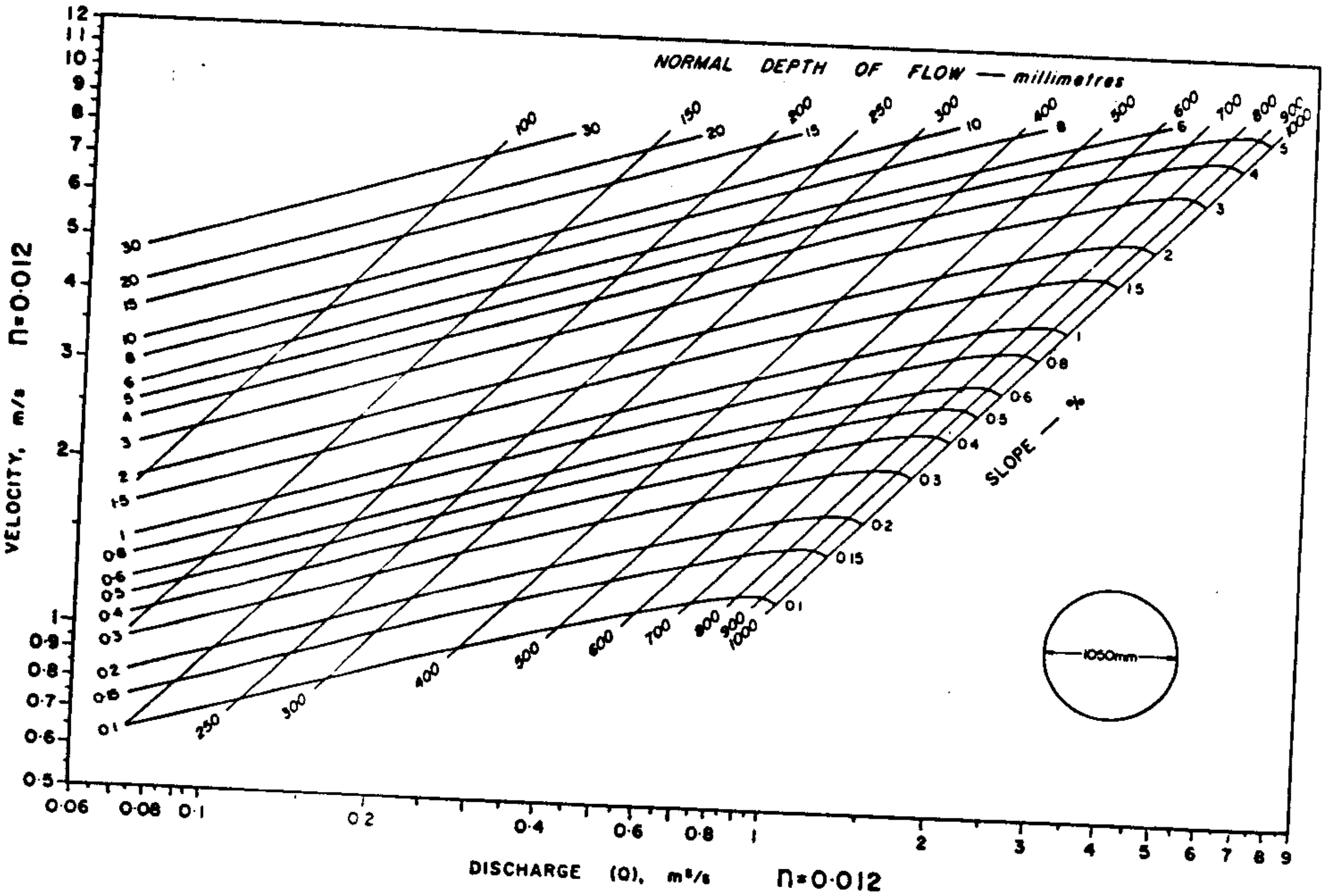
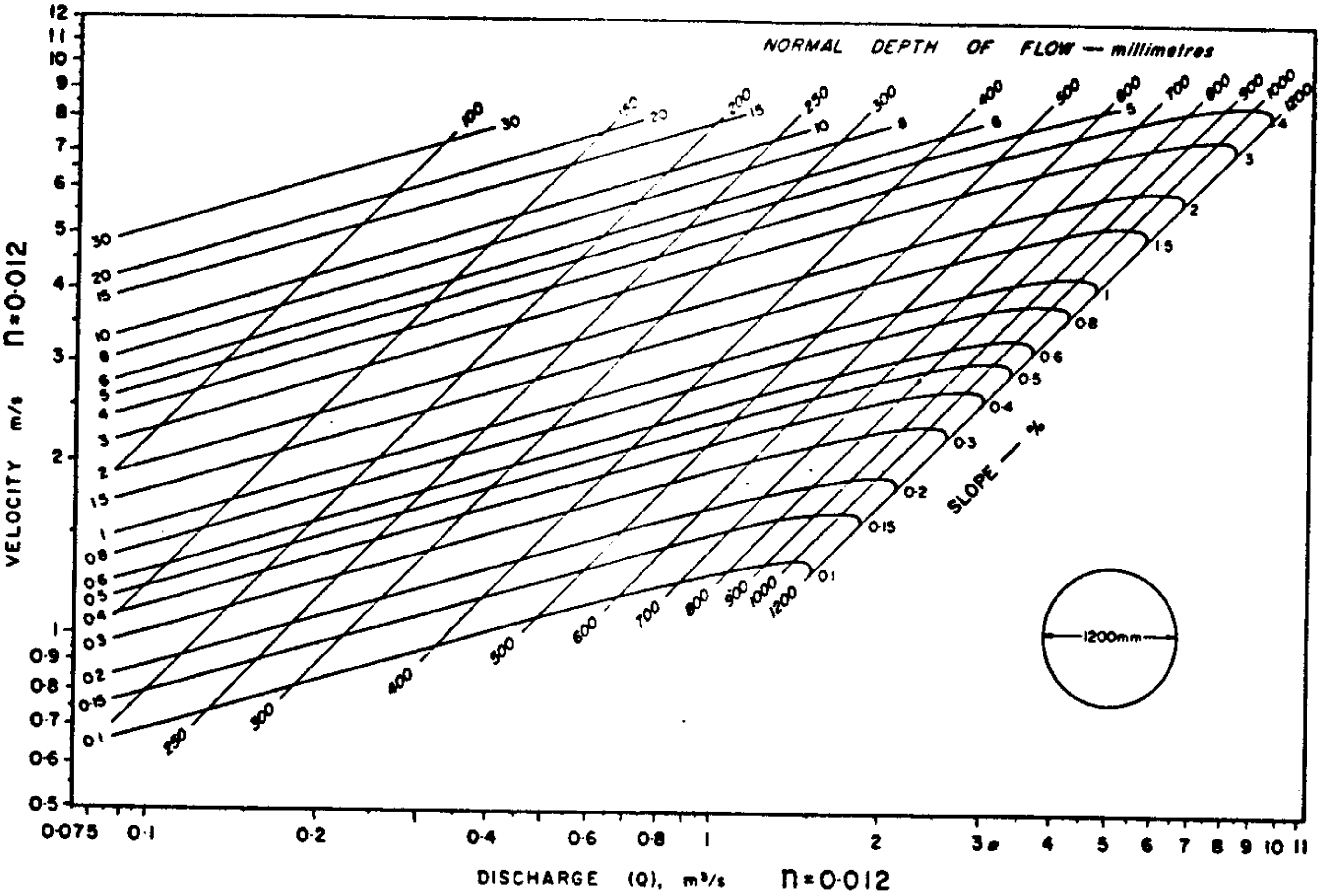

NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
1050mm
VELOCITY, m/s
n=0·012
DISCHARGE (Q), m³/s
n=0·012

Chart 3.8


NORMAL DEPTH OF FLOW — millimetres
VELOCITY m/s
DISCHARGE (Q), m³/s
n=0·012
n=0·012
SLOPE — %
1200mm

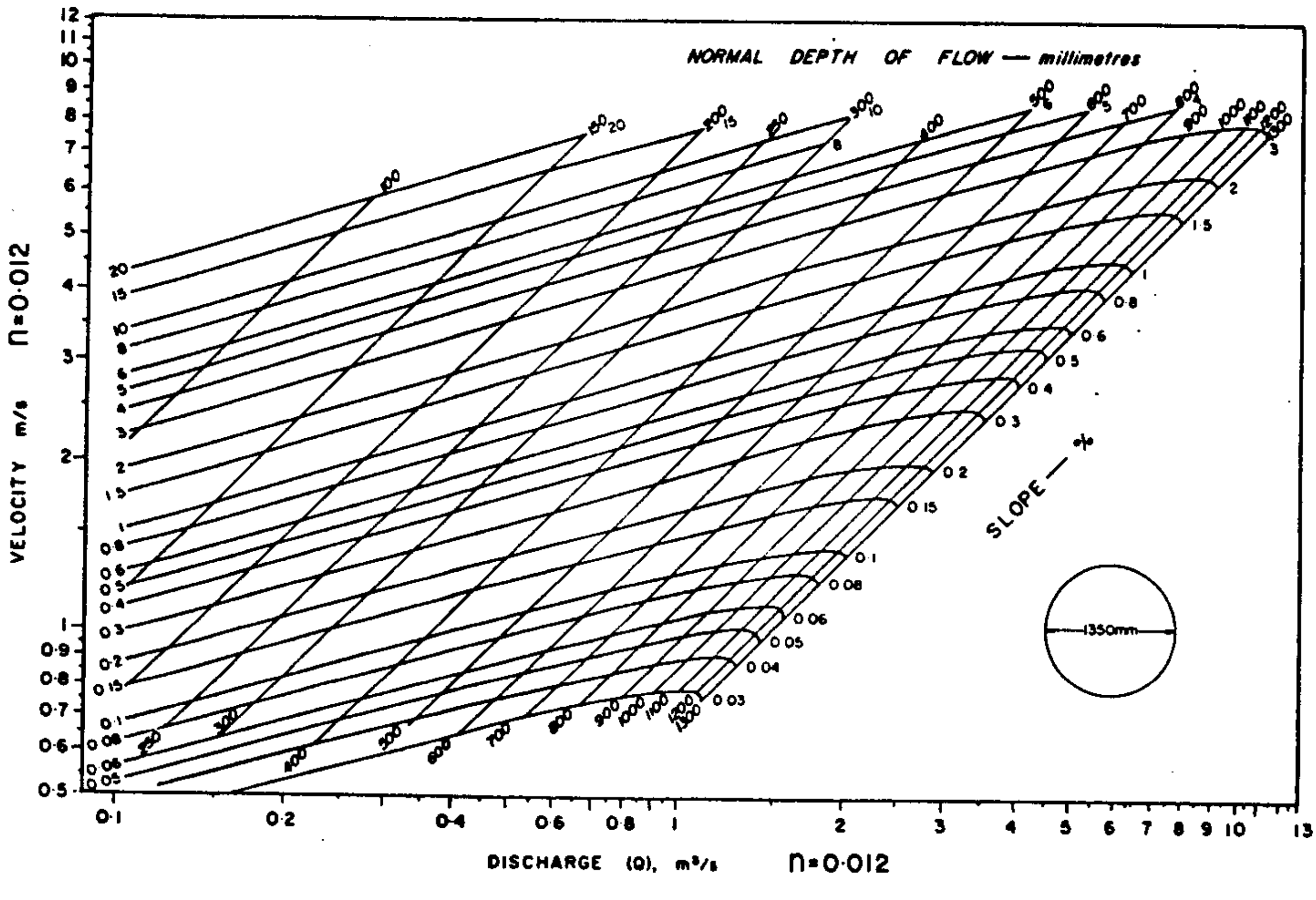
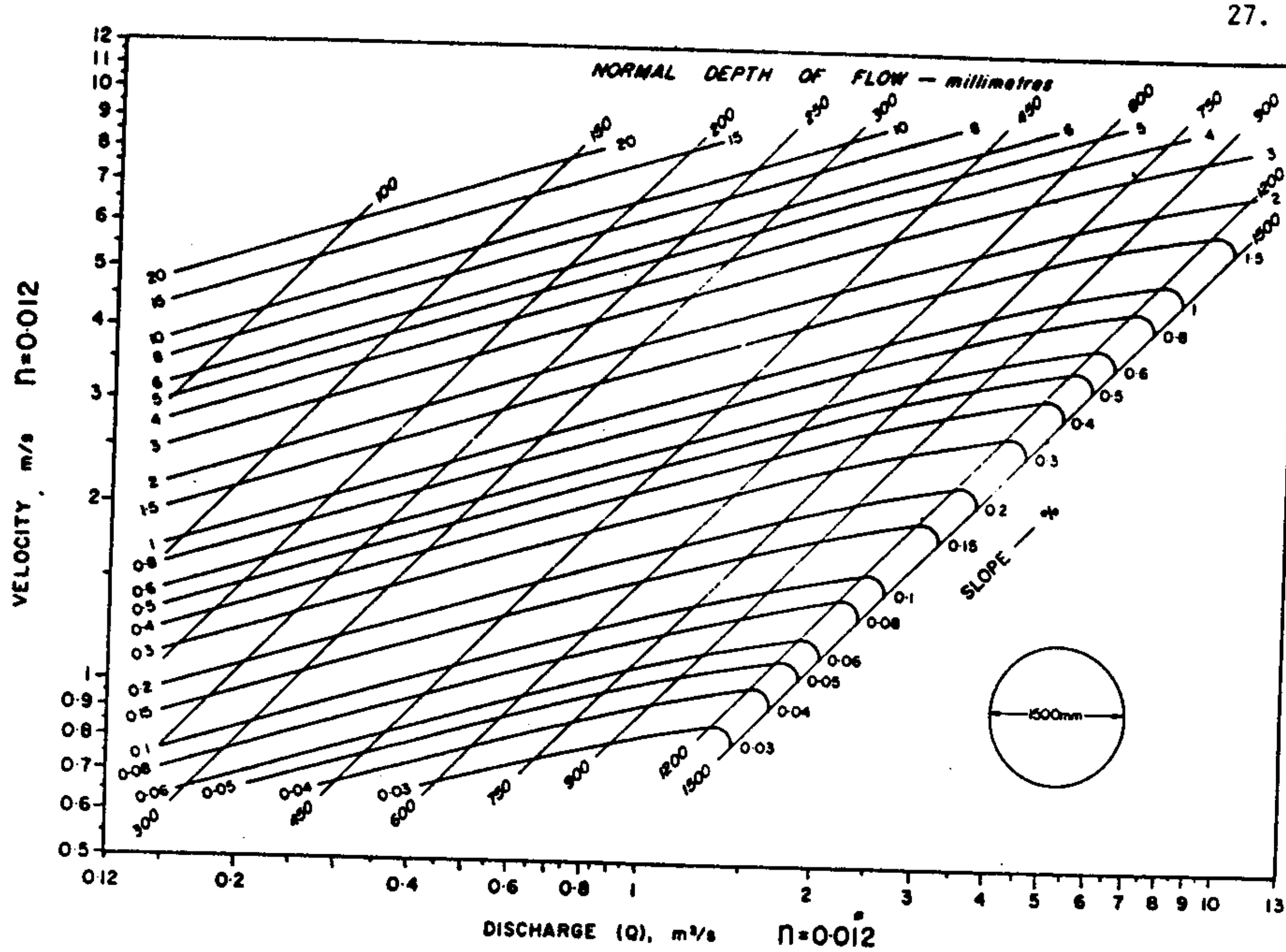

NORMAL DEPTH OF FLOW — millimetres
VELOCITY m/s
DISCHARGE (Q), m³/s
n=0·012
n=0·012
SLOPE — %
1350mm

Chart 3.9


NORMAL DEPTH OF FLOW — millimetres
VELOCITY, m/s
n=0·012
DISCHARGE (Q), m³/s
n=0·012
SLOPE — %
1500mm

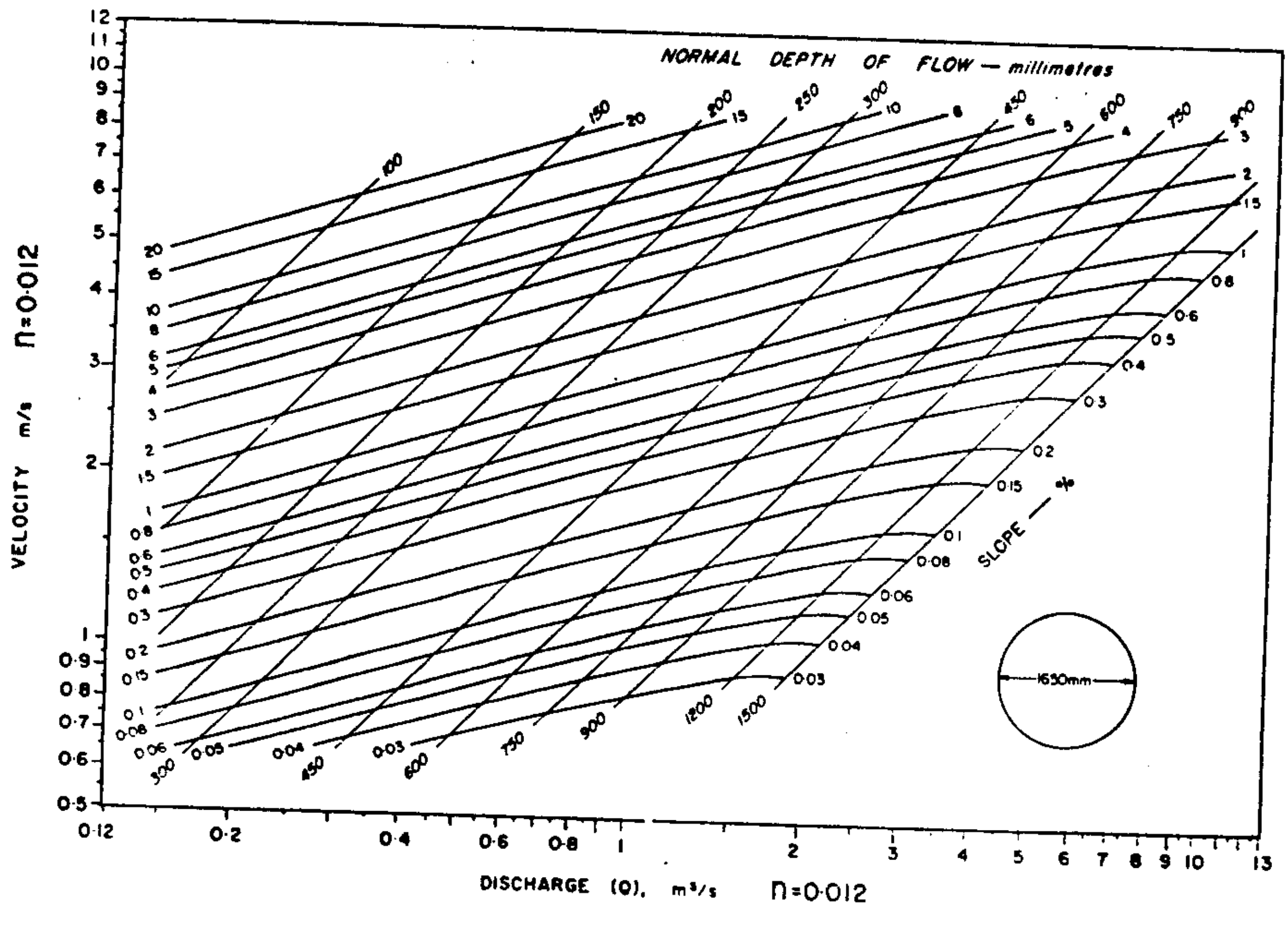
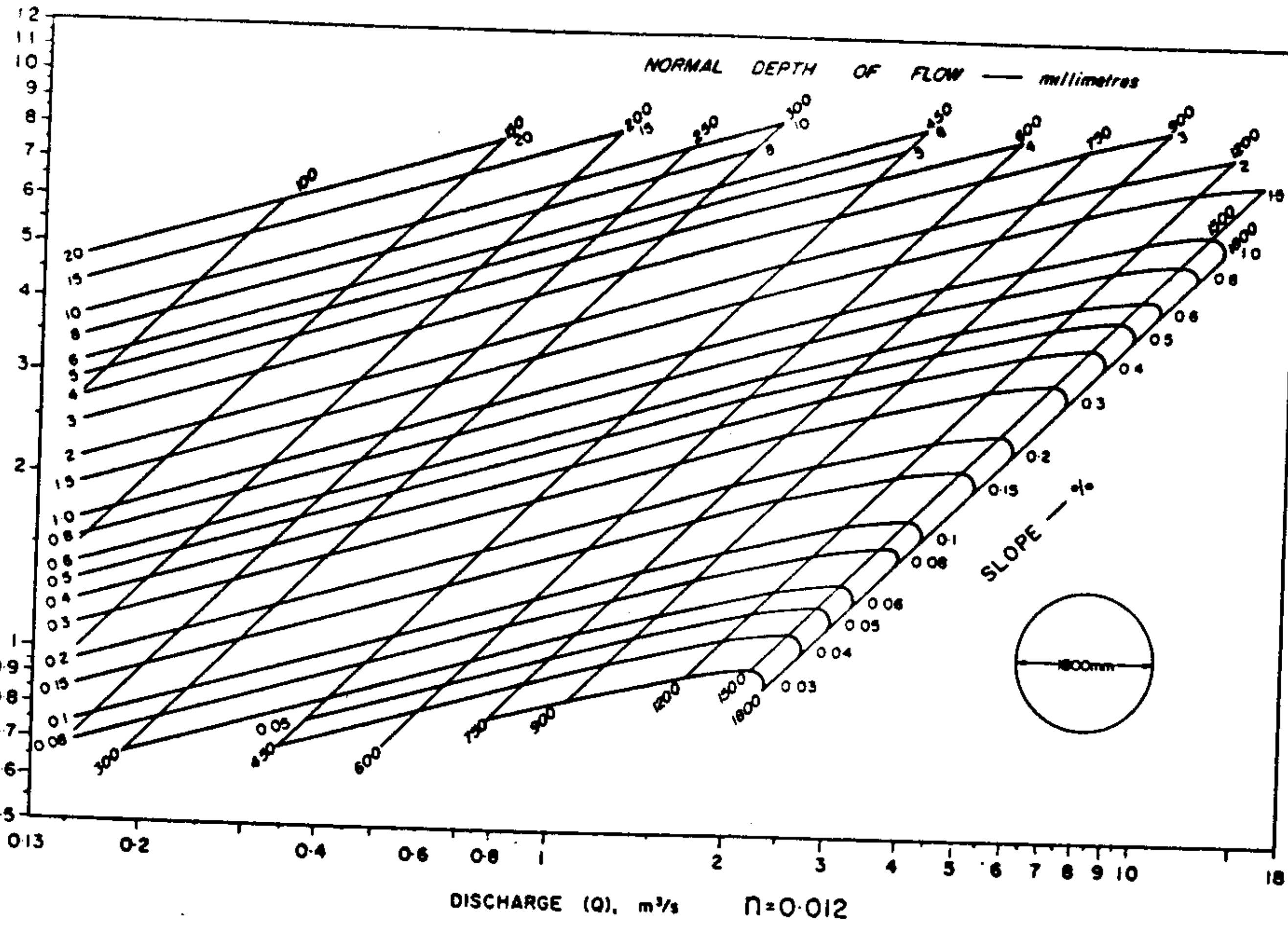

NORMAL DEPTH OF FLOW — millimetres
VELOCITY m/s
n=0·012
DISCHARGE (Q), m³/s
n=0·012
SLOPE — %
1650mm

Chart 3.10


NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
1800mm
DISCHARGE (Q), m³/s
n=0·012

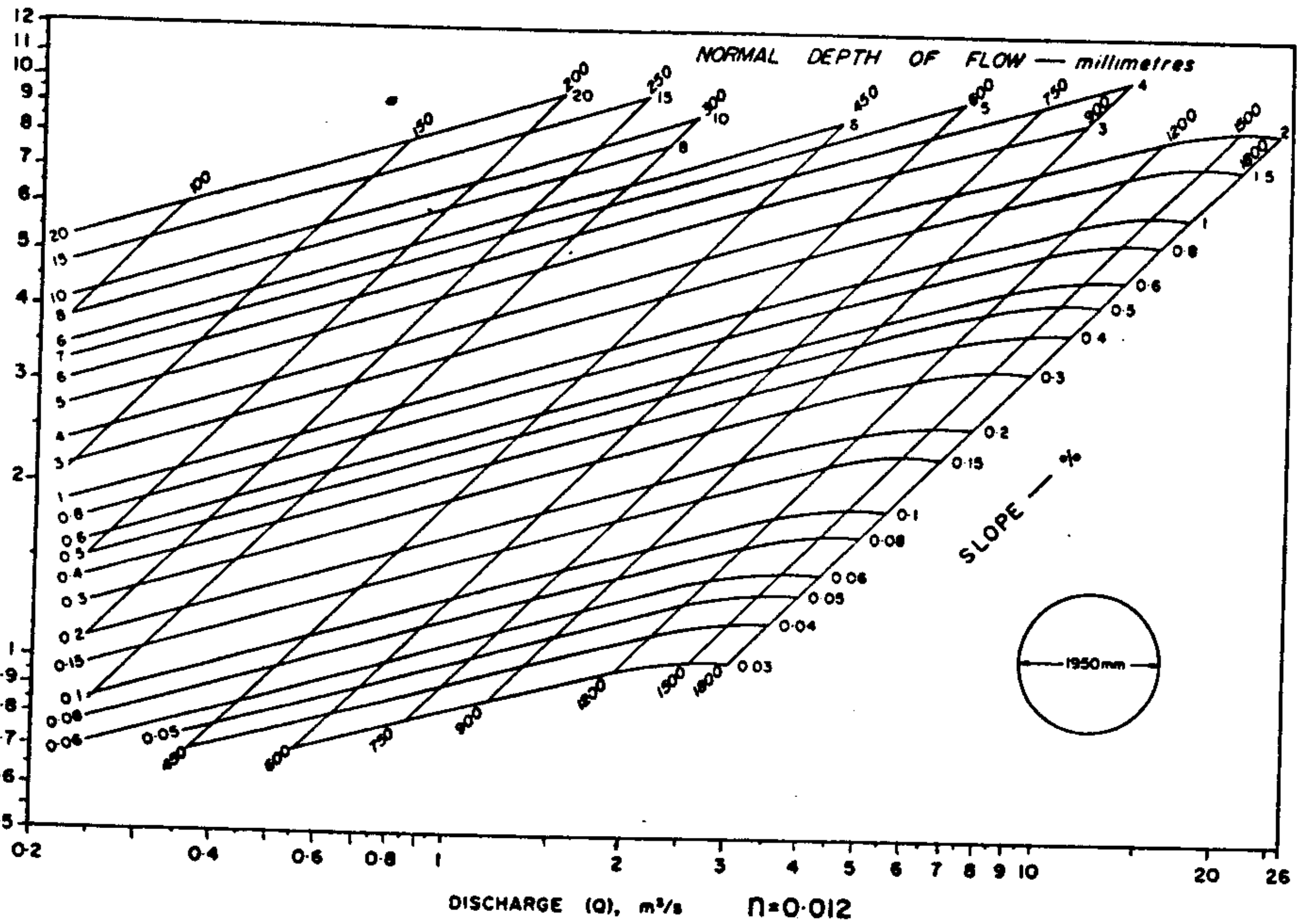
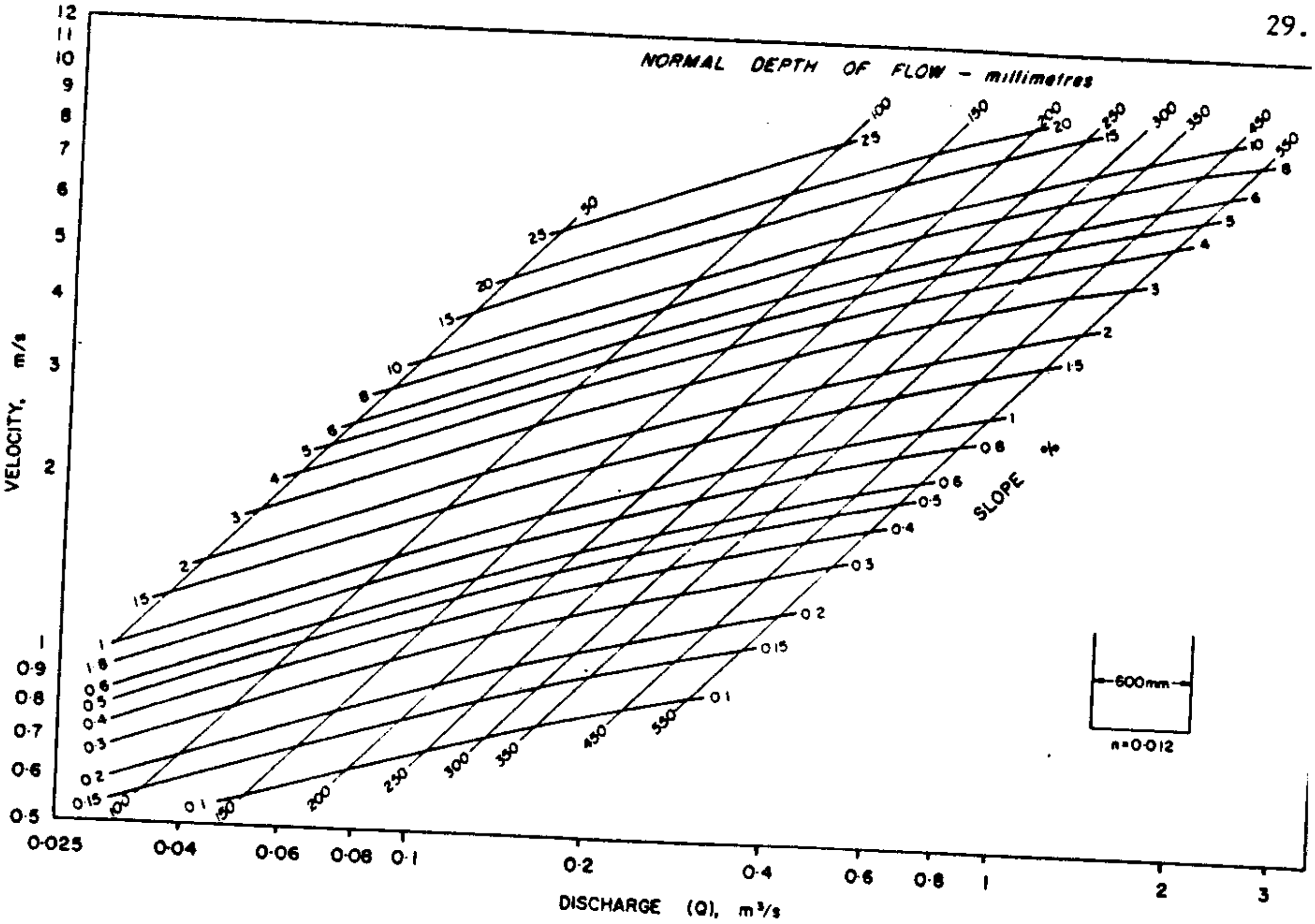

NORMAL DEPTH OF FLOW — millimetres
SLOPE — %
1950mm
DISCHARGE (Q), m³/s
n=0·012

Chart 3.11


NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE %
600mm
n=0·012

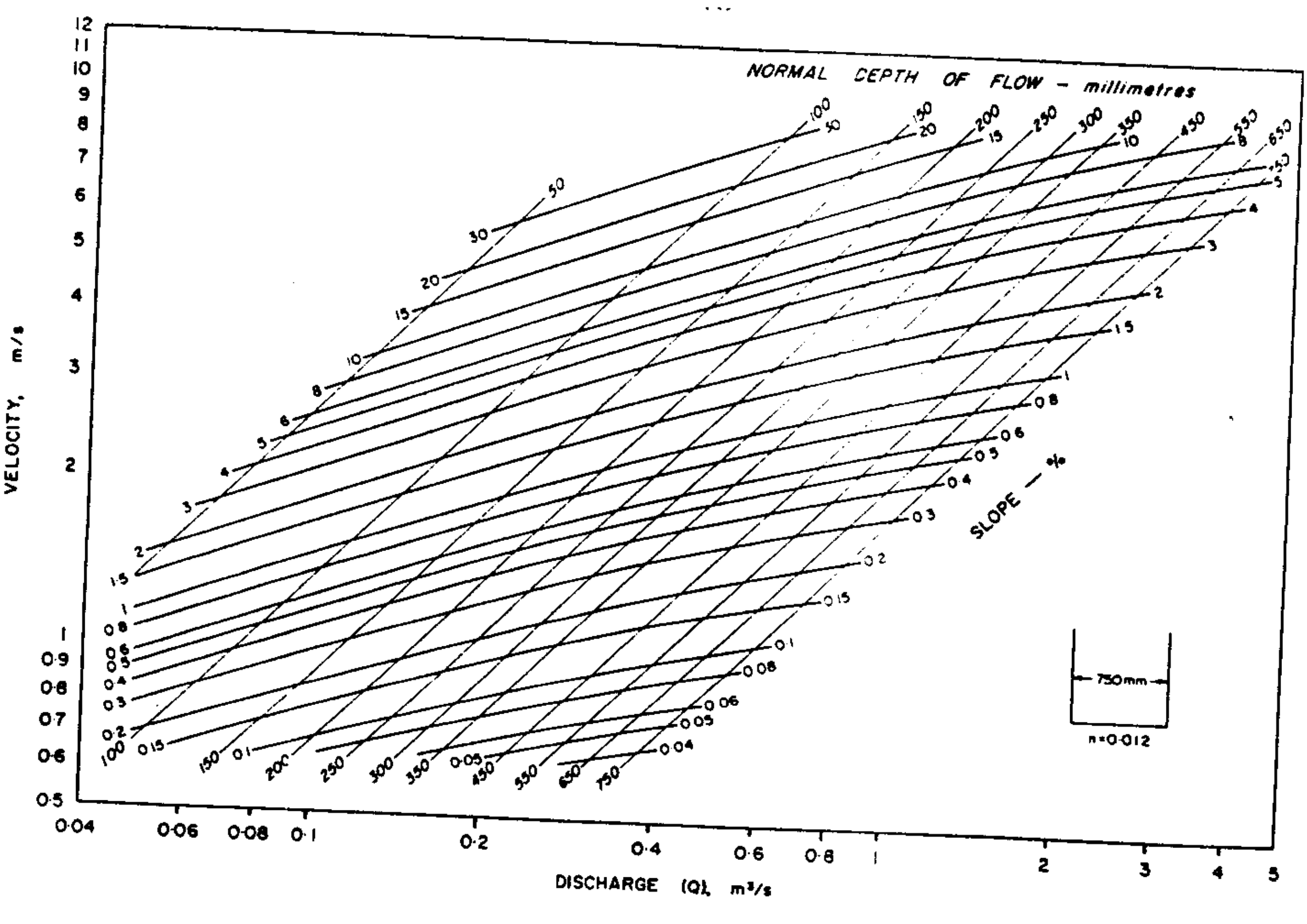
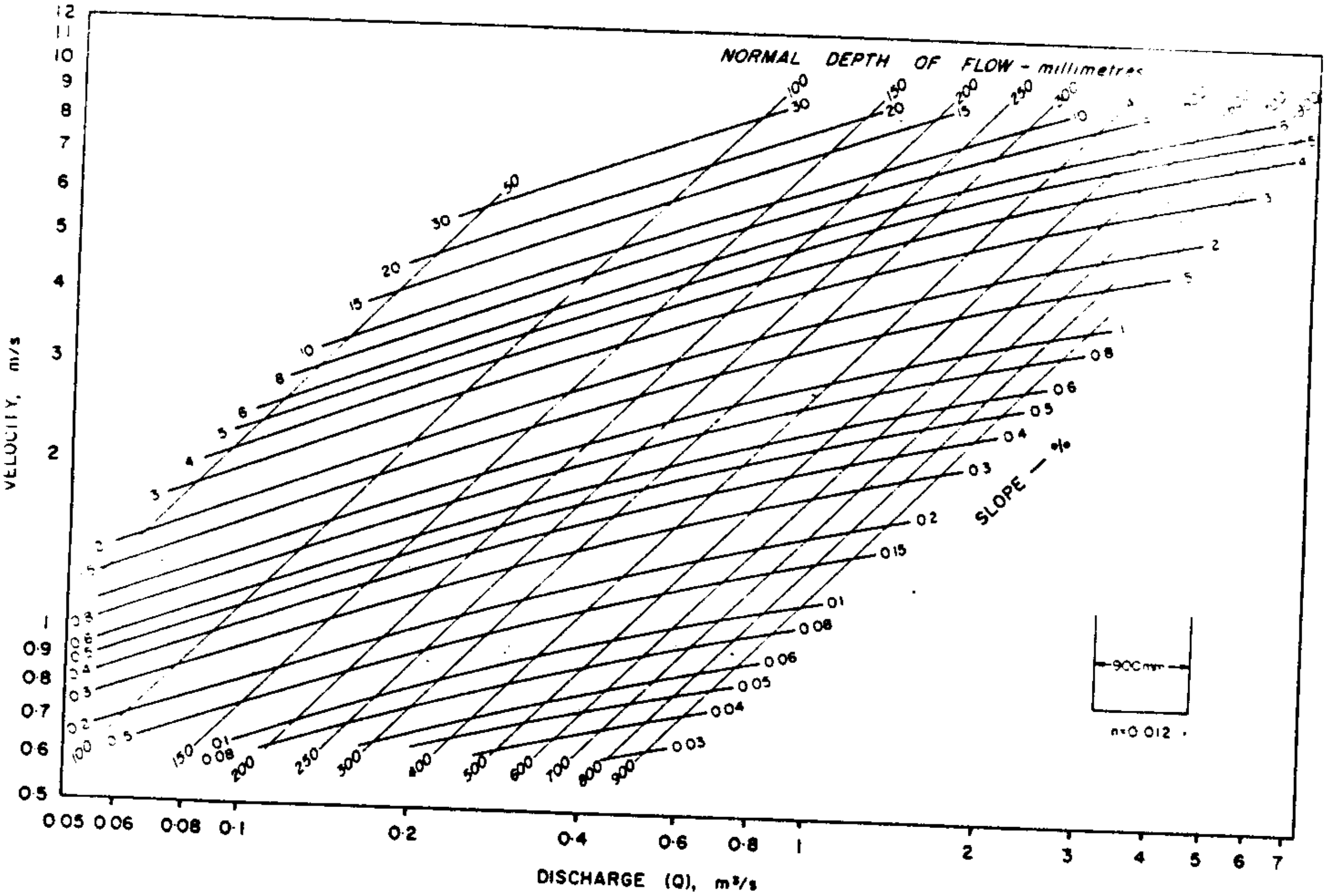

NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
750mm
n=0·012

Chart 3.12


NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
900mm
n=0.012

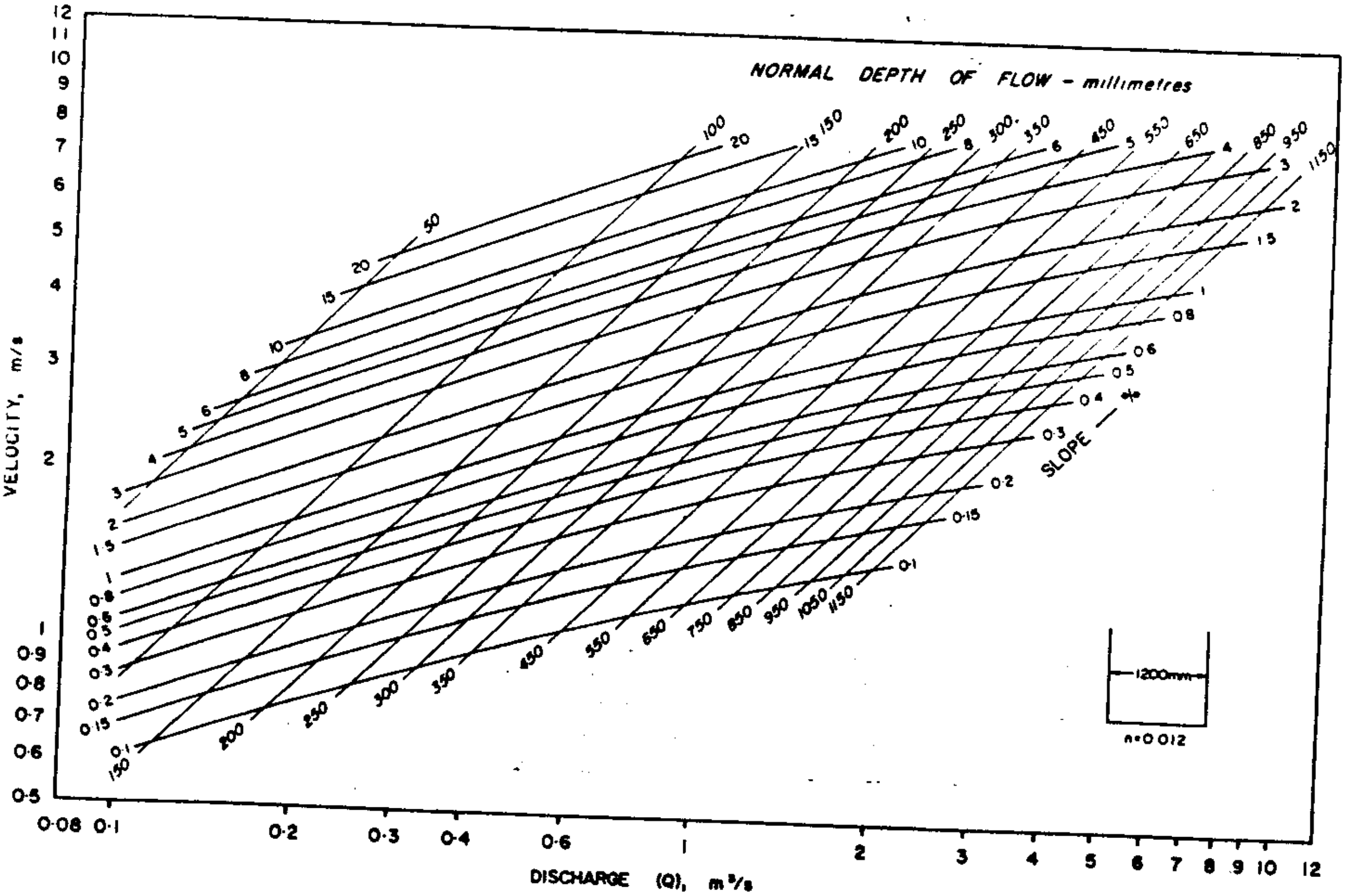
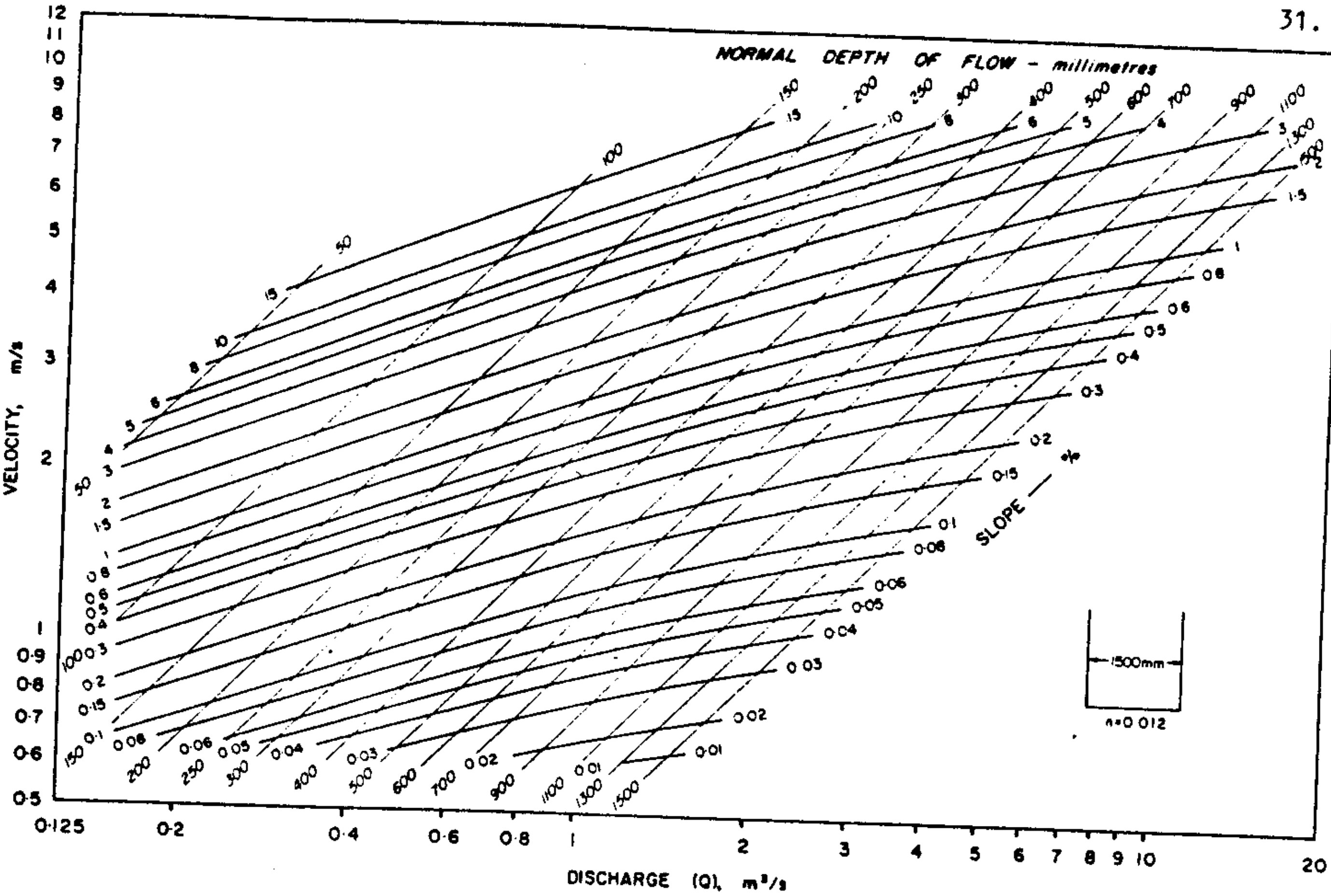

NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
1200mm
n=0.012

Chart 3.12


NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
1500mm
n=0.012

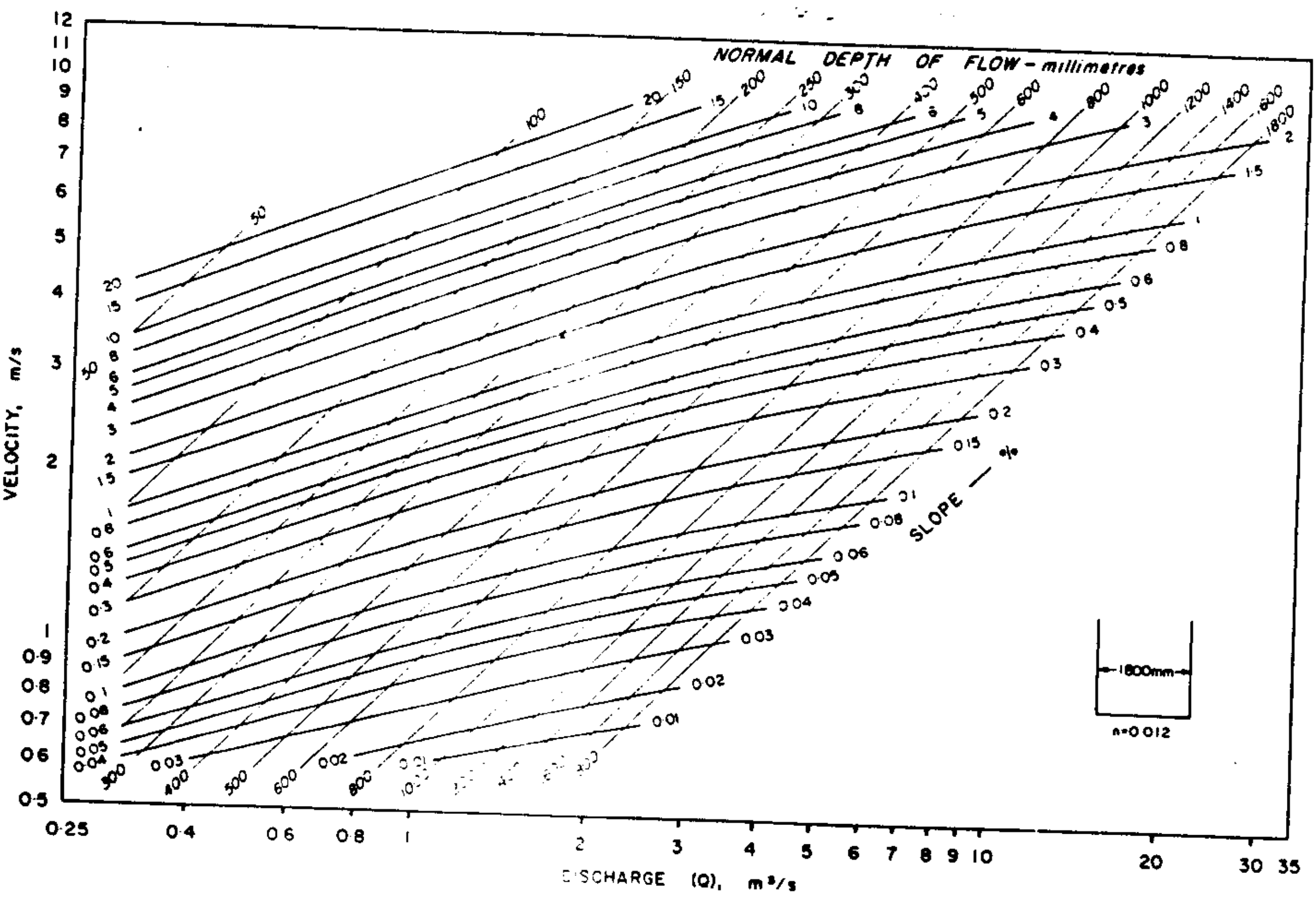
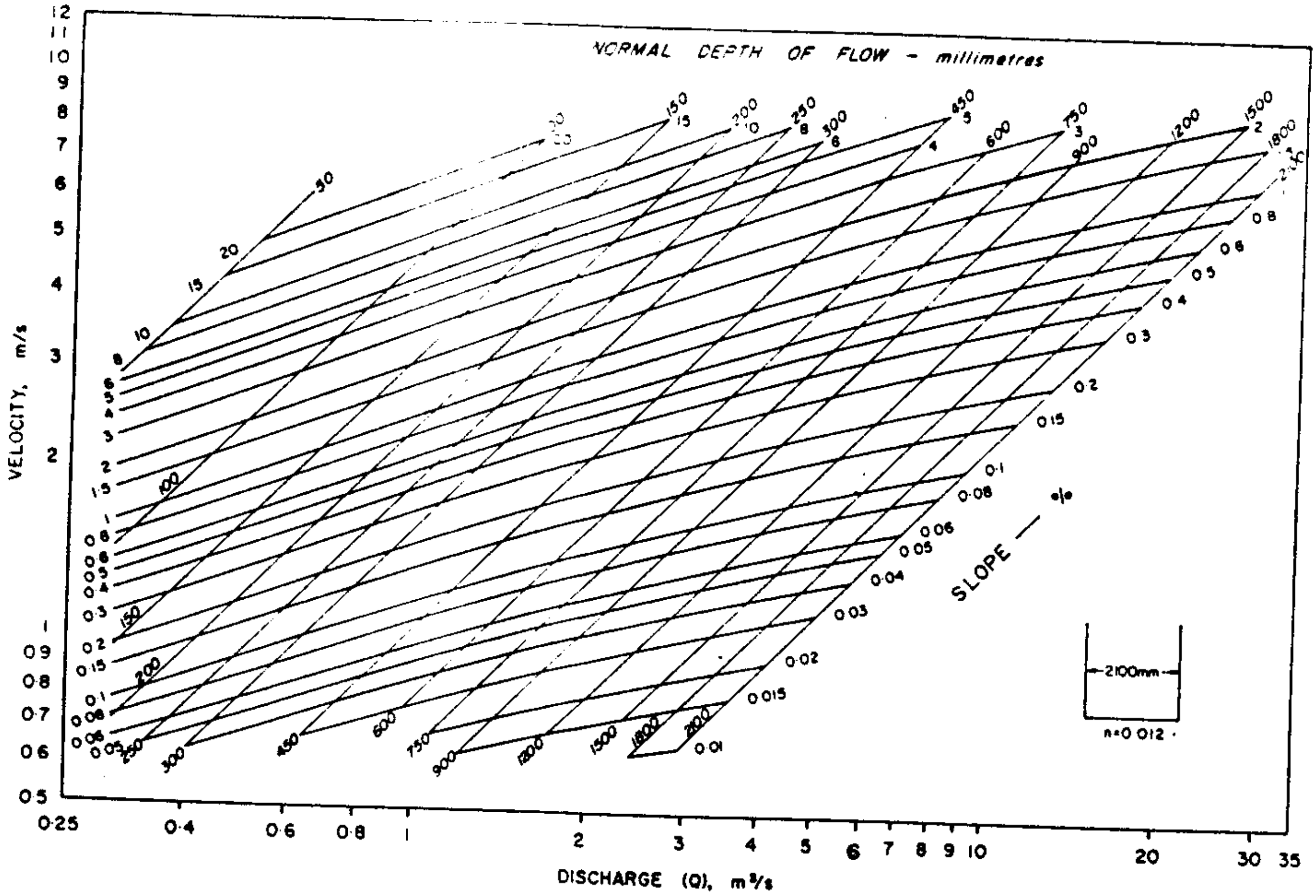

NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
1800mm
n=0.012

Chart 3.14


NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
2100mm
n=0.012

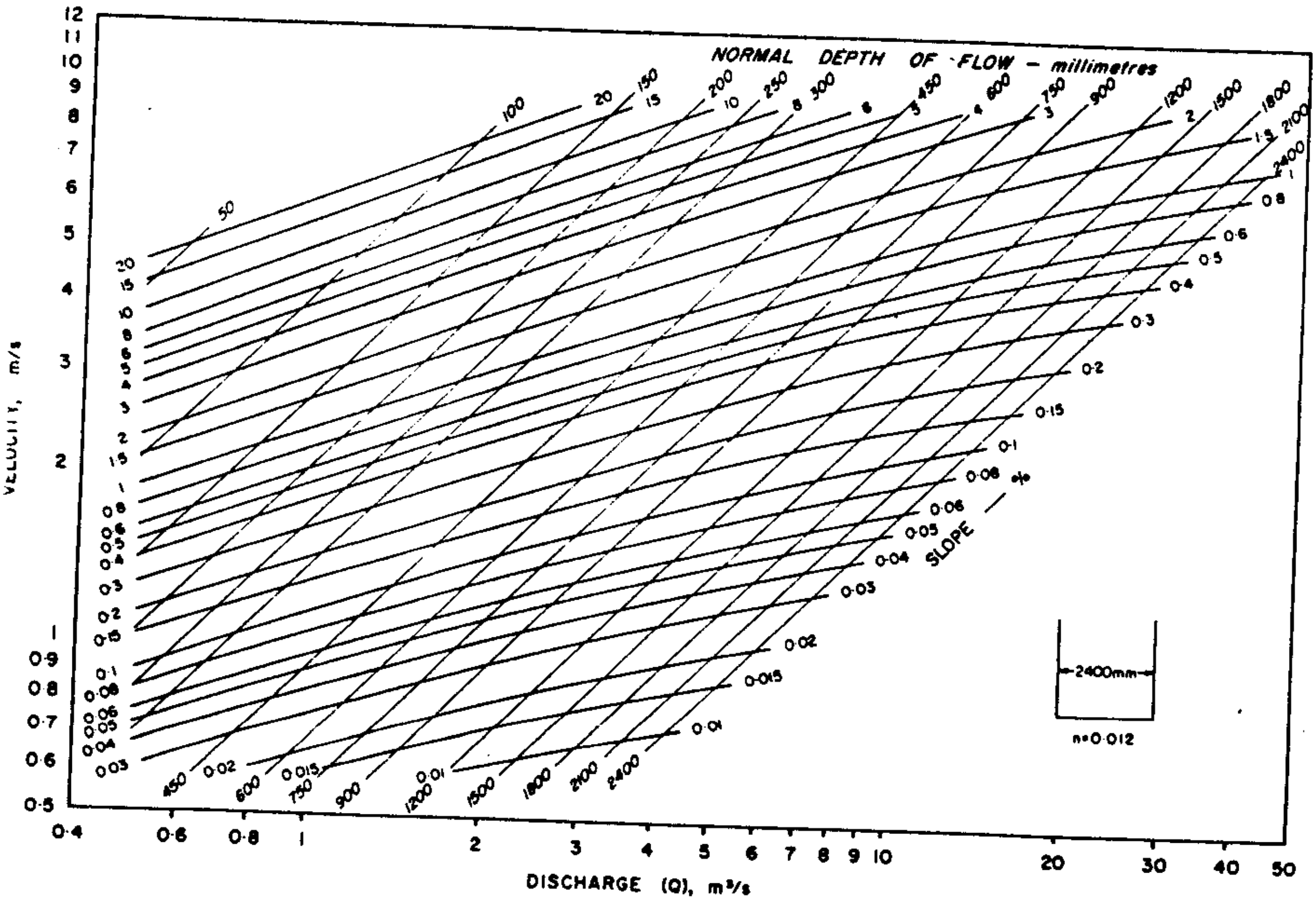
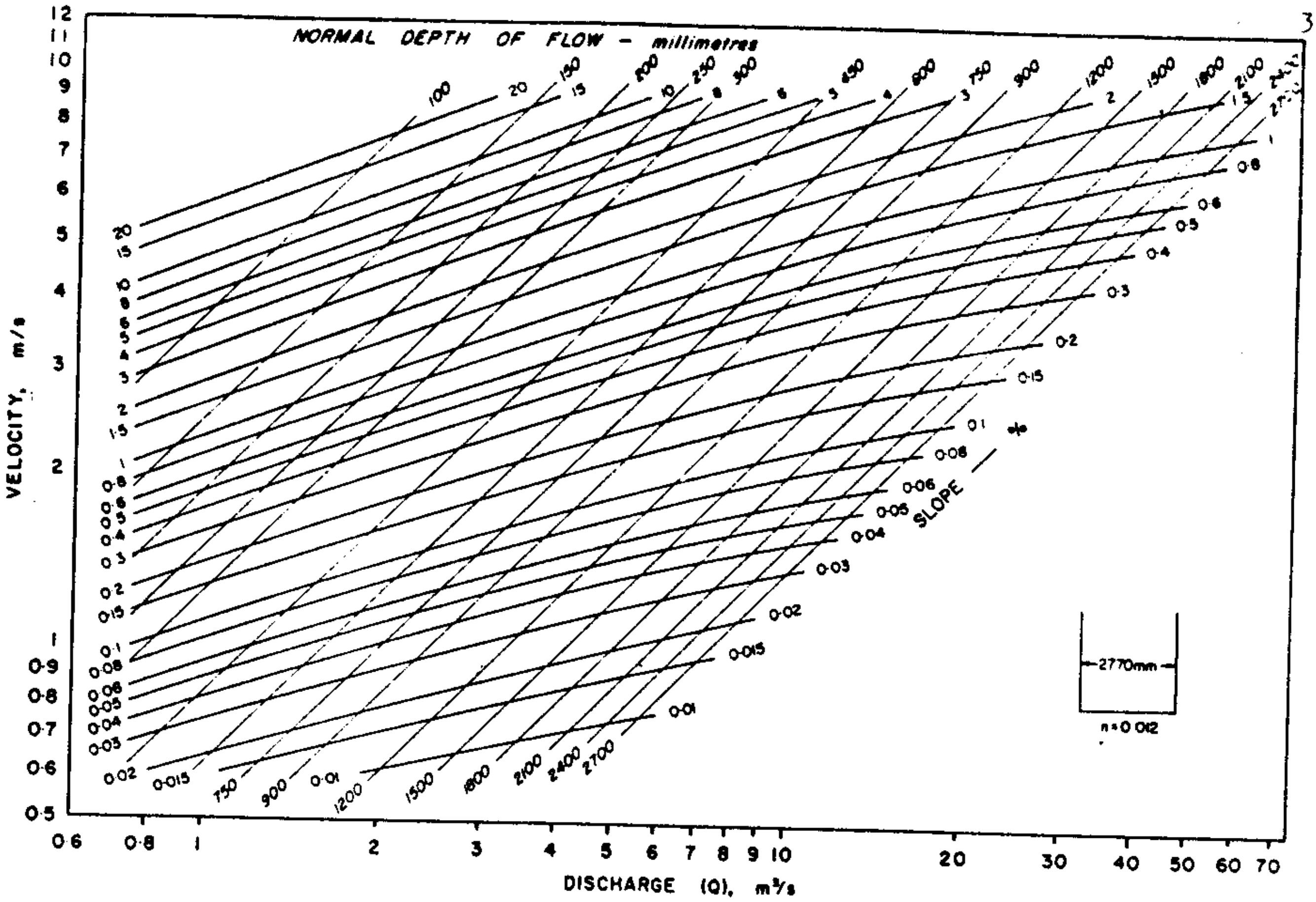

NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
2400mm
n=0.012

Chart 3.15


NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
2770mm
n=0.012

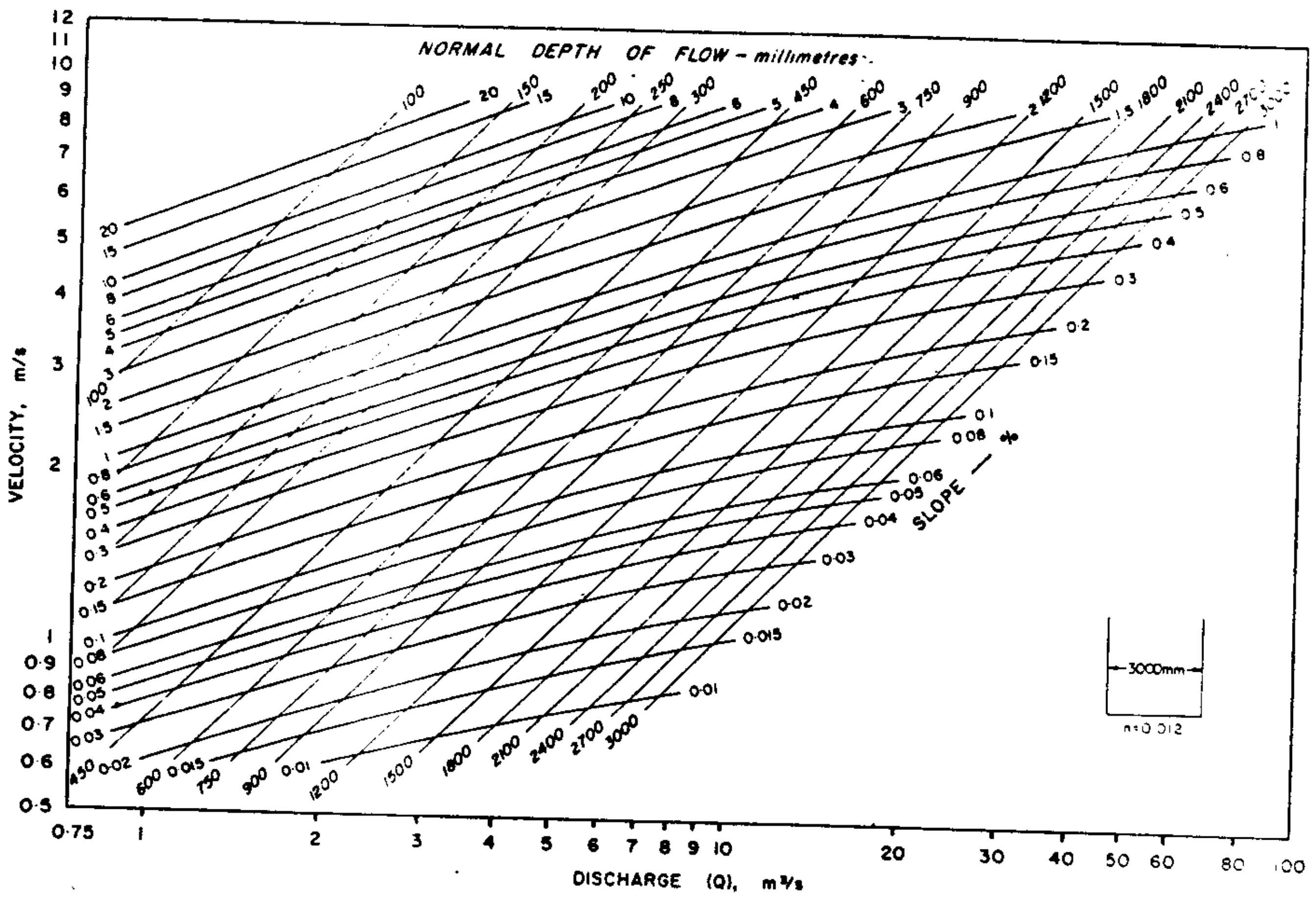
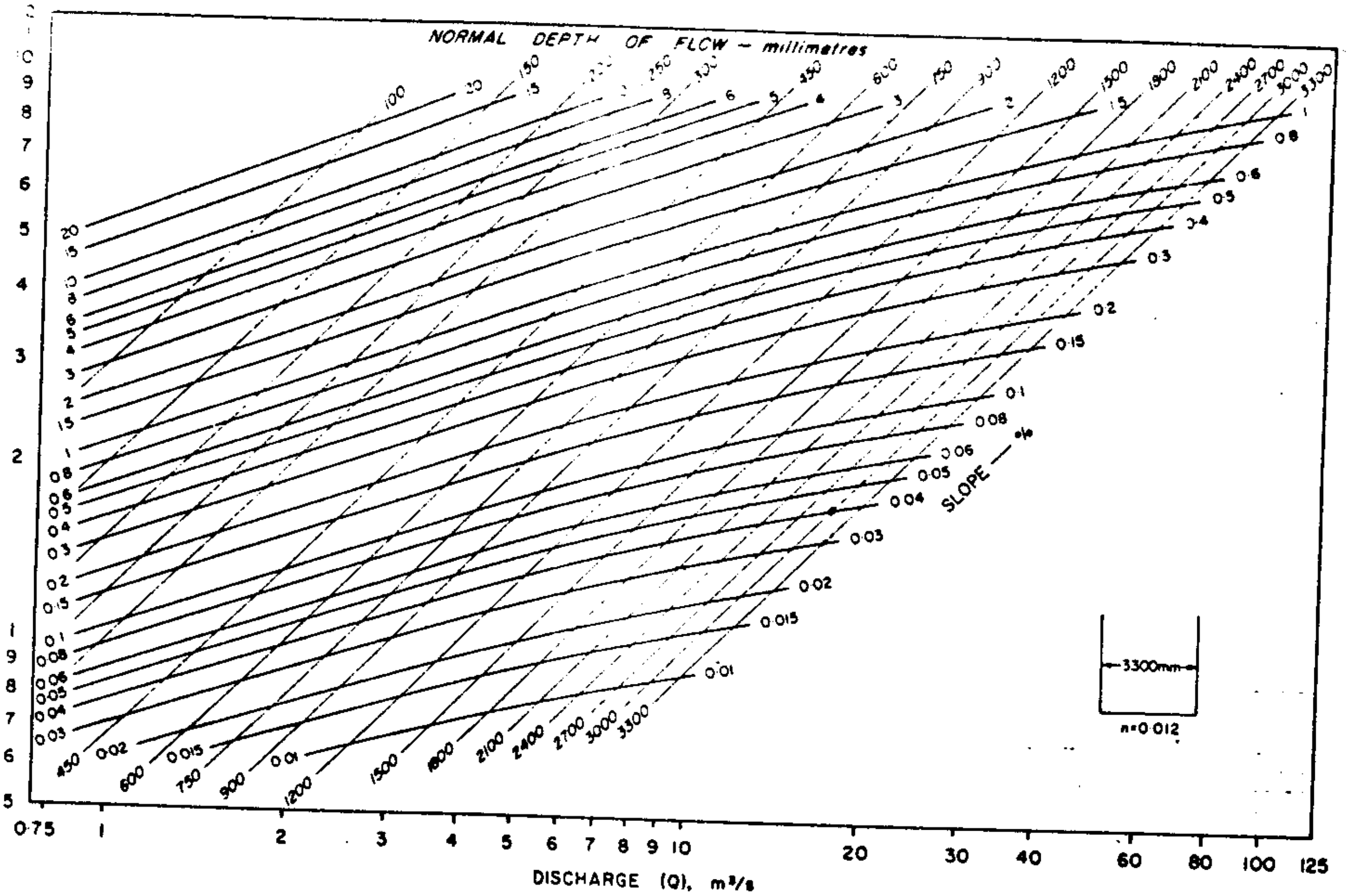

NORMAL DEPTH OF FLOW - millimetres
VELOCITY, m/s
DISCHARGE (Q), m³/s
SLOPE — %
3000mm
n=0.012

Chart 3.16


NORMAL DEPTH OF FLOW - millimetres
SLOPE — %
3300mm
n=0.012
DISCHARGE (Q), m³/s

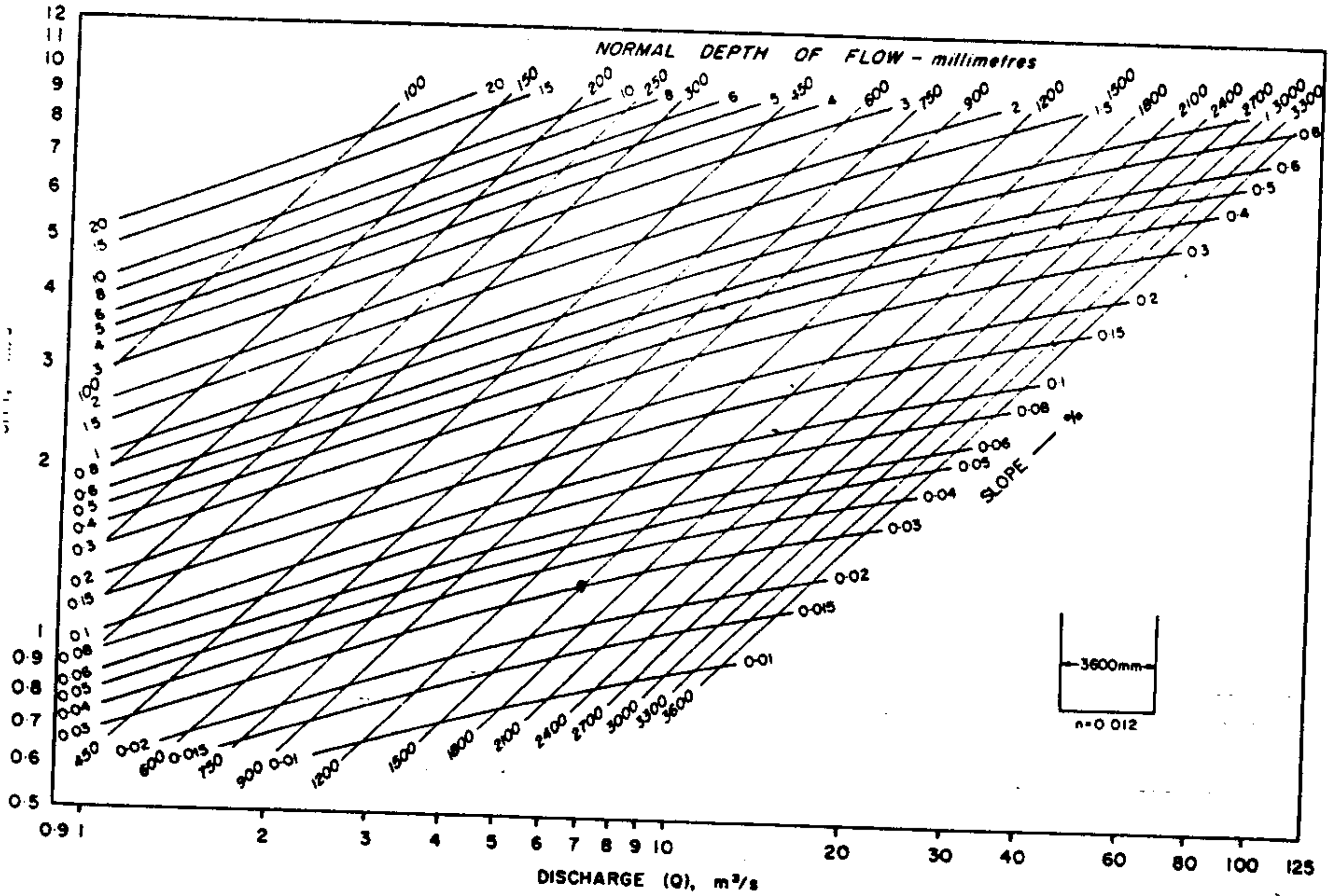

NORMAL DEPTH OF FLOW - millimetres
SLOPE — %
3600mm
n=0.012
DISCHARGE (Q), m³/s

Chart 3.17

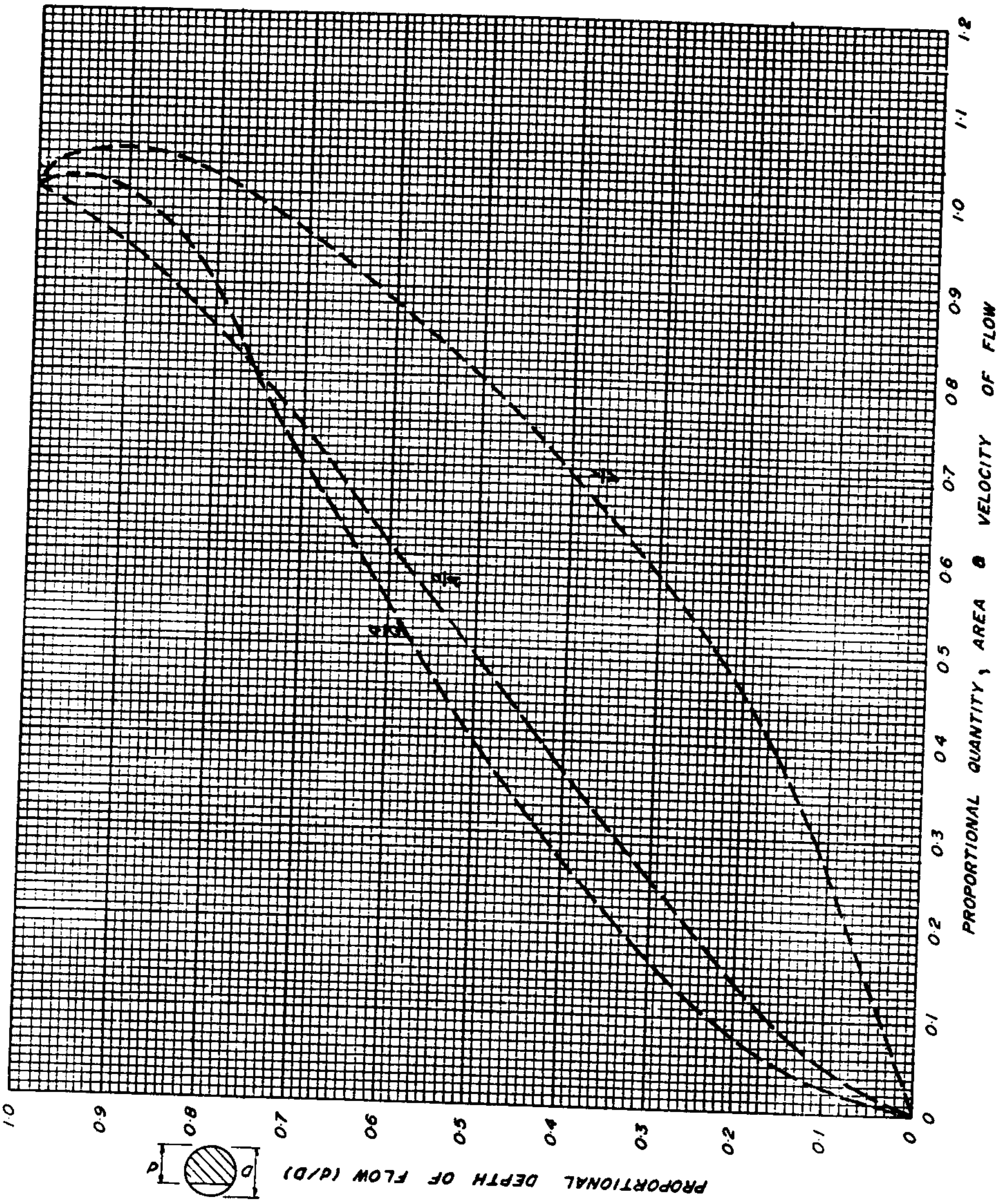
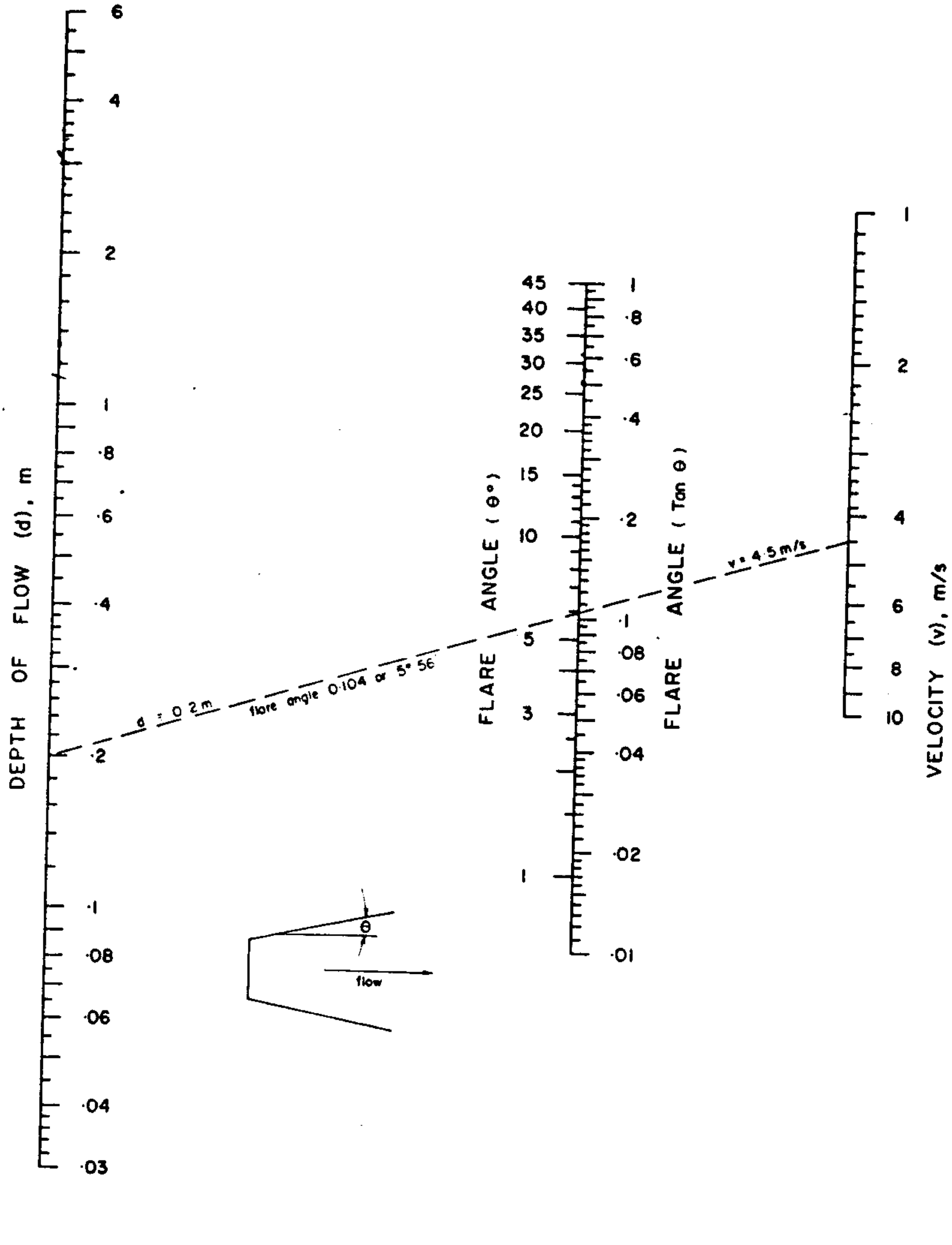

PROPORTIONAL QUANTITY, AREA & VELOCITY OF FLOW
PROPORTIONAL DEPTH OF FLOW (d/D)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2

Chart 3.18



Flare angles for divergent flow

Chart 3.19

SECTION 4 - FLOODWAY DESIGN

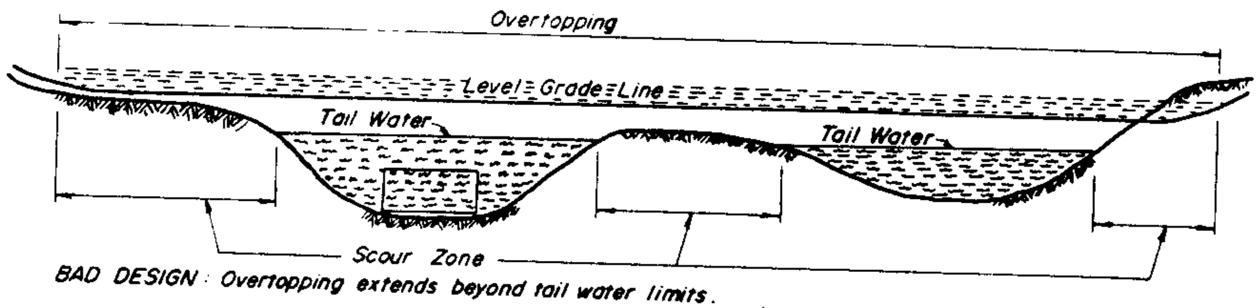
4.1 GENERAL

Floodway design is important in the economic prevention of erosion and scouring on the downstream side of a hydraulic structure.

Generally the greatest risk for scouring will occur when water overtops a floodway at points where there is little or no tailwater on the downstream side.

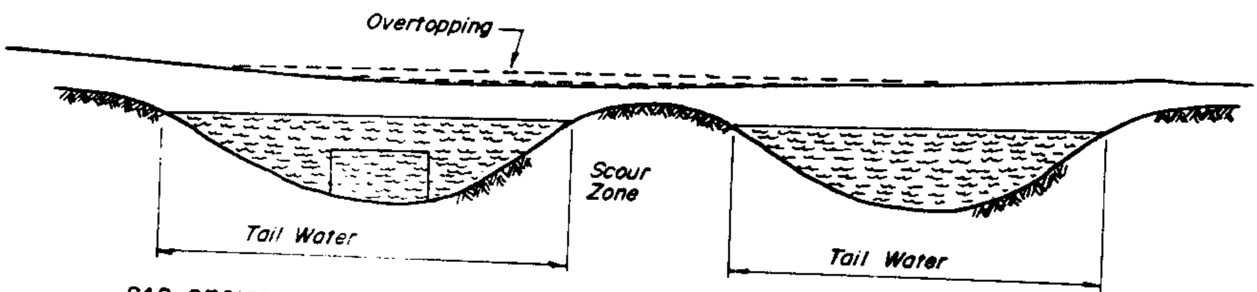
For trafficability, a level grade line is desirable to provide a long shallow length of flow, however, for reduced scour risk, a level grade line should be avoided.

The following diagrams depict example of both good and bad design practices for a particular situation.



BAD DESIGN: Overtopping extends beyond tail water limits.

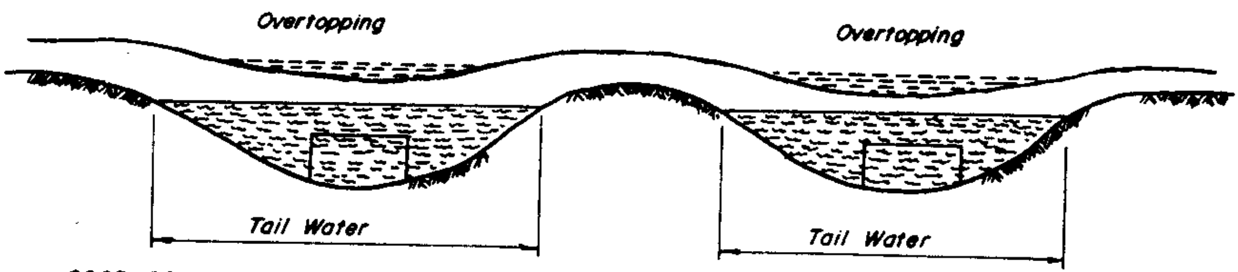
(a)



BAD DESIGN: Overtopping occurs at lowest point of V.C. where no tail water available.

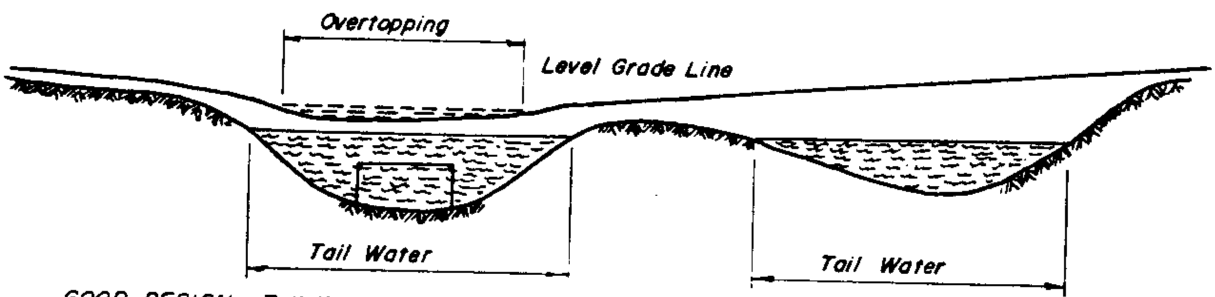
(b)

SECTION 4.2 (Cont.)



GOOD DESIGN: Tail Water exceeds width of overtopping.

(c)



GOOD DESIGN: Tail Water always exceeds width of overtopping

(d)

FIG. 4·1

4.2 GRADE LINE DESIGN

The grade line should be designed so as to provide:-

(a) A tailwater that extends wider than the length of roadway being overtopped at any stage of flow.

(b) A tailwater depth of at least 100mm at every point along the floodway before it overtops at that point.

(c) The lowest point of a sag VC should be at the location of maximum tailwater at the time of initial overtopping.

4.3 CONCRETE PROTECTION

Concrete protection shall be in accordance with S.W.D.S.#2, "Standard Causeway and Floodway Details".

(a) Upstream Protection - shall be provided to the full length of the floodway and consist of a concrete margin (shoulder), batter and cut-off wall.

In some instances, the Shire Engineer may reduce the batter and cut-off wall protection to a minimum of 5.0m to both sides of the culvert, however, a concrete margin for the full length of the floodway would still be required.

(b) Downstream Protection - shall be provided to the full length of the floodway and consist of a concrete margin (shoulder), batter, apron and cut-off wall.

(c) Apron Widths - shall be of 1.5 times the embankment height with a minimum width of 0.5m.

(d) Culvert Endwalls - shall not protrude above the line of the adjacent margin or batter.

4.4 DESIGN PROCEDURE

The design shall provide a maximum water surface drop of 450mm at the initial overtopping (e.g. shoulder level - TW level = 450mm).

(a) Determine the flow rate in the natural channel for a water level 450mm below the shoulder level (see Section #5.5 - Tailwater Depth Calculation).

(b) Design a culvert with this capacity based on an inlet head of 450mm.

4.5 FLOW DEPTH OVER EMBANKMENT

The depth of flow over long flat embankments and floodways may be determined in accordance with Section 14.13 of "Australian Rainfall and Runoff", however, in the absence of design information relating to flow depths in sag curves, the following derived equation based on a broad crest weir may be used as an approximation:-

SECTION 4.5 (Cont.)

FIG. 4.2

Let Q = flow over the embankment (m^3/sec)

A = area of flow (m^2)

y = depth of flow (m)

g = $9.8 m/sec^2$

Now $Q = VA$ therefore $V = Q/A$

and $V = \sqrt{gy}$

therefore $Q/A = \sqrt{gy}$

But (A) is a function of (y) and the following two (2) longitudinal grade sections of a floodway are considered.

Long. Section No.1

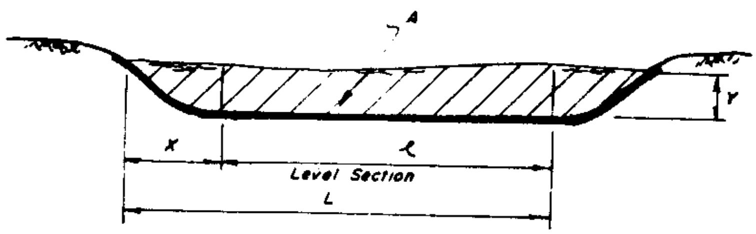


FIG 4.3

Assume the flow areas at each end of the level section approximates to triangles.

therefore $A = \ell y + 2\left(\frac{xy}{2}\right) = (\ell + x)y$

but $\ell + x = L$

$\therefore \quad A = Ly$

Long Section No. 2

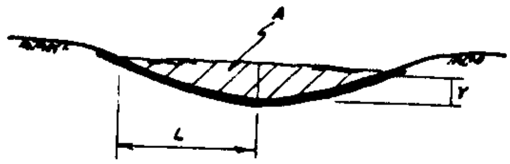


FIG. 4.4

Again, assume the flow areas is approximate to two (2) triangles:-

$$A = 2\left(\frac{Ly}{2}\right) = Ly$$

Subject to the above definition of length (L)

$Q/A = \sqrt{gy}$ and $A = yL$

therefore $\frac{Q}{yL} = \sqrt{gy}$

therefore $y = \frac{1}{g^{1/3}}\left(\frac{Q}{L}\right)^{2/3}$

let $g = 9.8\text{m/sec}^2$

$$\boxed{\text{therefore } y = 0.47(Q/L)^{2/3}}$$

where y = depth of flow (m)

Q = flow volume (m^3/sec)

L = "length" of flow as defined above (m)

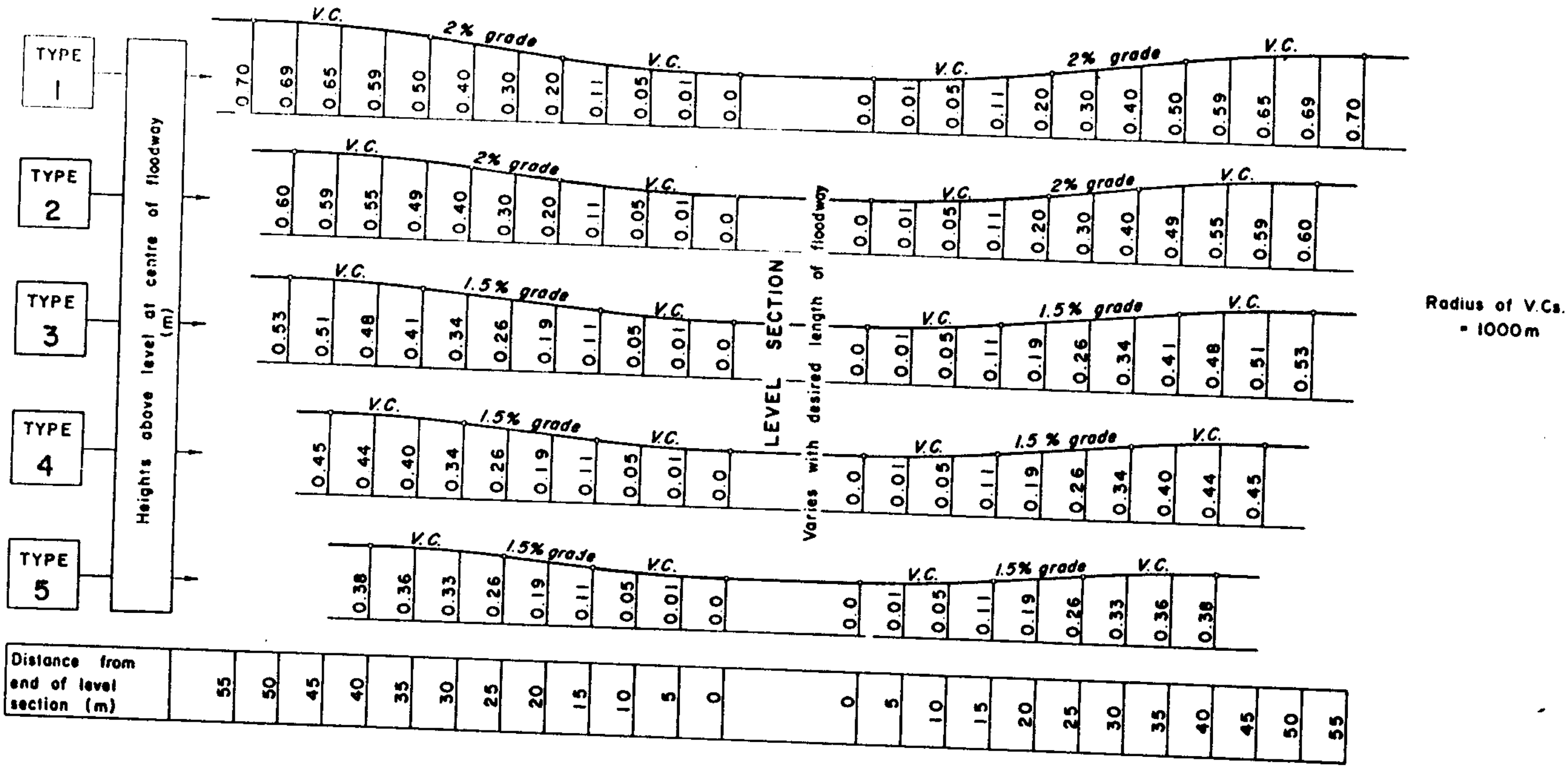
Alternatively in terms of (L)

$$\boxed{L = 0.32 \frac{Q}{(y)^{3/2}}}$$

4.6 CAUSEWAYS

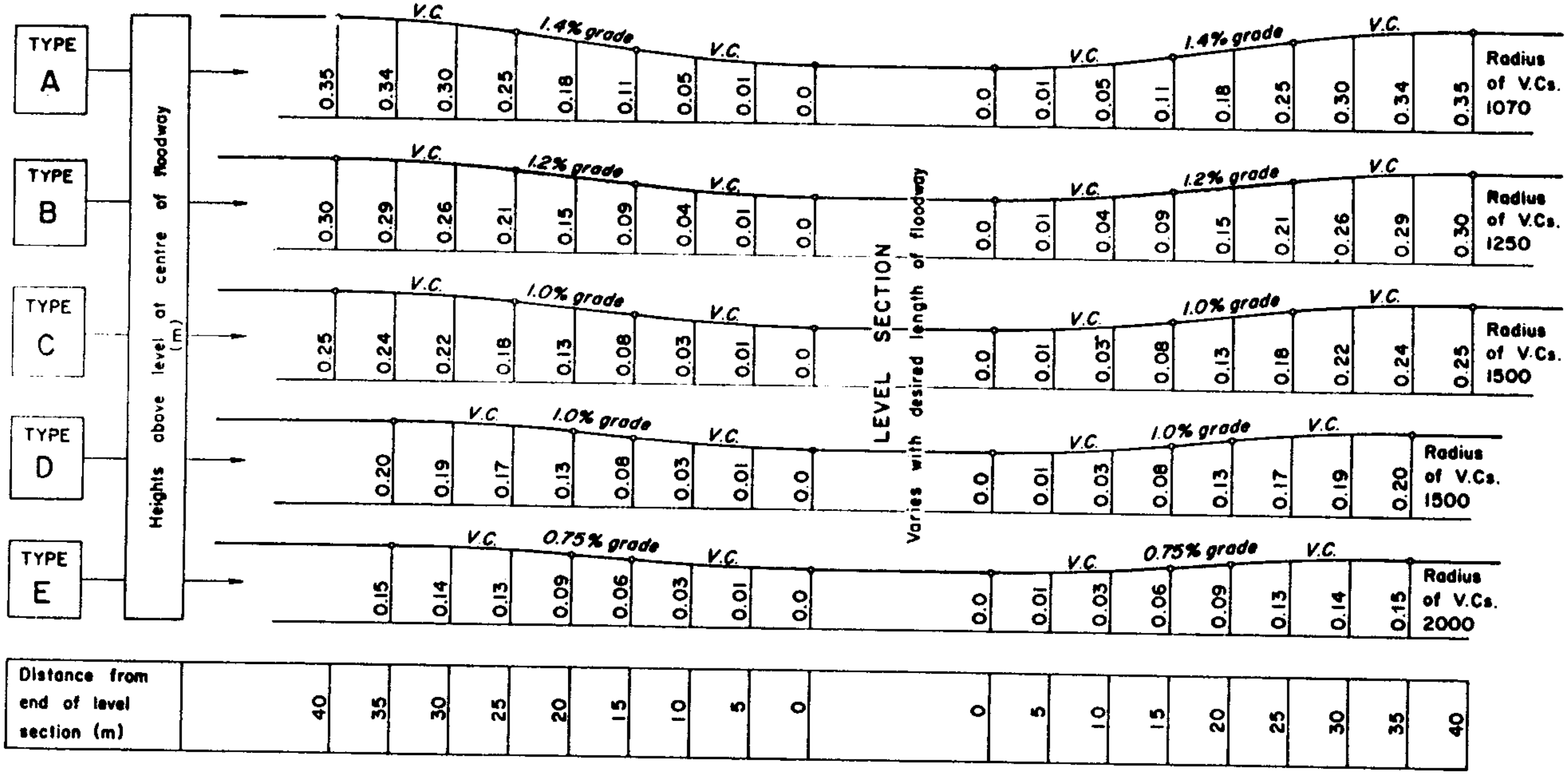
Where nominal flows only are catered for with a causeway structure, construction shall be in accordance with S.W.D.S.# "Standard Causeway and Floodway Details".

The following diagrams give ten (10) alternative profiles for causeway and approach grade lines.



STANDARD FLOODWAYS

Fig. 4.5



STANDARD FLOODWAYS IN
LOW FORMATION

SECTION 5 - OPEN CHANNEL FLOW

5.1 GENERAL

Open channel flow calculations may be required to determine stage-discharge curves for culvert tailwater depths, or to determine channel geometry for inlets and outlets to hydraulic stuctures to give acceptable flow velocities without causing erosion in the channel.

The following procedures are based on Manning's Formula and are the preferred design method.

5.2 MANNING'S FORMULA

The following formula is an empirical relationship of uniform flow:-

$$V = \frac{R^{0.667} S^{0.5}}{n}$$

from which $Q = AV$

where V = velocity (m/sec)

n = Manning's Roughness co-efficient

A = cross-sectional area of flow (m^2)

R = hydraulic radius $= \frac{\text{Area}}{\text{Wetted Perimeter}}$

S = slope of channel bed (m/m)

Q = flow (m^3/sec)

Solutions to this formula in nomograph form, may be obtained from Chart 5.1.

5.3 MANNING'S ROUGHNESS CO-EFFICIENT (n)

In applying this formaula, the roughness co-efficient "n" may be obtained from the tables 5.1 and 5.2 and the following copies of photographs.

5.4 MAXIMUM AND MINIMUM VELOCITY

(a) Maximum Velocity - refer to table in Section #6.2

(b) Minimum Velocity - The minimum non-silting velocity in the channel shall not fall below 0.5m/sec. at the Design Flow (Q_D).

n = 0.03

n = 0.040 – 0.045

n = 0.05

n = 0.06

n = 0.07

n = 0.08

Note: (a) Increase in grass, weeds, shrubs and trees with increase in "n" value.

(b) In general, growth of trees in photographs tend to look more dominant than when seen on a site inspection.

Natural streams in Queensland

ARTIFICIAL CHANNELS	
Conditions of Structure	n
1. Closed Conduits	
(a) Concrete pipe	0.012
(b) Corrugated metal pipe or pipe-arch (small corrugation)	
(i) plain or unpaved	0.024
(ii) paved invert - full flow	
25% circumference paved	0.021
15% circumference paved	0.018
Fully paved	0.012
(c) Structural plate pipe or pipe-arch	0.030-0.033
(d) Monolithic concrete (box culvert)	0.012
(e) Vitrified clay pipe	0.012
2. Open Channels - Lined	
(a) Concrete - Smooth forms or trowelled	0.012
(b) Bituminous Concrete	
(i) Smooth	0.013
(ii) Rough	0.016
3. Open Channels - Excavated (Straight alignment and natural lining)	
(a) Earth - uniform section	
(i) clean - new to weathered	0.016-0.020
(ii) with short grass, few weeds	0.022-0.027
(iii) in gravelly soil, clean	0.022-0.025
(b) Earth - fairly uniform section	
(i) no vegetation	0.022-0.025
(ii) grass, some weeds	0.025-0.030
(iii) dense weeds or plants in deep channel	0.030-0.035
(iv) sides clean, gravel bottom	0.025-0.030
(v) sides clean, cobble bottom	0.030-0.040
(c) Dragline excavated or dredged	
(i) no vegetation	0.028-0.033
(ii) light bush on banks	0.035-0.050
(d) Channels not maintained, weeds and brush uncut	
(i) dense weeds, high as flow depth	0.08-0.12
(ii) clean bottom, brush on sides	0.05-0.08
(iii) same, highest stage of flow	0.07-0.11
(iv) dense brush, high stage	0.10-0.14

TABLE 5.1

NATURAL CHANNELS		
Type of Channel		n
Main Channel		
1. Fairly Regular Section		
(a) Some grass and weeds, little or not brush		0.030-0.035
(b) Dense growth of weeds, depth of flow materially greater than weed height		0.035-0.05
(c) Some weeds, light brush on banks		0.035-0.05
(d) Some weeds, heavy brush on banks		0.05-0.07
(e) Some weeds, dense willows on banks		0.06-0.08
(f) Trees within channel with brances sub-merged at high stage	Add	0.01-0.02
2. Irregular Section, with pools, slight channel meander		
To (a) to (f) above as applicable	Add	0.01-0.02
3. Mountain Streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage		
(a) Bottom, gravel, shingle and few boulders		0.04-0.05
(b) Bottom, shingle with large boulders		0.05-0.07
Adjacent Flood Channels		
1. Pasture, no Brush		
(a) Short grass		0.030-0.035
(b) High grass		0.035-0.05
2. Cultivated Areas		
(a) No crop		0.03-0.04
(b) Mature row crops		0.035-0.045
(c) Mature field crops		0.04-0.05
3. Heavy Weeds, Scattered Brush		0.05-0.07
4. Light Brush and Trees		0.06-0.08
5. Medium to Dense Brush		0.10-0.16
6. Dense Willows, Summer, Not Bent Over By Current		0.15-0.20
7. Cleared Land with Tree Stumps (250 to 450 per hectare)		
(a) No sprouts		0.04-0.05
(b) With heavy growth of sprouts		0.06-0.08
8. Heavy Stand of Timber, A Few Fallen Trees Little Undergrowth		
(a) Flood depth below branches		0.10-0.12
(b) Flood depth reaches branches		0.12-0.16
Major Streams (Surface width at flood stage>30m)		
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of "n" may be somewhat reduced. The value of "n" for larger streams of mostly regular section, may be in the range		0.028-0.33

TABLE 5.2

5.5 TAILWATER DEPTH CALCULATION

(a) General

For culverts with outlet control, it is important that the tailwater level be calculated as accurately as possible. One possible exception is when the tailwater is known to be well below the critical depth at the culvert outlet.

For outlet channels that are reasonably uniform in cross-section, slope and roughness, the depth of flow can be derived from Manning's formula.

For streams with varying cross-sections such as main channel and flood plain, reference should be made to Chapter 4 of "Australian Rainfall and Runoff" (1987).

(b) Design Checks

A change in the design tailwater level can occur due to backwater from another creek or obstructions downstream, or even from the use of too low an "n" value in analysing the channel. The culvert design must be checked for these possibilities as a higher tailwater may result in unacceptable headwater levels.

(c) Stage-Discharge Curve and Tailwater Depth

By first deriving a simple stage-discharge curve for the channel, the tailwater depth can be determined using the Design Flow (Q_D) as per the following procedure and example.

(i) Required information

* Channel cross-sectional geometry
* Channel slope
* "n" value
* Design flow Q_D

(ii) Assume a depth of flow in the channel and determine:-

* Water Way Area (A =)
* Wetted Perimeter (WP)
* Hydraulic Radius ($R = \frac{A}{HP}$)

(iii) Calculate the flow velocity (V) from Manning's formula or nomograph Fig.5.1.

(iv) Calculate the discharge $Q = VA$

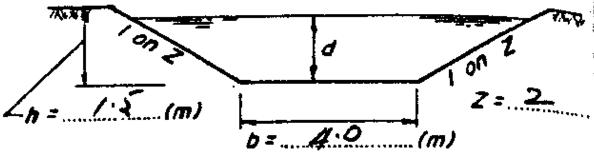
(v) Repeat the above procedure so that a curve can be drawn.

(vi) Using this curve and the design flow (Q_D), and check flow (Q_c), determine the tailwater depths.

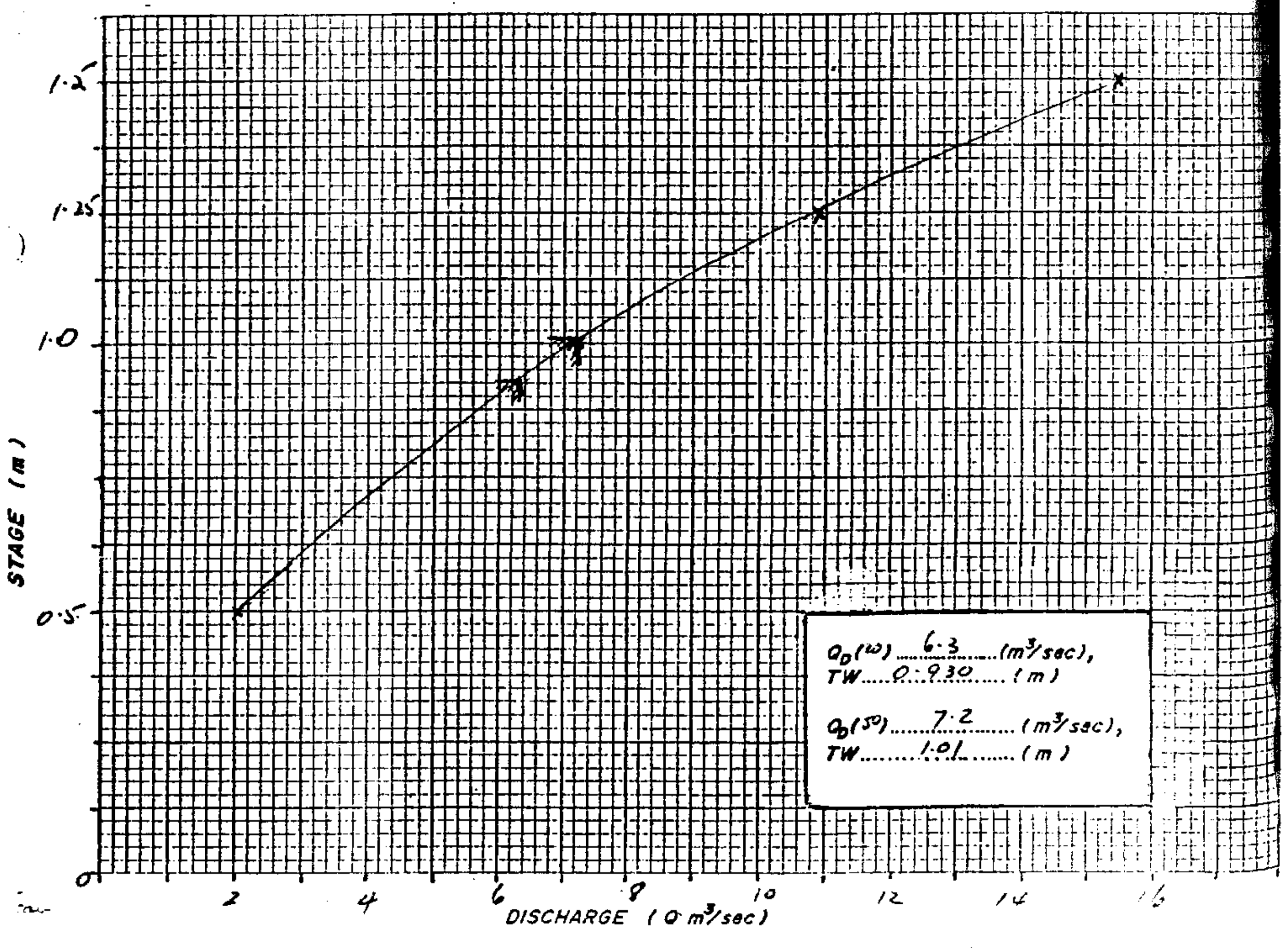
-o0o-

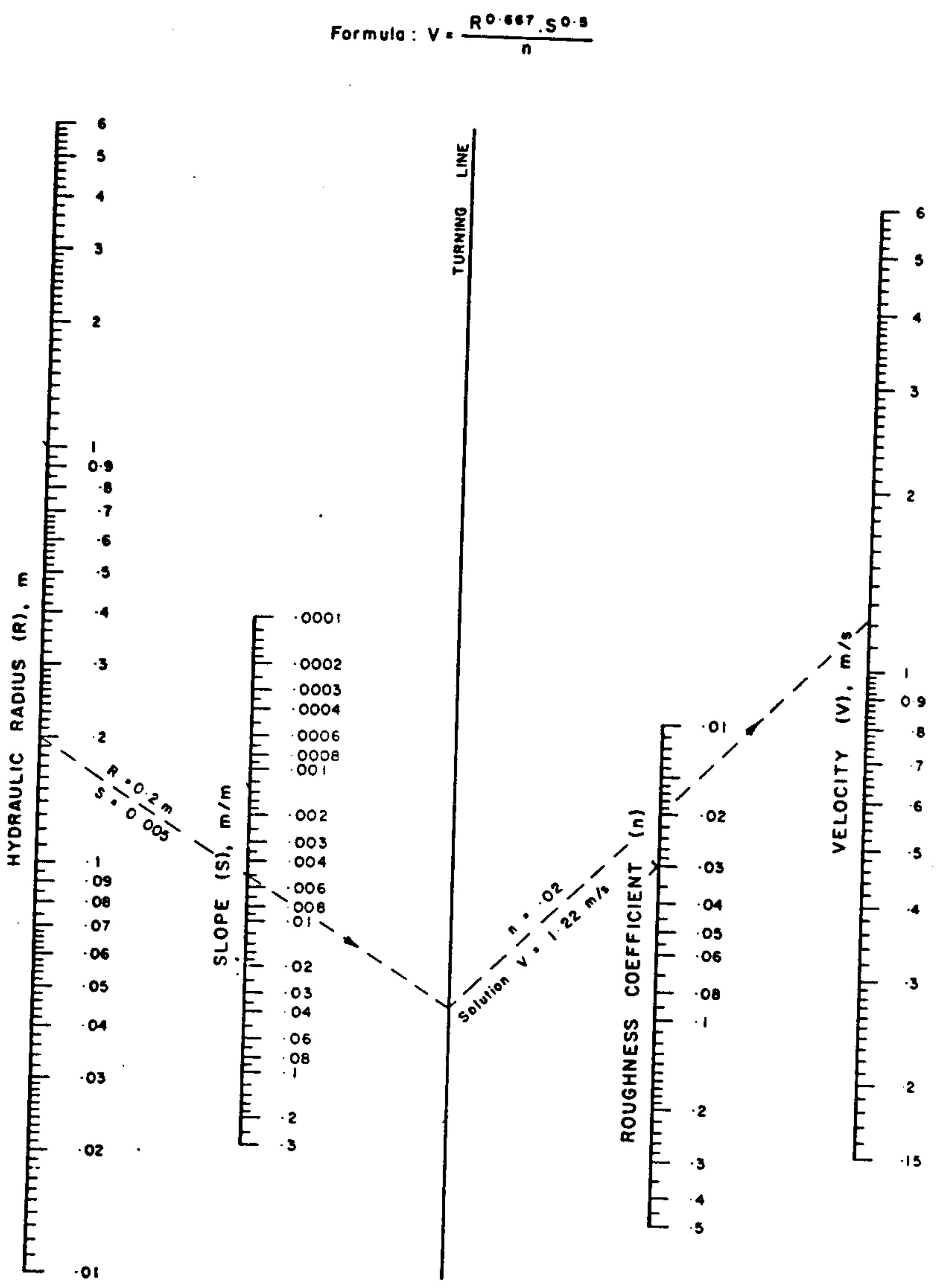
Name *Example* Chainage	OPEN CHANNEL STAGE - DISCHARGE	DESIGNED .... DATE .... CHECKED .... DATE ....

DESIGN FLOW Q_D (20) ....*6.3*.... (m^3/sec)
CHECK FLOW Q_C (50) ....*7.2*.... (m^3/sec)
CHANNEL SLOPE S = ...*0.002*.... (m/m)
MANNINGS ('n') ....*0.03*....



TRIAL	FLOW AREA	WETTED PERIM.	HYD. RADIUS	VELOCITY		COMMENTS
d	$A = bd + zd^2$	$W.P. = b + 2d\sqrt{1+Z^2}$	$R = A/W.P.$	$V = \frac{R^{0.667} S^{0.5}}{n}$	$Q = VA$	
(m)	(m^2)	(m)	(m)	(m/sec)	(m^3/sec)	
0.5	2.5	6.24	0.40	0.81	2.02	
1.0	6.0	8.47	0.71	1.18	7.11	
1.25	8.13	9.59	0.85	1.33	10.85	
1.5	10.50	10.71	0.98	1.47	15.45	





Analysis of uniform flow, Manning's formula

Chart 5.1

SECTION 6 - EROSION PROTECTION

6.1 GENERAL

Natural waterways often have erodable banks and bottoms, however those which have mild slopes are reasonably stable and will remain so unless flow velocities and existing natural protection is changed.

Hydraulically efficient structures generally operate at increased velocities to that in a natural waterway and any construction works will disturb existing natural protection for varying periods of time after its completion.

6.2 MAXIMUM PERMISSIBLE FLOW VELOCITIES

The maximum permissible velocities depend on the amount of grass cover and the soil type or particle size.

Where natural re-vegetation will ultimately provide sufficient stability against erosion, interim protection will usually be necessary. This may take the form of turfing the bottom of drains for the full width and part way up the batters, top soiling and planting a fast growing (2 weeks) cover to stabilise and protect the slower growing ultimate cover, or an open-jointed riprap filled with top soil and seeded.

The method used will depend on the time of year the works are carried out and also the natural soil type.

If natural re-vegetation will not provide adequate protection in the long term, other treatments will be necessary. The following table is to be used as a guide to determine the extent and/or combination of necessary protection works.

SECTION 6.2 (Cont.)

Cover Type/Material	Maximum Permissible Velocity
Well established grass providing full cover to soil	2.0-2.4m/sec
Meadow type grass with short pliant blades	1.5-1.8m/sec
Bunch grass with exposed soil between plants	0.6-1.2m/sec
Stiff-stemmed grasses that do not bend over under shallow flow	0.6-0.9m/sec
Earth without vegetation	
Silt, Clay, Sand	0.3m/sec
Gravel (6mm)	0.6m/sec
Gravel (25mm)	0.9m/sec
Firm Loam Soil	1.2m/sec
Gravel (50mm)	1.8m/sec
Boulders (100mm)	2.4m/sec
Boulders (150mm)	3.0m/sec
Boulders (300mm)	4.5m/sec

TABLE 6.1

6.3 GRASS LINED CHANNELS

(a) Maximum Velocities - in grass lined channels should be kept within reasonable limits to avoid scour problems.

(b) Channel Slope - should be from 0.3% to 0.6%, however, when the natural slope of ground greater than this, slope drops should be used.

(c) Curves - should be as large as possible, however as a guide, the minimum radius at the centre line of a curve shall be equal to twice the width of the top water level at design flow, with minimum radius of 30m.

(d) Cut-off Walls - at regular intervals in a grass lined channel is desirable, as they will safeguard against serious erosion in large run-offs or before the grass cover is sufficiently established.

SECTION 6.3 (Cont.)

These cut-off walls are usually reinforced concrete approximately 200mm wide and 450 to 600mm deep, extending the fall width of the channel bottom.

6.4 RIPRAP

(a) Locations - Riprap can be used upstream and downstream of hydraulic structures, on the banks, bends and bottoms of channels and other areas where erosion tendencies exist such as slope drops.

(b) Layer Thickness - should be about 1.5 times the largest size of rock used.

(c) Rock Size - the following graph indicates the size of rock to be used. Bottom velocity can be estimated as 0.7 of the average channel velocity.

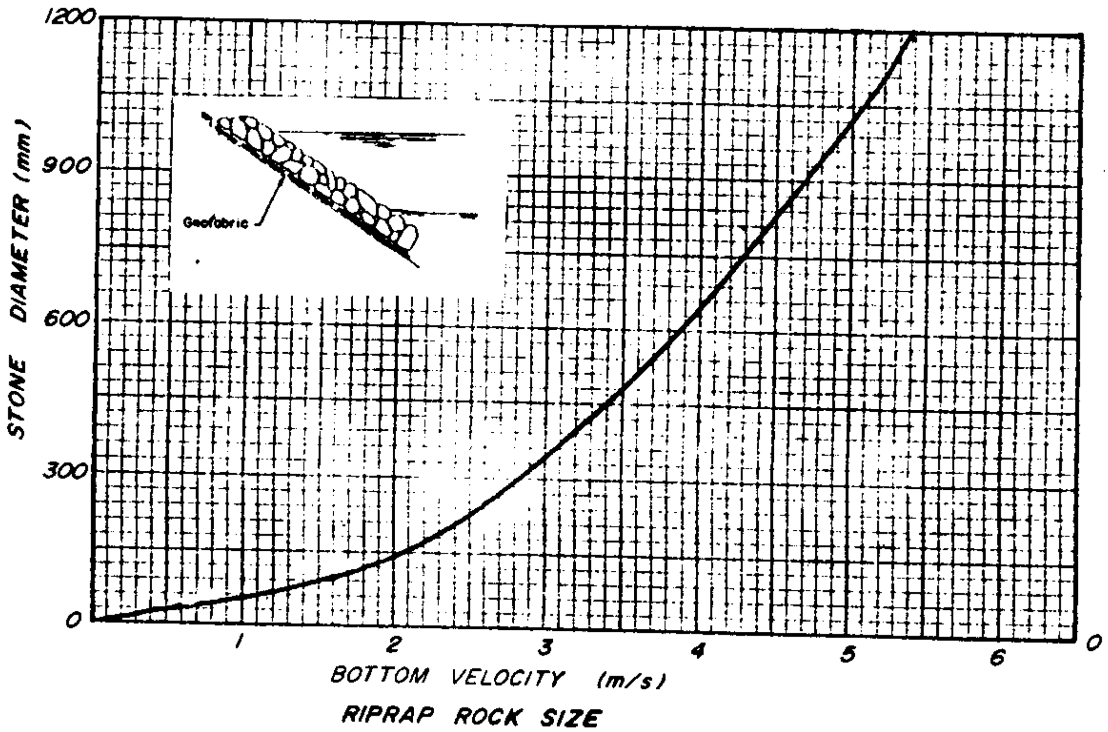


Fig. 6.1

(d) Disadvantage - this type of protection can present a maintenance problem.

6.5 STONEPITCHING

(a) Location - stone pitching can be used upstream and downstream of hydraulic structures, on the banks, bends and bottoms of channels and other areas where erosion tendencies exist such as slope drops.

(b) Rock - should be between 150-500mm in size and placed to form irregular joints. All rocks are to be interlocked and wedged with smaller size rock as necessary, so that no single rock may be easily dislodged and no large voids remain between rocks.

(c) Cement Mortar - shall be 10MPa/20 concrete.

(d) Disadvantage - this type of protection usually cracks and allows water under it to cause hidden erosion resulting in failure.

6.6 GABIONS

(a) Location - gabions can be used upstream and downstream of hydrailic structures, on the banks, bends and bottoms of channels and other areas where erosion tendencies exist such as slope drops.

(b) Advantages - smaller rock sizes can be used because the wire basket tends to make the mat act as a unit, but still having some flexibility.

(c) Materials - shall comply with Main Roads Specification M.R.S.11.03 Drainage, Retaining Structures and Protective Treatments.

6.7 CONCRETE LINED CHANNELS

(a) General - Concrete lining of channels is not normally required where the flow is sub-critical.

(b) Supercritical Flows - care should be taken to ensure that:-

(i) curves, which create energy problems are kept to a minimum;

(ii) the channel is free from obstructions which may cause oscillatory waves which extend down the channel (e.g. pipe entering should be cut flush);

SECTION 6.7 (Cont.)

(iii) construction joints are dowelled to prevent differential movement between sections. This would allow high velocity flows to enter the joints causing rapid scouring and high uplift forces and can lead to the failure of the channel lining;

(iv) unintended hydraulic jumps do not form.

(c) Materials

(i) Concrete shall be 20MPa/20 and a minimum thickness of 100mm.
(ii) Reinforcing shall be F62 mesh.

(d) Construction Joints - shall be at a maximum spacing of 20m.

(e) Weep Holes - shall be 90mm diameter spaced at 1800 crs both horizontally and vertically, with 300 x 300 x 150 no fines concrete blocks or geofabric backing at each hole.

Large channels may require a sub-soil drain to prevent hydrostatic uplift forces.

-o0o-

PART II

URBAN CATCHMENTS

SECTION 7 - URBAN DRAINAGE DESIGN BASIS

7.1 GENERAL

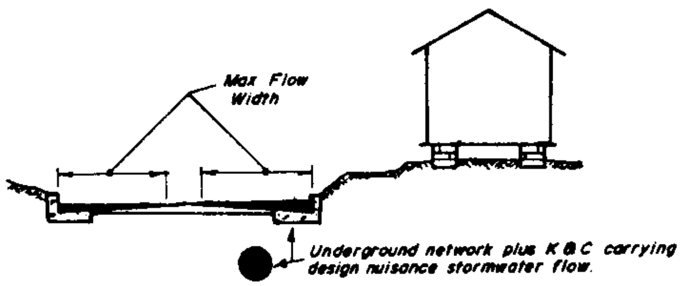
The following procedures are applicable to small urban drainage systems having a catchment area of less than 20 hectares and a rainfall/runoff time of 30 minutes or less.

The design of the more complex drainage systems for large urban areas, especially those involving storage or detention basins and hydrographs requiring the use of computer models are outside the intended scope of these procedures, however all urban drainage systems shall be designed on the basis of providing a dual system in accordance with the following.

7.2 MINOR SYSTEM

The Minor System consists of the kerb and channel, gully inlets and underground pipe network designed to totally cater for stormwater flows from storms up to a specified A.R.I. in the order of 1 in 5 or 1 in 10 depending on the development.

Generally the Minor System is designed on a nuisance or inconvenience basis, such that the stormwater is wholly contained on the roadway between the kerb and channel, and having a maximum specified width or depth of flow down a roadway (Fig. 7.1) or around a corner



MINOR DRAINAGE SYSTEMS

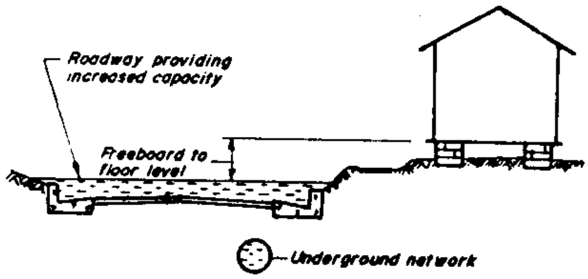
Fig. 7.1

Design A.R.Is for the minor system is given in Table 7.1.

7.3 MAJOR SYSTEM

The Major System consists of the Minor System plus roadway and overland flow paths (and detention basins) which cater for stormwater flows from storms with greater A.R.Is, in the order of 1 in 50 or 1 in 100 depending on the development.

Generally the Major System provides increased capacity by using for example, the full width of a roadway and some footpath at a greater depth of flow, plus overland flow paths.



MAJOR DRAINAGE SYSTEMS

Fig. 7.2

Design A.R.Is for the Major Systems are given in Table 7.2.

7.4 AVERAGE RECURRENCE INTERVALS (A.R.I.)

Average recurrence intervals for both the Minor and Major Systems shall be in accordance with the following tables.

Development	Minor System Design A.R.I.
Open Space/Recreation Areas	1 in 2 years
Residential	1 in 5 years
Commercial Industrial	1 in 10 years

Minor System Design A.R.I.

TABLE 7.1

SECTION 7.4 (Cont.)

Development	Minor System Design A.R.I.
All	1 in 100 years

Major System Design A.R.I.
TABLE 7.2

In addition to the above, specific important public premises shall be in accordance with the following:-

Development	Rare Flood Design A.E.P.
Floor levels of hospitals	1 in 500 years

7.5 DESIGN PROCEDURE

The design procedure is based on the following three steps which are explained in more detail in the relevant section.

Step 1 - Major System (Section 8) - investigates the roadway and overland flow routes and capacities based on the "gap" flow concept.

Step 2 - Minor System Surface Drainage (Section 9) - determines the runoff quantities via overland and gutter paths with gully interception based on maximum roadway flow widths or depths.

Step 3 - Minor System Pipe Network (Section 10) - determines the design of the underground pipe network and manholes based on the gully interception quantities.

(BLANK)

SECTION 8 - MAJOR SYSTEM DESIGN

8.1 DESIGN BASIS

By taking into account the capacity of the Minor System, the capacity of the overland flow component of the Major System can be derived. For example, a 1 in 100 A.R.I. capacity of the Major System could be made up of a Minor System with a 1 in 5 (viz) A.R.I. capacity on its own, plus an overland flow system with a 1 in 5 (viz) A.R.I. capacity on its own. This is the basis of the gap flow concept as derived in Section #8.3.

Design of the Major System essentially compares the expected major flow quantities with the capacities of the proposed roadways and is predominantly concerned with identifying areas where this roadway capacity is exceeded, especially if adjacent to properties which are lower than the road level.

Having identified such problem areas, the designer can investigate alternative solutions, for example, increasing the Minor System A.R.I. at that location during the detailed design of the pipe network in the next step.

Other options include increasing the roadway grade or width to increase capacity, allotment fill earthworks, alter the road network and allotment layout to provide open space and open channel flow, etc.

8.2 DESIGN GUIDELINES

Without limitation, the design of the Major System shall conform to the following:-

(a) Roadway lower than the allotments on both sides - (e.g. the allotments on both sides of the road drain to the road)

(i) Maximum flow depth at kerb and channel invert d_g = 200mm

(ii) Maximum flow width = road reserve boundary
(e.g. no flooding of the property)

(iii) Product of (kerb depth) x (average flow velocity)
= $0.40m^2$/sec. max.

SECTION 8.2 (Cont.)

(b) Roadway higher than the allotment on one side (the allotments on one side only drain to the road)

(i) Maximum flow depth at kerb and channel invert on the high side d_g = 200mm

(ii) Maximum flow width on the high side = road reserve boundary (e.g. no flooding of property)

(iii) Maximum flow depth at kerb and channel invert on the low side d_g = 125mm (e.g. no overtopping of channel)

Note:- The above criteria allows for flow from the high side across the road crown to the low side.

(iv) Product of (kerb depth) x (average flow velocity) = 0.40m^2/sec. max.

(c) Roadway higher than the allotments on both sides

(i) Maximum flow depth at kerb and channel invert d_g = 100mm

(ii) Maximum flow width = between kerb and channel only (e.g. confined to the roadway, no footpath flow)

(iii) Product of (kerb depth) x (average flow velocity) = 0.40m^2/sec. max.

(d) Open Space and Recreational Areas - the flow shall be contained within the area boundary.

(e) Unacceptable Practices - The following practices to increase roadway capacities will not be acceptable.

(i) Small levees on the footpath to contain major flows, unless constructed in accordance with Section 8.7.

(ii) Reducing the roadway crossfall below the standard minimum specified in Calliope Shire Council Road Design Standard.

SECTION 8.2 (Cont.)

(f) Detention Basins

(i) All detention basins shall be dry types.

(ii) The spillway shall be designed to cater for A.R.I. flows of 1 in 100 years without causing flooding to upstream private property.

(iii) Where the drainage system of an upstream subdivision joins the drainage system of a lower subdivision, an area of open space shall be set aside for the future construction of a detention basin. This area may be included as part of the open space area required by Council's Subdivisional By-Law.

As a guide, the area to be set aside can be estimated by the following formula.

$$V = 0.2\ At_v(I_{100} - I_1)$$

where V = Volume of storage in the detention basin (m^3) (from which the Area can be derived having regard to the developed levels in the surrounding area, the invert level of existing outlet pipework and an allowance of 1:6 for bank slopes).

A = Catchment Area (h_a)

t_v = Times of Concentration (mins) - (used to derive various I_{100} values as per the following example)

I_{100} = Intensity for ARI = 100 yrs using (t_v)(mm/hr)

I_1 = Intensity for ARI = 1 year using the time of concentration of the catchment (mm/hr)

Example

It should be noted that the time of concentration (t_c) for the catchment may not yield the maximum required storage volume, therefore various values of (t_v) are used to derive the associated I_{100} values, and the equation solved to give

SECTION 8.2 (Cont.)

a range of storage volumes. The required area should be based on the maximum storage volume as per the following, which is best developed in tabular form.

Let Catchment Area A = 11.7 hectares
Time of Concentration = 14 minutes
Intensity (1 year) $I_{14,1}$ = 70mm/hour

$$V = 0.2 \times 11.7\ t_v\ (I_{100} - 70)$$

therefore $V = 2.34\ t_v\ (I_{100} - 70)$

t_v	I_{100}	$(I_{100}-70)$	$V = 2.34\ t_v(I_{100}-70)$
10	250	180	4212m^3
20	180	110	5148m^3
30	150	80	5616m^3
40	125	55	5148m^3

Let greatest volume = 5700m^3. The time of concentration giving the largest volume is about 30 minutes, which is much greater than the actual time of concentration of 14 minutes used by the Rational Method.

8.3 "GAP" FLOW DERIVATION

Let Major System A.R.I. = m years
and Minor System A.R.I. = n years

From the Rational Formula

$$Q_m = K\ C_m\ I_m \quad \text{and} \quad Q_n = K\ C_n\ A\ I_n$$

therefore $Q_{gap} = Q_m - Q_n = KA(C_m I_m - C_n I_n)$

$$Q_g = KA\ C_m \left(I_m - \frac{C_n}{C_m} I_n\right)$$

SECTION 8.3 (Cont.)

But $C_n = F_n\, C_{10}$ and $C_m = F_m\, C_{10}$

therefore $C_n/C_m = F_n\, C_{10}/F_m\, C_{10} = F_n/F_m = F'$ say

therefore $Q_g = KAC_m(I_m - F'\, I_n)$

for Q_g in l/sec. $K = \dfrac{1}{0.36}$

therefore $Q_g = \dfrac{CA(I_m - F'\, I_n)}{0.36}$

where
Q_g = design gap flow (l/sec)
C = Co-efficient of runoff from Table 8.4 for ARI = m years
A = Catchment Area (h_a)
I_m = Intensity for the Major System ARI = m years
I_n = Intensity for the Minor System ARI = n years
F' = Conversion factor derived from Table 8.5

The above is based on the minor system operating without any blockage. Allowance for the minor system not operating at full capacity can be easily catered for as follows:-

$$Q_{gap} = K\, CA(I_m - bF'\, I_n)$$

where b = blockage factor (b = 0 (fully blocked) to 1 (fully open))

It is reasonable to assume that some debris/plastic, etc. will be caught in gully inlets during a large storm and would be less visible with the large flow depths in the gutters.

A blockage factor of 0.5 is recommended, however with side inlet gullies it is felt that this can be increased to 0.75, (75% operational) resulting in the following equation.

$$Q_{gap} = K\, CA(I_m - 0.75\, F'\, I_n)$$

8.4 FLOW TIMES

(a) Sheet Flow Times - are obtained from the Kinematic Wave Equation.

$$t = \frac{6.94\ (L.n)^{0.6}}{I^{0.4}\ S^{0.3}}$$

where
t = overland sheet flow time
L = flow path length (m) (L = 200m maximum)
n = surface roughness co-efficient (similar but not identical to Manning's "n")
I = rainfall intensity (mm/hr)
S = slope (m/m)

As the equation involves intensity which is time based, it has to be solved iteratively, however this can be done directly by a table of values of ($t\ I^{0.4}$) for the various times of concentration and A.R.Is.

Using the flow length, slope and values of "n" from Table 8.1 and the equation:-

$$t\ I^{0.4} = \frac{6.94(L.n)^{0.6}}{S^{0.3}}$$

values of ($t\ I^{0.4}$) can be easily calculated, and the time of concentration can be interpolated directly from Table 8.2.

It should be noted that the above is based on the assumption of no inflow to the area of sheet flow, therefore it is incorrect to add values of ($tI^{0.4}$) for a number of upstream sections. Reference should be made to Chapter 14, "Australian Rainfall and Runoff" in such situations.

Example 1

For ($tI^{0.4}$) = 59 A.R.I. = 10 years
t_1 = 7 mins. V = 59, V_1 = 57, V_2 = 63
therefore $t = 7 + \frac{59-57}{63-57} = 7.3$ mins.

Example 2

For $(tI^{0.4}) = 38$ A.R.I. = 10 years

$t_1 = 5$ mins., $V = 38$, $V_1 = 42$, $V_2 = 50$

therefore $t = 5 + \left(\frac{38 - 42}{50 - 48}\right) = 3.0$ mins.

This estimation is acceptable when the sheet flow time is a component of the overall flow time, however the minimum total time of concentration cannot be less than 5 minutes.

(b) Sheet Flow Length Limitation

The time adopted for travel in large pervious areas must recognise the limits of the overland flow phenomenon. It is a matter of field observation that "sheet" flow rarely travels more than 200m before entering a "wash" which then falls into the natural channel category.

(c) Overland Channel and Gutter Flow Times

Estimates of overland flow times are not highly accurate, and the gutter flow times to be added to these need not be calculated precisely. In many cases, the gutter flow times need only be estimated to the nearest half minute.

Where flow paths are mostly along gutters, it is likely that the total travel time will be less than 5 minutes, the shortest duration for which rainfall intensity data are available.

In Rational Method calculations it is customary to adopt a minimum duration of 5 minutes, so it is not necessary to calculate short times precisely.

Both overland and gutter flow times can be estimated from Chart 8.1.

(d) Roadway Flow Times

Roadway flow times can be estimated more accurately from Charts 8.3 and 8.4. It is suggested that these be used for flow times down hill of the first location where roadway capacities are investigated.

SECTION 8.4 (Cont.)

(e) Roof-to-Gutter Flow Times

Considerable uncertainty must surround roof-to-gutter flow times. In cases where roof water from both residential and commercial/industrial buildings is conveyed directly to the gutter or to a rear-of-allotment open channel, recommended travel times are given in Table 8.3. These times refer to the roof-to-drain times only, and gutter or channel flow times are to be treated as separate elements.

8.5 RUNOFF CO-EFFICIENTS

Weighted Runoff Co-efficients for the various land uses are given in Table 8.4. These values include allowances for impervious area fractions (f) and A.R.I. frequency factors.

8.6 EQUIVALENT IMPERVIOUS AREA

Sub-catchments having multiple land uses are catered for by adding the equivalent impervious areas of each land use within the sub-catchment. For example, if a sub-catchment has three land uses having runoff co-efficients of C_1, C_2, and C_3 and areas A_1, A_2, and A_3, the equivalent Impervious Area is given by:-

$$\Sigma CA = C_1A_1 + C_2A_2 + C_3A_3$$

where the total sub-catchment area $A = A_1 + A_2 + A_3$

In this manner, the equivalent impervious areas (ΣCA) of successive sub-catchments can be accumulated as design proceeds down the catchment.

8.7 INTERSECTION DESIGN CHECK

(a) Levee Height - If a flow is moving down-slope towards a "T" intersection and was abruptly stopped by a pond of water, the height of a levee which would be required to contain the pond can be estimated by the formula:-

$$h = v^2/2g$$

where h = levee height above top-of-kerb (m)
v = velocity of flow (m/sec)
g = $9.81 m/sec^2$

(b) Levee Heights 0 to 75mm - the standard footpath cross-sectional grade of 2.5% (in 40) for a width of 3.0m in accordance with R.D.S.21 shall be maintained.

(c) Levee Heights 75mm to 230mm - Levees shall be constructed such that the required levee height is located at the property boundary and the standard footpath maximum cross-sectional grades in accordance with R.D.S.21 shall be maintained. In addition, fill earthworks shall extend a minimum of 6.0m from the boundary into the property.

(d) Levee Heights above 230mm - shall be referred to the Shire Engineer and each situation will be assessed on its merits.

(e) Levee Length - shall be twice the width of the terminating road (at the full levee height) and located such that half the levee length is directly opposite the terminating road and half the length is located on the down slope side.

(f) Ponding - In some situations it may be necessary to carry out fill earthworks to allotments to eliminate any potential ponding of water caused by levee construction.

8.8 WIDE ROAD DESIGN CHECK

During the design procedure it should be recognised that with wide pavements the maximum allowable flow depth may be reached before the water level is high enough to flow over the crown, and therefore only the half-roadway capacities can be used in design.

8.9 OVER-CROWN FLOWS

Where runoff enters the roadway from one side only and over-crown flows are expected, it is essential that the capacity of the kerb and channel on the opposite side is not surcharged if it fronts allotments. In such cases, the full width roadway flow capacity may be estimated by assuming the maximum flow level on the high side equal to the designed crown level, plus the standard flow depth limits for the low sides.

This procedure utilises the fact that crowns are constructed as slight curves rather than apex points, which would allow flows to "equalise" across the crown.

8.10 DESIGN PROCEDURE

The design procedure is aided by the use in the Design Chart as per the following explanation and example based on Fig. 8.1.

Column 1 - Location	- generally viewed as a cross-section perpendicular to the flow direction and various nomenclature can be used.
Column 2 - Flow Path Description	- describes the land use through which the runoff flows.
Column 3 - Flow Length	- to note the flow lengths of the various flow paths elements.
Column 4 - Roughness Factor	- Surface Roughness Value from Table 8.1.
Column 5 - Slope	- to note the slope of the various flow path elements.
Column 6 - $(tI^{0.4})$	- calculates values of $(tI^{0.4})$ for use with Table 8.2 to determine flow times (Column 7) for sheet flow only.

Column 7 - Flow Times (tmins)	- to note the individual flow times of the various elements of the flow path. (See Section 8.4).
Column 8 - Total Time	- is the sum of the individual flow times of the flow path elements in Column 7.
Column 9 - Time of Concentration	- is the greatest of the total times of the various flow paths in column 8.
Column 10 - Major System Intensity	- Rainfall Intensity from Chart 2.1 for the Major System A.R.I. using the value in Column 9.
Column 11 - Minor System Intensity	- Rainfall Intensity from Chart 2.1 for the Minor System A.R.I.
Column 12 - Blockage, Frequency Factor	- allows for 25% blockage, and frequency factor (f^1) from Table 8.5.
Column 13 - Gap Intensity	- is the difference between columns 10 and 12.
Column 14 - Runoff Co-efficient	- is obtained from Table 8.4 for the various land uses.
Column 15 - Contributing Area	- area of the various land uses in the catchment.
Column 16 - Equivalent Impervious	- product of Columns 14 and 15.
Column 17 - Calculative Impervious Area (ΣCA)	- allows for upstream impervious areas to be carried down for calculation.

SECTION 8.10 (Cont.)

Column 18 - Gap Flow - calculates the gap flow which is to be compared with Column 22

(Columns 19 to 22 are primarily intended for roadway flow capacity. Overland flow or open channel flow should be investigated separately and the (Q) value inserted in Column 22).

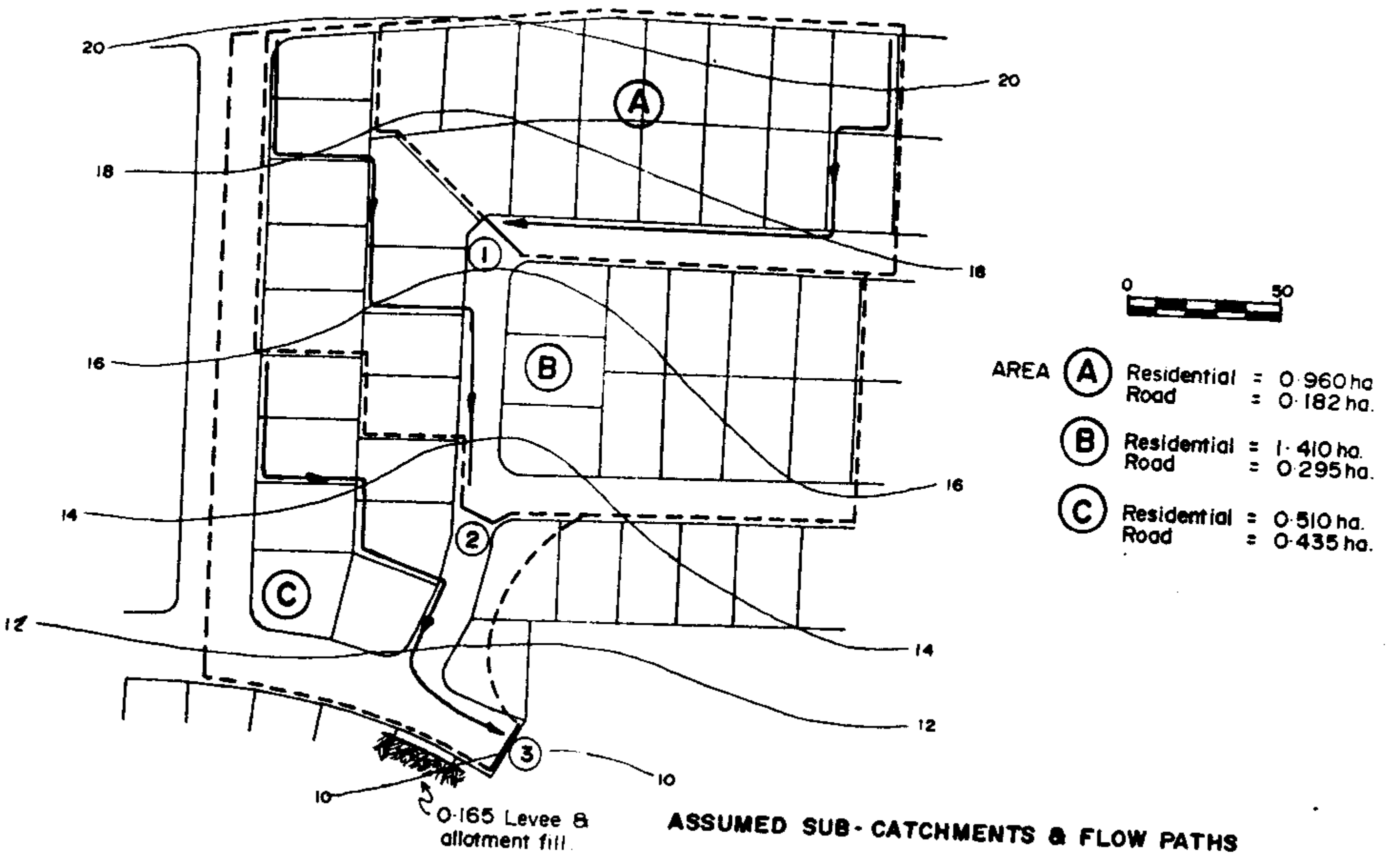
Column 19 - Escape Path Description - to note the roadway width and whether one or two sides can be considered in the capacity.

Column 20 - Maximum Allowable Flow Depth - is used for Chart 8.2. (See Section 8.2).

Column 21 - Roadway Grade - is also used for Chart 8.2.

Column 22 - Roadway Capacity - is derived from Chart 8.2 using Columns 20 and 21.

EXAMPLE



EXAMPLE CATCHMENT - MAJOR SYSTEM

Fig. 8.1

Major sytem ARI m 100 years
Major system ARI n 5 years

$F' = \frac{Fn}{Fm} = \frac{0.95}{1.20} = 0.79$
$f = 0.75F' = 0.59$ (Col. 12)

STORMWATER DRAINAGE

MAJOR SYSTEM DESIGN

Example

Sheet ... of ...
Designed ...
Checked ...
Signed ... / /

Description		Sub-Catchment Flow Times							Design Intensities				Design Flows					Escape Path Capacity				
(1) Location	(2) Flow Path Description (Land Use)	(3) Flow Length (L)	(4) Roughness Factor (n)	(5) Slope (S)	(6) $\frac{6.94 (Ln)^{0.6}}{S^{0.3}}$	(7) Component Flow Times (t)	(8) Total Times (t)	(9) Time of Concentration (tc)	(10) Major System Intensity (Im)	(11) Minor System Intensity (In)	(12) Block Freq factor (f) (0.59 In)	(13) Gap Intensity (Ig = Im - f In)	(14) Runoff Co-eff (C)	(15) Contributing Area (A)	(16) Equiv. Imp. Area (CA)	(17) Cumul. Equiv. Imp. Area (Σ CA)	(18) Gap Flow $Qg = \frac{(\Sigma CA) Ig}{0.36}$	(19) Escape Path Description	(20) Max. Allow. Flow Depth (dg)	(21) Roadway Grade	(22) Roadway Capacity QR	REMARKS
		m	–	m/m	mins mm/hr	mins.	mins.	mins.	mm/hr	mm/hr	mm/hr	mm/hr	–	ha	ha	ha	l/sec	–	m	%	l/sec	
(1)	Resid (A)	90	0.025	1:3/40 0.029	24	4.2							0.95	0.960	0.912							* high side depth crown height
	Road (A)	120	–	1.5/100		2.0	6.2	6.2	300	170	100	200	1.0	0.182	0.182	1.094	607	9m rd. 2 sides ✓	180*/125 ✓	1.5 / 3.0%	= 300 + 320 = 620 = 300 + 310 = 610	NB. - increase grade to 3%. O.K.
(2)	(1)→(2)	100	–	3/100 3%	100 m / 1.5 m/sec	1.1	7.3															
	Resid (B)	160	0.025	2.8/100 0.028	47	6.0							0.95	1.410	1.094 / 1.340							
	Road (B)	70	–	1.5/100	–	1.0	7.0	7.3	290	160	94	196	1.0	0.295	0.295	2.729	1486	8m rd. 2 sides	200/200	2.5	2200	O.K.
(3)	(2)→(3)	110	–	≈3%	110 m / 1.8 m/sec	1.0																
	(1)→(2)					7.3	8.3															
	Resid (C)	135	0.025	0.030	41	5.2							0.95	0.510	2.729 / 0.485							
	Road (C)	90	–	≈3%	40/7.8	≈1.0	6.2	8.3	275	150	89	186	1.0	0.435	0.435	3.649	1885	12m rd. 2 sides	180*/125	2.5	850 + 290 = 1140	NB. - other Options? Overland = 1885 - 1140 = 745.
Go Pipework ⇒ F' = 100/120 = 0.83, f = 0.83 × 0.75 = 0.63									275	180 × 0.63 = 113		162				3.649	1642					
Go Pipework ⇒ F' = 10/12 = 0.88, f = 0.88 × 0.75 = 0.66									275	200 × 0.66 = 132		143				3.649	1450	✓	180*/125	4%	1050 + 380 = 1430	

Option (1) - Go Pipework
Increase grade to 4%

Option (2) - 1.5 Pipework
Overland = 745 l/sec

CALLIOPE SHIRE COUNCIL

Surface Type	n
Paved Surface	0.015
Mown Surface	0.25
Thickly Grassed Surface	0.50

Overland Flow "n" Values

Table 8.1

Duration	A.R.I. (years)						
t(mins.)	1	2	5	10	20	50	100
5	33	36	40	42	45	48	51
6	38	43	47	50	53	57	59
7	44	49	54	57	60	65	68
8	49	54	60	63	67	72	76
9	54	60	66	70	74	80	84
10	59	66	73	76	81	87	91
11	64	71	79	83	88	94	99
12	69	76	84	89	94	101	106
13	74	82	90	95	101	108	113
14	78	87	96	101	107	115	120
15	83	92	102	107	113	121	127
16	88	97	107	113	120	128	134
17	92	102	113	118	126	134	141
18	97	107	118	124	131	141	147
19	101	112	123	129	137	147	154
20	105	116	128	135	143	153	160

Sheet Flow Times Related to Values of $tI^{0.4}$

Note:- (1) For use with the formula $t(I)^{0.4} = 6.94\frac{(L.n)^{0.6}}{S^{0.3}}$

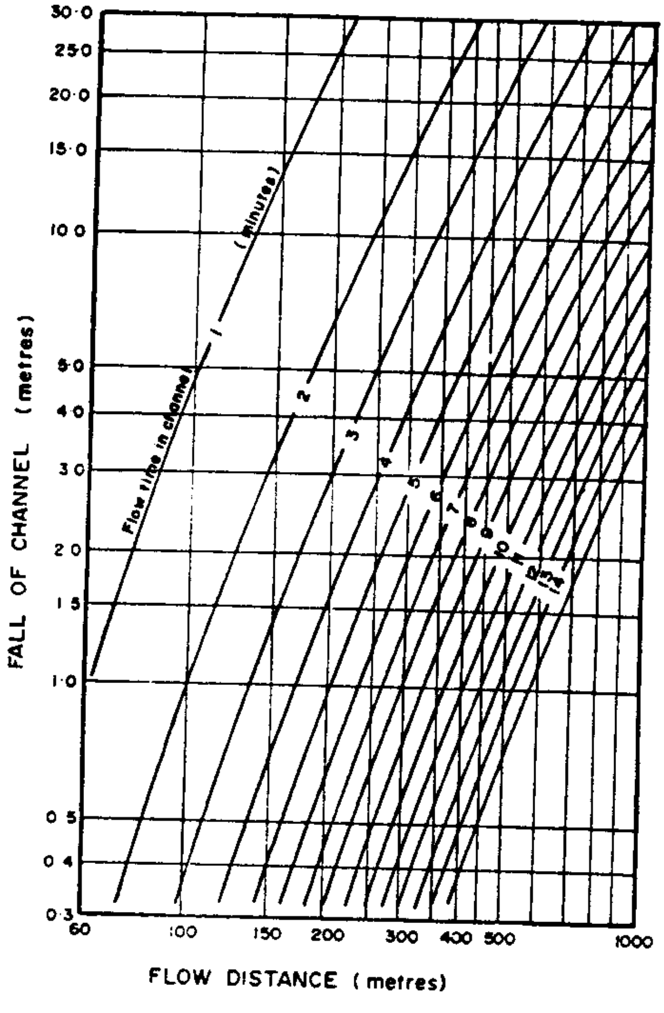
(2) Linear Interpolation $t = t_1 + \frac{(V-V_1)}{(V_2-V_1)}$

Table 8.2

Development Type		Roof-to-Drain Flow Times
Residential		5 minutes
Commercial/Industrial		
(a) Small	$< 2000m^2$	5 minutes
(b) Medium	$2000m^2$-$5000m^2$	10 minutes
(c) Large	$> 5000m^2$	15 minutes

Roof-to-Drain Flow Times

Table 8.3



NOTES:

1. Flow travel time (approximate) may be obtained directly from this chart for:
   * kerb-and-gutter channels
   * underground stormwater pipes

2. A multiplier, Δ, should be applied to values obtained directly from the chart in the following cases:
   * blade-cut earth roadside channels, well maintained and without driveway crossings $\Delta = 2$
   * natural channels $\Delta = 3$
   * grassed swales, well maintained and without driveway crossings $\Delta = 4$

Channel Flow Times

Chart 8.1

Land Use	Weight Runoff Co-efficients (C_w)	
	ARI = 50 years	ARI = 100 years
Parks/Open Space	0.8	0.85
Residential	0.9	0.95
Commercial/Industrial	1.0	1.0
Road Reserves (incl. footpaths)	1.0	1.0

Major System Weighted Runoff Co-efficients

Table 8.4

ARI (years)	1	2	5	10	20	50	100
Conversion Factor F_y	0.80	0.85	0.95	1.00	1.05	1.15	1.20

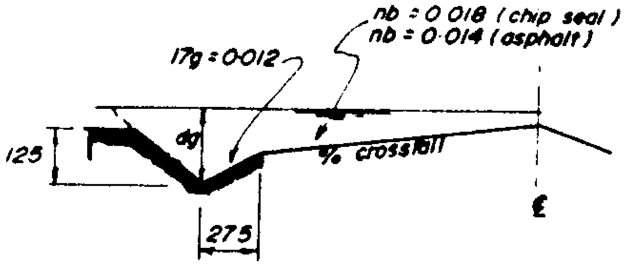
Example

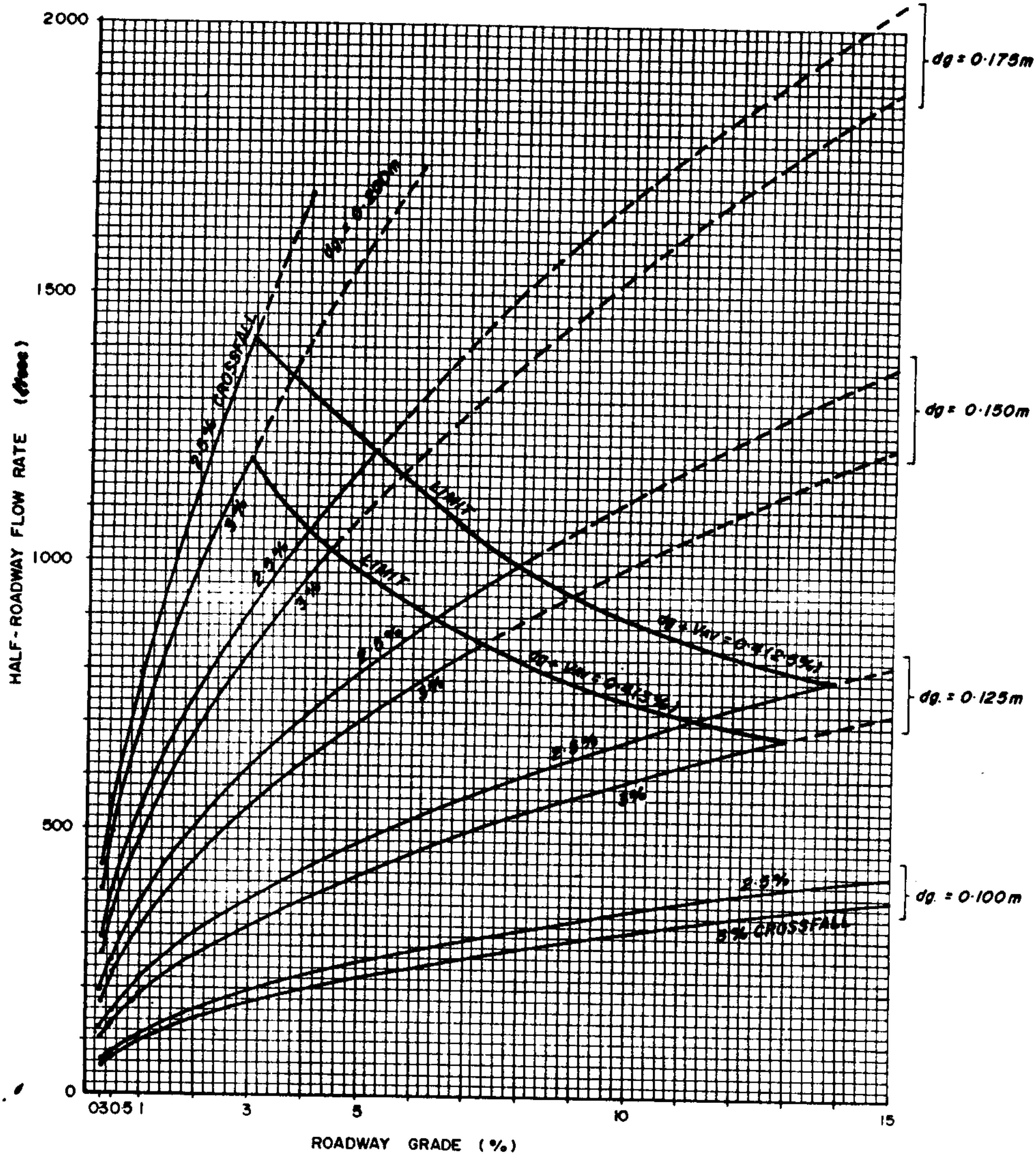
Major System ARI m = 100 years
Minor System ARI n = 5 years

$$F' = \frac{F_5}{F_{100}} = \frac{0.95}{1.20} = 0.79$$

Conversion Factors (F_y) for Various ARI's

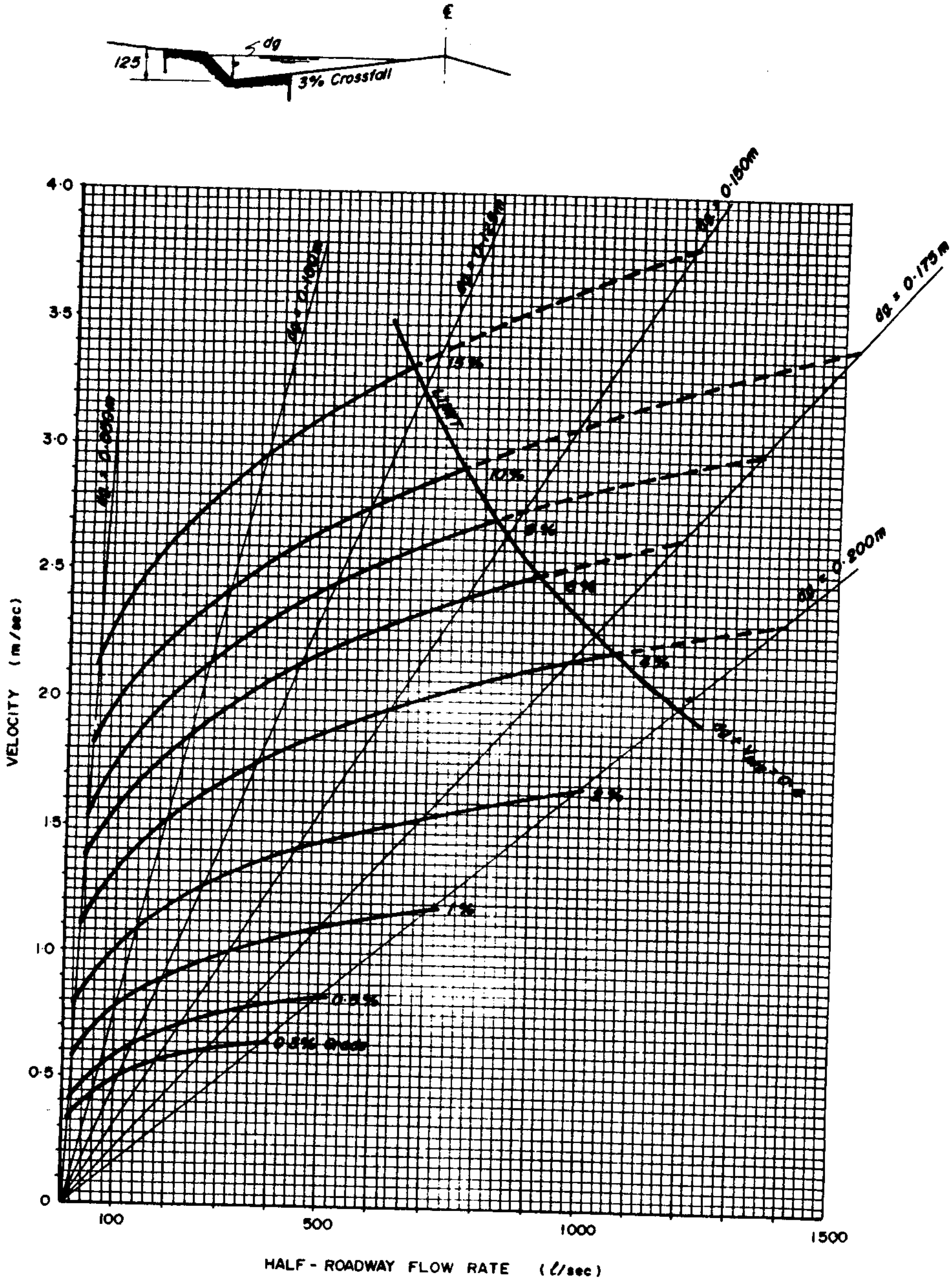
Table 8.5





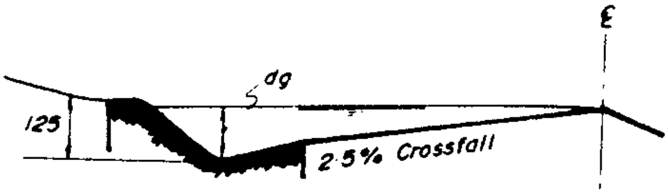
ROADWAY CAPACITIES - MAJOR SYSTEM

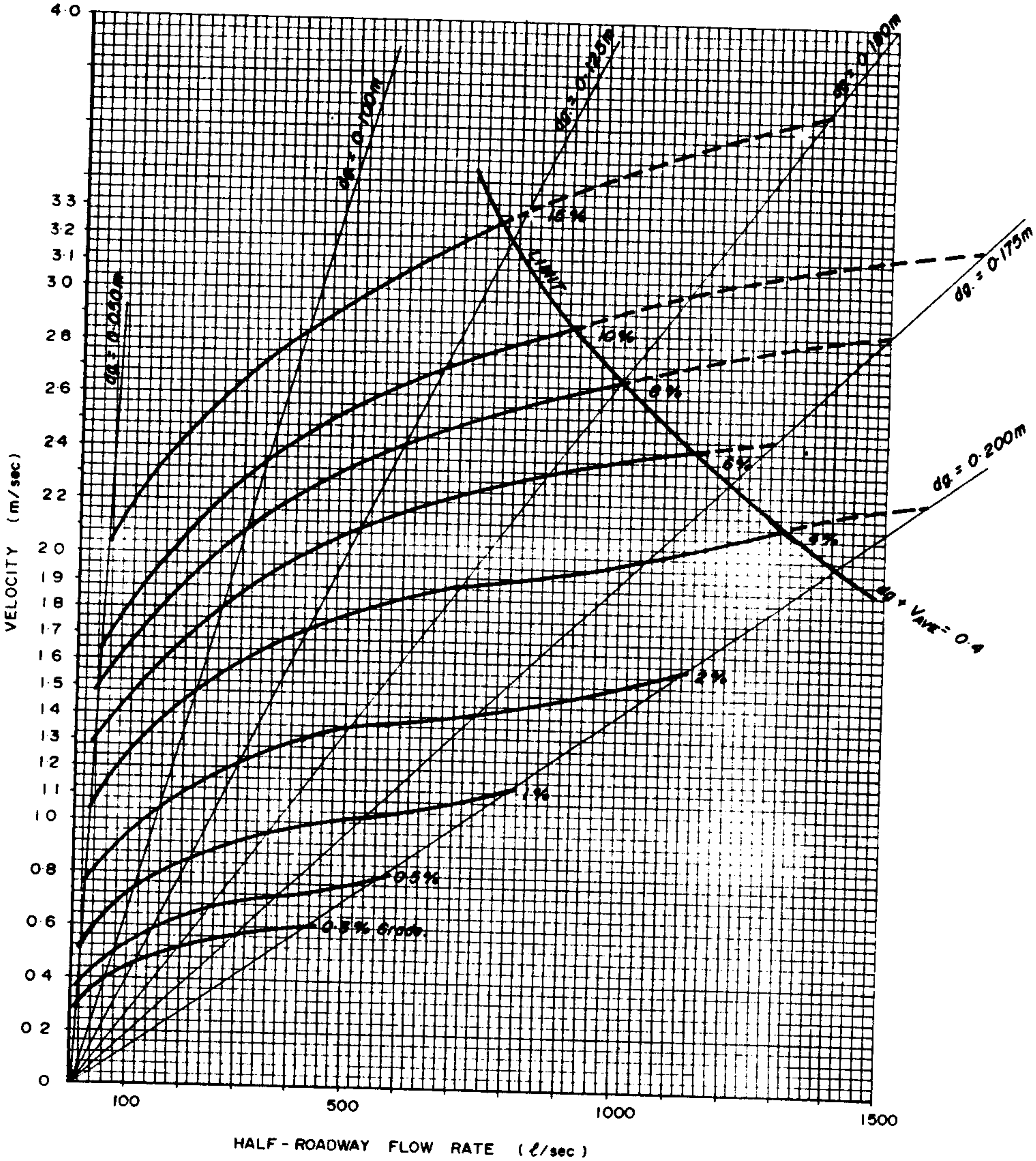
Chart 8.2



ROADWAY FLOW VELOCITIES (3% Crossfall)

Chart 8.3





ROADWAY FLOW VELOCITIES (2·5% Crossfall)

Chart 8.4

SECTION 9 - MINOR SYSTEM SURFACE DRAINAGE

9.1 DESIGN BASIS

The flow quantity to each gully box location is based on the runoff quantity from each individual gully box sub-catchment area plus by-pass flows from higher gully boxes.

The location and number of gully boxes is determined by the maximum allowable width of flow on the roadway or around a corner, and the gully box interception flow rates.

9.2 DESIGN GUIDELINES

Without limitation, design of the minor system surface drainage shall conform to the following:-

(a) Flow Criteria

(i) Maximum Flow Spread = 3.0m (including sags)
= 1.0m around corners

(ii) Maximum Flow into sags or at inlet points where flow bypass is not acceptable = 95% of gully capture

(iii) Concentrated Flow into kerb & channel = 20 l/sec.(max.)

(iv) Minimum Capture Rate of Gully Grate = 80%

(b) Gully Box Location

(i) Gullies should be located at the midpoint of allotment frontages to reduce the likelihood of conflict with service conduits and future driveways.

(ii) Gullies at intersections should be located at the tangent points of kerb radii.

9.3 PEAK RUNOFF DETERMINATION

The peak runoff is calculated by the Rational Method for both full area and part area cases (Section 9.6), and the highest value from each case is used for design purposes.

A.R.Is are obtained from Table 7.1 and the rainfall intensities corresponding to the flow times derived in Section #9.4 below are obtained from Chart 2.1.

9.4 SURFACE FLOW TIMES

Surface flow times are to be calculated in accordance with Section 8.4.

9.5 RUNOFF CO-EFFICIENTS

Weighted Runoff Co-efficients for the various land uses in a sub-catchment are given in Table 9.1 for various A.R.Is. These values include allowances for impervious area fractions (f) and A.R.I. frequency factors.

9.6 PARTIAL AREA EFFECTS

(a) Design Basis

The Rational Method requires that the critical storm duration (time of concentration) to be set equal to the total catchments longest travel time, however, the calculated peak discharge from the full-area in some situations will be less than the discharge of an impervious part-area having a shorter time of concentration.

These situations arise particularly when the catchment has multiple land uses and the impervious area is in the lower part of the catchment. Homogeneous land use catchments usually don't suffer from this partial effect.

The Partial Area calculation derives the discharge associated with these impervious part-areas in each catchment for comparison with the full-area discharge, and the largest value is used for design purposes.

SECTION 9.6 (Cont.)

To be included in the calculation, the impervious area should be directly connected to the gutter or stormwater system, e.g. roof and carpark runoff should be piped, or flow directly to the gutter over an impervious surface. If runoff from an impervious area (roof) flows across a pervious area (lawn) before entering the gutter, this impervious area should be included as part of the pervious area, not the impervious area.

(b) Time of Concentration and Intensity

By determining the flow paths and then calculating the flow times to the gully box or interception point of each individual directly connected impervious area, the partial area time of concentration (t_p) is determined as being the longest of these individual flow times, and from which the partial area design rainfall intensity (I_p) can be obtained.

(c) Contributing Area

As the time of concentration is taken as the longest flow time of all directly connected impervious areas, it follows that all directly connected impervious areas will contribute to the discharge.

It also follows that a portion of the various pervious areas may also contribute to the discharge quantity. This can be investigated by determining the flow paths and calculating the total flow times (t_t) for each individual pervious area. Note that the total flow time (t_t) is usually made up of a component flow time across the pervious area plus a component flow time (t_i) along an impervious surface (gutter).

For any pervious area with a total flow time (t_t) less than (t_p), its total area will contribute to the discharge, and for any pervious area with a flow time greater than (t_p), only a portion of its area will contribute to the discharge. This proportion of the area which contributes is estimated by the formula:-

$$P = \frac{(t_p - t_i)\,A}{(t_t - t_i)} \qquad \text{(Equ. 9.1)}$$

where P = the proportion of the pervious area contributing to the discharge (h_a)

t_p = partial-area time of concentration (mins.)

t_t = total flow time of the particular pervious area (mins.)

t_i = that component of the full-area flow time along an impervious surface (mins.)

A = total area of the pervious area (h_a)

In practice, the (t_i) term is usually omitted and also the area (A) is expressed as the equivalent impervious area, giving a more convenient expression in terms of equivalent impervious area.

$$P' = \frac{t_p(CA)}{t_t} \qquad \text{(Equ. 9.2)}$$

Because the Runoff Co-efficients for impervious and pervious are relatively close, full-area discharges will usually be the largest.

(d) Runoff Co-efficients

Weighted runoff co-efficients for pervious and impervious surfaces and the various A.R.Is are given in Table 9.1.

(e) Discharge

The partial-area discharge is calculated by the Rational Method in the normal manner by summing the equivalent impervious areas of all the individual impervious areas and contributing portions of the pervious areas (ΣCA), and using (I_p) as the rainfall intensity.

SECTION 9.6 (Cont.)

(f) Examples

The following examples will demonstrate the above.

Example 1

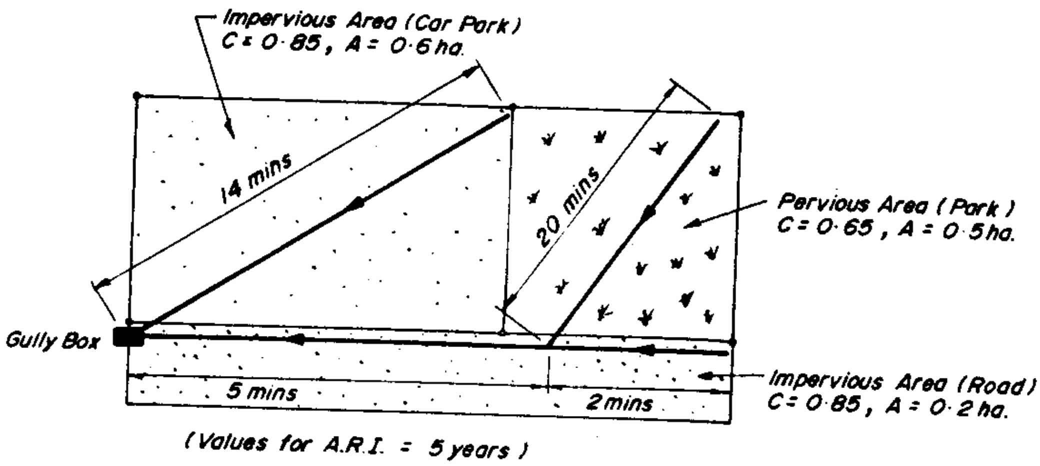


Fig. 9.1

From the above diagram, both the full-area and part-area discharges can be calculated.

Full-Area Discharge

Full-Area time of concentration (t_f) = 25 mins.
Full-Area Intensity I_f = 95 mm/hr. (ARI 5 years)

Equiv. Imp. Area → Carpark (CA) = 0.85 x 0.6 = 0.51
→ Road (CA) = 0.85 x 0.2 = 0.17
→ Park (CA) = 0.65 x 0.5 = 0.33
ΣCA = 1.01

$$\text{Discharge } Q_{full} = \frac{(\Sigma CA)I}{0.36} = \frac{1.01 \times 95}{0.36} = \underline{266 \text{ l/sec}}$$

Partial Area Discharge

Impervious → Road Flow Time = 7 mins.
→ Car Park Flow Time = 14 mins.
Therefore Partial Area time of concentration (t_p) = 14 mins.
Partial Area Intensity I_p = 125mm/hr.
Pervious Area t_t = 25 mins.

Equiv. Imp. Area → Car Park (CA) = 0.85 x 0.6 = 0.51

→ Road (CA) = 0.85 x 0.2 = 0.17

→ Park (Part CA) = $\left(\frac{14}{25}\right)$ x 0.65 x 0.5 = 0.18

ΣCA = 0.86

Discharge $Q_{part} = \frac{(\Sigma CA)I}{0.36} = \frac{0.86 \times 125}{0.36} = 298$ l/sec.

In this case the Partial Area discharge of 298 l/sec. is used for design purposes.

For comparison, equation 9.1 can be used where t_i = 5 mins.

→ Car Park + Road (CA) = 0.51 + 0.17 = 0.68

→ Park (Part CA) = 0.65 x $\left(\frac{14-5}{25-5}\right)$ x 0.5 = 0.15

0.83

$$Q_p = \frac{0.83 \times 125}{0.36} = 288 \text{ l/sec.}$$

which is not significantly different and for all practical purposes, the additional work is not warranted.

Example 2

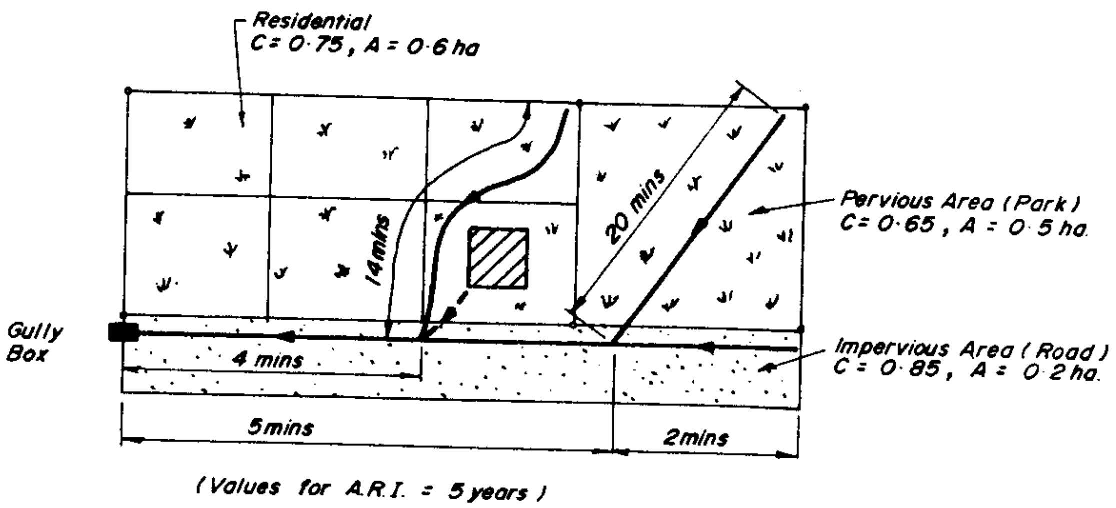


Fig. 9.2

SECTION 9.6 (Cont.)

Full Area Discharge

Full Area time of concentration (t_f) = 25 min.
Full Area Intensity (I_f) = 95mm/hr. (ARI 5 years)

Equiv. Imp Area	→ Road	(CA)	= 0.85 x 0.2	=	0.17
	→ Residential	(CA)	= 0.75 x 0.6	=	0.45
	→ Park	(CA)	= 0.65 x 0.5	=	0.33
			ΣCA	=	0.95

Discharge $Q_{full} = \frac{(\Sigma CA)I}{0.36} = \frac{0.95 \times 95}{0.36} = 251$ l/sec.

Partial Area Discharge

Impervious Area → Road flow time = 5 + 2 = 7 mins.
→ House flow time = 5 + 4 = 9 mins.
(Note:- Table 8.3 Standard House Roof-to-drain flow time = 5 mins.)
Therefore: Partial Area time of concentration (t_p) = 9 mins.
Partial Area Intensity (I_p) = 150mm/hr.
Resid. pervious area t_c = 18 mins.
Park pervious area t_t = 25 mins.

Equiv. Imp. Area	→ Road	(CA)	= 0.85 x 0.2	=	0.17
	→ Resid.	(Part CA)	= (9/18)x0.75x0.6	=	0.23
	→ Park	(Part CA)	= (9/25)x0.65x0.5	=	0.12
			ΣCA	=	0.52

Discharge $Q_{part} = \frac{(\Sigma CA)I_D}{0.36} = \frac{0.52 \times 150}{0.36} = 217$ l/sec.

In this case the full area discharge of 251 l/sec. is used for design purposes.

9.7 FLOW SPREAD

The flow spread and flow depth related to gutter flow rates are given in charts 9.1 and 9.2

9.8 GULLY INLET CAPACITIES

Gully inlet capacities for both sag and on-grade situations are given in charts 9.3 to 9.6.

9.9 DESIGN PROCEDURE AND EXAMPLE

Gully Box Location and Spacings

The location of gully boxes and spacings can be approximated as a "first trial" by the following method.

(a) Determine the maximum flow rate in the kerb and channel based on the maximum allowable flow width (3.0m) and the kerb and channel grade.

e.g. Crossfall = 3%, Grade = 0.6%
Therefore Q = 100 l/sec. (Chart 9.1)

(b) Determine a trial Intensity

e.g. assume t_c = 5 minutes, A.R.I. = 5 years
Therefore I = 180mm/hr.

(c) Convert the catchment to an approximate rectangular area giving -

Area A = x L
where x = distance from catchment boundary to the road centreline and is known.
L = length of the catchment and is the distance to the first gully box or gully box spacing - which is to be calculated.

e.g. let x = 40m
therefore A = 40L

(d) From the Rational Method

$$Q = \frac{CIA}{0.36}$$

For Residential catchments and A.R.I. = 5 years, C = 0.75

$$\text{Therefore } Q = \frac{0.75 \times 180 \times (40L/10{,}000)}{0.36}$$

$$Q = 1.5L$$

SECTION 9.9 (Cont.)

For Q = 100 l/sec. from (a)

$$L = \frac{100}{1.7} = 66.7m$$

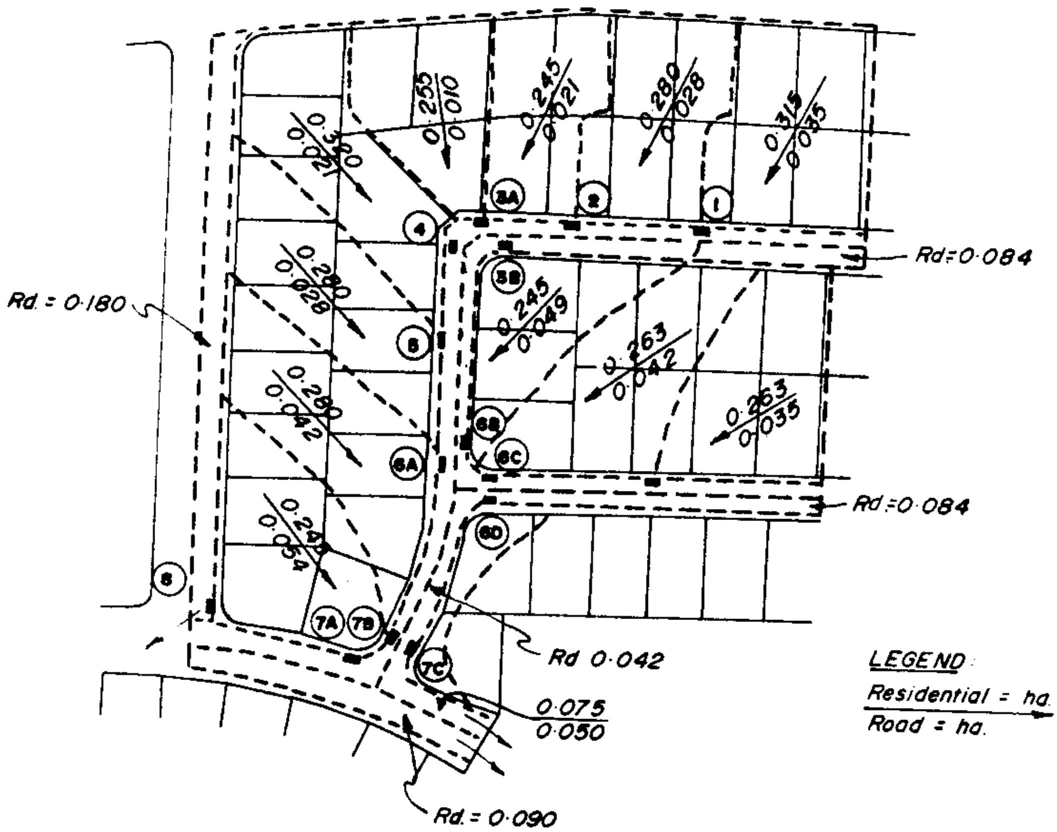
(e) If the gully intercepts 90 l/sec., the by-pass quantity is 100 -90 = 10 l/sec.

(f) Spacing to the next gully box

$$L = \frac{(100 - 10)}{1.5} = 60m$$

(g) The above process can be continued down the catchment

The subsequent design procedure is similar to that for the Major System design and is aided by a Design Chart as per the following example, using the layout in Fig. 8.1.



MINOR SYSTEM - SUB-CATCHMENTS, AREAS AND GULLY BOX LOCATIONS

Fig. 9.3

Minor System A.R.I. n = 5 years

STORMWATER DRAINAGE

MINOR SYSTEM SURFACE FLOWS

Example.

Sheet ...... of ......
Designed ....
Checked ....
Signed 1.1

(1) Gully Box	(2) Flow Path Description (Land Use)	(3) Flow Length (L)	(4) Roughness Factor (n)	(5) Slope (S)	(6) $L^{0.4}$ = $\frac{6.94(Ln)^{0.6}}{S^{0.3}}$	(7) Component Flow Times	(8) Total Flow Times	(9) Time of Concentration	(10) Intensity (In)	(11) Runoff Co-eff (C)	(12) Area (A)	(13) Equiv. Imp. Area (CA)	(14) Cumul. Equiv. Imp Area (Σ CA)	(15) Q = (Σ CA)In / 0.36	(16) By-Pass Flow from [ ]	(17) Design Flow	(18) Gutter Grade	(19) Flow Width	(20) Inlet Type	(21) Inflow	(22) Bypass Flow to Pit [ ]	REMARKS
		m	-	m/m	mins mm / hr.	mins	mins	mins	mm / hr	-	ha	ha	ha	l/sec	l/sec	l/sec	m/m	m		l/sec	l/sec	
1	Resid.	90	0.025	0.024	34	4.2																
	Road	30		1.5%		≈0.6	4.8	5.0		0.75	0.315	0.236										
2	Resid.	As	(1)	above					180	0.80	0.035	0.028	0.264	132	—	132	1.5%	<3.0	IC2T	110	22[2]	
	Road	40		2.5%		4.2				0.75	0.280	0.210										
						≈0.6	4.8	5.0														
3A	Resid	90	0.025	0.017	39	4.8			180	0.80	0.028	0.022	0.232	116	22[1]	138	2.5%	2.8	IC2T	110	25[3A]	
		30	—	3%						0.75	0.245	0.184										
3B	Road	120		3%		0.4	5.2	5.2	180	0.80	0.024	0.017	0.301	101	29[2]	129	3%	2.6	IC2T	100	29[4]	
4	Resid	90	0.025	0.047	28			5.0	180	0.80	0.084	0.067	0.067	34	—	34	3%	1.5	IC0T	34	—	Check conclu. is!
	Road	20	—	3%		3.3				0.75	0.255	0.191										
5	Resid	150	0.025	0.033	43	0.3	3.6	≈5.0	180	0.80	0.010	0.0[illegible]	0.199	100	29[3A]	129	3%	2.5	IC2T	98	31[5]	Check Grade is?
	Road	10				5.1				0.75	0.32[illegible]	0.24[illegible]										
6A	Resid	As	above			≈0.1	5.2	≈5.2	180	0.80	[illegible]	[illegible]	0.257	128	51[4]	159	1.5%	≈3.0	IC2T	170	39[6A]	
	Road	As	above							0.75	0.210	[illegible]										
6B	Resid.	110	0.025	0.027	39	4.7	4.7	5.2	180	0.80	[illegible]	[illegible]	0.232	116	39[5]	155	1.5%	≈3.0	IC2T	120	35[7B]	
	Road							≈5.0	180	0.75	0.245	0.184										
9	Resid	As	(1)	above						0.80	0.049	0.039	0.223	117	31[6C]	143	SAG	2.7	IC1T	143	—	
	Road							≈5.0	180	0.75	0.263	0.197										
6C	Resid									0.80	0.035	0.028	0.225	113	—	113	2%	2.4	IC2T	90	23[6C]	
	Road							≈5.0	180	0.75	0.243	0.193										
6D	Road									0.80	0.042	0.034	0.216	108	23[7]	131	2%	2.7	IC2T	100	31[6B]	
7B	Resid							≈5.0	180	0.80	0.081	0.067	0.067	34	—	34	2%	1.7	IC0T	34	—	
	Road									0.75	0.280	0.110										
8	Road							≈5.0	180	0.80	0.042	0.034	0.244	122	35[6A]	157	2.5%	2.9	IC2T	110	47[7A]	
7A	Resid							≈5.0	180	0.80	0.180	0.144	0.144	72	—	72	4%	2.0	IC2T	≈60	12[7A]	Bypass to exist system
	Road							≈5.0	180	0.75	0.245	0.184										
										0.80	0.054	0.043	0.227	113	12[8]							
7C	Road														47[7B]							
								≈5.0	180	0.80	0.042	0.034	0.034			172	SAG	2.8	IC1T	172	—	
														17	—	17	2.5%	<1.0	IC	17	—	

Option (B) Re Pipe Size – From Manual Pipe Flow Chart ⇒ Required Additional Gully Box Inflow = 1493 − 1120 = 373
Q's Inflow Gully Box (7A) = 172 } = 6.45 l/sec
From Chart 9.6 ⇒ for Q = 645 ⇒ IC1T, d = 260 mm N.G.
⇒ IC2T, d = 210 mm O.K.
Provide Additional Trough for Option 1

(BLANK)

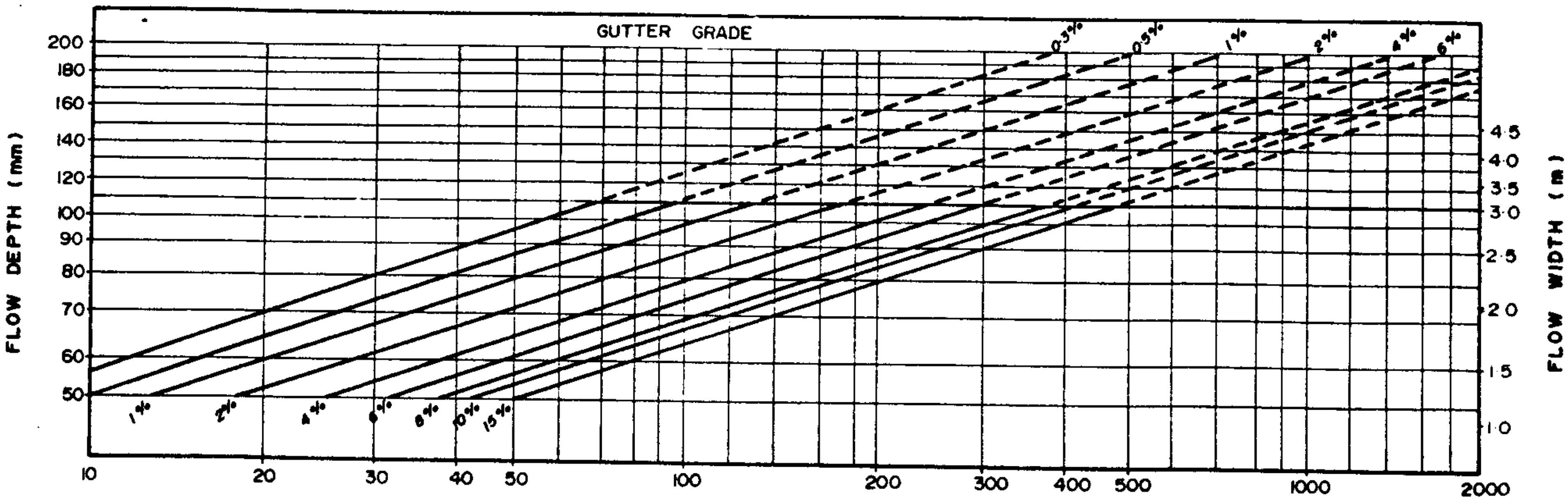
Land Use	Impervious function(f)	Weighted Runoff Co-Efficients C_w			
		A.R.I.			
		2	5	10	20
1. Parks/Open Space (≃Pervious Area)	0.1	0.60	0.65	0.70	0.75
2. Residential	0.50	0.70	0.75	0.80	0.85
3. Road Reserve (including footpath)	0.85	0.75	0.80	0.85	0.90
4. Commercial/Industrial	0.85	0.75	0.80	0.85	0.90
5. Impervious Area (Road - excluding footpath)	1.0	0.80	0.85	0.90	0.95

Minor System Weighted Runoff Co-efficients (C_w)

NOTE: Values for 1. Pervious Areas and 5. Impervious Areas are essentially Basic Runoff Co-efficients (C) for the catchment surface.

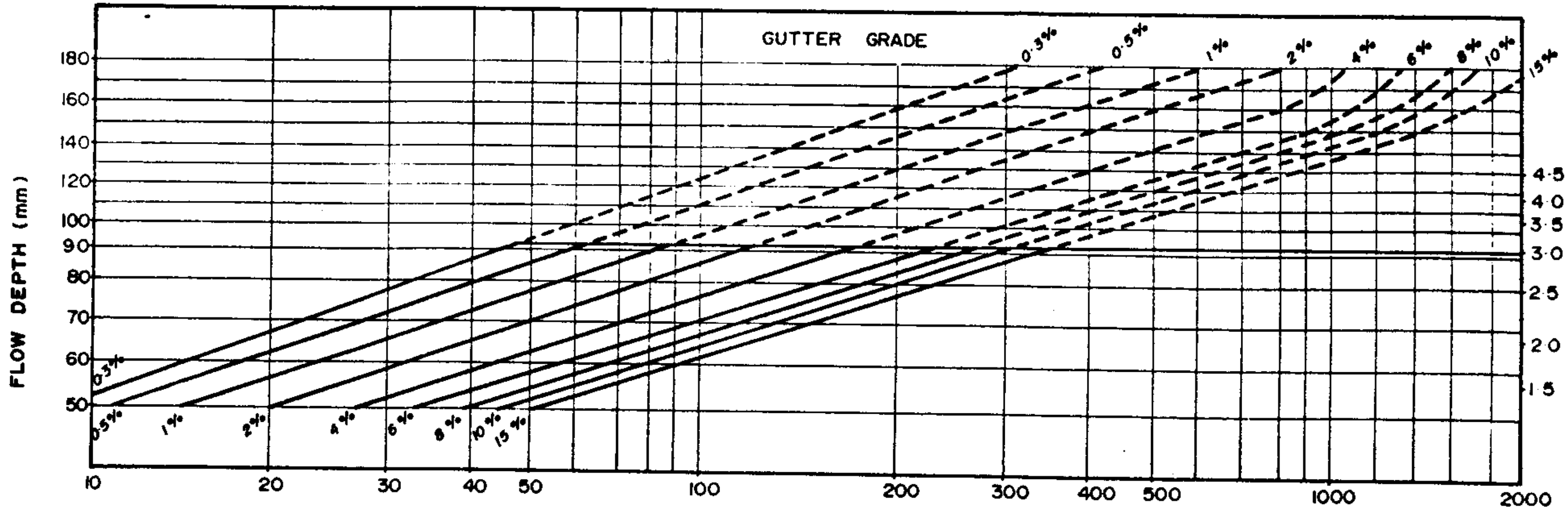
Table 9.1

Chart 9.1



GUTTER FLOWRATE (ℓ/sec)
3% CROSSFALL

Chart 9.2



GUTTER FLOWRATE (ℓ/sec)

2·5 % CROSSFALL

Chart 9.3

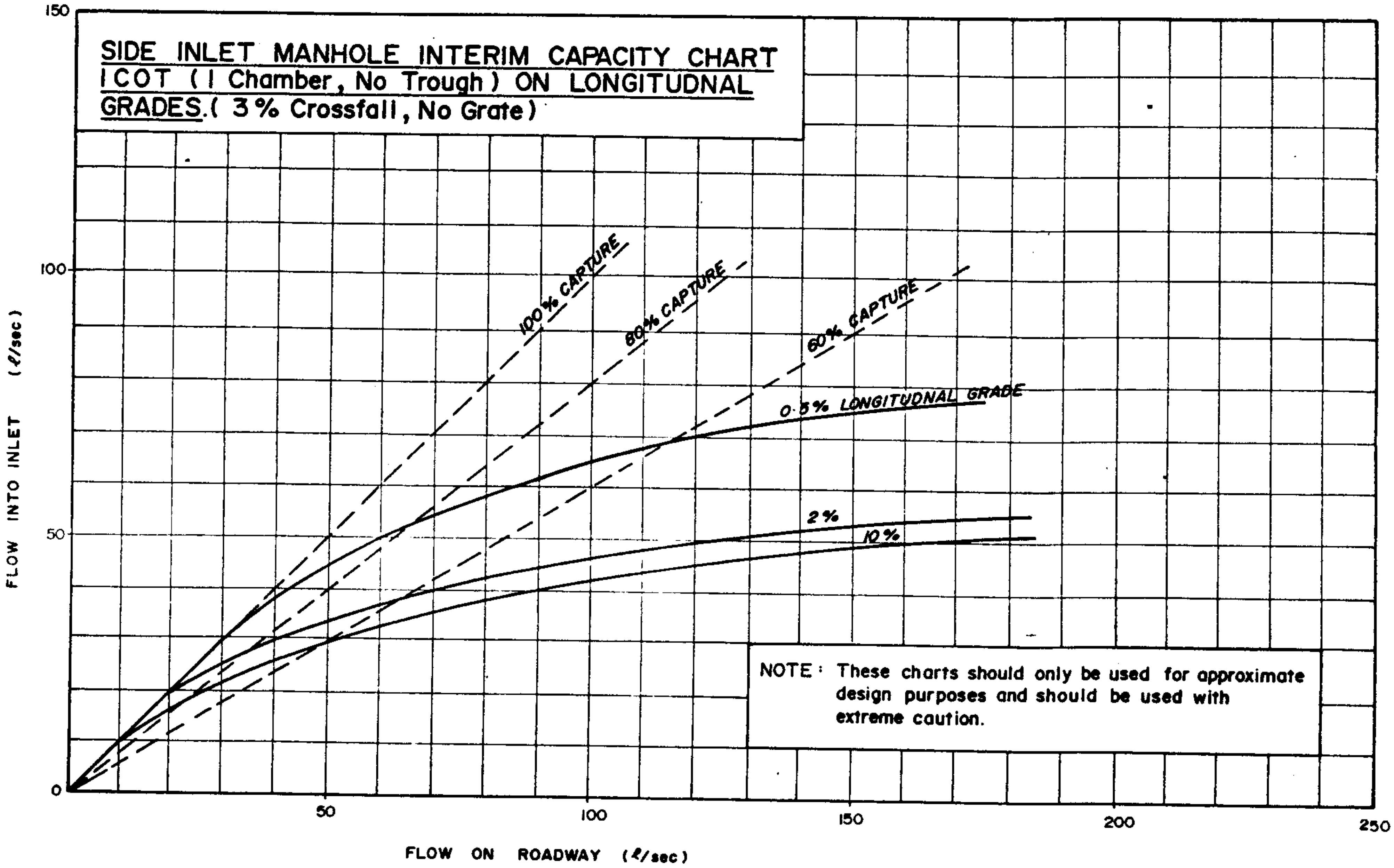

SIDE INLET MANHOLE INTERIM CAPACITY CHART
ICOT (1 Chamber, No Trough) ON LONGITUDNAL
GRADES.(3% Crossfall, No Grate)
100% CAPTURE
80% CAPTURE
60% CAPTURE
0.5% LONGITUDNAL GRADE
2%
10%
NOTE: These charts should only be used for approximate design purposes and should be used with extreme caution.
FLOW INTO INLET (ℓ/sec)
FLOW ON ROADWAY (ℓ/sec)
0
50
100
150
200
250

Chart 9.4

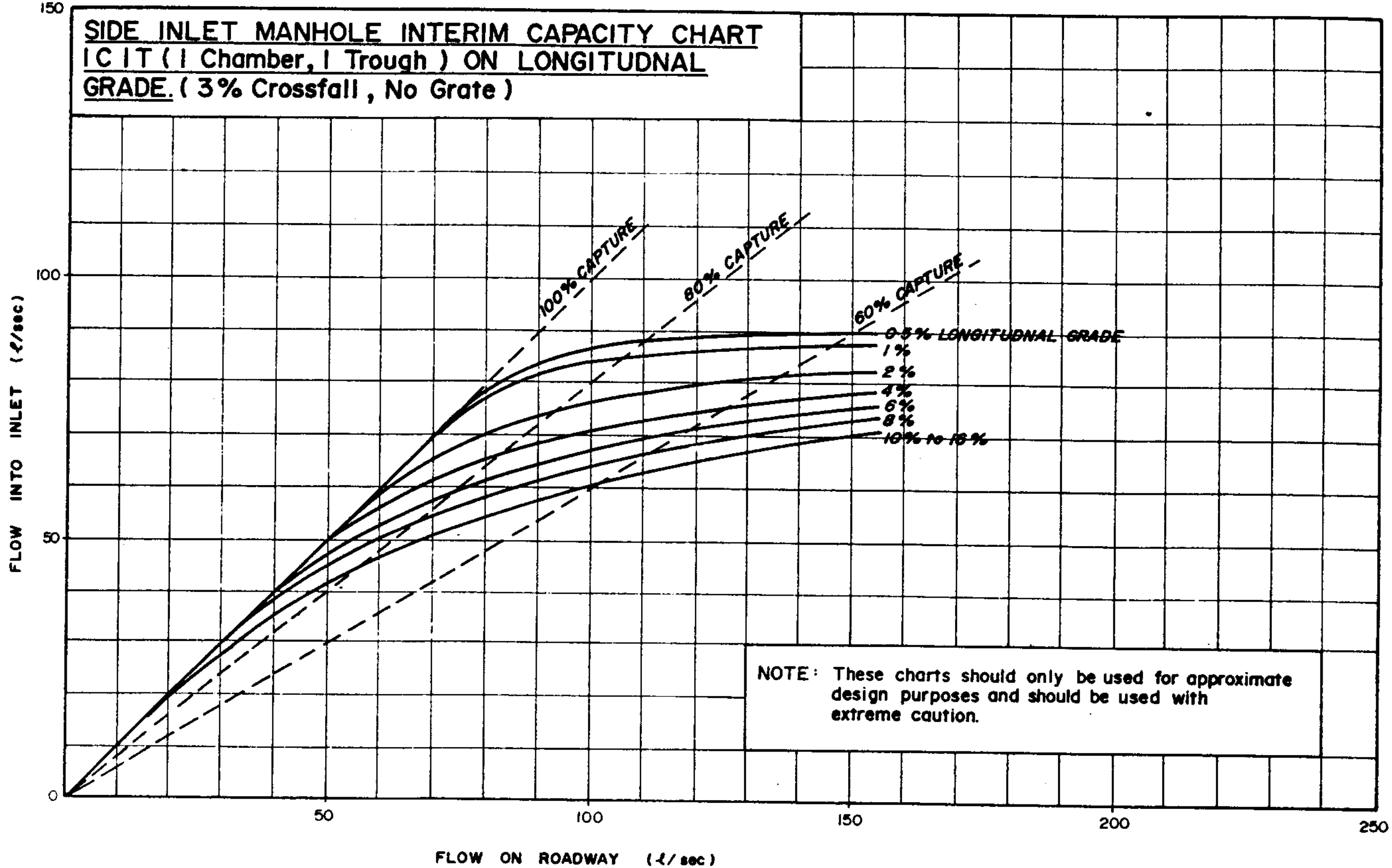
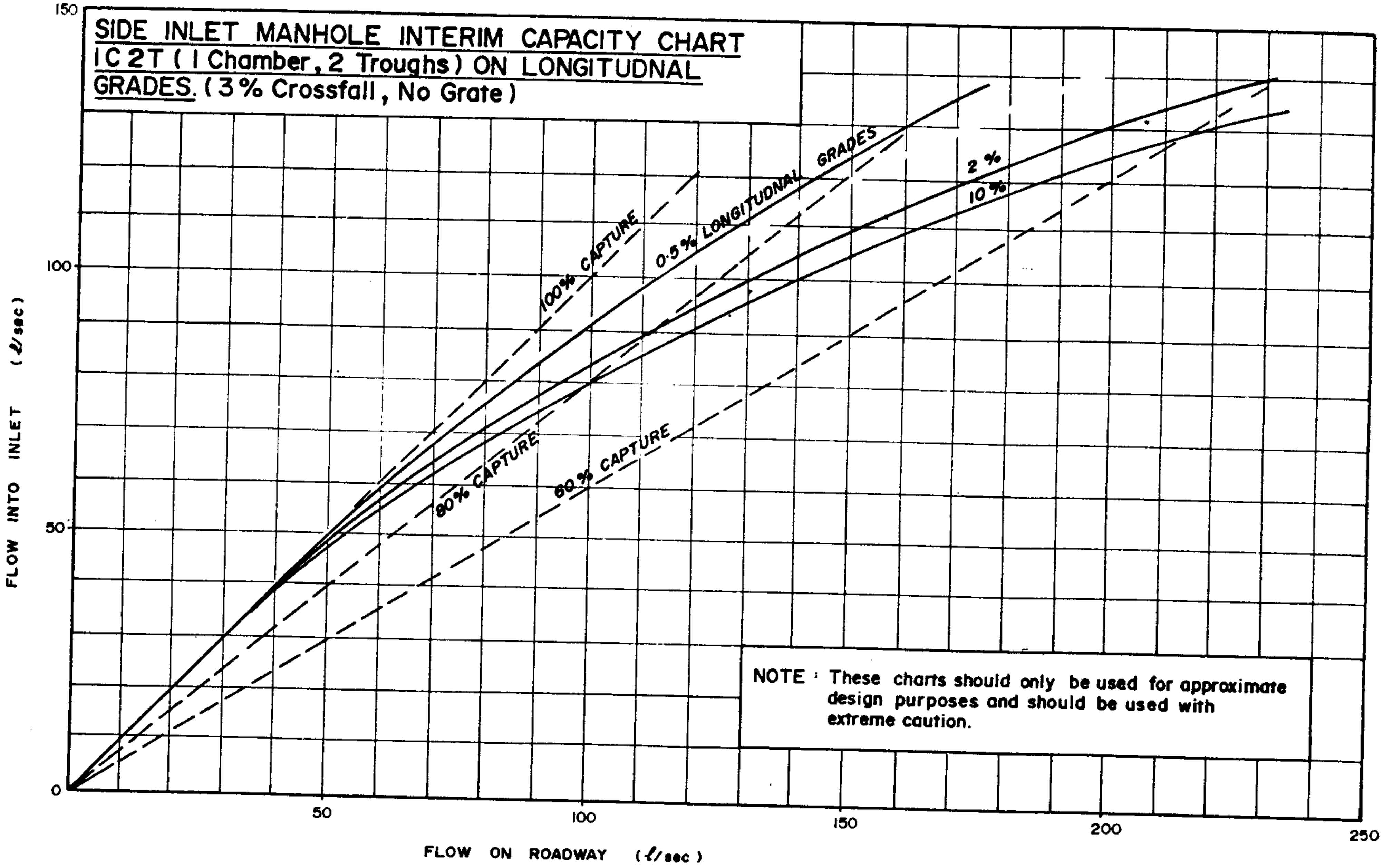

SIDE INLET MANHOLE INTERIM CAPACITY CHART
1C1T (1 Chamber, 1 Trough) ON LONGITUDNAL
GRADE. (3% Crossfall, No Grate)
FLOW INTO INLET (ℓ/sec)
FLOW ON ROADWAY (ℓ/sec)
0
50
100
150
200
250
100% CAPTURE
80% CAPTURE
60% CAPTURE
0.5% LONGITUDNAL GRADE
1%
2%
4%
6%
8%
10% to 18%
NOTE: These charts should only be used for approximate design purposes and should be used with extreme caution.

Chart 9.5



SIDE INLET MANHOLE INTERIM CAPACITY CHART
IC2T (1 Chamber, 2 Troughs) ON LONGITUDNAL
GRADES. (3% Crossfall, No Grate)
100% CAPTURE
80% CAPTURE
60% CAPTURE
0.5% LONGITUDNAL GRADES
2 %
10 %
NOTE : These charts should only be used for approximate design purposes and should be used with extreme caution.
FLOW INTO INLET (ℓ/sec)
FLOW ON ROADWAY (ℓ/sec)
0
50
100
150
200
250

SIDE INLET MANHOLE INTERIM CAPACITY CHART FOR SAG VERTICAL CURVE (No Grate)

NOTE : These charts should only be used for approximate design purposes and should be used with extreme caution.

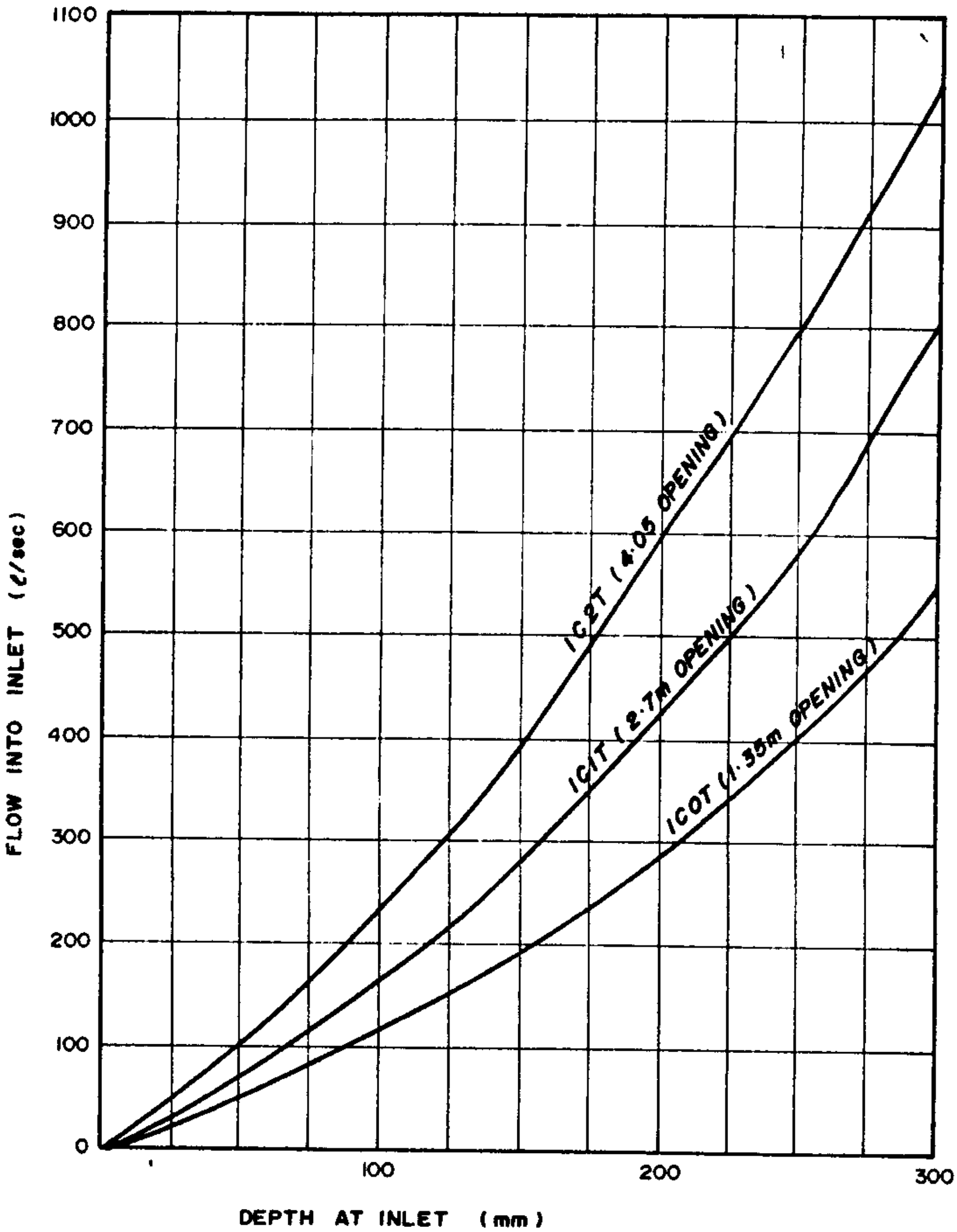


Chart 9.6

SECTION 10 - MINOR SYSTEMS PIPE NETWORK

10.1 DESIGN BASIS

Design flows for gully box inlets are calculated for each particular sub-catchment, however, the design flows for mainline pipes are calculated for the total catchment draining to each pipe. Unless all gully box catchments have identical times of concentration, it is not correct just to sum all these flows, as this will over estimate the flow rates and thus the pipe sizing.

Pipe flow rates are estimated by accumulating "equivalent impervious areas" (ΣCA) for the various sub-catchments and applying the appropriate (I) values corresponding to the times of concentration at the various points along the drainline using the Rational Formula.

10.2 DESIGN GUIDELINES

Without limitation, the design of pipe networks shall comply with the following:-

Pipes

(a) Minimum Scour Velocity - 0.5m/sec. at Design Flow.
(b) Minimum Pipe Size = 375mm diameter.
(c) Pipe sizes should not decrease in the downstream direction unless approved by the Shire Engineer.

Pipes Under Kerb and Channel

(d) Pipes up to a maximum of 600mm diameter may be laid under the kerb and channel to connect gully boxes (refer S.W.D.S.4).
(e) Gully boxes connected by pipes under the kerb and channel, shall be designed for maximum water levels to be 0.450m below the kerb and channel invert level (e.g. as for Manholes).
(f) Gully boxes with outlet pipes under the kerb and channel shall have a maximum of two (2) inflow pipes.
(g) Maximum distance between gully connected by pipes under the kerb and channel shall be 120m (e.g. as for manholes).

Gully Boxes

(h) Gully Box Water Levels = 0.150m (min.) below gutter invert.
(i) Gully Box Outlet Pipes = Obvert to be 0.700m below top of gutter.

Manholes

(j) Manhole Water Levels = 0.450m (min.) below road (or top cover) level.
(k) Fall across Manhole = 0.050m (min.).
(l) Consecutive Manhole Water Levels = shall differ by at least 0.100m in the downstream direction.
(m) Manhole Location = at all changes in grade, alignment pipe size and pipe junctions.
(n) Manhole Spacings = 120m maximum for 1050mm diameter pipes and less.

10.3 PIPE DESIGN FLOWS

(a) Time of Concentration - The surface travel time component can be obtained from the Minor System Surface Flow Design Chart and by adding the flow times of upstream pipework will give the time of concentration.

Mainline pipe work flow times will be short and can be accumulated in increments 0.5mins.

As the primary pipe work (gully box to manhole) is usually in the order of 10m long, flow times will be very small and can be ignored.

(b) Catchment Area - The catchment sub-areas for each mainline pipe can also be obtained directly from the Minor System Surface Flow Chart.

(c) Flows - Both full-area and partial-area flows should be investigated and the greater value adopted for design.

10.4 MINIMUM GRADE ANALYSIS

The Minimum Grade Analysis is used to derive "first round" or "first trial" pipe diameters while working from the top of the catchment to the bottom or outfall. Pipes are assumed to be flowing full and manhole head losses are ignored in this process. Primary pipework (gully box inter-connection and gully outlet pipework) can also be ignored during the first round pipe sizing.

The analysis is carried out using Chart #10.1

10.5 PIPE DESIGN - HYDRAULIC GRADE LINE ANALYSIS

The application of Hydraulic Grade Line Analysis initially assumes the pipes are flowing full and under pressure, and requires a manhole to manhole headloss calculation commencing from the outfall of the drainage system.

(a) Application - Manhole to Manhole (or Gully Box)

The headloss between manholes is made up of two components as per the following diagram:-

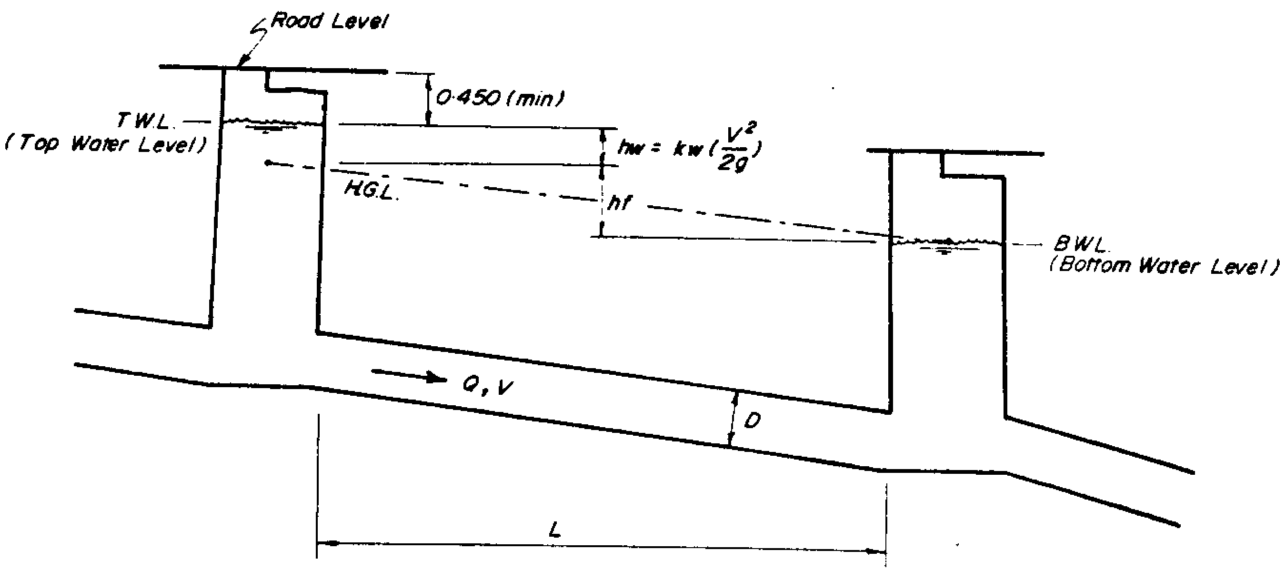


Fig. 10.1

SECTION 10.5 (Cont.)

$$H_T = h_f + K_w \frac{V^2}{2g} \qquad \text{Eqn. \#10.1}$$

where h_f = pipe friction headloss from the Darcy-Weisbach formula $L_f = 0.022 \frac{L}{D}\left(\frac{V^2}{2g}\right)$(metres)

L = Length of pipe (m)

D = Pipe diameter (m)

K_w = Manhole headloss co-efficient from Table #10.1

V = Flow Velocity in the pipe

g = 9.81m/sec.

(b) Application - Manhole to Outfall

(i) Outlet Submerged - If the downstream end of the pipe is submerged, the B.W.L. and downstream end of the Hydraulic Grade Line is located directly above the discharge end of the pipe and is approximately the free surface water level of the pond the pipe is discharging into.

(ii) Outlet Not Submerged - If the outlet is not submerged, the B.W.L. and downstream end of the Hydraulic Grade Line is assumed to be at the pipe obvert (top of pipe).

(c) Tests

Calculations from manhole to manhole moving in an upstream direction will give B.W.L's (T.W.L. of downstream pipe) as a design basis for each section of pipe. Having calculated (H_{TOT}) for each section, the following tests can be applied.

(i) Minimum Headloss Test - in accordance with design guideline #10.2(h), the water level difference or headloss between consecutive manholes shall be a minimum of 0.100m.

therefore $H_{TOT} = h_f + h_w \geq 0.100\text{m}$ Eqn. #10.2

SECTION 10.5 (Cont.)

(ii) Manhole Overflow Test - in accordance with design guideline #10.2(f), manhole water levels shall be 0.450m(mm) below the road level.

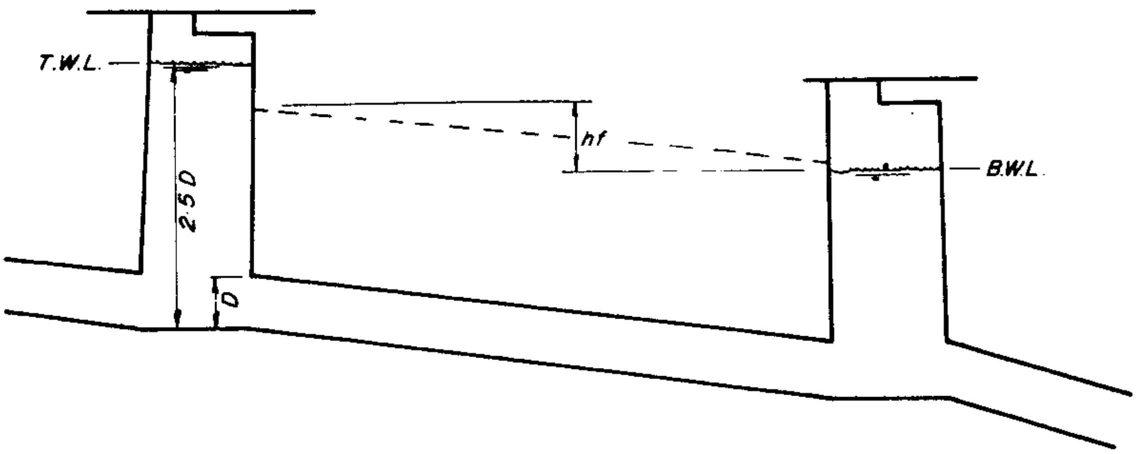
therefore T.W.L. = B.W.L. + H_{TOT} > 0.450m below the road level Eqn.#10.3

Similarly, gully boxe water level = 0.150m(min.) in accordance with design guidline #10.2(d).

(iii) Pipe Obvert Level Test

Test (ii) above implies that full barrel flow in the discharge pipe of diameter (D) can be achieved without overflow occuring at the upstream pit, however it does not indicate at what depth the invert of the pipe must be to achieve this.

The test is based on the comparison of the quantity (h_f + 1.5D) against the allowable head difference (T.W.L. - B.W.L.) as illustrated in Fig. 10.2.



Test (h_f + 1.5D) $\geqslant$ (T.W.L. - B.W.L.) Eqn. #10.4

Fig. 10.2

An arbitrary invert depth limit of 2.5D below T.W.L. is applied, which is based on the test conditions under which much of the information in Tables #10.1 and #10.2 has been obtained.

Result 1

If $(h_f + 1.5D) \geq (T.W.L.-B.W.L.)$, e.g. if the Eqn. #10.4 is satisfied, it follows that the invert of the upstream end of the pipe may be installed above (T.W.L.-2.5D) without losing full barrel flow.

Limit of upstream invert level

$$\text{Pipe I.L. (upstream)} \leq B.W.L. + h_f - D \qquad \text{Eqn. \#10.5}$$

Water level in upstream manhole

$$T.W.L. = B.W.L. + h_f + K_w\left(\frac{V^2}{2g}\right)$$

or

$$T.W.L. = B.W.L. + H_T \qquad \text{Eqn. \#10.6}$$

Result 2

If $(h_f + 1.5D) < (T.W.L.-B.W.L.)$ e.g. Eqn. #10.4 fails, it follows that the invert of the upstream end of the pipe needs to be installed lower than (T.W.L.-2.5D), which is undesirable, and the alternative is to design for part-full outflow from the upstream manhole.

Limit of upstream invert level

$$\text{Pipe (Upstream) I.L.} \leq T.W.L. - 1.5K_w\left(\frac{V^2}{2g}\right)$$

or

$$\text{Pipe (upstream) I.L.} \leq T.W.L. - 1.5h_w \qquad \text{Eqn. 10.7}$$

(refer Table 10.1 - Code 19)

SECTION 10.5 (Cont.)

Water level in upstream manhole

T.W.L. = the level used for test (i), (ii) and Eqn. 10.7 above.

Normally, the upstream pipe invert level given by eqn. 10.7 is above that given by equ. 10.5. However, the inter-relationship between these levels can sometimes be reversed. The situations which lead to such reversal are those where large flows must be passed through multi-pipe junction pits or where large flows are subject to severe direction changes at pits. In cases where the upstream invert level given by equ. 10.5 is higher than that given by eqn. 10.7, the former level should be adopted and B.W.L. (upstream) computed by eqn. 10.6, i.e. treat Result 1 as the design situation.

10.6 PRIMARY PIPEWORK

Primary pipework consists of the pipes connecting gully boxes to manholes, and geometry is usually very similar between installations.

(a) Pipework Size Determination

Based on Hydraulic Grade Line Analysis with the following gully box constraints:-

(i) Outlet pipework = 20m maximum length
(ii) Floor level = 0.700 below top of kerb and channel
(iii) Maximum water level = 0.150m below kerb and channel invert level

the following formula is derived -

$$\frac{H_{TOT}}{Q^2} = \frac{0.0826}{D^4}\left(\frac{0.022L}{D}\right) + 4$$

where H_{TOT} = total head loss (m)
Q = pipe flow (m^3/sec.)
D = pipe diameter (m)
L = pipe length (m)

Based on this formula, Chart 10.2 is a plot of $\frac{H_{TOT}}{Q^2}$ vs (D) for both L = 5m and 20m.

The available head (H_{TOT}) is calculated from the manhole water levels (derived in the main drain line design) and the kerb and channel invert level at the gully box location with the 0.150m maximum gully box water level allowance. Gully box flow rate Q is obtained from the Minor System Surface Flow calculations.

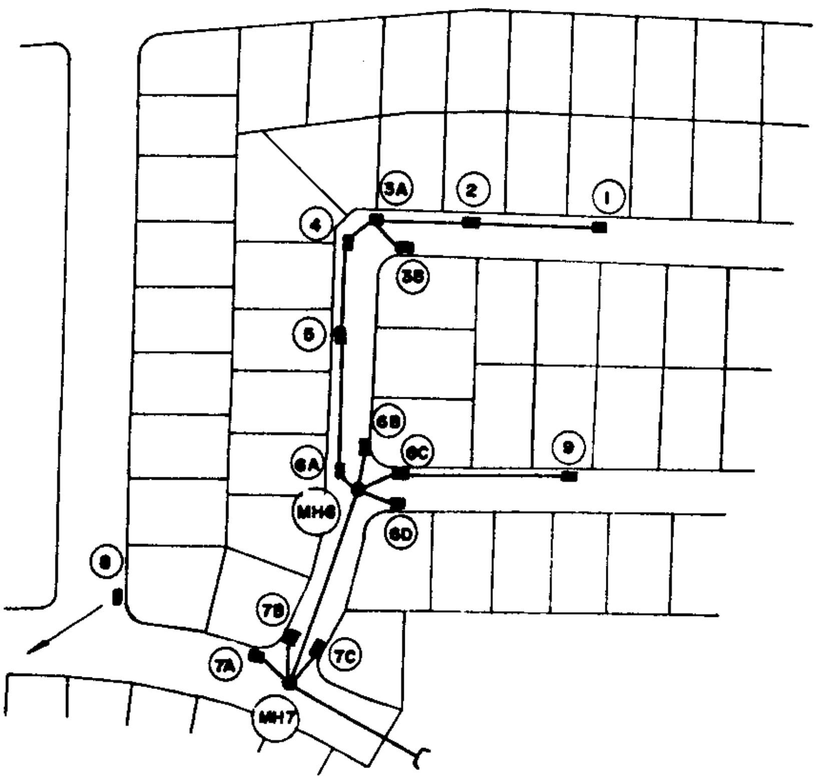
Example calculations are given on Chart 10.2.

(b) Double Gullies Inter-Connection Pipes

Double gullies inter-connection pipes shall be the same diameter as the gully outlet pipe.

10.7 DESIGN PROCEDURE

The design procedure is aided by Design Charts as per the following examples:-



MINOR SYSTEM PIPE NETWORK

MAINLINE PIPE FLOWS

Sheet ___ of ___

PROJECT Example

Designed ....
Date ....

Checked ....
Date ....

PIPE	FULL - AREA: SURFACE FLOW TIME	PIPE FLOW TIME	TIME (tc)	INTENSITY	Σ CA	$Q = \frac{\Sigma CA\ I}{0.36}$	PARTIAL - AREA: SURFACE FLOW TIME	PIPE FLOW TIME	TIME (tc)	INTENSITY	Σ CA	FLOW Q	ADOPTED FLOW	REMARKS
	mins	mins	mins	mm/hr	ha.	l/sec	mins	mins	mins	mm/hr	ha.	l/sec	l/sec	
1→2	4.8	-	≈5.0	180	0.264	132			≈5.0	180	0.264	132	132	
2					0.232									
2→3A	✓	0.5	5.3	180	0.496	248			≈5.0	180	0.496	248	248	
3B					0.067									
3A					0.201									
3A→4	✓	1.0	5.8	175	0.764	371			≈5.0	180	5/5.8×0.764 =0.659	329	371	
4					0.199									
4→5	✓	1.5	6.3	170	0.963	454			≈5.0	180	5/6.3×0.963 =0.764	382	454	
5					0.257									
5→6A	✓	2.0	6.8	170	1.220	576			≈5.0	180	5/6.8×1.220 =0.897	448	576	
6A					0.232									
6A→MH6	✓	2.5	7.3	165	1.452	665			≈5.0	180	5/7.3×1.452 =0.995	497	665	
9→6C	4.8	-	4.8	180	0.225	113			≈5.0	180	0.225	113	113	
6C					0.216									
6C→MH6	✓	0.5	5.3	180	0.441	220			≈5.0	180	5/5.3×0.441 0.416	208	220	
6B					0.223									
6D					0.067									
6A→MH6	4.8	3.0			1.452									
MH6→MH7	4.8	1.0	7.8	155	2.183	939			≈5.0	180	5/7.8×2.183 =1.399	700	939	

MAINLINE PIPE FLOWS

PROJECT

Designed ....
Date ....

Checked ....
Date ....

PIPE	FULL - AREA: SURFACE FLOW TIME	FULL - AREA: PIPE FLOW TIME	FULL - AREA: TIME (tc)	FULL - AREA: INTENSITY	FULL - AREA: Σ CA	FULL - AREA: $Q = \frac{\Sigma CA\, I}{0.36}$	PARTIAL - AREA: SURFACE FLOW TIME	PARTIAL - AREA: PIPE FLOW TIME	PARTIAL - AREA: TIME (tc)	PARTIAL - AREA: INTENSITY	PARTIAL - AREA: Σ CA	PARTIAL - AREA: FLOW Q	ADOPTED FLOW	REMARKS
	mins	mins	mins	mm/hr	ha.	ℓ/sec	mins	mins	mins	mm/hr	ha.	ℓ/sec	ℓ/sec	
MH6-MH7	4.8	3.0	7.8		2.183									
7A					0.227									
7B					0.244									
7C					0.034									
MH7→out	4.8	3.5	8.3	150	2.688	1120			≈5.0	180	5/8.3 × 2.688 = 1.619	809	1120	
Option C. - No Pipework.														
MH7→out			8.3	200	2.688	1493			≈5.0	240	1.619	1079	1493	

STORMWATER DRAINAGE

PIPE NETWORK DESIGN H.G.L. ANALYSIS

Example

Sheet ...... of ......
Designed ......
Checked ......
Signed 1.1

1 Manhole	2 Outlet Pipe Flow (l/sec)	3 Length (m)	4 Slope (m/m)	5 Trial D (mm)	6 M.H. Lid R.L. (or KBC I.L.) (m)	7 TWL = R.L. - 0.45 (m)	8 B.W.L. (m)	9 Velocity V = Q/A (m/sec)	10 Kw Code (-)	11 Kw (-)	12 $V^2/2g$ (m)	13 $hw = Kw(V^2/2g)$ (m)	14 $hf = \frac{0.022L}{D}(V^2/2g)$ (m)	15 HT = hf + hw, test (i) = [HT > 0.1?] (m)	16 T.W.L. - B.W.L., test (ii) [> HT?] (m)	17 hf + 1.5 D, test (iii) = [≥ TWL-BWL.?] (m)	18 Pipe U/S I.L. [test (iii)=Yes] I.L ≤ BWL+hf-D; [test (iii)=No] I.L ≤ TWL-1.5hw (m)	19 T.W.L. [test (iii)=Yes] = BWL + HT; [test (iii)=No] = TWL = Col. 7 (m)	20 Adopted D (mm)	21 Adopted T.W.L. & B.W.L. (m)	REMARKS
1	132	40	2.5%	375φ	16.900	16.750	15.339	1.12	1	✓	0.064	0.256	0.150	0.406	1.411 (yes)	0.713 (No)	≤ 16.566	16.750	375φ	T = 16.750 B = 15.339	U/S IL = 16.900 + 0.15 - 0.7 - 0.375 = [illegible] < 16.[illegible]; D/S IL = 14.834 + 0.05 = 14.884 Slope = 2.7%
2	248	30	2.7%	375φ	16.000	15.550	14.750	2.26	3	0.5	0.261	0.130	0.459	0.589	0.800 (yes)	1.022 (yes)	≤ 14.834	15.339	375φ	T = 15.339 B = 14.750	U/S IL = 16.000 + 0.15 - 0.7 - 0.375 = 15.0[illegible] → 14.834; D/S IL = 14.250 + 0.05 = 14.300 Slope = 1.8%
3A	371	10	4%	375φ	15.200	14.750	13.732	3.37	20	0.5	0.580	0.290	0.310	0.636 (yes)	1.018 (yes)	0.903 (yes)	≤ 14.315	14.750	375φ	T = 14.750 B = 13.732	U/S IL = 15.200 + 0.15 - 0.7 - 0.375 = 14.150 < 14.315; D/S IL = 13.094 + 0.05 = 13.144 Slope = 4.7%
4	454	40	1%	525φ	14.300	13.850	13.242	2.1	20	0.5	0.225	0.113	0.377	0.490 (yes)	0.608 (yes)	1.165 (yes)	≤ 13.094	13.732	525φ	T = 13.732 B = 13.242	U/S IL = 14.300 + 0.15 - 0.7 - 0.525 = 13.[illegible] → 13.094; D/S IL = 12.540 + 0.05 = 12.590 Slope = 1.26%
5	576	40	1%	600φ	13.800	13.350	12.841	2.0	3	0.5	0.204	0.102	0.299	0.401 (yes)	0.509 (yes)	1.199 (yes)	≤ 12.540	13.242	600φ	T = 13.242 B = 12.841	U/S IL = 13.800 + 0.15 - 0.7 - 0.6 = 12.650 → 12.540; D/S IL = 11.975 + 0.05 = 12.025 Slope = 1.3%
6A	665	10	1%	600φ	13.400	12.950	12.750	2.3	3	0.5	0.270	0.135	0.099	0.234 (yes)	0.200 (No)						
			Try	750φ	✓	✓	✓	1.5	✓	✓	0.115	0.057	0.034	0.091 (OK)	0.200 (yes)	1.159 (yes)	≤ 12.034	12.841	750φ	T = 12.841 B = 12.750	U/S IL = 13.400 + 0.15 - 0.75 = 11.95 < 12.034; D/S IL = 11.850 + 0.05 = 11.900 Slope = 0.75%
9	113	55	2%	375φ	15.500	15.050	13.750	1.03	1	4	0.054	0.215	0.174	0.389 (yes)	1.300 (yes)	0.737 (No)	≤ 14.725	15.050	375φ	T = 15.050 B = 13.750	U/S IL = 15.500 + 0.15 - 0.7 - 0.375 = 14.350 < 14.725; D/S IL = 13.138 + 0.05 = 13.188 Slope = 1.9%
6C	220	15	7%	375φ	14.200	13.750	12.750	2.0	2	2	0.204	0.405	0.180	0.585	1.00 (yes)	0.743 (No)	≤ 13.138	13.750	375φ	T = 13.750 B = 12.750	U/S IL = 14.200 + 0.15 - 0.7 [illegible] = 13.138; D/S IL = 11.850 + 0.05 = 11.900 Slope = [illegible]
MH6	939	70	3%	600φ	13.200	12.750	10.656	3.3	15	2	0.562	1.123	1.442	2.565 (yes)	2.094 (No)						
			Try	750φ	✓	✓	✓	2.1	✓	2	0.230	0.461	0.472	0.933 (yes)	2.094 (yes)	1.597 (No)	≤ 12.060	12.750	750φ	T = 12.750 B = 10.656	U/S IL = 13.200 - 0.750 - 0.6 (hw) = 11.850 < 12.060; D/S IL = 9.265 + 0.05 = 9.315 Slope = 3.6%

Example

STORMWATER DRAINAGE

PIPE NETWORK DESIGN H.G.L. ANALYSIS

Sheet .... of ....
Designed ....
Checked ....
Signed 1.1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
Manhole	Outlet Pipe Flow	Length	Slope	Trial D	M.H. Lid R.L. (or K&C I.L.)	TWL = R.L. − 0·45	BWL	Velocity $V = Q/A$	Kw Code	Kw	$V^2/2g$	$hw = Kw\,(V^2/2g)$	$hf = \frac{0 \cdot 022L}{D}\,(V^2/2g)$	HT = hf + hw test (i) = [HT > 0·1?]	TWL − BWL test (ii) [> HT?]	hf + 1·5 D test (iii) = [≥ TWL−BWL?]	Pipe U/S I.L. [test (iii)=Yes] I.L ≤ BWL + hf − D [test (iii)=No] I.L ≤ TWL − 1·5 hw	T.W.L. [test (iii)=Yes] = BWL + HT [test (iii)=No] = TWL = Col. 7	Adopted D	Adopted T.W.L. & BWL.	REMARKS
	l/sec	m	m/m	mm	m	m	m	m/sec	-	-	m	m	m	m	m	m	m	m	mm	m	
MH7	1120	50	2.5%	600	11.700	10.750	9.550	3.9	15	2	0.776	1.552	1.423	2.975 (yes)	1.200 (No)						Bd. Pipe Obvert = 9.550 (say)
			Try	750	✓	✓	✓	2.5	✓	2	0.319	0.638	0.468	1.106 (yes)	1.200 (yes)	1.594 (yes)	≤ 9.268	10	750 ø	T = 10.656 B = 9.550	D/S IL = 9.550 − 0.750 = 8.800 U/S IL = 11.2 − 0.750 − 0.6 = 9.850 (9.268) T/S IL = (1.100) Slope = 2.94%
	Option (1) − Rev Pipe work																				
MH2	1493	50	2.5%	900	11.200	10.750	9.550	2.35	15	2	0.282	0.564	0.345	0.909 (yes)	1.200 (yes)	1.695 (yes)	≤ 8.995	10.457	900 ø	T = 10.459 B = 9.550	U/S IL = 11.2 − 0.9 − 0.6 = 9.700 = (8.995) D/S IL = 9.550 − 0.9 = (8.650) Slope = 0.7%
				750	✓	✓	✓	3.4	✓	✓	0.582	1.164	0.854	2.018	1.200 (No)						

PRIMARY PIPEWORK DESIGN

Sheet __ of __

PROJECT Example	Designed .... Date	Checked .... Date ....

(1) GULLY BOX	(2) K B C I.L.	(3) T.W.L. = I.L. - 0.150	(4) B.W.L. = Manhole T.W.L.	(5) H TOT. T.W.L. - B.W.L.	(6) Q = GULLY INFLOW	(7) $\frac{H\ TOT.}{Q^2}$	(8) PIPE (D)	(9) D/S I.L. = I.L. - L.S.	(10) H/S I.L. = (2) - 0.575 - D	(11) L = PIPE LENGTH	(12) PIPE SLOPES
	m	m	m	m	l/sec	m/l/sec^2	mm	m	m	m	%
3B	15.400	15.250	14.750	0.500	34	432×10^{-6}	375ϕ	14.450	14.30	15	1%
6B	13.500	13.350	12.750	0.600	143	29×10^{-6}	450ϕ	12.475	11.9	15	3.8%
6D	14.200	14.050	12.750	1.300	35	1124×10^{-6}	375ϕ	13.250	11.9	15	9%
7A	11.200	11.050	10.656	0.394	172	13×10^{-6}	450ϕ	[illegible].175	9.318	15	5.7%
7B	12.000	11.850	10.656	1.194	110	98×10^{-6}	375ϕ	11.050	9.318	20	11.5%
7C	12.000	11.850	10.656	1.194	17	4131×10^{-6}	375ϕ	11.050	9.318	20	11.5%

Flow Chart for Concrete Pipe Lines Flowing Full

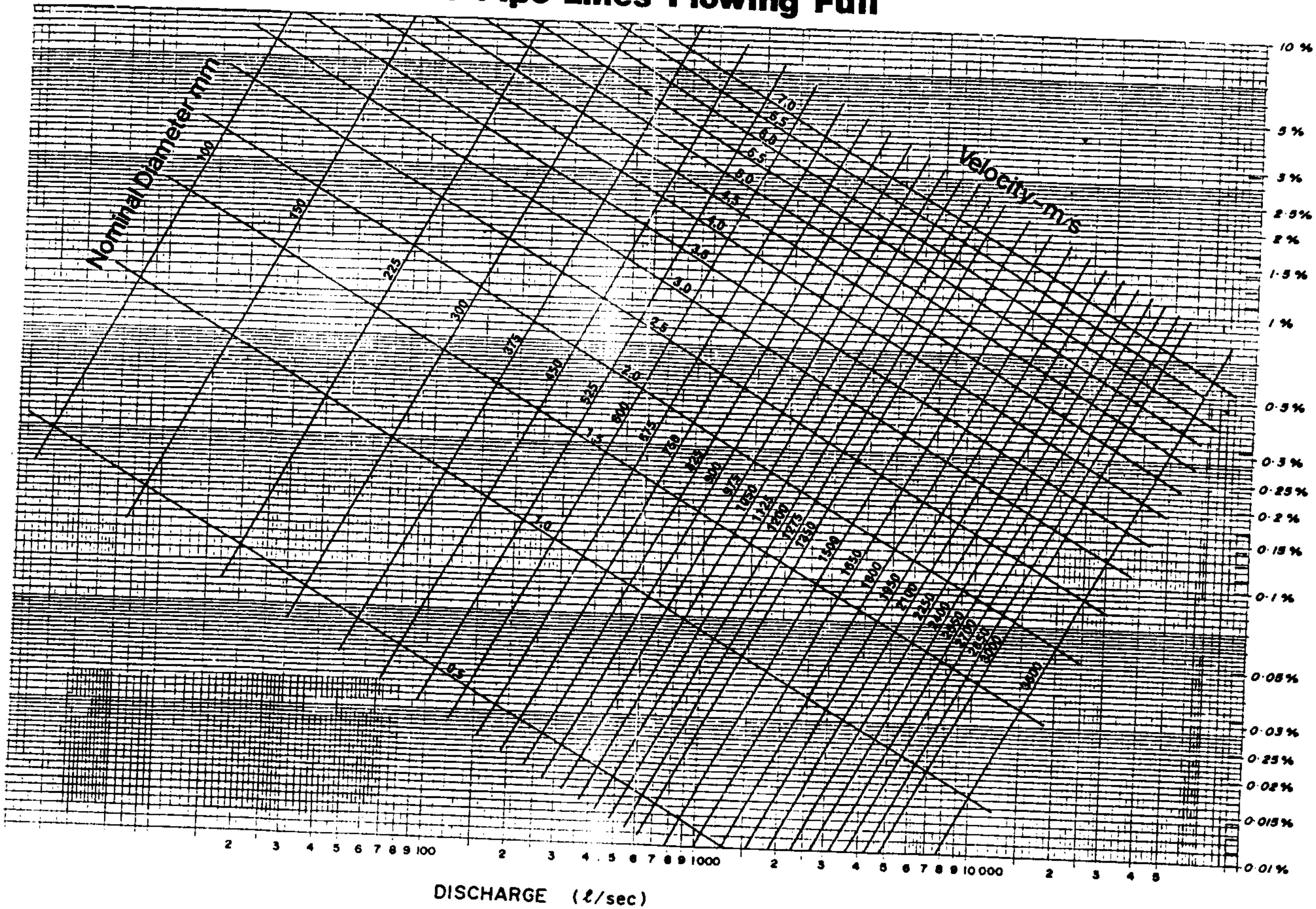
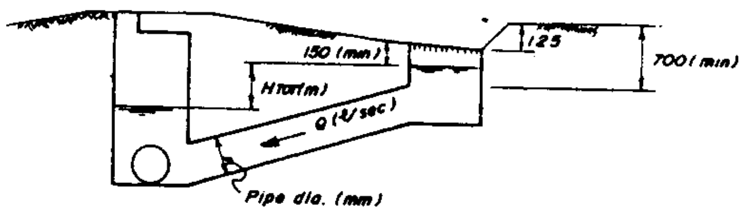


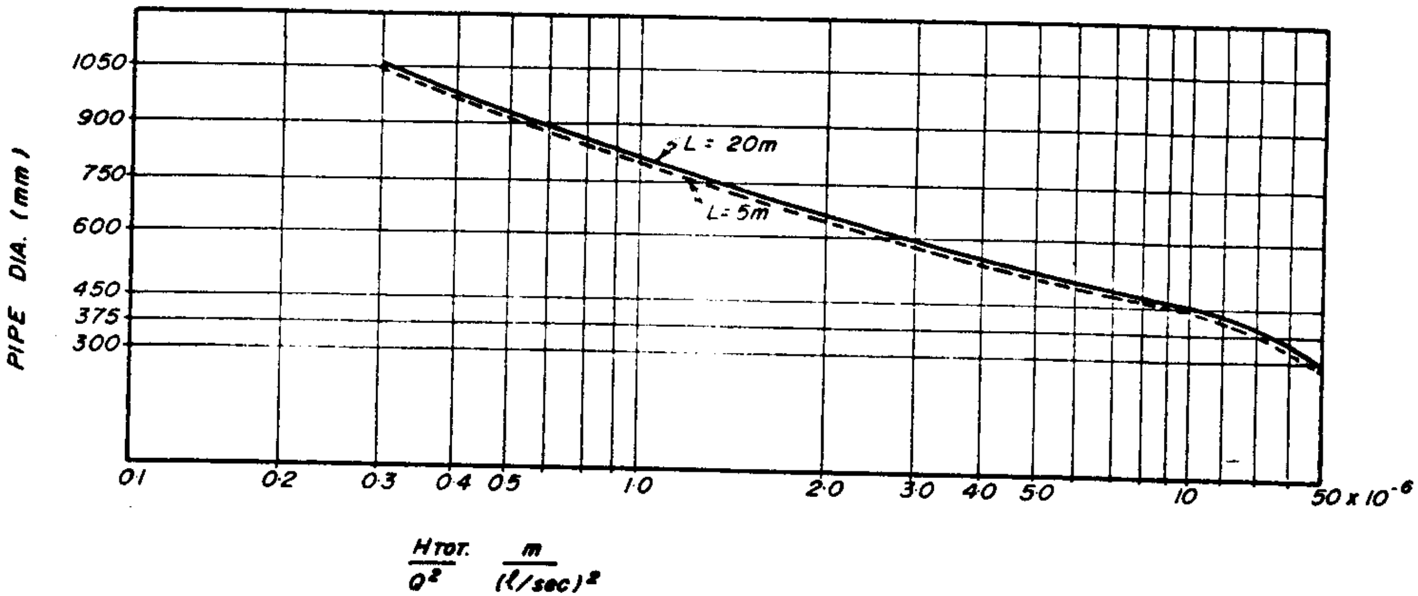
Chart 10.2

Example

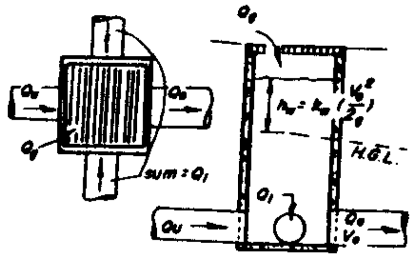
(a) $\frac{H_{TOT}}{Q^2} = \frac{0.15}{(100)^2} = 1.5 \times 10^{-5} = 15 \times 10^{-6}$ = 450ϕ pipe

(b) $\frac{H_{TOT}}{Q^2} = \frac{0.1}{(200)^2} = 2.5 \times 10^{-6}$ = 750ϕ pipe





PRIMARY PIPEWORK DESIGN CHART



1. INLET/MANHOLES WITH GUTTER FLOW

Code	Description	$Q_u \approx$	$Q_1 \approx$	$Q_g \approx$	$K_w =$
1	Inlet pit with single pipe outflow	-	-	Q_o	4.0
	Inlet on through pipeline				
2	$Q_u \approx Q_g$	$Q_o/2$	-	$Q_o/2$	2.0
3	$Q_u \approx Q_o$	Q_o	-	some	0.5
	Inlet on through pipe with lateral(s)				
4	$Q_u >> Q_1$	Q_o	some	some	0.5
5	$Q_u > Q_1$	$Q_o/2$	some	$Q_o/2$	1.5
6	$Q_u \approx Q_1$	$Q_o/2$	$Q_o/2$	some	1.5
7	$Q_u << Q_1$	some	Q_o	some	2.0
8	$Q_u < Q_1$	some	$Q_o/2$	$Q_o/2$	2.5
9	Inlet on 'L' pipe junction i.e. $Q_u = 0$	-	Q_o	some	2.5
	Inlet on 'T' pipe junction i.e. $Q_u = 0$				
10	opposed laterals	-	Q_o	some	3.0
11	offset laterals	-	Q_o	some	2.5

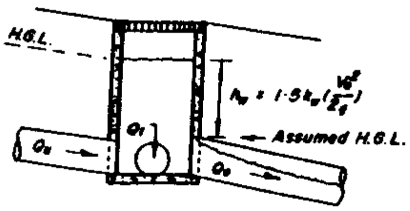
2. MANHOLES WITHOUT GUTTER FLOW

Code	Description	$Q_u \approx$	$Q_1 \approx$	Q_g	$K_w =$
12	Junction pit on through pipeline i.e. $Q_u = Q_o$	Q_o	-	-	0.2
	Junction pit on through pipe with lateral(s)				
13	$Q_u >> Q_1$	Q_o	some	-	0.5
14	$Q_u \approx Q_1$	$Q_o/2$	$Q_o/2$	-	1.0
15	$Q_u << Q_1$	some	Q_o	-	2.0
16	Junction pit on 'L' pipe junction i.e. $Q_u = 0$	-	Q_o	-	2.0
	Junction pit on 'T' pipe junction i.e. $Q_u = 0$				
17	opposed laterals	-	Q_o	-	2.5
18	offset laterals	-	Q_o	-	2.0

Note: Circular pits have slightly improved performance, however these should still be used in such cases.

Table 10.1

3. PART-FULL OUTFLOW FROM MANHOLES



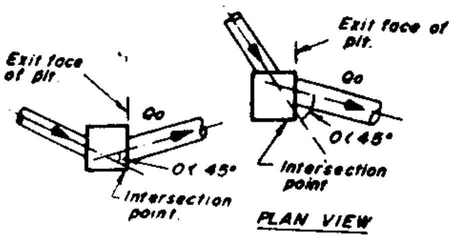
Part - full outflow from a junction pit

Code	Description	
19	Part-full cut flow	1.5 Kw

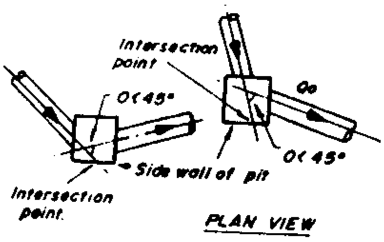
where $V_o = Q_o/A_o$

A_o = discharge pipe full area

4. MANHOLES WITH SINGLE ENTRY/EXIT PIPES



Code	Description	Q_g	KW
20	For $\Theta < 45^o$	0 or small	0.5
21		$\approx Q_o/2$	1.5



Code	Description	Q_G	Kw
22	$\Theta > 45^o$	all values	1.5

APPROXIMATE VALUES OF CO-EFFICIENTS (Kw) Ref. 3

Table 10.1 (cont.)

5. DROP MANHOLES WITH LENGTH ENTRY AND EXIT PIPES
(both submerged under design flow conditions)

Code	Description	Q_B	Kw
23	$\theta < 45^{\circ}$ - rectangular pits	0	2.0
24	- circular pits	0	1.5
25	$\theta > 45^{\circ}$ - rectangular pits	0	2.5
26	- circular pits	0	2.0

6. ALL OTHER CASES

Code	Description	Kw
27	For all flows	3.0

APPROXIMATE VALUES OF CO-EFFICIENT Kw

Ref. 3

Table 10.1 (cont.)

SECTION 11 - OUTFALLS

11.1 GENERAL

In the design of a drainage system, it is essential that appropriate receiving water levels be determined. In steep terrain, the backwater affects only a short distance from the stream outlet, however, in flat country these effects may be extensive and the size and cost of the drainage system may be sensitive to the levels adopted.

11.2 OPEN CHANNELS

Where the system outlet discharges into a natural open channel that is essentially an extension of the system, tailwater levels shall be calculated for A.R.I. = 100 years or the pipe obvert level, whichever is the greater. This ensures that the minor system can operate as designed in conjunction with major system flows.

11.3 CREEKS

For creeks that have catchment areas up to 10 times the catchment area of the drainage system, levels shall be calculated for A.R.I. = 100 years, or the pipe obvert, whichever is the greater

11.4 RIVERS

The drainage system discharge is not likely to affect the receiving water levels, and also its flow hydrograph will pass before the river reaches its peak, however, subsequent storm events are possible before the river hydrograph passes the drainage system outlet.

In such cases, statistical analysis methods from Chapter 10, "Australian Rainfall and Runoff" should be utilized. In any case, the pipe obvert level shall be taken as the minimum tailwater level.

11.5 TIDAL OUTLETS

Ocean outlets including tidal reaches of rivers etc., shall be designed for tailwater levels in accordance with the following or the pipe obvert level, whichever is the greater.

Tailwater Level = 4.0m A.H.D.

STORMWATER DESIGN STANDARD DRAWINGS

S.W.D.S.	1	End Walls and Aprons to Pipe and Box Culverts
	2	Standard Floodway and Causeway Details
	3	Stormwater Manholes
	4	Under Kerb Pipes with Sub-soil Drainage
	5	Side Inlet Manhole Components
	6	Side Inlet Manhole Cast Iron Cover and Frame
	7	Side Inlet Manhole Load Test Procedure
	8	Side Inlet Manhole Construction Details

DESIGN CHARTS

(a) Pipe and Box Culverts
(b) Stage Discharge
(c) Major System Design
(d) Minor System Surface Flows
(e) Mainline Pipe Flows
(f) Pipe Network Design
(g) Primary Pipework Design

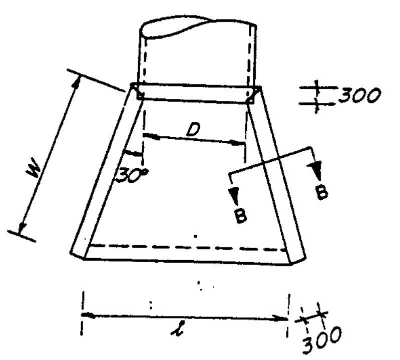
R E F E R E N C E S

1. I.E. Aust (1987) "Australian Rainfall & Runoff"

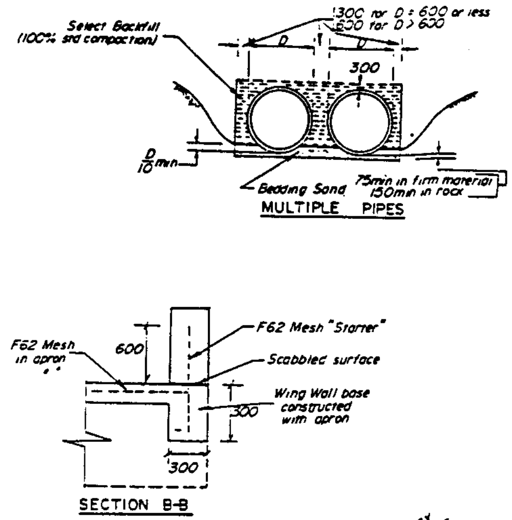
2. M.R.D. "Urban Road Design Manual" - Vol. 2

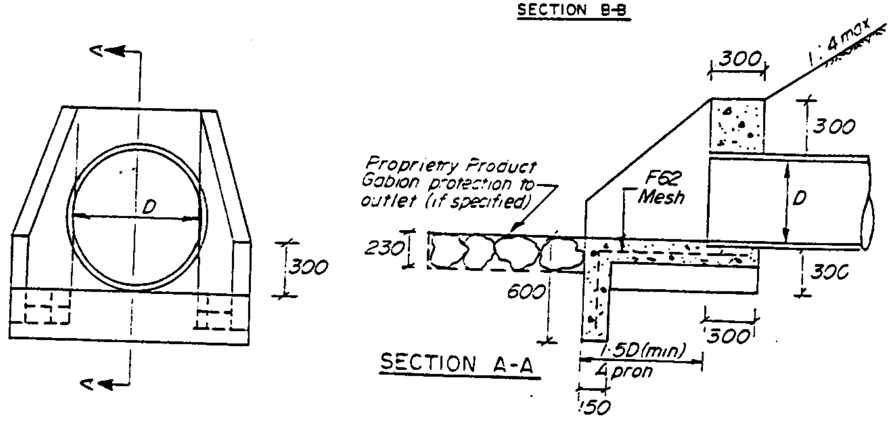
3. Argue, J.R. (1987) "Storm Drainage Design in Small Urban Catchments" A.R.R.B. Special Report No.34

4. M.R.D. "Standard Drawings Manual" - Vol. 1



NOTES :-
(1) Concrete = 20 MPa/20
(2) Precast Headwalls ⩽ 750 ⌀ Pipes

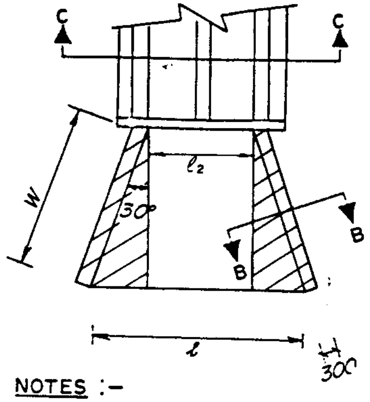




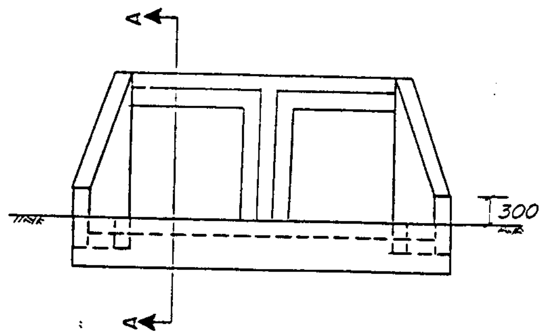
PIPE CULVERTS

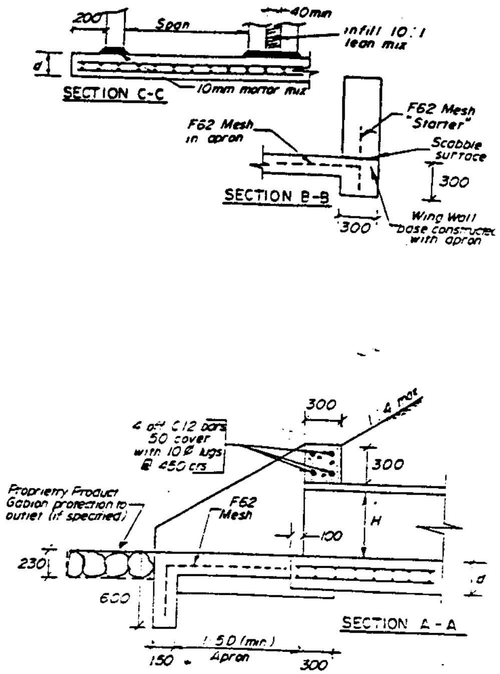
BOX CULVERTS

Pipe ⌀	w	ℓ	1.5D	Concrete Qtys per end (m³) Apron + cut off wall + 2 Wing Walls + Headwall	each addit pipe Headwall + Apron + Cut off wall
900	1273	4500	1350	1.72	1.3
1200	1697	6000	1800	3.1	1.81
1500	2121	7500	2250	4.11	2.64
1800	2545	9000	2700	5.5	3.4
2100	2970	10500	3150	7.0	4.25
2400	3394	12000	3600	8.8	5.2



NOTES :-
(1) Concrete = 20 MPa/20
(2)





Culvert Height (D)	w	ℓ	1.5D	Concrete Qtys per end (m³) Qty per shaded AREA	Qty per M span (ℓ_2)
600	1220	2220	900	0.9m³	0.3m³/m
750	1470	2470	1125	1.15m³	0.33m³/m
900	1730	2730	1350	1.46m³	0.36m³/m
1200	2250	3250	1800	2.1m³	0.43m³/m
1500	2770	3770	2250	2.9m³	0.5m³/m
1800	3300	4300	2700	3.86m³	0.57m³/m
2100	3810	4810	3150	4.8m³	0.63m³/m
2400	4330	5330	3600	6.1m³	0.7m³/m
2700	4850	5850	4050	7.5m³	0.77m³/m
3000	5370	6370	4500	8.5m³	0.83m³/m
3300	5890	6890	4950	10.5m³	0.91m³/m
3600	6410	7410	5400	12.2m³	0.97m³/m

SLAB DETAILS

D = Height of opening
B = Span = internal width

B Span	Thickness of slab, d	Reinforcing fabric type	Position of fabric in slab
300	120	F718	Single layer on centre line
375	120	"	
450	120	"	
600	120	"	
750	130	F818	
900	140	F1018	
1200	150	F1118	
1520	160	F818	Single layer at top and bottom 40 cover
1830	170	F818	
2130	180	F918	
2440	190	F918	
2740	200	F1018	
3050	210	F1018	
3350	220	F1118	
3650	[illegible]	F11[illegible]	

(A) QUANTITY CALCULATIONS

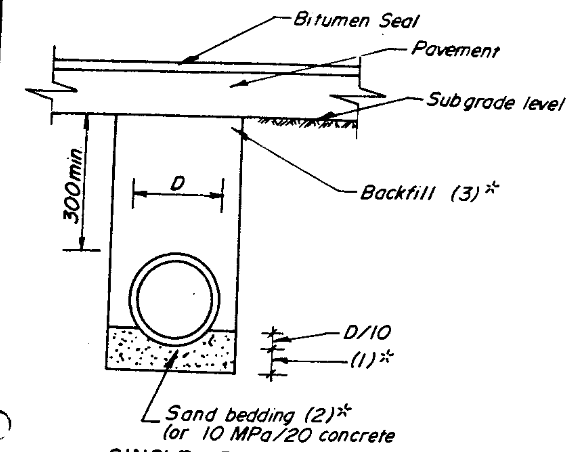
REVISIONS | APPROVED | DATE

CALLIOPE SHIRE COUNCIL

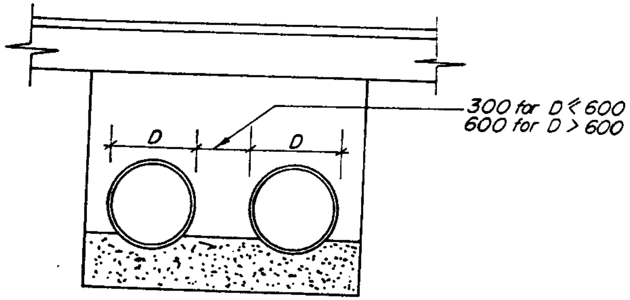
END WALLS & APRONS TO PIPE & BOX CULVERTS

SCALE : N.T.S.
DATE : 25th AUGUST, '89
APPROVED :
DESIGNED :
CHECKED :
DRAWN : C.H.
TRACED

S.W.D.S. 1.



SINGLE PIPE



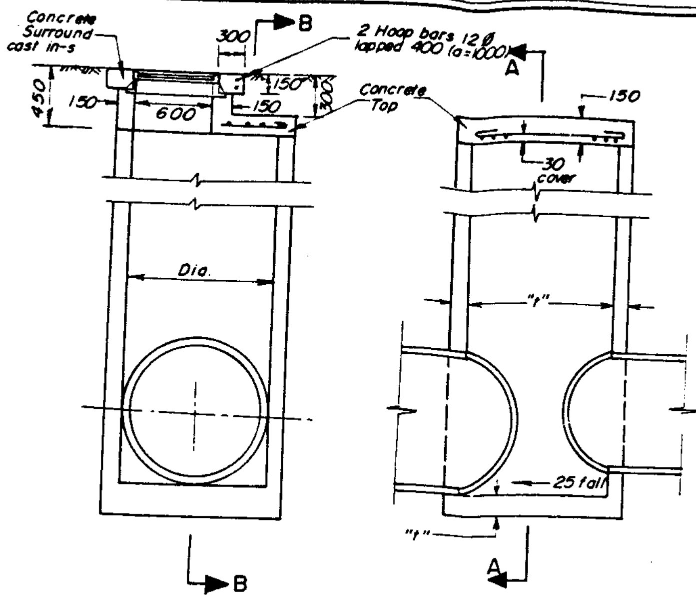
MULTIPLE PIPES

NOTES :-

(1.)* 75 min. in firm material
150 min. in rock

(2)* Sand bedding - sharp, clean, siliceous sand max. particle size = 5mm.

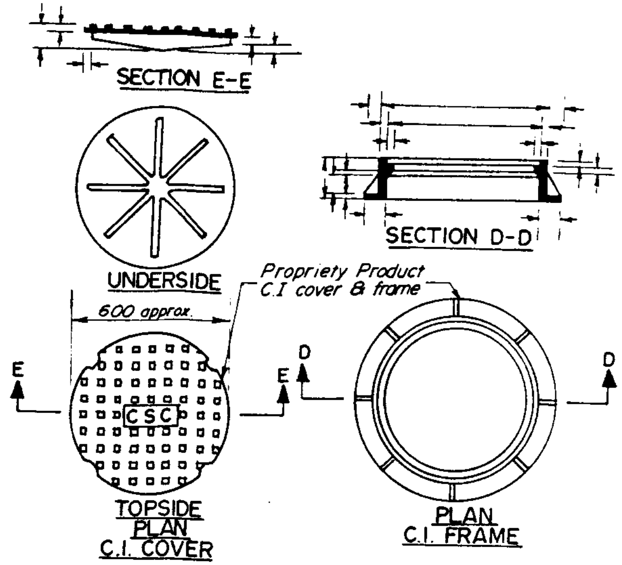
(3)* Backfill material - gravel / loam, free from vegetable matter, particle size = 10mm (max.), linear shrinkage = 8% (max.), compacted to subgrade level at 100% std. compaction in 100mm (max.) layers, or cement stabilized sand (20 : 1 mix)



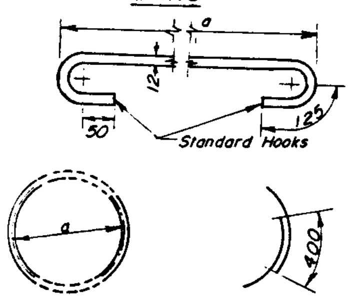
SECTION A-A

SECTION B-B

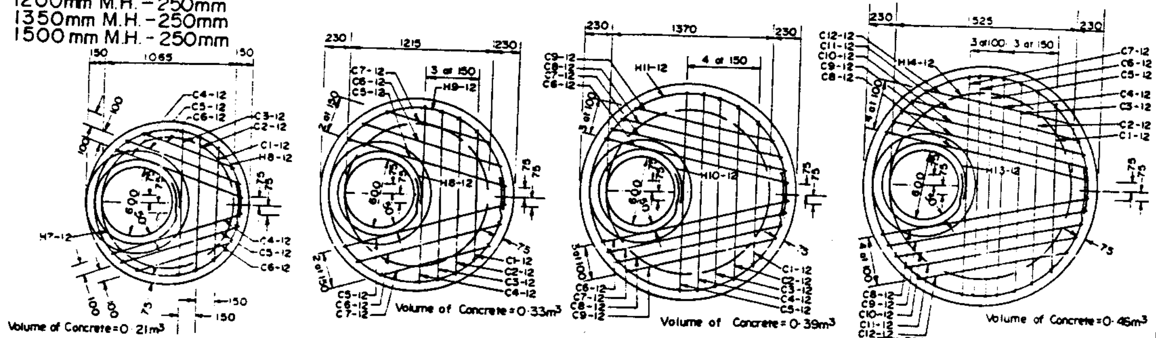
NOTES :- 1. Reinforcing steel shall be plain round structural grade steel bars.
2. Concrete 20MPa/20
3. Wall thickness "t" : 1050mm M.H. - 150mm
1200mm M.H. - 250mm
1350mm M.H. - 250mm
1500mm M.H. - 250mm



COVER AND FRAME DETAIL
ROADWAYS & FOOTPATHS



REINFORCING SHAPES



REINFORCING SCHEDULE : 1050 DIA MANHOLE

MARK	DIA	a	OVERALL LENGTH	Nº OFF	TOTAL LENGTH
C1-12	12	950	1180	1	1180
C2-12	12	1140	1370	1	1370
C3-12	12	1245	1475	1	1475
C4-12	12	1140	1370	2	2740
C5-12	12	1020	1250	2	2500
C6-12	12	825	1055	2	2110
H7-12	12	685	2600	1	2600
H8-12	12	1220	4270	1	4270
					TOTAL = 16245
					WEIGHT = 6.6 kg

REINFORCING SCHEDULE : 1200 DIA MANHOLE

MARK	DIA	a	OVERALL LENGTH	Nº OFF	TOTAL LENGTH
C1-12	12	1220	1450	1	1450
C2-12	12	1420	1650	1	1650
C3-12	12	1525	1750	1	1750
C4-12	12	1560	1800	1	1800
C -12	12	1475	1700	2	3400
C6-12	12	1320	1550	2	3100
C7-12	12	1065	1300	2	2600
H8-12	12	685	2590	1	2590
H9-12	12	1525	5230	1	5230
					TOTAL = 23570
					WEIGHT = 21·5 kg

REINFORCING SCHEDULE : 1350 DIA MANHOLE

MARK	DIA	a	OVERALL LENGTH	Nº OFF	TOTAL LENGTH
C1-12	12	1295	1520	1	1520
C2-12	12	1510	1750	1	1750
C3-12	12	1640	1880	1	1880
C4-12	12	1700	1930	1	1930
C5-12	12	1700	1930	1	1930
C6-12	12	1625	1850	2	3700
C7-12	12	1550	1780	2	3560
C8-12	12	1435	1680	2	3360
C9-12	12	1285	1520	2	3040
H10-12	12	685	2590	1	2590
H11-12	12	1675	5720	1	5720
					TOTAL = 30980
					WEIGHT = 28·3 kg

REINFORCING SCHEDULE : 1500 DIA. MANHOLE

MARK	DIA	a	OVERALL LENGTH	No. OFF	TOTAL LENGTH
C1-12	12	1345	1600	1	1600
C2-12	12	1600	1830	1	1830
C3-12	12	1740	1980	1	1980
C4-12	12	1830	2060	1	2060
C5-12	12	1855	2080	1	2080
C6-12	12	1865	2110	1	2110
C7-12	12	1855	2080	1	2080
C8-12	12	1790	2030	2	4060
C9-12	12	1725	1960	2	3920
C10-12	12	1625	1850	2	3700
C11-12	12	1485	1730	2	3460
C12-12	12	1295	1520	2	3040
H13-12	12	685	2590	1	2590
H14-12	12	1830	6200	1	6200
					TOTAL = 40718
					WEIGHT = 37·1 kg

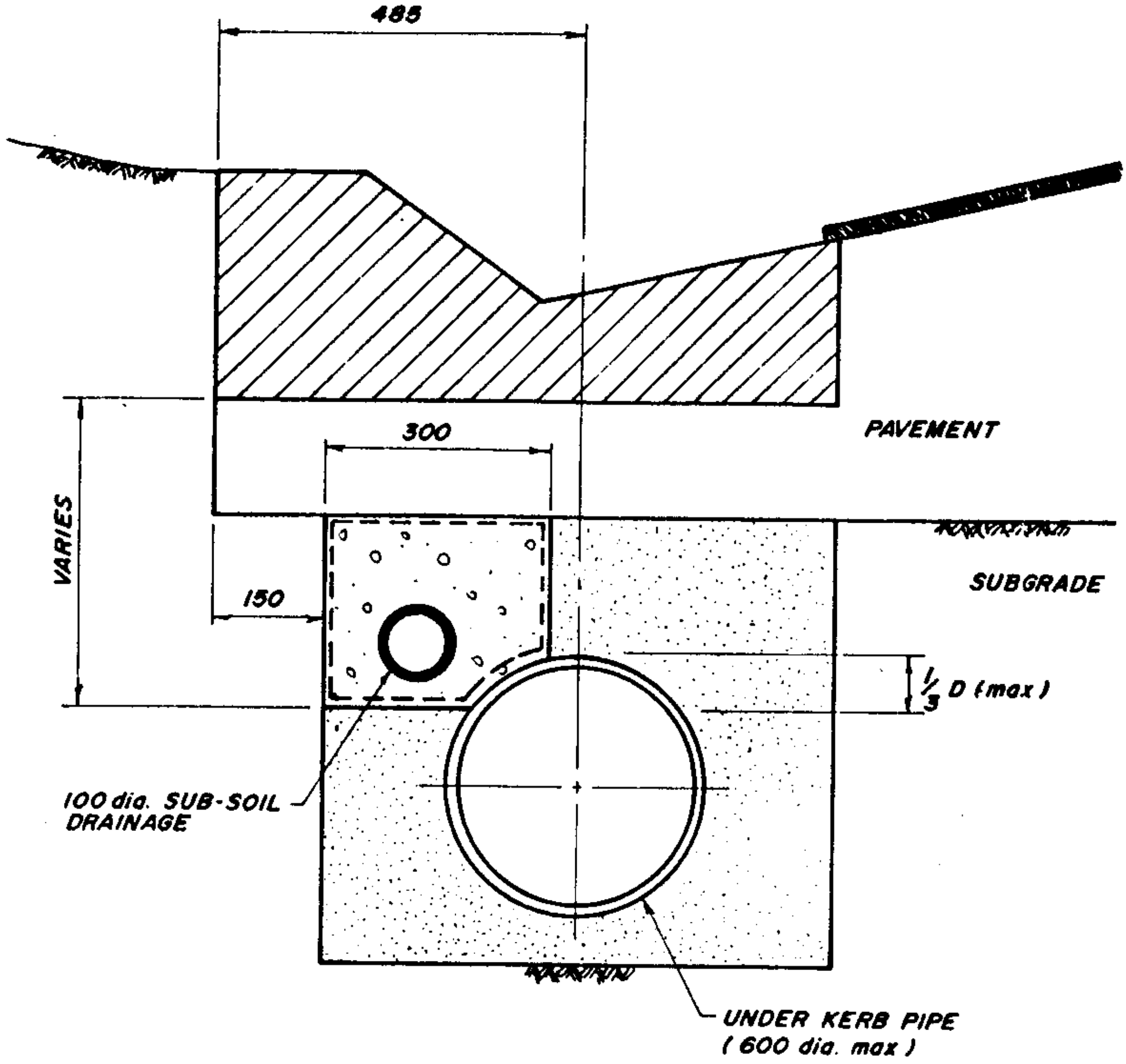
REVISIONS	APPROVED	DATE

CALLIOPE SHIRE COUNCIL

STORMWATER MANHOLES AND PIPE BEDDING DETAILS

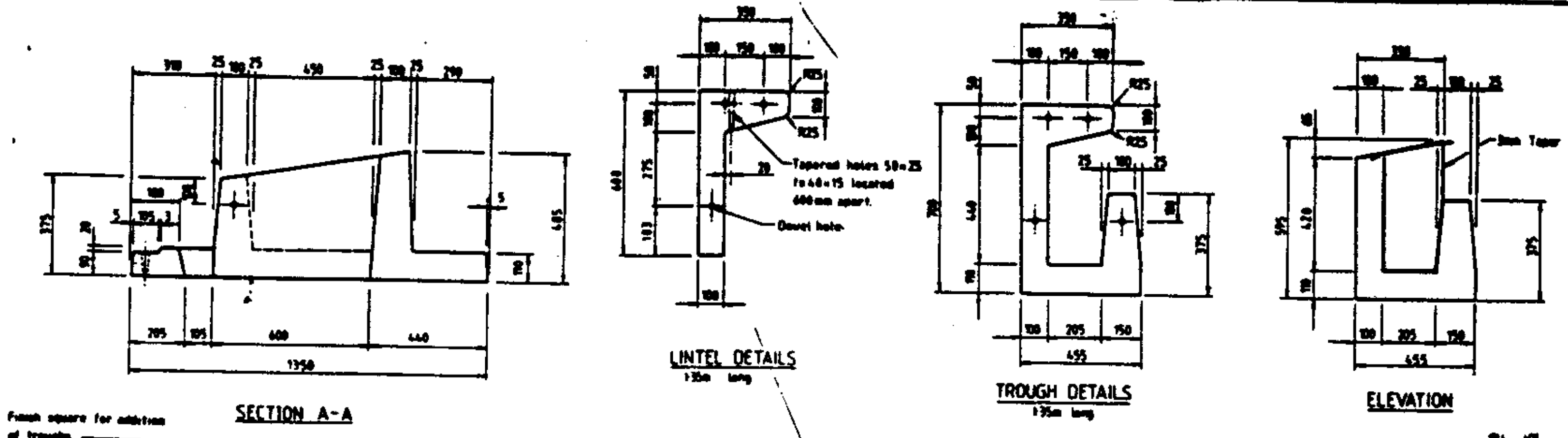
SCALE : N. T. S. DATE : *30th AUGUST, '89*
APPROVED :
DESIGNED :
CHECKED :
DRAWN : C.H.
TRACED :

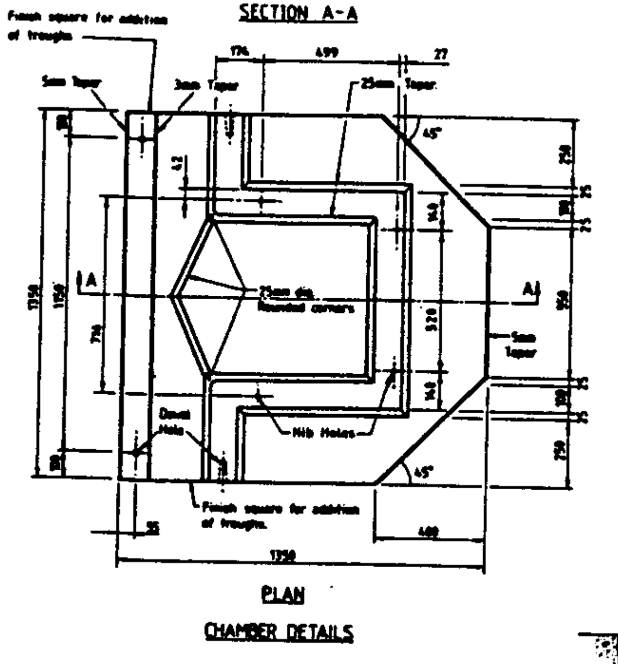
S.W.D.S. 3

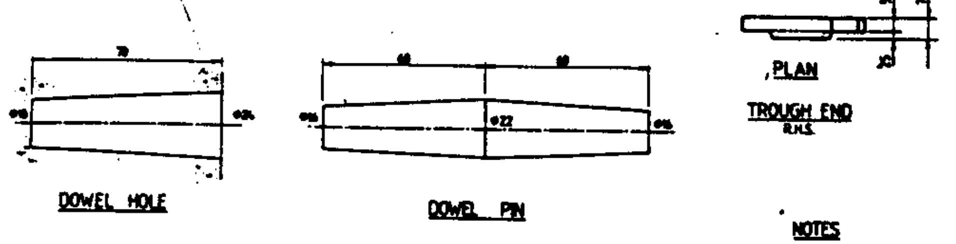


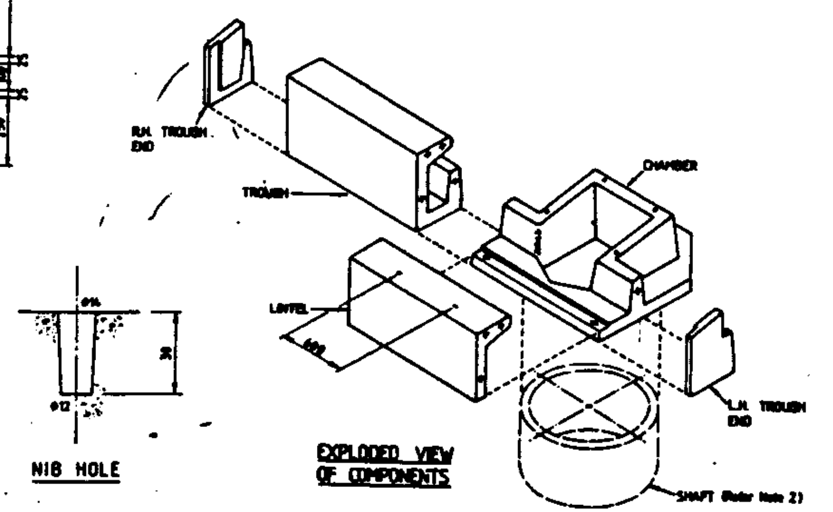
NOTES: - Sub-soil drainage installation in accordance with R.D.S. 28.
- Pipe installation in accordance with S.W.D.S. 3.

CALLIOPE SHIRE COUNCIL	SCALES: NONE		DATE: 15th AUGUST, '89
	APPROVED:		
UNDER KERB PIPES WITH SUB-SOIL DRAINAGE	DESIGNED:	S.W.D.S. 4.	
	CHECKED:		
	DRAWN: D.P.W.		
	TRACED:		









NOTES

1. Alternative arrangements for various components may be approved subject to the external dimensions and overall external dimensions remaining unchanged.
2. The standard shaft external diameter shall be 1050mm nominal either insitu or precast. Where larger manholes are used, converter slabs may be required between manhole shaft and chamber.
3. Components shall comply with test loading criteria in L.G.E.A. Drawing No. SW-07
4. Minimum concrete cover shall be 30mm. Where the cover is less than 40mm, the maximum aggregate size shall be 10mm.
5. Minimum concrete characteristic compressive strength = 32MPa
6. The cast iron frame shall be supplied fixed to the chamber by the chamber manufacturers.

A	ORIGINAL ISSUE		9/87
	REVISIONS	APPROVED	DATE

CALLIOPE SHIRE COUNCIL

SIDE INLET MANHOLE COMPONENTS

L.G.E.A. (Qld) STANDARD DRAWING SW-05

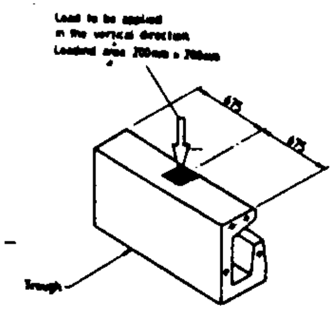
S.W.D.S. - 5.

A

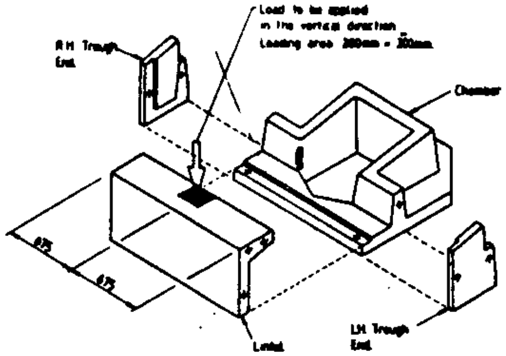
COMPONENT	CHAMBER	LINTEL	TROUGH
Proof Load	90 kN	50 kN	50 kN
Ultimate Load	[illegible] kN	75 kN	75 kN

LOADING CRITERIA

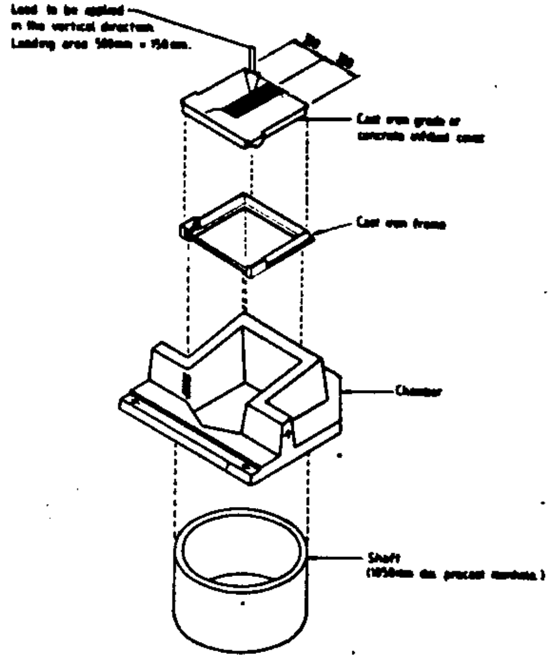
TABLE A



TROUGH TEST



LINTEL TEST



CHAMBER TEST

INSPECTION AND TESTING

NOTES

1. INSPECTION

   All materials, processes of manufacture, and finished components shall be liable to inspection and approval by the purchaser.

2. LOAD TEST REQUIREMENTS

   Proof Load Test. From each twenty components, which shall be called a batch and suitably identified as such, of each component type, the purchaser may select one component for proof load testing.

   When so tested, every component shall sustain the load given in Table A without developing a test crack greater than that defined in Note 3 below, and on removal of the load, no crack caused by the load shall be greater than that defined in Note 4 below as a residual test crack.

3. A test crack is defined as one into which the point of a test-crack measuring gauge conforming to the details given in Figure 88(a) of AS1597, Part 1 1974, may be inserted to a depth of 3mm over a length of at least 300mm at intervals not exceeding 50mm.

4. A residual test crack is defined as one into which the point of a residual test-crack measuring gauge conforming to the details given in Figure 88(b) of AS1597, Part 1 1974, may be inserted to a depth of 3mm over a length of at least 300mm at intervals not exceeding 50mm.

5. Ultimate Load Test. The purchaser may select the required number of components for ultimate load testing.

   When so tested, every component shall sustain the load given in Table A.

6. RETESTS

   Load Test. Should any component fail to meet the load test requirements, the purchaser shall select two further components from the batch represented by the component that failed. These components shall be subjected to the proof load test (Note 2) or the ultimate load test (Note 5) or both tests, as specified by the purchaser.

   If both of the components meet the load test requirement(s), the batch shall be deemed to comply with the test requirement(s).

   If one of the retest components fails, the purchaser may reject the batch or may request that every component in the batch be subjected to the proof load test.

7. ACCEPTANCE

   All the components represented by those components which have been tested and found to comply with the requirements specified herein shall be accepted subject to final inspection in accordance with Note 1.

   Components which have been subjected to the ultimate load test may be accepted for use by the purchaser provided they comply in all respects with this Standard Drawing.

METHOD FOR THE LOAD TESTING OF COMPONENTS

8. APPARATUS

   The following apparatus is required:-

   a) Testing machine. A machine capable of applying to a component the forces specified in Table A without undue deformation or yielding of the machine. The machine shall be mounted over a sand-bed complying with the requirements of paragraph (e) below.

   b) Load-measuring device. The testing machine shall be fitted with a load-measuring device capable of indicating the applied load to within ± 2 per cent of the correct load within the range of use. The device shall have been calibrated within 1 year by an independent authority acceptable to the purchaser, and the calibration certificate shall be made available for inspection by the purchaser on request.

   c) Bearing blocks. Hardwood bearing blocks shall be 500mm long, 150mm wide and 100mm deep and 200mm long, 200mm wide and 100mm deep.

   d) Rubber pads. Rubber pads shall have the same plan area as the bearing block and shall be 20-25mm thick of Shore durometer hardness 40-60.

   e) Bed. A sand-bed of sufficient area to accommodate the component. The top and bottom surfaces of the sand-bed shall be level and the sand shall be not less than 75mm deep. Alternatively a concrete slab bed may be used.

   f) Crack measuring gauges. Leaf gauges of the dimensions shown in Figure 88(a) and (b) of AS1597, Part 1 1974.

9. LOCATING COMPONENT IN TESTING MACHINE

   The test components shall be positioned on the bed with the component sections properly assembled as in service, or propped with restraints as described below.

   A bed of mortar not more than 5mm thick or a bituminous felt membrane may be used to protect the bearing surfaces of the component during the application of the test load.

   The orientation and position of the component under the loading beam of the testing machine shall be appropriate to the loading conditions specified in Note 10.

10. APPLICATION OF TEST LOAD

    The test load shall be applied vertically to the component through a wooden bearing block and rubber pad, with the loading beam and bearing block parallel with a vertical plane through the longitudinal axis of the component. The position of the load on the component shall be as shown on this Standard Drawing.

    For proof load testing the load shall be increased uniformly so that the specified value is reached in 5 minutes. While subjected to the proof load the component shall be examined for cracks which, if present, shall be checked by means of the crack-measuring gauge. The load shall then be released and the component again examined to check whether all test cracks have closed.

    The ultimate test load shall be applied at the same uniform rate as the proof load until the component sustains the specified ultimate load.

A	ORIGINAL ISSUE		9/87
	REVISIONS	APPROVED	DATE

CALLIOPE SHIRE COUNCIL

SIDE INLET MANHOLE

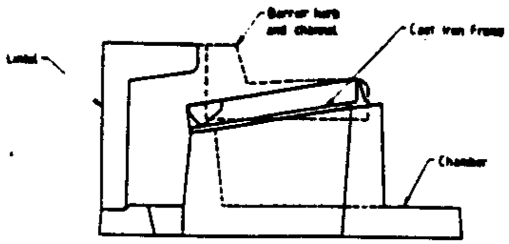
LOAD TEST PROCEDURE

L.G.E.A. (Qld) STANDARD DRAWING

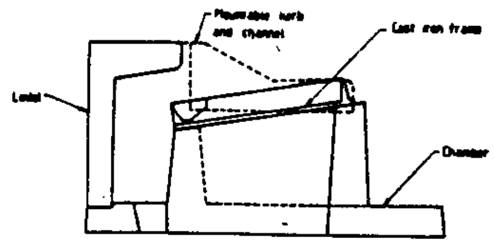
SW-07

S.W.D.S. - 7.

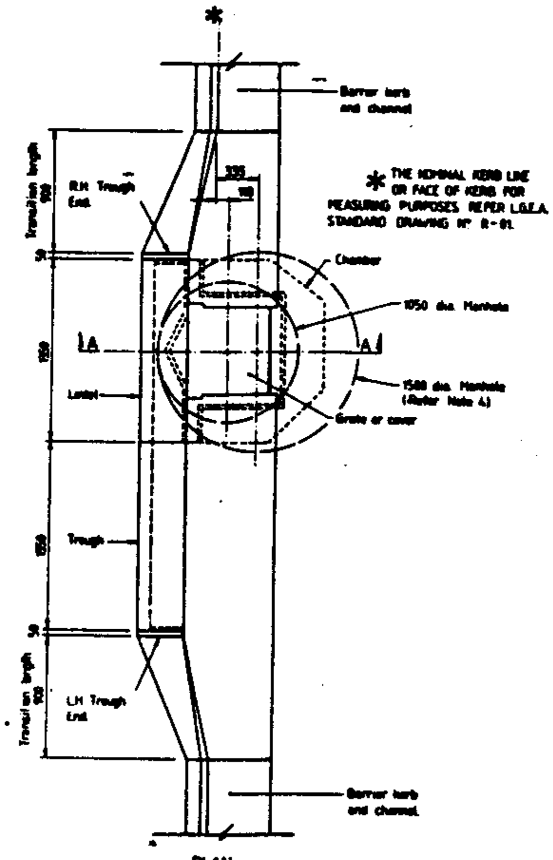
A



SECTION A-A



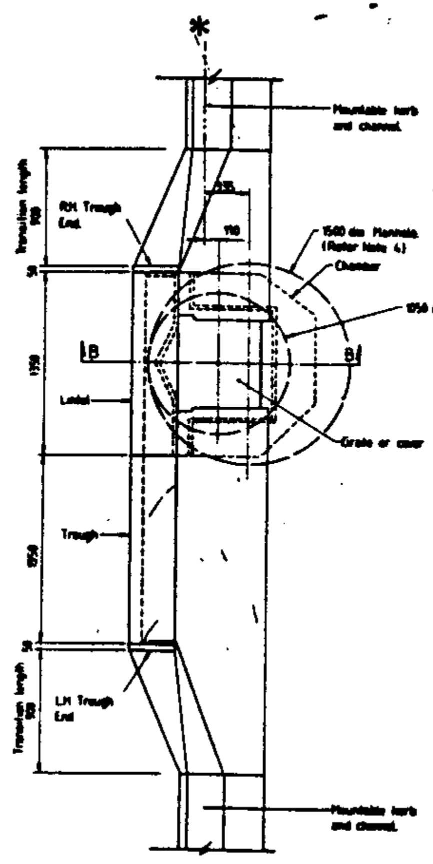
SECTION B-B



PLAN

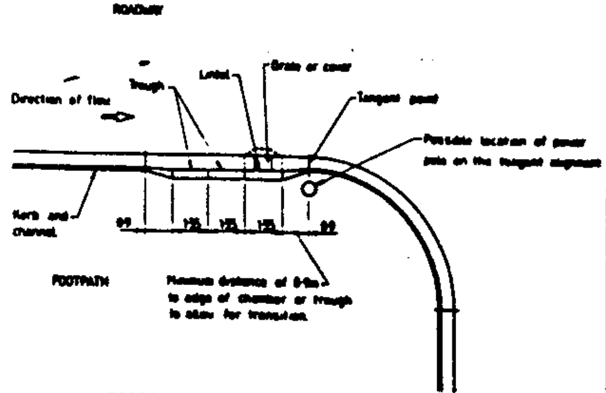
BARRIER KERB AND CHANNEL

* THE NOMINAL KERB LINE OR FACE OF KERB FOR MEASURING PURPOSES REFER L.G.E.A. STANDARD DRAWING Nº R-01.



PLAN

MOUNTABLE KERB AND CHANNEL



POSSIBLE LAYOUT AT CORNERS

NOTES

1 Precast components shall be connected by means of dowel pins. If specified by the Engineer approved corrosion protection shall be applied to dowel pins.

2 If ordered by the Engineer joints between components shall be sealed (with approved mortar or taping) to prevent ingress of backfill material.

3 An approved manhole converter slab shall be used under the chamber for manhole shaft diameters larger than 1050mm.

4 Stormwater pipes parallel to kerb and channel shall be located no closer to the footpath than the back face of kerb unless directed by the Engineer.

SYSTEM	NO SHAFT	SHAFT 'H' 450	SHAFT 'H' 600	SHAFT 'H' 900	SHAFT 'H' 1200
	0TCS0	0TCS4/5	0TCS6	0TCS9	0TCS12
	1TCS0	1TCS4/5	1TCS6	1TCS9	1TCS12
	2TCS0	2TCS4/5	2TCS6	2TCS9	2TCS12
	3TCS0	3TCS4/5	3TCS6	3TCS9	3TCS12
	4TCS0	4TCS4/5	4TCS6	4TCS9	4TCS12

TERMINOLOGY TABLE

A	ORIGINAL ISSUE		9/87
	REVISIONS	APPROVED	DATE

CALLIOPE SHIRE COUNCIL

SIDE INLET MANHOLE
CONSTRUCTION DETAILS
BARRIER KERB AND CHANNEL
MOUNTABLE KERB AND CHANNEL

L.G.E.A. (Qld) STANDARD DRAWING SW-08

S.W.D.S. - 8.

A B

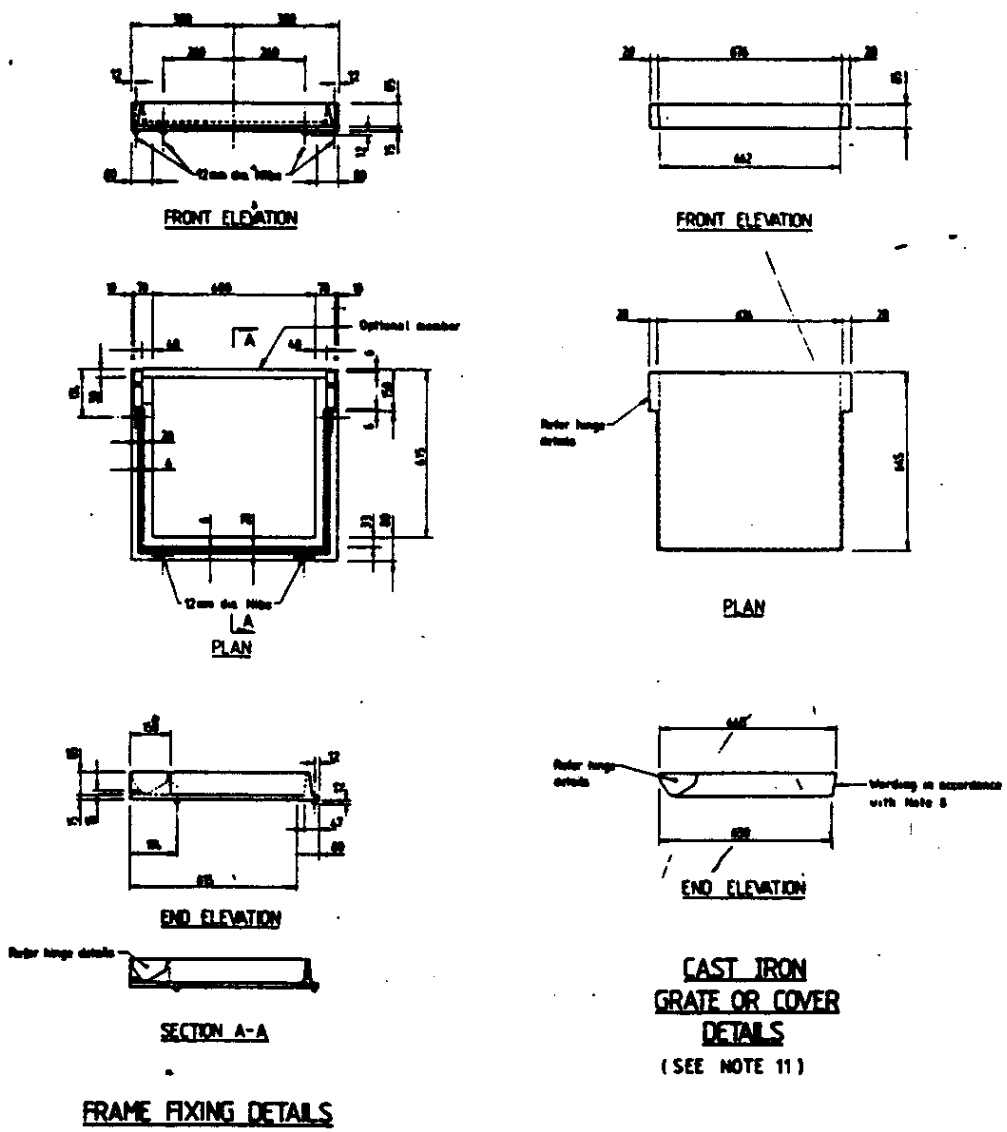
STORMWATE DRAINAGE

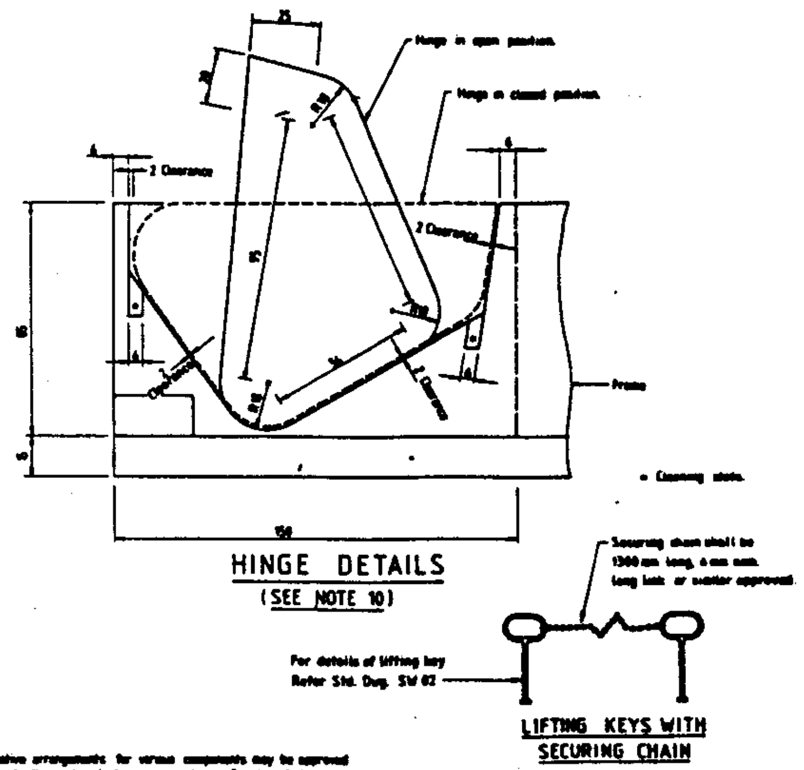
PRIMARY PIPEWORK DESIGN

Sheet........ of..........
Designed ....
Checked ....

Signed ....
........../..../......

Gully Box	K B C . I.L.	T.W.L. = I.L. − 0·150	B.W.L. = Manhole T.W.L.	H TOT T.W.L. − B.W.L.	Q = Gully In Flow	$\frac{H\,TOT}{Q^2}$	Pipe D	U/S I.L. = (2) − 0·575 − D	min D/S I.L. = (Manhole)	L = Pipe Length	Slope (S)
	m	m	m	m	l/sec	$m/(l/sec)^2$	mm	m	m	m	%





NOTES

1. Alternative arrangements for various components may be approved subject to the sectional dimensions and overall external dimensions remaining unchanged.
2. Cast iron shall comply with AS 1830 - 1986 for Grey Cast Iron having min. Grade T220
3. Grate or cover type to be specified in construction drawings.
4. Grate or cover shall comply with the requirements of BS 497 - 1976
   Minimum Design (Working) Load - 70 kN
   Minimum Test (Proof) - 210 kN
   Covers shall be tested prior to concrete infilling
5. All components shall be manufacture in accordance with a Quality Assurance System complying with the requirements of AS 1822 & AS 1823
6. All completed frames and lids shall be dipped in hot bitumen.
7. Concrete infill covers shall be filled with concrete, characteristic compressive strength = 25 MPa.
8. The words "SECURE LID WITH CHAIN TO LINTEL" shall be adequately displayed on the front face of grate or cover.
9. The frame shall be supplied fixed to the chamber by chamber manufacturer.
10. THE HINGE DETAIL SHOWN ON THIS DRAWING IS SUBJECT TO A PATENT APPLICATION Nº 77950/87 FOR HUMES LIMITED.
    HUMES LIMITED WILL GRANT NON-EXCLUSIVE LICENCE AGREEMENTS FOR THE MANUFACTURE AND MARKETTING OF THE HINGES.
11. Different grates and covers may be specified. Cast iron grate or cover required shall be specified when ordering Side Inlet Manhole components.

A	ORIGINAL ISSUE		9/87
	REVISIONS	APPROVED	DATE

CALLIOPE SHIRE COUNCIL

SIDE INLET MANHOLE
CAST IRON GRATE, COVER AND FRAME

L.G.E.A. (Qld) STANDARD DRAWING
SW-06

S.W.D.S. - 6.

A

PIPE NETWORK DESIGN H.G.L. ANALYSIS

Sheet.......... of..........
Designed..........
Checked..........
Signed........../..........

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
Manhole	Outlet Pipe Flow	Length	Slope	Trial D	M.H. Lid R.L. (or K&C I.L.)	TWL = R.L. − 0·45	B.W.L.	Velocity $V = Q/A$	Kw Code	Kw	$V^2/2g$	$hw = Kw\,(V^2/2g)$	$hf = \frac{0{\cdot}022L}{D}\,(V^2/2g)$	H_T = hf + hw test (i) = [H_T > 0·1 ?]	T.W.L. − B.W.L. test (ii) [> H_T ?]	hf + 1·5 D test (iii) = [≥ T.W.L.−B.W.L. ?]	Pipe U/S I.L. [test (iii) = Yes] I.L. ≤ B.W.L. + hf − D [test (iii) = No] I.L. ≤ T.W.L. − 1·5 hw	T.W.L. [test (iii) = Yes] = B.W.L. + H_T [test (iii) = No] = T.W.L. = Col. 7	Adopted D.	Adopted T.W.L. & B.W.L.	REMARKS
	l/sec	m	m/m	mm	m	m	m	m/sec	-	-	m	m	m	m	m	m	m	m	mm	m	

CALLIOPE SHIRE COUNCIL

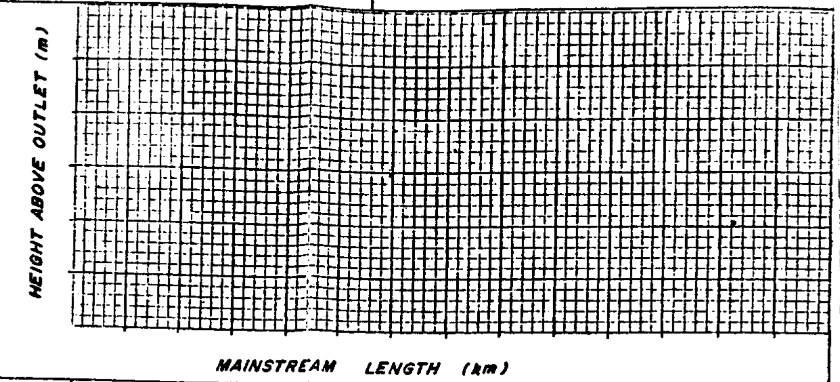
CULVERT DESIGN

Water Course Name ....
Structure Chainage ....

Designed .... Date ....
Checked .... Date ....

DESIGN FLOWS

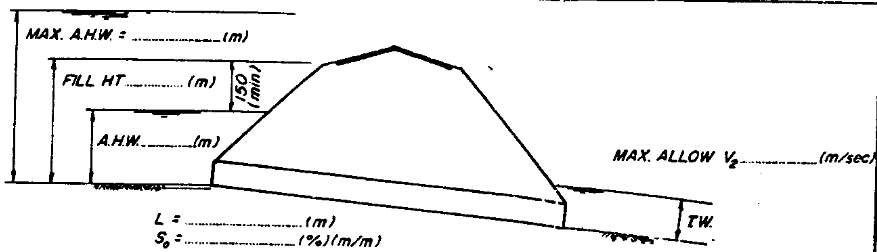
CATCHMENT AREA	MAINSTREAM SLOPE				TIME OF CONC.	A.R.I.	INTENSITY	CO - EFF OF RUNOFF	DESIGN FLOW
A	CATCHMENT HEAD LEVEL R_1	OUTLET LEVEL R_2	MAINSTREAM LENGTH L	$Se = \frac{R_1 - R_2}{L}$	$tc = \frac{91\,L}{A^{0.1}\,Se^{0.2}}$		$I_{t,y}$	C	$Q = 0.0028\ C.I_{ty}.A.$
ha	m	m	km	m/km	mins	years	mm/hr		m³/sec
									Q( ) =
									Q( ) =
									Q( ) =

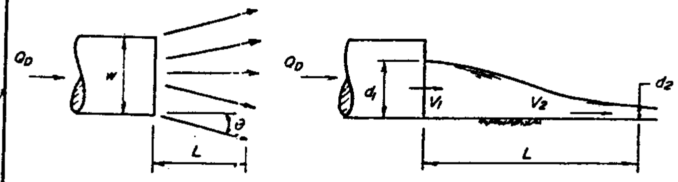


FLOW INFORMATION

Q_D( ) = ... (m³/sec)
TW = ... (m)

Q_C( ) = ... (m³/sec)
TW = ... (m)





Q	PIPE (D) or B x D	Q/B	A	HEADWATER CALCULATION: INLET CONTROL		OUTLET CONTROL $HW = H + h_o - LS_o$							CONTROLLING HW	CULVERT OUTLET VEL/DEPTH				CHANNEL VEL/DEPTH		$d = (d_1 + d_2)/2$	$V = (V_1 + V_2)/2$	FLARE ANGLE	W	PROTECTION LENGTH	STANDARD PROTECTION LENGTH 1.5 D	REQUIRED PROTECTION LENGTH L - 1.5D	COMMENTS
				HW/D	HW	H	TW	dc	$\frac{dc + D}{2}$	h_o	LS_o	HW		d_1	d_1/D	Prop. Area of Flow P	$V_1 = Q/PA$	d_2	V_2			$Tan\,\theta = \frac{\sqrt{9.8d}}{3V}$		$L = \left(\frac{Q_D}{V_2 d_2} - w\right)/2\tan\theta$			
m³/sec		m³/sec/m	m²		m	m	m	m	m	m	m	m		m			m/sec	m	m/sec	m	m/sec		m	m	m	m	

Minor System A.R.I.

n = ... years

STORMWATER DRAINAGE

MINOR SYSTEM SURFACE FLOWS

Sheet .... of ....

Designed ....

Checked ....

Signed ..../../.......

		Sub-Catchment Flow Times							Design Flow								Gully Inlet Design					
(1) Gully Box	(2) Flow Path Description (Land Use)	(3) Flow Length (L)	(4) Roughness Factor (n)	(5) Slope (S)	(6) $\frac{t 10\cdot 4 = 6\cdot 94(Ln)^{0\cdot 6}}{S 0\cdot 3}$	(7) Component Flow Times	(8) Total Flow Times	(9) Time of Concentration	(10) Intensity (In)	(11) Runoff Co-off (C)	(12) Area (A)	(13) Equiv. Imp. Area (CA)	(14) Cumul. Equiv. Imp Area (Σ CA)	(15) $Q = \frac{(\Sigma CA)In}{0\cdot 36}$	(16) By-Pass Flow from [ ]	(17) Design Flow	(18) Gutter Grade	(19) Flow Width	(20) Inlet Type	(21) Inflow	(22) Bypass Flow to Pit [ ]	REMARKS
		m	-	m/m	mins mm / hr.	mins	mins	mins	mm / hr	—	ha	ha	ha	l/sec	l/sec	l/sec	m/m	m		l/sec	l/sec	

CALLIOPE SHIRE COUNCIL

Major sytem ARI m..........years

Major system ARI n..........years

$F' = \frac{Fn}{Fm} = \text{—} = ...$

$f = 0{\cdot}75F' = ...$ (Col. 12)

STORMWATER DRAINAGE

MAJOR SYSTEM DESIGN

Sheet.......... of..........

Designed ....

Checked..........

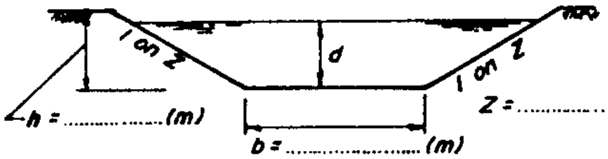
Signed ..../../..

Description		Sub-Catchment Flow Times							Design Intensities				Design Flows					Escape Path Capacity				REMARKS
(1) Location	(2) Flow Path Description (Land Use)	(3) Flow Length (L)	(4) Roughness Factor (n)	(5) Slope (S)	(6) $t1\ 0{\cdot}4 = \frac{6{\cdot}94(Ln)^{0{\cdot}6}}{S^{0{\cdot}3}}$	(7) Component Flow Times (t)	(8) Total Times (t)	(9) Time of Concentration (tc)	(10) Major System Intensity (Im)	(11) Minor System Intensity (In)	(12) Block., Freq. factor (f) (.......In)	(13) Gap Intensity ($Ig = Im - f\ In$)	(14) Runoff Co-off (C)	(15) Contributing Area (A)	(16) Equiv. Imp. Area (CA)	(17) Cumul. Equiv. Imp. Area (Σ CA)	(18) Gap Flow $Qg = \frac{(\Sigma CA)Ig}{0{\cdot}36}$	(19) Escape Path Description	(20) Max. Allow. Flow Depth (dg)	(21) Roadway Grade	(22) Roadway Capacity QR	
		m	–	m/m	mins. mm / hr.	mins.	mins.	mins.	mm/hr	mm/hr	mm/hr	mm/hr	–	ha	ha	ha	l/sec	–	m	%	l/sec	

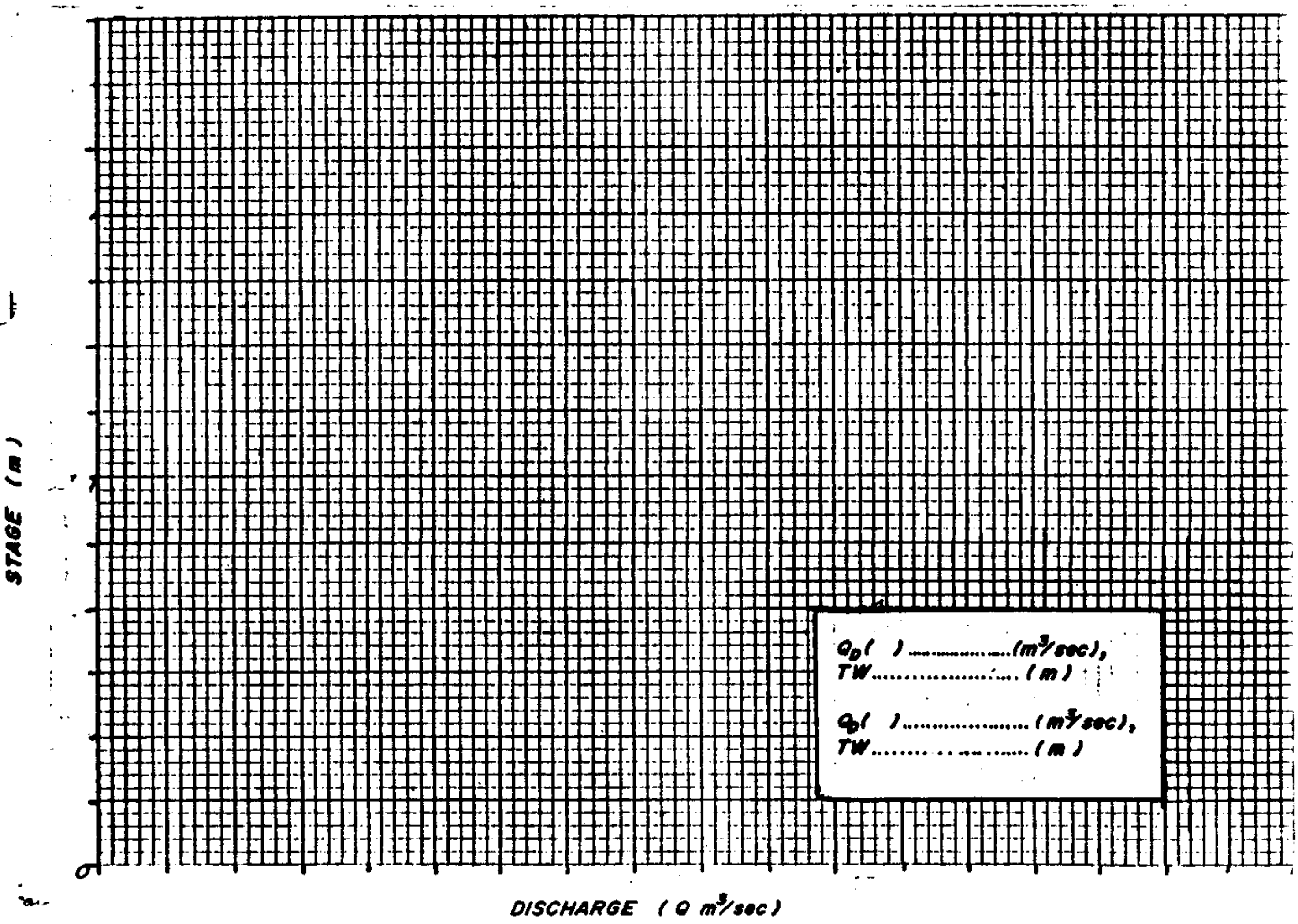
CALLIOPE SHIRE COUNCIL

Name .... Chainage	**OPEN CHANNEL** **STAGE - DISCHARGE**	DESIGNED .... DATE .... CHECKED .... DATE ....

DESIGN FLOW Q_D ( ) .... (m^3/sec)
CHECK FLOW Q_C ( ) .... (m^3/sec)
CHANNEL SLOPE S = ... (m/m)
MANNINGS ('n') ....



TRIAL	FLOW AREA	WETTED PERIM.	HYD. RADIUS	VELOCITY		COMMENTS
d	$A = bd + zd^2$	$W.P. = b + 2d\sqrt{1+Z^2}$	$R = A/W.P.$	$V = \frac{R^{0.667} S^{0.5}}{n}$	$Q = VA$	
(m)	(m^2)	(m)	(m)	(m/sec)	(m^3/sec)	



CALLIOPE SHIRE COUNCIL

STORMWATER DRAINAGE

MAIN LINE PIPE FLOWS

Sheet........of..........
Designed..................
Checked...................

Signed....................
......../..../.......

Pipe	Full Area						Partial Area						Adopted Flow	REMARKS
	Surface Flow Time	Pipe Flow Time	Time (tc)	Intensity	Σ CA	$Q = \frac{\Sigma CAI}{0.36}$	Surface Flow Time	Pipe Flow Time	Time (tc)	Intensity	Σ CA	Flow Q		
	mins	mins	mins	mm/hr	ha	l/sec	mins	mins	mins	mm/hr	ha	l/sec	l/sec	