ENGINEERING DESIGN SPECIFICATION

D5

STORMWATER DRAINAGE DESIGN

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STORMWATER DRAINAGE DESIGN

GENERAL

D5.01 SCOPE

1. The work to be executed under this Specification consists of the design of stormwater drainage systems for urban and rural areas. ***Scope***

D5.02 OBJECTIVES

1. The objectives of stormwater drainage design are as follows:

(i) To ensure that inundation of private and public buildings located in flood-prone areas occurs only on rare occasions and that, in such events, surface flow routes convey floodwaters below the prescribed velocity/depth limits.

(ii) To provide convenience and safety for pedestrians and traffic in frequent stormwater flows by controlling those flows within prescribed limits.

(iii) Retain within each catchment as much incident rainfall and runoff as is possible and appropriate for the planned use and the characteristics of the catchment.

2. In pursuit of these objectives, the following principles shall apply: ***Design Principles***

(i) New Developments are to provide a stormwater drainage system in accordance with the "major/minor" system concept set out in Chapter 14 of Australian Rainfall & Runoff, 1987 (ARR 1987); that is, the "major" system shall provide safe, well-defined overland flow paths for rare and extreme storm runoff events while the "minor" system shall be capable of carrying and controlling flows from frequent runoff events.

(ii) Redevelopment – Where the proposed development replaces an existing development, the on-site drainage system is to be designed in such a way that the estimated peak flow rate from the site for the design average recurrence interval (ARI) of the receiving minor system is no greater than that which would be expected from the existing development. The 1% AEP post development runoff should also be calculated so as to determine the allowable site discharge.

D5.03 REFERENCE DOCUMENTS

Standards

- Australian Rainfall and Runoff
  A Guide to Flood Estimation. Inst. of Eng. Aug 1987.
- AS3725 Loads on Buried Concrete Pipes
- Queensland Urban Drainage Manual, Volumes 1 and 2, 1993.
- Sangster, WM, Wood, HW, Smerdon, ET and Bossy, HG.
- Pressure Changes at Storm Drain Junction,
  Engineering Series, Bulletin No, 41, Eng.
  Experiment Station, Univ. of Missouri 1958.
- Stormwater Drainage – Hydraulic Design of Underground Pipe Networks
  (Wollongong City Council)

- Hare, CM.
  Magnitude of Hydraulic Losses at Junctions in Piped Drainage Systems.
  Transactions, Inst. of Eng. Aust., Feb. 1983.
- Department of Main Roads (NSW)
- Model Analysis to determine Hydraulic Capacities of Kerb Inlets and Gully Pit Gratings, 1979.
- Austroads Bridge Design Code, 1992
- Concrete Pipe Association of Australia
- Concrete Pipe Guide, charts for the selection of concrete pipes to suit varying conditions.
- Henderson, FM.
  Open Channel Flow, 1966.
- Chow, Ven Te
  Open Channel Hydraulics, 1959.
- John Argue – Australian Road Research Board Special Report 34
- Stormwater drainage design in small urban catchments: a handbook for Australian practice.
- Australian National Conference On Large Dams, Leederville WA.
- ANCOLD 1986, Guidelines on Design Floods for Dams.
- Austroads – Waterway Design (1994)
- Shoalhaven City Council Interim Flood Policy, 1998

HYDROLOGY

URBAN DRAINAGE

D5.04 DESIGN RAINFALL DATA

1. The rainfall data to be used is obtained from the rainfall pattern taken from Intensity/Frequency/Duration (IFD) relationships supplied by the Australian Bureau of Meteorology for the urban area. (See figures D5.04.1–D5.04.23).

2. Design Average Recurrence Interval (ARI) – For design under the "major/minor" concept, the design ARIs to be used are given below. ***Average Recurrence Intervals***

3. Recurrence intervals for minor events depend on the zoning of the land being serviced by the drainage system. The minor system design ARIs are detailed below:–

   - 10 years for commercial/industrial area "minor" systems
   - 5 years for residential area "minor" systems
   - 5 years for rural residential area "minor" systems
   - 25% of the 1 year ARI peak flow for parks and recreation area "minor" low flow systems.

4. In addition, where a development is designed in such a way that the major system flows involve surcharge across private property, then the underground system (both pipes and inlets) shall be designed to permit flows into and contain flows having an ARI of 100 years from the upstream catchment which would otherwise flow across the property. A surcharge path shall be defined for systems even where 100 year ARI flows can be maintained within the system. Allowance to be made for 50% inlet blockage. Easements or drainage reserves are to be provided in private property over pipe systems and surcharge paths. ***Surcharge paths easements***

D5.05 CATCHMENT AREA

1. The catchment area of any point is defined by the limits from where surface runoff will make its way, either by natural or man made paths, to this point. Consideration shall be given to likely changes to individual catchment areas due to the full development of the catchment. ***Catchment Definition***

2. Where no detailed survey of the catchment is available, 1:4000 orthophoto maps are to be used to determine the catchments and to measure areas.

3. Catchment area land use shall be based on current available zoning information or proposed future zonings, where applicable. ***Land Zoning***

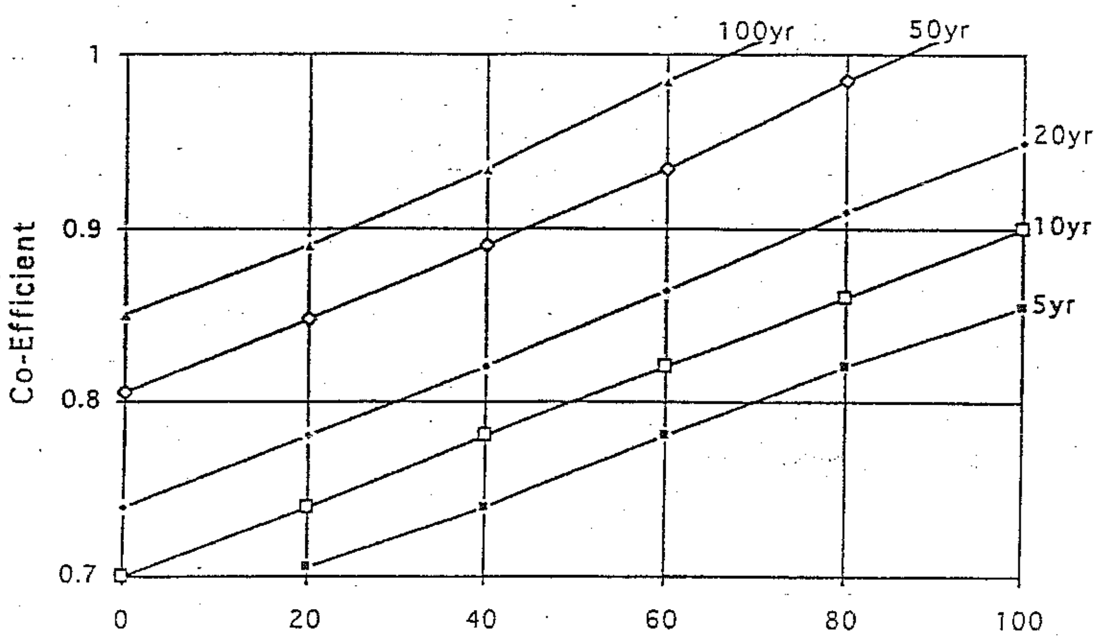
D5.06 RATIONAL METHOD

1. The Rational Method model employing statistical design rainfall data is to be employed in determining the peak flow rate. This model uses the formula – ***Rational Method***

   Q = CyIA/360.

   WhereQ is design flow rate ($m^3/_{sec}$)
   Cy is a runoff co-efficient
   I is rainfall intensity (mm/h)
   A is catchment area (ha)

2. The runoff co-efficient to be adopted for design shall be determined from Table 5.1. ***Runoff Coefficients***



% impervious area

Determination of Runoff Coefficient

Table 5.1

3. Actual percentages of impervious area may be used as long as they take into account the ultimate development of the site, but should not be less than those values shown in Table 5.2.

Land Use	Percentage Impervious Area
Residential 2(a)	40%
Half width road reserve	95%
Medium Density residential 2(b)	80%
Commercial areas	90%
Industrial areas	80%
Public recreation areas	25%

Percentage Impervious Area For Land Use Type

Table 5.2

4. Times of Concentration – The time of concentration of a catchment is defined as the time required for storm runoff to flow from the most remote point on the catchment to the outlet of the catchment. ***Times of Concentration***

5. Where the flow path is through areas having different flow characteristics or includes property and roadway, then the flow time of each portion of the flow path shall be calculated separately.

6. The maximum time of concentration in an urban area shall be 20 minutes unless sufficient evidence is provided to justify a greater time. ***Roof Flow***

For flows from roofs and other surfaces, which drain quickly through downpipes and underground drains, a single response time of 5 minutes should be nominated.

7. Flow paths to pits shall be representative of the fully developed catchment considering such things as fencing and the likely locations of buildings and shall be shown for each collection pit on the catchment area plan. Consideration shall be given to likely changes to individual flow paths due to the full development of the catchment.

8. For time of overland flow the kinematic wave equation should be used. ***Kinematic wave equation***

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

where t is overland flow time (minutes)

L is flow path length (m)
n* is retardance co-efficient
I is rainfall intensity (mm/h)
S is slope (m/m)

9. Surface roughness coefficients "n*" shall generally be derived from information in Chapter 14 of ARR&R 1987. Values applicable to specific zoning types and overland flow path types are given below: ***Overland Flow Retardance***

Flow across bush land	0.30
Flow across Parks	0.17
Flow across Rural Residential land	0.30
Flow across Residential (2a)	0.21

Flow across Residential (2b)	0.11
Flow across Industrial	0.06
Flow across Commercial	0.04
Flow across Paved Areas	0.01
Flow across Asphalt Roads	0.02
Flow across Gravel Areas	0.02

10. Where overland flow is concentrated, naturally or by design into an earth or grass lined channel Manning's formula should be used. ***Mannings Formula***

$Q = A.V = AR^{0.67}\ S^{0.5}/n$

Q is flow rate ($m^3/5$)
A is cross sectional area of flow (m^2)
V is velocity (m/s)
R is hydraulic radius (m)
S is longitudinal slope (m/m)
n is Mannings roughness co-efficient

11. Gutter flow times can be estimated from design aids or the relationship derived from hydraulic models. ***Gutter Flow Times***

12. Where there are significant pipe flows, times need to be considered in estimating the total travel time. ***Pipe Flow Times***

13. The discharge at the outlet of each sub-area may be calculated using the Rational Method. A plan of the adopted sub-area/s shall be submitted. ***Partial & Full Area Effects***

The calculations shall be set out in tabular form similar to Fig D5.06.13. Alternative tabulation sheets may be used as long as the basic data input values and output results are included. The following points should be noted: ***Form of Calculations***

(i) Total flow time (Column 12) is calculated by summing the separate flow times over different surfaces.

(ii) The rainfall intensity adopted (Column 13) corresponds with the total flow time.

(iii) A "C" value (Column 15) and sub-area (Column 14) is calculated for each land use type.

(iv) The discharge for the sub-catchment (Column 18) is calculated by summing the C.As (Column 17) and multiplying by 1/360.

Rainfall intensity 'I' is obtained from the Intensity Frequency – duration data as supplied in figures (D5.04.1–D5.04.23) for a given duration and ARI of 5 and 100 years for the minor and major storm events respectively. ***Rainfall Intensities***

Catchments need to be checked where "partial area" effects occur giving a greater flow rate when applied to a lower part of the catchment with a concentration time less than the full - area travel time. ***Partial Area Effects***

HYDRAULICS

D5.07 PRELIMINARY LAYOUT

1. The determination of drainage paths will require a site survey to be carried out. Existing drainage systems shall be analysed using either reliable work-as-executed plans or alternatively, if these are not available, a field survey will be necessary. ***Site Survey***

Once the survey information is available, a proposed drainage network is defined taking into account rainfall data applicable to the site, times of concentration, gutter and pit capacities.

The width of flow in the gutter is to be restricted to 2.1 metres for the 1 in 5 year ARI event. Maximum spacing of pits is 80 metres. Pits at road intersections shall be located at the upstream tangent points of kerb returns.

An approximate procedure for locating pits is detailed in "Technical Note 2" in ARR (Page 300).

Catchment areas for each pit can then be defined once the pipe layout has been established. A site inspection should be carried out to determine the effect of features which could alter the assumed catchment boundaries and flow paths such as existing or proposed fencing, retaining walls or other structures. Inlet pits should be located a minimum of 6 metres from allotment boundaries prolongation or directly opposite the boundary. ***Pit Location***

Sub-area discharges can be calculated using the procedures detailed in D5.08. Major system flow paths should be identified at this stage and analysed using the procedures detailed in D5.11.

2. Hydraulic calculations shall generally be carried out in accordance with Australian Rainfall and Runoff 1987 and shall be undertaken by a qualified person experienced in hydrologic and hydraulic design. The calculations shall substantiate the hydraulic grade line adopted for design of the system and shown on the drawings. ***Qualified Person***

3. The overall aim of the Major/Minor approach is to ensure that hazardous situations do not arise on streets and footpaths, and that all buildings in urban areas are protected against flood waters to a similar standard to that applying in zones adjacent to rivers. See Fig. D5.07.3. ***Major/Minor System Hydraulics***

Approval will not be given unless these standards are met. The design shall cater for the ultimate anticipated level of development of a catchment.

4. Downstream water surface level requirements are given as follows:– ***Downstream Control***

(i) For a submerged outlet discharging into an open channel the water surface in the channel immediately downstream of the outlet shall be determined by using Manning's Equation to find the normal depth. This water level becomes the control point, unless there is an obvious downstream control which makes it necessary to perform a backwater analysis.

(ii) For a free outlet, the control point is taken as the greater of tailwater depth or $\frac{dc + D}{2}$ where dc = critical depth in pipe and D = diameter of pipe.

(iii) If a new system connects to an existing system, and there will be significantly increased discharge in the existing system, the existing system shall be checked to assess its adequacy in carrying the increased discharge. This checking shall extend down the system until either the increase in discharge becomes insignificant (less than 5%) or the outlet is reached.

D5.08 MINOR SYSTEM CRITERIA

1. Gutter flow widths are to be limited to a maximum of 2.1 metres for the minor system flow. Flows on one side of a street are to be contained by the crown of the road. ***Gutter Flow Widths***

2. The water surface in drainage pits shall be limited to 0.150m, below the gutter invert for inlet pits and 0.150m below the underside of the lid for junction pits. ***Water Surface Limits***

3. Minimum conduit sizes are given below: ***Conduit Sizes***

   - The minimum pipe size shall be 375mm diameter.
   - The minimum box culvert size shall be 600mm wide x 300mm high.

   Pipe sizes in a network should not be less than those of pipes upstream. This need not apply to pipes larger than 600mm diameter, unless maintenance problems are anticipated. For interallotment drainage pipes a minimum diameter of 150mm is to be used.

4. To prevent excessive deposition of sediments in pipes the limiting slope is to be 1% for normal drainage lines and 1% for interallotment drains. A lower pipe slope may be acceptable under certain conditions ***Limiting Slopes***

5. The allowable maximum and minimum depths of pipes which are established from considerations of earth pressures and forces due to superimposed loads are to be determined from the appropriate charts provided in reference document *Concrete Pipe Association of Australia, Concrete Pipe Selection and Installation.* For interallotment drainage pipes a minimum cover of 400mm is to be used ***Pipe Cover***

6. Pipe manufacturers allowable pipe deflection angles and mitre bends may be used for changes in direction in the horizontal plane only where the use of junction pits is not required. Appropriate headlosses are to be taken into account ***Curved Pipelines***

7. Minimum and maximum velocity of flow in stormwater pipelines shall be 0.6m/sec and 8m/sec respectively ***Velocity Limits***

8. Where the grade of the pipe exceeds 15% concrete anchor blocks shall be provided as a minimum at every third collar. ***Anchor Blocks***

D5.09 PITS

1. Inlet Pits shall be spaced so that the gutter flow width is limited in accordance with this specification and so that the inlet efficiency is not affected by adjacent inlet openings. Preference shall be given to the location of drainage pits at the upstream side of allotments at the upstream end of kerb returns on tangent points. ***Spacing***

2. Other pits shall be provided:

   - To enable access for maintenance.
   - At changes in direction, grade, level or class of pipe.
   - At junctions.
   - Where gutter flow exceeds 2.1m wide on road
   - Clear of driveways
   - Such that the depth of flow in the low side kerb and gutter does not exceed the crest level of driveways to properties below road level.

3. The maximum recommended spacing of pits where flow widths are not critical are given in Table 5.3:

	Pipe Size (mm)	Spacing (m)
Generally	less than 1200	80
	1200 or larger	100
In tidal influence	all	80

Table 5.3

4. To calculate the capacity of any type of inlet the first step involves determining the quantities and characteristics of the flow approaching the inlet. ***Inlet Capacity***

5. Gutter flow capacities for both a 150mm high standard kerb, 130mm high roll kerb and layback kerb may be obtained from the graphs in Fig D5.09.5a–D5.09.5c for various longitudinal grades. For specific cases discharges should be calculated and characteristics of the flow at the pits determined. ***Gutter Flow Capacities***

6. Pit inlet capacities may be obtained from graphs in Fig D5.09.6a–D5.09.6d. The capacities on grade are given in terms of low captured from the flow approaching. The graphs also take into account pit blockages of up to 20% unless noted otherwise. ***Pit Inlet Capacities***

7. Maximum sag pit capacities may be obtained based on a depth of ponding up to the top of kerb. The graphs in Fig D5.09.6b and D5.09.6c may be used to obtain capacities. In analysing existing systems it is recommended that if the graphs provided do not specifically address the configuration then an appropriately proportioned combination may be used, unless specific calculations are provided. A reducing factor introduced to both the lintel and grate flow may give a conservative value of a special combination inlet's capacity. However, the assumptions made in specifying the reducing factor are to be substantiated. ***Say Pit Capacities***

8. A grate in a sag operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the kerb is not included in computing P. ***Inlets at Sags***

Weir operation continues to a depth of about W/3 (where W is the smallest dimension of the grate, ie width rather than length) above the top of the grate and the discharge intercepted by the grate is:

$Q = 1.66Pd^{1.5} m^3/_s$ ....(1)

Where $d < 0.12m$

and $P = L + 2W$ or $2(L + W)$ depending on type

When the depth at the grate exceeds 0.9W the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$Q_g = 0.67A\,(2gd)^{0.5}$ ....(2)

A = Clear area of grate

$d > 0.43m$

Between the depths of 0.33W and 0.9W the grate capacity adopted should be the lower of the two equations.

Because of the vortices and the tendency of trash to collect on the grate, the clear opening or perimeter of a grate inlet should be at least twice that required by the equations above in order to remain below the design depth over the grate. Where danger of clogging is slight a factor of safety less than two might be used, but is to be substantiated. Where combination inlets are used, the grate need only be as large as given by the equations above because the kerb opening provides the safety factor from clogging. Therefore, for a 150mm kerb the maximum capacity of a 900 x 450mm Weldlok grate in a sag will be as follows:

Q_g $= 1.66 \times 1.8 \times 0.15^{1.5} m^3/s$
$= 0.174/FOS$
$= 0.087\ m^3/s$ (factor of safety = 2)

9. The capacity of a lintel (kerb inlet) in a sag depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. When the water depth exceeds about 1.4 times the height of the entrance, the inlet operates as a special orifice defined as a large lateral aperture. Between weir-type operation and orifice-type operation the capacity is indeterminate. The discharge resulting from weir flow is: ***Lintels at a Sag Point***

$Q_L = 1.66Ld^{1.5} m^3/s$..(3)
Where $d \leq 1.4h$ and L = Length of inlet depending on type

When the depth at the lintel exceeds 1.4h the lintel begins to operate as a large lateral aperture (relatively large orifice at the side) and the discharge intercepted by the lintel is:

$Q_L = 0.67A[zg(d-h/z]^{0.5}$..(4)

Q_L = inlet flow
d = depth of flow
L = inlet width
A = opening area
g = acceleration due to gravity $9.8 m/s^2$

Note: Similarly, a Factor of Safety should be used to determine actual inflow.

10. Kerb inlets on grades are generally inefficient. As the grade increases the effectiveness of the inlet to intercept flow decreases. The inlet can be made more effective by depressing the inlet, increasing the cross fall at the inlet or by providing deflectors at the inlet. Grades above about 5% are said to be steep. ***Inlets on Grade***

11. Grates tend to be more efficient at capturing flow on grades than lintels, however, they are also more prone to blockages. Hence, a grate on its own has substantial by-pass flows. ***Grates on Grade***

12. Flow on steeper grades tends to overshoot a kerb opening such as a lintel, however, increasing cross fall, depressing the inlet or the use of deflectors tend to improve the performance significantly. ***Lintels on Grade***

13. All new developments shall use a combination inlet of grate and lintel and it is recommended that the use of deflectors be encouraged on steeper grades where possible. The overshoot as described above is less likely to occur due to the standing wave formed by the turbulence created by the grate. ***Combined Inlets on Grade***

14. Deflectors have been found to be extremely efficient on steeper slopes, say greater than 5%. They tend to train the flow of water toward kerb inlets thereby greatly improving the overall inlet performance. Therefore, it is regarded that the use of deflectors on steep slopes is good engineering practice where this will not adversely effect the movement of traffic. ***Effect of Deflectors***

15. Any inlet pit that is proposed to be used can be analysed by the principles set out above or by following procedures as set out in ARR. ***Pits Not Covered by Graphs***

16. Pit crossfalls should be no less than 30mm. Changes in diameter should be graded obvert to obvert. ***Pit Grade***

17. Where the depth of the pit exceeds 1.2 metres, standard galvanised or other approved step-irons are to be provided at a spacing of 300mm to provide access for inspection and cleaning. Refer Standard Drawing No. 263707. ***Step Irons***

18. Every endeavour is to be made to maintain flow velocities through pits. Excessive drops will not be permitted except where designed to dissipate energy/velocities. ***Pit Velocity***

19. Pits are to be located and constructed in accordance with Council's Standard Drawings nos 263701–263708 and GG78.62. Precast pits may be used if the prior approval to the type and design is obtained from Council. ***Pit Standard Drawing***

20. In medium density, commercial or community title development where accessways slope towards street footways, a collection grate and drainage pit shall be provided to prevent flows (ie zero discharge) up to the Design Storm, crossing the footpath. ***Driveway Discharge***

21. Calculation of flowrates in Rural – Residential subdivisions should be in accordance with Chapter 5, "Estimation of Peak Flows for small to medium sized Rural Catchments" of "Australian Rainfall and Runoff 1987". Pipe drainage systems are required under all road reserves. ***Rural Residential Subdivisions***

Culverts and other drainage structures are to be designed for a flood frequency of 1:5 years. Special attention must be given to scour protection where flows exceed the design frequency. In some rural subdivisions, where lots are likely to be isolated by rising of local streams, flood free access will be required if approval is to be granted to the subdivision. (Reference to D5.11.2)

22. Road culverts should be designed in accordance with culvert hydraulics theory, ie the culvert capacity is determined by the flow conditions, depending on whether inlet control or outlet control governs. ***Culvert Design***

Recommended design procedures are contained in Section 3 of the Concrete Pipe Association of Australia's publication: "Hydraulics of Precast Concrete Conduits – Hydraulic Design Manual". Nomographs in figures D5.09.22.1–D5.09.22.4 provide culvert capacities for both pipe culverts and box culverts for both inlet and outlet control conditions. Entrance loss coefficients are provided in the following table 5.4.

Design of Entrance	Ke
Pipe Culverts	
Pipe projecting from fill, square cut end	0.5
Socket end	0.2
Headwall with or without wingwalls, square end	0.5
Socket end	0.2
Pipe mitred to conform to fill slope, precast end	0.5
Field cut end	0.7

Design of Entrance	Ke
Box Culverts	
No wingwall, headwall parallel to embankment, square edge on three edges	0.5
Three edges rounded to 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel, square edge at crown	0.4
Crown rounded to 1/12 culvert height	0.2
Wingwalls at 10° to 30° to barrel, square edge at crown	0.5
Wingwalls parallel (extension of sides) square edge at crown	0.7

Entrance Loss Coefficient
An alternative is the use of Austroads – *Waterway Design (1994)*

Table 5.4

D5.10 HYDRAULIC LOSSES

1. Drainage pipe systems shall be designed as an overall system, with due regard to the upstream and downstream system and not as individual pipe lengths. Drainage pipeline systems shall generally be designed as gravity systems flowing full at design discharge, but may be pressurised with the use of appropriate pits and joints. Pipe friction losses and pipe sizes in relation to discharge shall be determined using the Colebrook–White formula with the acceptable roughness co-efficients being 0.6mm for concrete pipes and 0.06mm for FRC pipes. See figure D5.10.1. ***Pipe System*** ***Pit Friction***

2. Going from larger upstream to smaller downstream conduits is only permitted where the smaller pipe is 600mm diameter or greater as blockage is unlikely to occur. In going from smaller to larger pipes benching shall be provided in pits to enable a smooth flow transition. ***Contraction/Expansion Losses***

3. The energy losses at pits once pipes are pressurised shall be determined from the Missouri charts for pit energy losses or from computer programmes based on the University of Missouri Studies. See Figures D5.10.3.1 to D5.10.3.28 or Hare Charts. Benching in pits may be used to assist in reducing pit losses where desirable. ***Pit Losses***

4. Drop pits occur where there is a substantial difference between the invert of the inlet and the obvert of the outlet. Usually, this results in the water losing all of its forward velocity and momentum. Very large headlosses can be expected due to the need to accelerate the water from rest and due to the turbulence caused by the incoming water. When the obvert of the outlet pipe is at a or greater than $D_o/4$ (D_o = diameter of outlet pipe) below the invert of the upstream pipe, the inflow shall be regarded as grate flow.

5. Energy losses at pipe bends is to be taken into consideration. ***Bend Losses***

6. Openings at pits must not be large enough to admit a child. Grates and depressions associated with inlets should not be hazardous to road users, particularly cyclists. ***Safety at Pits***

7. The effects of pit and pipe blockages on the safety of people and property is to be taken into consideration. ***Blockages***

D5.11 MAJOR SYSTEM CRITERIA

1. Major system flows are flows in excess of the minor system capacity. These major flows shall be catered for by providing suitable escape routes such that they do not present a danger to life and property. These overland flow paths shall have a capacity to carry major system flows up to the 1:100 year ARI. The following over land flow paths may be used to act as a major system flow route. ***Definition***

   (i) Roadways including footpaths

   (ii) Pathways

   (iii) Parkland or open space

   (iv) Drainage reserve

   Consideration must be given to continuity of the overland flow path and as such where, for example, a roadway acting as an overland flow path discharges stormwater to a pathway, park, drainage reserve, etc the footpath should have a reverse crossfall to facilitate the overland flow. Other obstructions, such as fences, shall not traverse these flow paths. ***Continuity Overland Flow***

2. In the design of these overland flow paths the actual flow depth must be taken into account when establishing an appropriate design value of V*D. See Fig D5.11.2 below. (From Research Report No. 69 *Safety Aspects of Designing Roads and Floodways Urban Water Research Association of Aust, Nov 1993*). ***Safety Velocity/ Depth Criteria***

   It is a Council requirement that all properties are protected against the 1:100 year ARI flood. As such, drainage designs shall take account of the existing flood behaviour and at the very least ensure that it is not made worse by any proposed works.

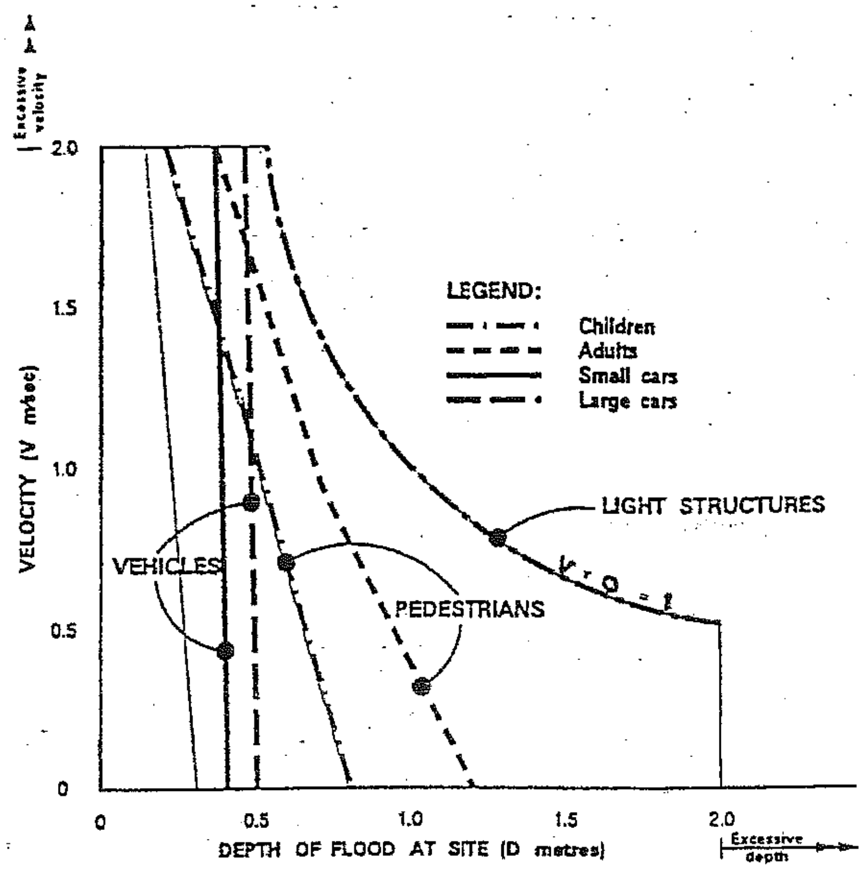


Fig. D5.11.2

3. Freeboard requirements for floor levels and levee bank levels from flood levels in open channels, roadways and stormwater surcharge paths are given below: ***Freeboard***

Generally:–

(i) A minimum freeboard of 0.3m shall be provided between the 100 year flood level and floor levels on structures and entrances to underground car parks and top of bank. A higher freeboard may be required in certain circumstances.

(ii) Where the road is in fill or overtopping of kerbs and flow through properties may occur a 100mm freeboard shall be provided between the ponding level of water in the road and the high point in the footpath. Driveway construction in these instances needs to consider this requirement.

In Surcharge Paths:–

(iii) A minimum freeboard of 0.3 shall be provided between the 100 year flood level and floor levels on structures and entrances to underground car parks.

In Open Channels:–

(iv) A minimum freeboard of 0.5m shall be provided between the 100 year flood level and floor levels on structures and entrances to underground car parks.

4. Table 5.5 below gives road carriageway flow capacities for a 5 metre, 6 metre, 7 metre and 12 metre road with varying longitudinal slopes. The apparent discrepancy in the capacity of the roadway widths is explained as follows ***Road Reserve Flow Capacity***

(i) The most efficient hydraulic cross section is one which is rectangular.

(ii) As the roadway widens the hydraulic cross-section changes from almost rectangular at 5m wide to two distinct separate triangles.

(iii) By this, at a fixed depth of 150mm on the roadway, the cross sectional flow area increases marginally by widening the road. However, the hydraulic radius decreases as does the conveyance of the cross-sections. Therefore, the capacity of the cross section of the 8m roadway.

(iv) Roadway capacities are calculated by Manning's equation for a compound cross-section.

If the roadway alone does not provide the required capacity, the entire width of the road reserve may be used as the flood path subject to velocity and depth restrictions obtained from Figure shown in Section D5.11.2.

Longitudinal Slope	FLOW CAPACITY (L/s)				
	Width of Road				
%	5m	6m	7m	8m	9–13m
1	650	680	675	640	Flow divided by Crown adopt 50% 8m flows each side
2.5	1025	1080	1065	1015	
5	1450	1525	1510	1440	
7.5	1770	1860	1840	1750	
10	2050	2155	2130	2030	
12.5	2240	2410	2380	2270	
15	2510	2640	2610	2480	
16.7	2650	2800	2770	2640	

Note: Calculations assume roughness "n" = .013 and kerb profiles in accordance with SCC or Dept of Housing Drawings. Other profiles require Council approval.

Maximum Flow Capacities of Roadways with Standard Kerb & Gutter, 3% Crossfall and Depth of Ponding 130mm

Table 5.5

Longitudinal Slope	FLOW CAPACITY (L/s)			
	Width of Road			
%	5m	6m	7m	8m –13m
1	505	515	500	Flow divided by Road Crown Adopt 50% 7m flows each side.
2.5	800	815	790	
5	1130	1155	1120	
7.5	1380	1410	1370	
10	1600	1630	1585	
12.5	1790	1820	1770	
15	1960	1995	1940	
16.7	2080	2120	2060	

Note: Calculations assume roughness "n" = .013 and kerb profiles in accordance with SCC or Dept of Housing Drawings. Other profiles require Council approval.

Maximum Flow Capacities of Roadways with Layback Kerb & Gutter 3% Crossfall Ponding Depth 130mm

Table 5.6

	FLOW CAPACITY (L/s)			
Longitudinal Slope	***Width of Road***			
%	***5m***	***6m***	***7m***	***8m–13m***
1	500	510	495	Flow divided by Road Crown Adopt 50% 7m flows each side.
2.5	795	805	780	
5	1125	1145	1105	
7.5	1375	1395	1350	
10	1590	1615	1560	
12.5	1775	1805	1745	
15	1945	1975	1910	
16.7	2065	2180	2030	

Note: Calculations assume roughness "n" = .013 and kerb profiles in accordance with SCC or Dept of Housing Drawings. Other profiles require Council approval.

Maximum Flow Capacities of Roadways with Roll Top Kerb & Gutter 3% Crossfall Ponding Depth 130mm

Table 5.7

6. Technical Note 6 and 9 in ARR details the recommended methods of analysing major systems flows. The hydrological and hydraulic design data shall be presented as shown in Figure D5.06.13. ***Form of Calculations***

D5.12 TRUNK DRAINAGE

1. Trunk drainage systems serve catchments larger than 15 hectares and can include the following elements: ***General***

- Natural watercourses or artificial open channels. For the purpose of this document, the term "natural watercourse" is considered to be a defined path which stormwater follows and includes channels and any overbank flow path. The provision of channel lining or enclosure by pipes, etc. does not in any way diminish the fact that the flow path is a natural route for the stormwater.
- Culverts and road crossings exceeding 900mm in diameter.
- Naturally occurring ponds and lakes exceeding 1000 m3 capacity.
- Transition and hydraulic structures.
- Artificial detention storages exceeding 1000 m3 capacity.

2. An appropriate runoff routing model shall be used to estimate design flowrates at suitable points along a trunk drainage system. It is necessary to estimate runoffs using storm patterns of different durations to determine the critical storm. Such patterns are obtained form Chapters 2 and 3 of ARR. The model should reflect the existing conditions of the catchment and as provided by rainfall intensity charts Figure D5.04.1 to Figure D5.04.23.

   The model chosen shall be calibrated against a recorded storm event if available, otherwise the model shall be compared with at least one other runoff routing model and/or the Probabilistic Rational Method. If calibration is to be made against the Probabilistic Rational Method, then once calibration is achieved, model parameters shall be adjusted to take into account the extent of urbanisation.

The model eventually chosen shall be justified for its adoption including all parameters.

3. Hydraulic design of trunk drainage systems shall proceed with the use of flood surface profile calculations as appropriate. These calculations may be done manually or by the use of appropriate computer models. Any assumptions made in the hydraulic analysis design are to be clearly stated and shown to be justified. ***Hydraulic Design***

The hydraulic design shall take into account obstructions to flow such as buildings and fences. If sections of the trunk drainage system are prone to blockage by debris, the effects of a minimum 50% blockage shall be investigated. That is, what is the likely rise in flood level? This information will assist in determining whether sufficient freeboard is still available to habitable floor levels.

The data required for the hydraulic design which includes channel Cross sections, dimensions of controls, obstructions, etc, shall be obtained. This data shall reflect the existing characteristics of the channel. Unrealistic approximations are not to be made on this data.

A sufficient number of sections both within the site and extending sufficiently upstream and downstream of the site shall be analysed in order to reflect flood behaviour and take account of impacting control structures.

In the design of new channels, a freeboard of 0.3 metres shall be provided above the 1 in 100 year ARI top water level.

4. Where the product of average Velocity and average flow Depth for the design flow rate is greater than $0.4m^2/s$, the design will be required to specifically provide for the safety of persons who may enter the channel. ***Safety***

5. Maximum side slopes on grassed lined open channels shall be 1 in 4, with a preference given to 1 in 6 side slopes (particularly in the vicinity of schools), channel inverts shall generally have minimum cross slopes of 1 in 20. ***Side Slopes***

6. Low flow provisions in open channels (man-made or altered channels) will require low flows to be contained within a pipe system. Minimum pipe size is to be 375mm in diameter. Subsurface drainage shall be provided in grass lined channels to prevent waterlogging of the channel bed. Low flow recurrence intervals are given in Section D5.04 ***Low Flows***

7. Maximum flow velocities in grass lined channels shall be restricted to 2m/s. Designs shall ensure that flow is subcritical. Supercritical flow shall be avoided, however where this cannot be prevented as in the case of an existing open channel, fencing to the satisfaction of the Subdivision and Development Manager, design shall be provided for the length of the open channel where supercritical flow occurs. ***Flow Velocities Channels***

8. Adequate scour protection shall be designed for all discharge points into and out of the channel. It shall also be provided at any point along the channel where there is a significant change in flow conditions. ***Scour Protection***

9. Modifications to natural watercourses are generally not allowed as they can adversely impact on a number of issues including: ***Watercourse Modification***

- hydraulic function
- channel pattern and form
- long-term channel stability

- aesthetic appearance
- aquatic and bankside habitat diversity
- water quality

Proposals involving modifications to watercourses will need to include in addition to a hydraulic assessment, an assessment of their environmental impact and approvals and permits will also be required form the Department of Land and Water Conservation. The NSW Department of Planning manual "Better Drainage, A Guideline for the Multiple Use of Drainage Systems" should be used when watercourse modification is proposed.

D5.13 MAJOR STRUCTURES

1. All major structures shall be designed for the 100 year ARI storm event without afflux in urban areas. Some afflux and upstream inundation may be permitted in certain rural and urban areas provided the increased upstream flooding is minimal and does not inundate private property. ***Afflux***

2. A minimum clearance of 0.3m between the 100 year ARI flood level and the underside of any major structure superstructure is required to allow for passage of debris without blockage. ***Freeboard***

3. All bridges shall be designed for the 100 year ARI flood intensity without afflux in urban areas. ***Bridges***

4. Certified structural design shall be required on bridges and other major culvert structures and may be required on some specialised structures. Structural design shall be carried out in accordance with AUSTROADS (1992) Bridge Design Code.

5. All culverts shall be designed for 1% probability flood intensity without afflux in urban areas. ***Culverts***

6. Culverts (either pipe or box section) shall be designed in accordance with Section D5.09.22. Standard concrete headwalls for 300 to 900 diameter pipes are shown in Standard Drawing ref no. 263714 (Fig. D5.Misc.10).

7. Site Information required for Major Drainage structures ***Survey***

   To determine that the structure is compatible to the site, the following information is required:

- Contour plan at 1:100 extending minimum 30 metres upstream and downstream and showing:

  (a) Contours well above the known or estimated highest flood level
  (b) Location of road centre line including chainages
  (c) Location of proposed structure
  (d) Location of foundation test bores
  (e) Location and value of Bench Mark

- Longitudinal Section of road centre line at 1:100 showing:

  (a) Accurate profile of stream bed
  (b) Location of structure
  (c) Proposed road design levels, deck level, soffit level
  (d) 1:5, 1:20 and 1:100 flood levels (so that capacity and stability can be assessed)

- Characteristic cross sections at 1:100 (natural) minimum 10 metre intervals for minimum 30 metres upstream and downstream with 1:5, 1:20 calculated or observed flood levels shown.

- Longitudinal section of stream extending 30 metres upstream and downstream at 1:100 (natural) showing levels at critical changes of grade and the location of the proposed structure.

Checklist Rural Drainage Structures

Pipe Culverts

- Sufficient length in multiples of 1.22 to support batters with standard headwall or inlet pits.
- Location L & R of centre line.
- Invert levels prominent.
- Grade of pipe.
- Sufficient cover for class of pipe.
- Number of cells/diameter/length/RR joint.
- Extent of erosion control at inlet/outlet.
- Headwall design or refer to RTA Standard Drawing (or 900 x 900 inlet pit in cut with 300mm openings facing incoming flow).
- Conformation with existing stream bed. (Show existing RL's at centreline and 10m upstream/downstream.

Box Culverts and Causeways

- As above
- Sections both ways
- Full dimensions including reinforcement placement
- Slab details
- Kerb details
- Protection of batters

D5.14 RETARDING BASINS

1. For each ARI a range of storm events shall be run to determine the peak flood level and discharge from the retarding basin. Storm patterns shall be those given in AR&R 1987 Volume II. ***Critical Storm Duration***

2. The critical storm duration with the retarding basin is likely to be longer than without the basin. A graph showing the range of peak flood levels in the basin and peak discharges from the basin shall be provided for the storms examined.

3. Flood Routing should be modelled by methods outlined in AR&R 1987. ***Routing***

4. The high level outlet to any retarding basin shall have capacity to contain a minimum of the 100 year ARI flood event. Additional spillway capacity may be required due to the hazard category of the structure. The hazard category should be determined by reference to ANCOLD (1986). ***High Level Outlet***

5. The spillway design shall generally be in accordance with the requirements for Open Channel Design in this Specification.

6. Pipe systems shall contain the minor flow through the Retarding Basin wall. Outlet pipes shall be rubber ring jointed with lifting holes securely sealed. Pipe and culvert bedding shall be specified to minimise its permeability, and cut off walls and seepage collars installed where appropriate. ***Low Flow Provision***

7. The low flow pipe intake shall be protected to prevent blockages.

8. Freeboard – Minimum floor levels of dwelling shall be 0.5m above the 100 year ARI flood level in the basin. ***Freeboard at Dwellings***

9. Public Safety Issues – Basin design is to consider the following aspects relating to public safety. *Safety Issues*

- Side slopes are to be a maximum of 1 in 6 to allow easy egress. Side slopes of greater than 1 in 4 may require handrails to assist in egress.
- Water depths shall be, where possible, less than 1.2m in the 20 year ARI storm event. Where neither practical or economic greater depths may be acceptable. In that case the provision of safety refuge mounds should be considered.
- The depth indicators should be provided indicating maximum depth in the basin.
- Protection of the low flow intake pipe shall be undertaken to reduce hazards for people trapped in the basin.
- Signage of the spillway is necessary to indicate the additional hazard.
- Basins shall be designed so that no ponding of water occurs on to private property or roads.
- All outflow drainage pipes shall be fitted with anti-seepage collars. The embankment shall be provided with a cut off wall as deemed appropriate.
- No planting of trees in basin walls is allowed.
- No basin spillway is to be located directly upstream of urban areas.
- Basin to be certified by an appropriate engineer for stability and overtopping.
- Submission of design plans to the Dam Safety Committee is required where any of these guidelines are not met or Council specifically requires such submission. See also D7.18 for such requirements.
- Basin design to be certified by an appropriate engineer for stability and overtopping.

STORMWATER DETENTION

D5.15 STORMWATER DETENTION

1. Installation of Stormwater Detention is required on redevelopment sites within the City where under capacity drainage systems exist. A redevelopment site is defined as a site which used to have or was originally zoned to have a lower density development than is proposed. Site specific information will be provided by Council's Strategic Drainage Engineer. *Redevelopment*

INTERALLOTMENT DRAINAGE

D5.16 INTERALLOTMENT DRAINAGE

1. Interallotment drainage shall be provided for every allotment which does not drain directly to its street frontage or a natural watercourse.

2. Interallotment drainage shall be contained within an easement having a minimum width of 1.0 metre but depending on pipe size. See Section D5.18. The easement shall be in favour of the benefiting allotments.

3. Pipe Capacity – The interallotment drain shall be designed to accept concentrated drainage from buildings and paved areas on each allotment for flow rates having a design ARI the same as the "minor" street drainage system. A slope junction capped off and documented on a work as executed plan to be provided for each allotment.

4. In lieu of more detailed analysis, the following areas of impervious surface are assumed to be contributing runoff to the interallotment drain:–

Impervious Area

Developer Type	% of Lot Area
Residential (2a)	40
Residential (2b)	70
Industrial	80
Commercial	90

5. Pipes shall be designed to flow full at the design discharge without surcharging of inspection pits.

6. *Pits* Interallotment drainage pits shall be located at all changes of direction or at approximately 70 metre intervals. Pits shall be constructed of concrete, with 100mm thick walls and floor and have a minimum 600 x 600 internal dimensions. Pits shall have a 100mm concrete lid finished flush with the surface of works.

7. *Minimum Grade* The interallotment drainage shall have a minimum longitudinal gradient of 0.5%.

8. *Pipe Type* The interallotment drainage shall be constructed from rubber ring jointed pipes of either reinforced concrete pipe or UPVC pipe which shall conform respectively to the requirements of AS 1712, 1342 and 1254. In public road and recreation reserves where vehicle loads may be encountered, reinforced concrete pipe only, shall be used.

9. *Sewer* Where interallotment drainage and sewer mains are laid adjacent to each other they are to be spaced both horizontally and vertically as follows.

   (i) horizontally – 1.5 metres between pipe centre lines.

   (ii) vertically – a minimum of 300mm clearance between the two pipelines is required.

   Where adjacent pipelines are located within the one property the pipeline of least depth is to be located nearest the property boundary

10. Where sewer mains are in close proximity to interallotment drainage lines they are to be shown on the interallotment drainage plan and similarly interallotment drainage lines are to be shown on sewer plans and longitudinal sections. ***Drainage Plan***

DETAILED DESIGN

D5.17 CONDUITS

1. Stormwater pipelines may be constructed from reinforced concrete pipes, fibre reinforced concrete pipes or UPVC pipes manufactured and installed in accordance with relevant Australian Standards. ***Specifications***

AS2032 Installation of UPVC Systems (1977)
AS2033 Installation of polyethylene pipe systems (1980)
AS2566 Plastics pipelaying design (1982)
AS3725 Loads on Buried Concrete Pipes (1989)
AS4060 Loads on Buried Vitrified Clay Pipes (1992)
Concrete pipe selection and installation, Concrete Pipe Association of Australia
MR Form No. 25, Specification for the Construction of Standard Concrete Pipe Culverts and Drains

All pipes are to be new and first quality. All pipelines shall be rubber ring jointed.

2. Pipe Bedding and Cover – Pipe Bedding and Cover Requirements shall be determined from the Concrete Pipe Association "Concrete Pipe Guide" or AS 3725 Loads on Buried Pipes and or Council's Standard Plan ref 263731. All pipes shall have a minimum cover of 450mm under any road pavement and 300mm under grassed areas except where land under a footpath connecting to a kerb. ***Bedding Cover***

D5.18 DRAINAGE EASEMENTS

1. Where it is intended to create drainage easements required in a subdivision, a notation should appear on the engineering drawings and subdivision plan creating the easement or easements pursuant to Section 88B of the Conveyancing Act, 1919 as amended. ***Easements***

2. Where stormwater drainage conveyed in a drainage easement, piped or otherwise, such as referred in (1) above would discharge onto land other than an existing drainage easement or public place, it shall be the responsibility of the developer to obtain a drainage easement through such land, sufficient in dimension to convey the drainage to an easement, natural watercourse or public place and to transfer easement rights therefore to Council. Depending upon the location of the development within a catchment, the developer will be responsible for stormwater discharged from the proposed development onto other lands. The plan of survey will not be released until the above requirements have been complied with, and all fees and contributions have been paid. ***Discharge Private Lands***

3. Interallotment Drainage – 1.0 metres minimum and should be at least 0.5 metres from lot boundary. ***Width Easement***

Piped Drainage Easement Width = (1.5 x depth of trench) + diameter of pipe.

Minimum easement width is 3.0 metres.

Open channel width = top width of 100 year design flow with freeboard + 1 metre, but dependent on maintenance requirements.

4. Council will not permit building over a drainage easement. Footings, piers and other foundations in the immediate vicinity of Council's drainage lines are to be founded at or below the invert level of the pipe, or on solid rock outside the easement. All other foundations are to be located so that the building is founded outside the "zone of influence" of Council's drainage. Council must be able to repair or remove at any time the drainage line without affecting the stability of the building.

Building Adjacent Easements

Zones of Influence

Where building construction involves displacement piles the application is to be referred to Council's Maintenance Manager. Piles adjacent to the drainage line need to be cored below the invert level to prevent heaving of the ground affecting the line. Engineers design details are required to show the design of footings and specified clearances. Council's requirements will be given by listing them with the conditions of Building Approval.

D5.19 PIT DESIGN

1. Pits shall be designed with benching to improve hydraulic efficiency and reduce water ponding. Typical pit designs and other pit design requirements are provided in Plans ref. 263701 to 263708, other pits are to be constructed in accordance with RTA and Department of Housing Standards. Safety and safe access are important considerations in pit design. Approved precast pits to be installed as per manufacturers specifications.

Pit Design

D5.20 RURAL DRAINAGE DESIGN

1. Rural drainage design is to be undertaken in conjunction with the 1987 edition of "Australian Rainfall and Runoff" (AR&R) and the Austroads publication "Waterway Design" (AWD) published in 1994 as a guide to the hydraulic design of bridges, culverts and floodways.

Introduction

These two documents form the basis for Council's requirements. The average recurrence interval (ARI) to be adopted will be specified generally at the time of subdivision or development approval however this should be confirmed with Council's Subdivision & Development Manager prior to the submission of detailed designs.

Hydrology

2. The calculation of peak flows shall generally be in accordance with the Probabilistic Rational Method as described in Chapter 5 of AR&R. It is noted that this method is applicable to catchments up to 250km^2. Larger catchments should be the subject to the alternative methods described in AR&R and designers must consult with Council staff as to an acceptable method prior to detailed designs being submitted.

Rational Method

3. Similarly if alternatives to the Rational Method, including computer programs, are proposed for the smaller catchments the acceptability of these programs should be cleared by Council staff prior to submission and that the proposed criteria are relevant to local conditions.

Where the discharge calculated by these alternative methods is appreciably less than that indicated by the Rational Method the difference will have to be substantiated

As used in design the formula of the Rational method is:

$$Q_Y = 0.278\, C_Y .\, I_{tc,\,Y} .\, A$$

Where Q_Y = peak flow rate (m^3/s) of average recurrence interval (ARI) of Y years

C_Y = runoff coefficient (dimensionless) for ARI of Y years
A = area of catchment (km^2)
$I_{tc,Y}$ = average rainfall intensity (mm/h) for design duration of t_c hours and ARI of Y years.

4. The elements of the equation are explained further as follows:–

Runoff Coefficients

(i) Determine the ARI of "Y" years to be adopted from the criteria described in the D5.04

(ii) Determine which zone the catchment is situated in from Fig 15 (Rainfall Intensity Zones, see 4(c)). Adopt the C10 Value as shown in the following table.

Zone	C_{10}	Zone	C_{10}
Jamberoo	1.00	Wandandian	0.80
Kangaroo Valley	0.70	Clyde Watershed	0.80
Berry	1.00	Sussex Inlet	0.75
Nowra	0.95	Milton	0.95
Calymea Creek	0.85	Bawley Point	0.85
Huskisson	0.80		

Frequency factors

(iii) Adopt the frequency factor appropriate to the ARI from the following table.

Frequency Factors – FF_y	Below 500m AHD	Above 500m AHD
FF_1	0.62	0.89
FF_2	0.78	0.92
FF_5	0.90	0.95
FF_{10}	1.00	1.00
FF_{20}	1.10	1.05
FF_{50}	1.12	1.17
FF_{100}	1.18	1.24

Eg. Catchment in Nowra Zone below 500m ARI 5 yrs $C_5 = 0.95 \times 0.90 = 0.85$.

Catchment Area

5. The catchment area is identified and the area calculated from a contour plan and in this regard the 1:25000 topographic maps published by the Central Mapping Authority are acceptable. A copy of catchment plan must be submitted with the design.

Rainfall Intensity

6. The average rainfall intensity ($It_c y$) is derived after first determining the time of concentration (t_c). Australian Rainfall and Runoff provide for catchments up to 250km^2 for eastern NSW.

$t_c = 0.76\ A^{0.38}$ where t_c is in hours and A is the catchment area in km^2.

Worked Example

7. Determine the design flood from the following catchment –

Location Kangaroo Valley (below 500m)
Area = 2km^2 (200 ha)
Stream length = 2.3 km
Stream slope (weighted) = 3% = 30m/km
Average Recurrence Interval = 1 in 20 years

$C_{20} = 0.70 \times 1.10 = 0.77$
$t_c = 0.76 \times (2)^{.38} = 0.99\text{hrs}$

adopt t_c 1 hr
$I_{20} = 75.8$ mm/hr

$Q = 0.278 \times 0.77 \times 75.8 \times 2$
$= 32.45\ m^3/sec$

D5.21 STORMWATER DISCHARGE

1. Adequate provision is to be made for scour protection at all drainage inlets/outlets. Energy dissipaters shall also be provided where discharge velocities are greater than 2 metres per second. All inlets and outlets shall be provided with silt control measures. ***Scour***

2. Headwalls and aprons shall be constructed in accordance with Council's Specifications or as shown on the approved plan, with due attention to workmanship and reinforcement and general requirements for concrete. In addition, handrails, guard-rail, trash racks or other safety devices may be required. ***Headwalls***

3. Gabion (or similar) or reno mattress structures are to be provided where permanent scour protection is required. These structures are to be constructed in accordance with manufacturers specifications. All structures are to be placed on a layer of geotextile fabric. ***Erosion Control***

4. Where a pathway is to act as an overland flow path, full width concrete paving may be required. Consult Council at preliminary design stage. Details of water way calculations are to be provided to ensure that the 100 year event is contained within the pathway reserve. ***Concrete Pathway***

   Pathways are to extend across the full width of the footpath and be of such shape to satisfactorily convey water to the proposed outlet without causing erosion, scouring or the like. Footpath crossfall to be reversed at overland flowpath to direct major storm flow down design overland flowpath. ***Overland Flowpath***

5. Drainage outlet pipes under pathways/overland flow paths shall generally extend a minimum of 5 metres into Council's reserve. This is to allow both pedestrian and vehicular access where applicable. Pathways/overland flow paths are to extend to the limits of the pipeline. The provision of an approved hand rail or treated timber log fencing suitable for in ground use will be required around the outlet headwall. ***Outlets Under Pathways to Reserves***

D5.22 MANAGEMENT OF STORMWATER FROM DEVELOPMENT SITES

1. The guidelines contained in this section apply to all developments within the City. It is the responsibility of applicants to submit satisfactory detail and as required in these guidelines and manual. ***General***

   Detached plans showing the proposed method of stormwater disposal are to be submitted with the Development/Building Application as approval will not be granted to any work commencing on site until the stormwater disposal system has been approved. All drainage designs, investigations shall be prepared in accordance with all relevant sections of this Code.

2. Property drainage discharge from a development can be discharged into the kerb and gutter by two 100mm diameter pipes where the calculated flow does not exceed the pipes capacity. ***Property Discharge***

   Connection into kerb shall be by way of an approved adapter.

Connection

underground system

3. Where the calculated stormwater discharge from the Development exceeds the capacity of two 100mm diameter pipes then discharge into the kerb and gutter may be provided by means of a concrete converter. The maximum discharge allowable by this method is 55l/sec construction is to be in accordance with Standard Drawing Plan ref. 263726. ***Into kerb and gutter***

4. Where the total discharge from any development exceeds 55 l/s, the disposal of all concentrated/paved surface runoff and roof discharge shall be by means of a suitably designed and constructed pipeline to the nearest available Council pit. The minimum RC pipe size shall be 375mm diameter. ***Connection to Council's Drainage System***

5. On development sites where the provision of onsite detention basins are impractical and the magnitude of the stormwater discharge is such that it is greater than that prior to development, the capacity of the drainage system into which the stormwater from the development discharges shall be checked in accordance with D5.07(3). If part of the drainage system is found to not have the required capacity, then that part shall be augmented/amplified to take the additional flow. ***Amplification Council's System***

6. When a property downstream of the development site must be traversed with a drainage pipeline in order to reach an appropriate point of discharge an easement to drain water must be obtained over the downstream property. Building consent will not be issued if an easement is not obtained. This applies to new subdivisions, residential, medium density developments, dual occupancies, commercial and industrial developments on land which naturally slopes away from its existing road frontage. ***Easement Downstream Properties***

7. Generally, no obstruction to the overland flow of stormwater runoff form adjacent properties on the high side of the development shall be permitted. Allowance shall be made of the overland flow component and adequately catered for. ***Flow Adjacent Properties***

8. Where connection must be made to Council's underground drainage system, the minimum pipe size crossing the footpath directly in front of the allotment is to be 150mm in diameter. ***Connection to Council Drains***

9. Sumps and inlet pits shall conform with the standards shown in Standard Drawings ref no. 263701 to 263708. Alternatives to cast-in-situ concrete may be acceptable subject to approval from the Subdivision and Development Manager. ***Inlet Pits***

   Heavy duty cast iron or fabricated galvanised steel grates shall be provided on all surface inlet pits in areas with vehicular traffic.

   Paved areas and driveways falling towards Council's footpath shall be provided with a full driveway width grated box drain at the property boundary, draining into the internal system and of minimal internal dimension of 100mm. See Standard Drawing ref no. 263717 (Fig. D5.Misc.11).

10. At development application stage where a building application will later be required, the following information will be required: ***DA Stage***

    - A concept site drainage layout (1:100 scale). This plan shall also show how the proposed drainage system is integrated with the proposed landscape plan for the site.
    - Flood study if applicable.
    - Proposed point/s of discharge.

- Written agreement from downstream property owners to provide an easement to drain water if applicable.
- Calculations of velocities and flows leaving the site both prior and after development.
- Proposed floor levels.
- Survey of the site providing sufficient information to be able to assess the application, which includes lot boundaries, buildings, easements, services etc.
- all levels to be related to Australian Height Datum (AHD).

12. At building application stage the following information will be required: ***BA Stage***

- Catchment plan (which includes site area plus other areas draining to site).
- Hydrologic and hydraulic calculations.
- Except for developments discharging less than 55l/sec, longitudinal section of pipelines showing calculated flows, velocity, size and class of pipe, grade, invert levels, services and ground levels.
- Where connection is to be made to an existing underground drainage system, a hydraulic grade line analysis shall be carried out and the pressure line plotted on the longitudinal section.
- Details, dimensions and location of pits, grates, onsite detention storages, weirs, outlet structures and scour protection.
- The details of any special features affecting runoff such as stormwater concentration from or to adjoining lands.
- Plan (1:100 scale) showing proposed and existing floor, ground and pavement levels to AHD. Note that sufficient levels shall be shown on the plan to enable its proper construction. It is insufficient to show arrows to indicate a fall in the pavement.

13. Both Development and Building applications shall be prepared in accordance with the Australian Technical Drawing Standard AS1100. ***Engineering Submission***

Flood studies shall be clearly and concisely presented.

Calculations to be clearly prevented in a logical sequenced manner. Calculations will not be accepted from floppy disc without hard copy.

D5.23 DEVELOPMENT SITES AFFECTED BY FLOODING

1. Flood studies shall be prepared by suitable qualified civil engineers. The flood study shall be prepared in accordance with the relevant Sections of this manual. The 100 year Average Recurrence Interval (ARI) flood event shall be adopted to assess the effects of flooding on the proposed development site and adjacent properties. In certain circumstances it may be necessary to assess the effects from lesser storm events or in fact the Maximum Flood Level (MFL). This will be required where the MFL is likely to be higher than the floor level set under D5.23 (2). ***Flood Studies***

2. Developments/Subdivisions with watercourses traversing the site require special attention and care. Depending on the size of the watercourse any development in the vicinity may require special approval from State Government Departments, such as Department of Land and Water Conservation and the Environment Protection Authority. Council is willing to accept an improvement to watercourses to alleviate flooding and scour provided adequate attention is paid to the environmental effects of the proposal and the requirements of State Government Departments are met.

Flood studies in support of subdivision proposal, must be prepared by a suitably qualified and experienced civil engineer who holds current registration to NPER3. A study must address the flow characteristics of the watercourse for the 100 Yr ARI Flood Event and take into account all upstream and downstream features and structures which may affect flows through the site.

The report submitted to Council must include the following information to allow proper assessment –

- locality sketch
- catchment plan
- survey information
- sketches and plans used for analysis
- data and calculations of hydrologic/hydraulic analysis
- 100 yr ARI flood levels over the site
- assessment of flood hazard and sensitivity analysis for changes in flood levels
- contour plan showing extent of inundation on the site
- assessment of scour potential of watercourses
- onsite detention proposals as required
- conclusions and recommendations resulting from analysis

Flood Warning Signs

Flood warning signs are to be provided, as required by Council, at regular internals along channel systems, in and abutting detention basins, at road crossings and points of pedestrian congregation or movement such as pathways, cycleways, parking areas, amenities blocks etc. Refer Standard Drawing No. SD 131.

Minimum Flood Level

3. Habitable floor levels are to be 0.5 metres above 1:100 year ARI flood level for residential development in floodways and 0.3 metres in flood storage and flood fringe areas. Structures shall be flood proofed to at least 0.5 metres above the 1:100 year ARI flood level in accordance with the Floodplain Development Manual.

Parking Area Levels

4. Parking area levels are to be set to limit 1:100 year ARI flood flow velocity and depth to within the vehicle stability limits as provided in Table 5.3.

Filling Flood Liable Land

5. In general it is undesirable to fill land or construct buildings which are located within the 1:100 year ARI level.

Within those areas affected by flooding filling is not to be undertaken unless Development Approval has been obtained form Council. Filling operations must include adequate provision for drainage of surface water and be so placed and graded in order to prevent the shedding of surface water direct to adjoining properties. Drainage of adjoining lots must not be impeded, whether surface or subsurface flow.

Subdivisions Flood Prone Land

6. Creation of new residential lots by subdivision will not be permitted in floodways. Creation of new lots by subdivision in designated flood storage areas is permissible provided that the cumulative total of all filling is not more than 1.0% of the available water storage volumes in that basin as determined by Council.

Creation of new residential lots by subdivision is permissible in designated flood fringe areas.

D5.24 MISCELLANEOUS

1. Subsoil drainage in Pipe Trenches – Subsoil Drainage shall be provided in pipe trenches as outlined below.

2. In cases where pipe trenches are backfilled with sand or other pervious material, a 1.5m length of subsoil drain shall be constructed in the bottom of the trench immediately upstream from each pit or headwall. The subsoil drain shall consist of 100mm diameter agricultural pipes, butt jointed, with joints wrapped with hessian or slotted PVC pipe. ***Subsoil Drain at Pits***

3. The upstream end of the subsoil drain shall be sealed with cement mortar, and the downstream end shall discharge through the wall of the pit or headwall.

4. The CBR method of pavement design is based on the assumption that the water table is at least 0.5 metres below subgrade level. To achieve this subsurface drains at this level should be provided to intercept side-hill seepage, high water table or isolated springs are present or if it is considered appropriate for the protection or draining of the road pavement, kerb and gutter or private property. Risers for cleaning out the drains shall be provided at the end of lines and at intermediate points such that the distance between pits and/or risers shall not exceed 60 metres. ***Pavement Subsoil Drains***

   The absolute minimum grade shall be 0.5%. Subsoil drains shall discharge into gully pits, junction pits or culverts.

5. Termination of kerb and gutter and associated scour protection – kerb and gutter shall be extended to drainage pit or natural point of outlet. Where outlet velocity is greater than 2.5 metres per second or where the kerb and gutter discharge causes scour, then protection shall be provided to prevent scour and dissipate the flow. ***Kerb & Gutter Termination***

DOCUMENTATION

D5.25 SUBDIVISION ENGINEERING PLANS

1. Catchment Area Plans shall be drawn at scales of 1:500 to 1:1000 for urban catchments and 1:4000 or 1:25000 for rural catchments, unless alternative scales are specifically approved by Council and shall show contours, direction of grading of kerb and gutter, general layout of the drainage system with pit locations, catchment limits and any other information necessary for the design of the drainage system. ***Drawing Scales***

2. The drainage system layout plan shall be drawn at a scale of 1:500 and shall show drainage pipeline location, drainage pit location and number and road centreline chainage, size of opening and any other information necessary for the design and construction of the drainage system.

3. The plan shall also show all drainage easements, reserves and natural water courses. The plan may be combined with the road layout plan.

4. The drainage system longitudinal section shall be drawn at a scale of 1:500 horizontally and 1:50 vertically and shall show pipe size, class and type, pipeline and road chainages, pipeline grade, hydraulic grade line location of underground services and any other information necessary for the design and construction of the drainage system.

5. Open channel cross sections shall be drawn at a scale of 1:100 natural and shall show the direction in which the cross sections should be viewed. Reduced levels are to be at Australian Height Datum, (AHD), unless otherwise approved by Council where AHD is not available. ***Open Channels***

6. Special Details including non-standard pits, pit benching, open channel designs and transitions shall be provided on the design drawings at scales appropriate to the type and complexity of the detail being shown.

7. Work as Executed Plans shall be submitted to Council upon completion of the drainage construction and prior to release of the linen plan. The detailed design plans may form the basis of this information, however, any changes must be noted on these plans. ***Work-as-Executed Plans***

D5.26 EASEMENTS AND AGREEMENTS

1. Evidence of any Deed of Agreement necessary to be entered into as part of the drainage system will need to be submitted prior to any approval of the engineering plans. Easements will need to be created prior to or shown on the survey plan of subdivision.

2. Where an agreement is reached with an adjacent landowner to increase flood levels on his property or otherwise adversely affect his property, a letter signed by all the landowners outlining what they have agreed to and witnessed by an independent person shall be submitted prior to any approval of the engineering plans.

D5.27 COMPUTER PROGRAM FILES AND PROGRAM OUTPUT

1. Computer program output may be provided as long as summary sheets for Hydrological and Hydraulic calculations in accordance with this Specification are provided with plans submitted for checking and with final drawings.

2. Copies of final computer data files, for both hydrological and hydraulic models shall be provided for Council's data base of flooding and drainage information in formats previously agreed with Council.

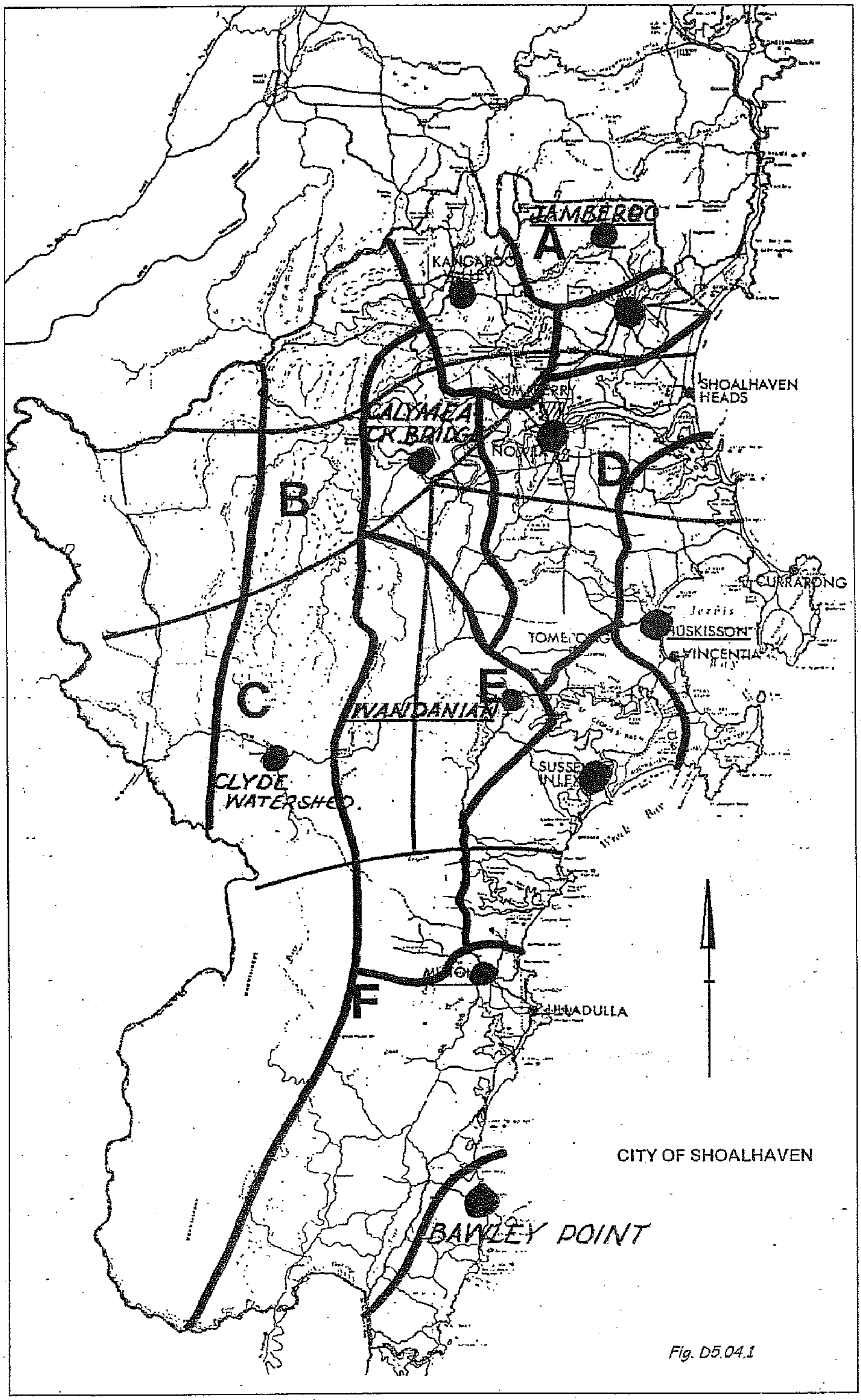

JAMBEROO
A
KANGAROO VALLEY
SHOALHAVEN HEADS
BOMADERRY
CALYMEA CK BRIDGE
NOWRA
D
B
CURRARONG
Jervis
HUSKISSON
VINCENTIA
TOMERONG
E
C
WANDANIAN
CLYDE WATERSHED.
SUSSEX INLET
Wreck Bay
MILTON
F
ULLADULLA
CITY OF SHOALHAVEN
BAWLEY POINT

Fig. D5.04.1

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 34.675 S 150.725 E * NEAR JAMBEROO

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

RAW DATA 53.30, 13.04, 5.10, 122.21, 33.11, 11.95, 0.000, 1HG1

PREPARED BY — HYDROLOGY BRANCH — BUREAU OF METEOROLOGY — MELBOURNE

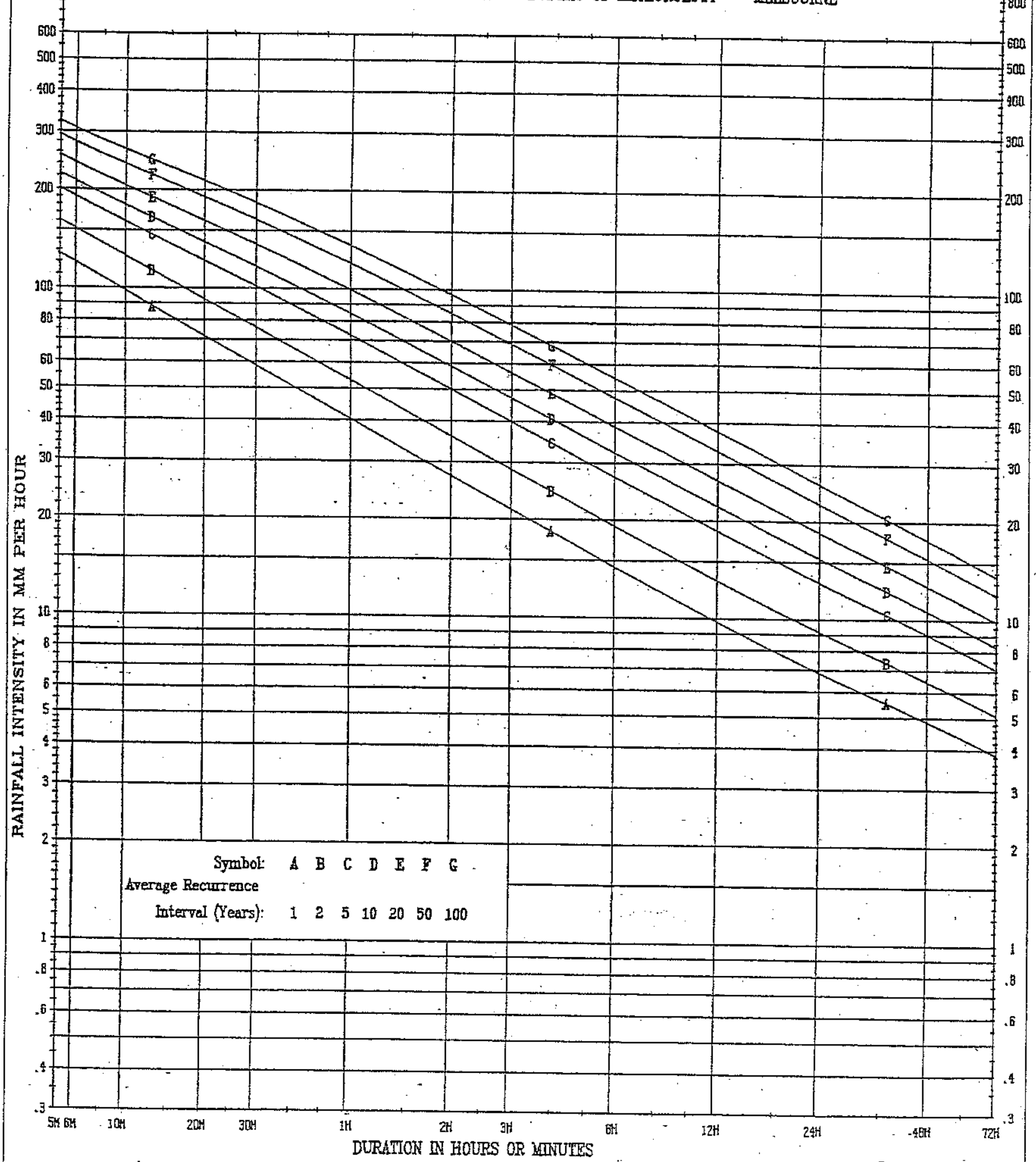


Fig. D5.04.2

LOCATION 34.675 S 150.725 E * NEAR.. JAMBEROO

ISSUED 2ND MARCH 1989 REF. -FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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

I = INTENSITY IN MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.7029	-0.5527	-0.0258	0.00378	0.001396	0.0003515	-0.0001274
2	3.9776	-0.5424	-0.0279	0.00459	0.001351	0.0002082	-0.0001018
5	4.2888	-0.5134	-0.0334	0.00577	0.001384	-0.0000464	-0.0000607
10	4.4407	-0.4986	-0.0360	0.00644	0.001375	-0.0001826	-0.0000381
20	4.6062	-0.4863	-0.0384	0.00706	0.001381	-0.0003046	-0.0000176
50	4.7938	-0.4723	-0.0411	0.00776	0.001384	-0.0004468	0.0000052
100	4.9179	-0.4629	-0.0430	0.00816	0.001402	-0.0005339	0.0000195

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	128.	163.	203.	226.	256.	296.	326.
0.100	120.	153.	192.	214.	243.	281.	309.
0.167	98.7	126.	161.	181.	207.	241.	267.
0.333	71.9	93.2	122.	140.	162.	192.	214.
0.500	58.7	76.6	102.	118.	137.	164.	184.
1.000	40.6	53.4	72.9	84.8	100.	121.	137.
2.000	27.4	36.2	50.4	59.2	70.3	85.6	97.4
3.000	21.6	28.7	40.2	47.5	56.6	69.2	79.0
6.000	14.4	19.2	27.3	32.4	38.9	47.8	54.7
12.000	9.82	13.1	18.7	22.3	26.8	33.0	37.9
24.000	6.91	9.21	13.1	15.5	18.6	22.8	26.2
48.000	4.89	6.48	9.04	10.7	12.7	15.5	17.7
72.000	3.87	5.13	7.09	8.32	9.91	12.0	13.7

USER DATA 53.30, 19.09, 5.10, 122.21, 53.11, 11.94, 0.000, [illegible]

Fig. D5.04.3

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 34.725 S 150.525 E * NEAR.. KANGAROO VALLEY

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.–FN2912

(RAW DATA 41.46, 10.33, 3.47, 90.95, 22.85, 6.94, 0.020, 1HG)

PREPARED BY –– HYDROLOGY BRANCH –– BUREAU OF METEOROLOGY –– MELBOURNE

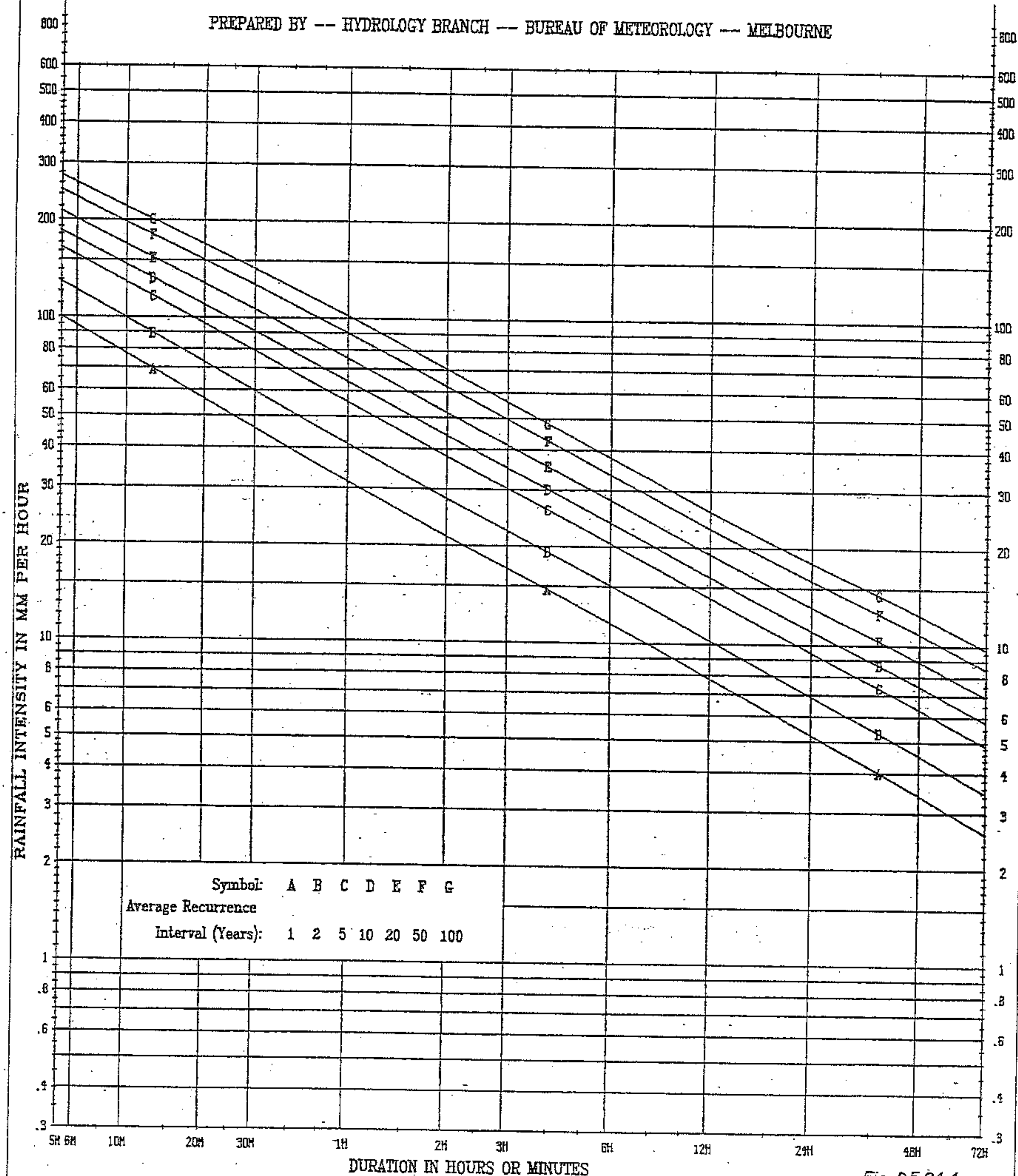


Fig. D5.04.4

LOCATION 34.725 S 150.525 E * NEAR.. KANGAROO VALLEY ISSUED 2ND MARCH 1989 REF.-FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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 MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.4559	-0.5531	-0.0135	0.00741	-0.001044	-0.0001711	0.0000219
2	3.7260	-0.5475	-0.0171	0.00697	-0.000571	-0.0001298	0.0000014
5	4.0220	-0.5336	-0.0250	0.00664	0.000256	-0.0001319	-0.0000179
10	4.1668	-0.5263	-0.0291	0.00648	0.000704	-0.0001254	-0.0000305
20	4.3275	-0.5206	-0.0327	0.00642	0.001098	-0.0001296	-0.0000404
50	4.5092	-0.5138	-0.0368	0.00623	0.001552	-0.0001263	-0.0000529
100	4.6300	-0.5093	-0.0394	0.00610	0.001856	-0.0001187	-0.0000620

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	101.	130.	166.	186.	214.	250.	277.
0.100	94.8	122.	156.	176.	202.	236.	262.
0.167	77.8	100.	130.	147.	169.	199.	221.
0.333	56.6	73.5	96.5	110.	128.	152.	170.
0.500	46.1	60.0	79.7	91.4	107.	127.	143.
1.000	31.7	41.5	55.8	64.5	75.8	90.9	103.
2.000	21.5	28.2	38.2	44.3	52.1	62.7	70.8
3.000	17.1	22.5	30.4	35.3	41.5	49.9	56.5
6.000	11.6	15.2	20.6	23.8	28.1	33.7	38.1
12.000	7.85	10.3	14.0	16.2	19.1	23.0	26.0
24.000	5.26	6.94	9.51	11.1	13.2	15.9	18.1
48.000	3.43	4.56	6.36	7.51	8.96	10.9	12.5
72.000	2.62	3.48	4.91	5.81	6.96	8.51	9.77

(RAW DATA 41.45, 10.39, 3.47, 90.95, 22.05, 8.94, 0.020, 1HG1)

Fig. D5.04.5

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 34.775 S 150.700 E * NEAR.. BERRY

* ENSURE THE COORDINATES ARE THOSE REQUIRED,
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

RAW DATA 49.06, 10.68, 3.39,107.79, 25.90, 6.51, 0.000, 2HG

PREPARED BY — HYDROLOGY BRANCH — BUREAU OF METEOROLOGY — MELBOURNE

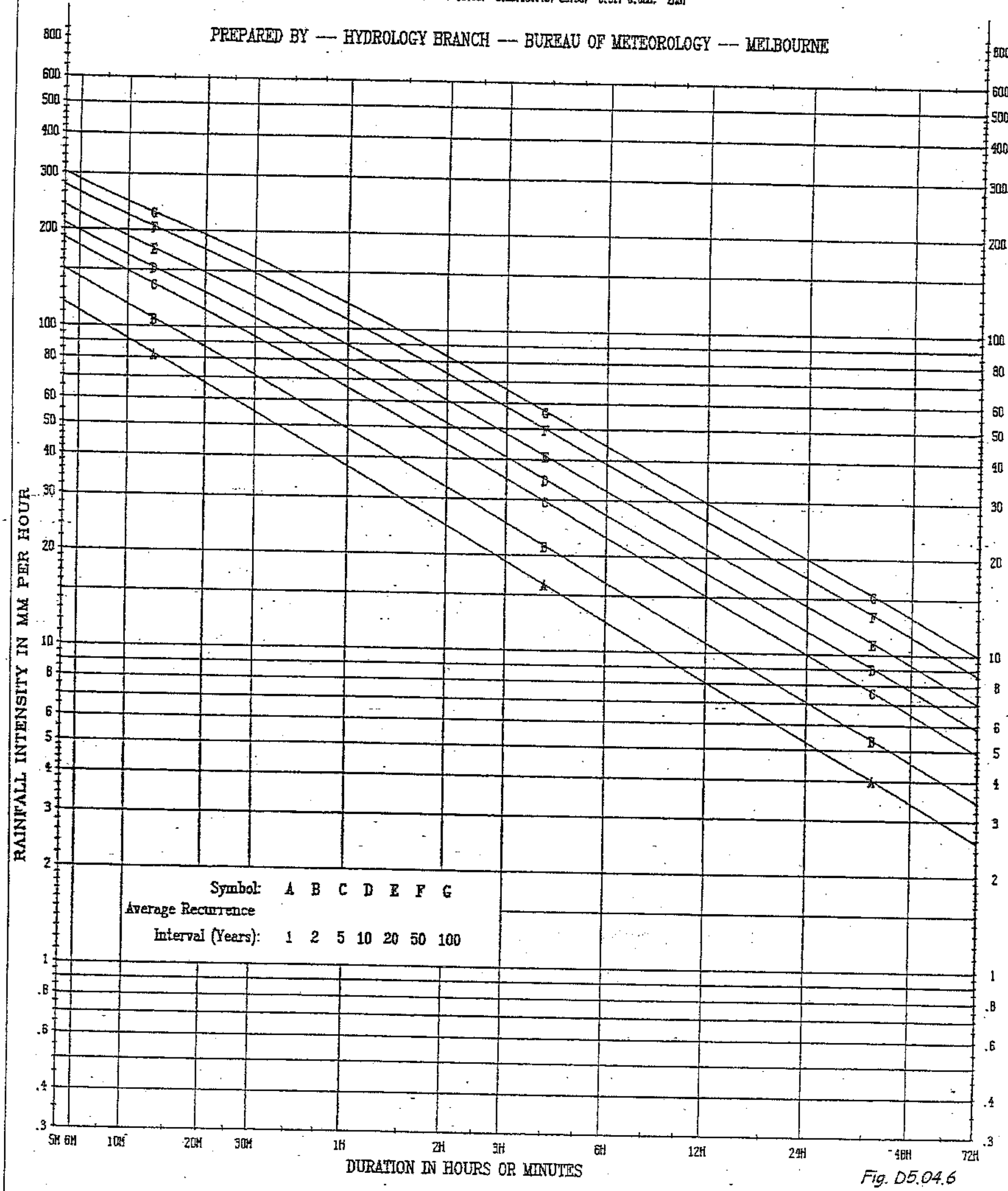


Fig. D5.04.6

LOCATION 34.775 S 150.700 E * NEAR. BERRY

ISSUED 2ND MARCH 1989 REF.–FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED. SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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

I = INTENSITY IN MILLIMETRES PER HOUR

T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.6170	-0.5802	-0.0353	0.00757	0.000938	-0.0002075	-0.0000266
2	3.8868	-0.5711	-0.0375	0.00785	0.001102	-0.0002635	-0.0000226
5	4.1819	-0.5457	-0.0420	0.00812	0.001385	-0.0003717	-0.0000109
10	4.3261	-0.5325	-0.0445	0.00830	0.001543	-0.0004276	-0.0000055
20	4.4846	-0.5212	-0.0465	0.00828	0.001702	-0.0004568	-0.0000046
50	4.6649	-0.5094	-0.0489	0.00863	0.001834	-0.0005346	0.0000047
100	4.7835	-0.5012	-0.0502	0.00865	0.001924	-0.0005556	0.0000066

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	118.	151.	189.	210.	238.	275.	303.
0.100	111.	141.	177.	197.	225.	260.	286.
0.167	91.1	117.	148.	166.	190.	221.	245.
0.333	66.9	86.5	112.	128.	147.	174.	194.
0.500	54.6	71.0	93.5	107.	124.	147.	165.
1.000	37.2	48.8	65.5	75.6	88.6	106.	120.
2.000	24.5	32.3	44.1	51.3	60.6	73.1	82.7
3.000	19.1	25.2	34.6	40.4	47.9	57.9	65.7
6.000	12.3	16.3	22.7	26.7	31.8	38.6	44.0
12.000	8.03	10.7	15.0	17.7	21.1	25.8	29.5
24.000	5.27	7.02	9.91	11.8	14.1	17.3	19.8
48.000	3.40	4.54	6.44	7.66	9.21	11.3	13.0
72.000	2.55	3.41	4.86	5.78	6.96	8.54	9.85

Fig. D5.04.7

BSW DATA 49.08, 10.68, 3.39,107.73, 25.90, 6.51, 0.000, 2KG

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 34.875 S 150.625 E * NEAR.. NOWRA

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

(RAW DATA 44.75, 8.72, 2.82, 94.89, 20.80, 6.19, 0.010, 2HG)

PREPARED BY --- HYDROLOGY BRANCH --- BUREAU OF METEOROLOGY --- MELBOURNE

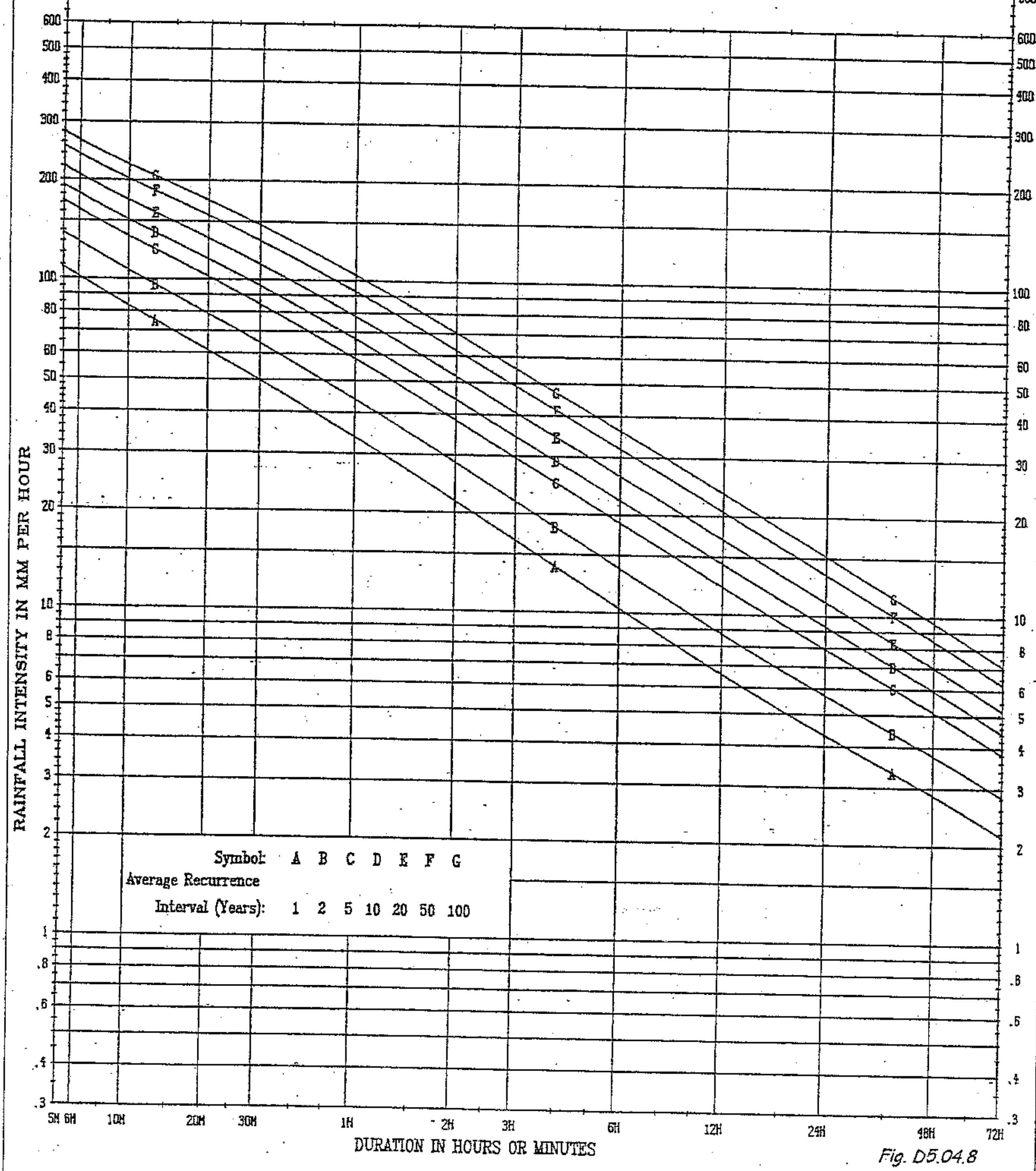


Fig. D5.04.8

LOCATION 34.875 S 150.625 E * NEAR.. NOWRA ISSUED 2ND MARCH 1989 REF.–FN2912

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

* ENSURE THE COORDINATES ARE THOSE REQUIRED. SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

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

I = INTENSITY IN MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.5248	-0.6022	-0.0532	0.00634	0.003124	-0.0000276	-0.0001159
2	3.7891	-0.5946	-0.0528	0.00699	0.002867	-0.0001280	-0.0000917
5	4.0721	-0.5728	-0.0528	0.00816	0.002458	-0.0003437	-0.0000466
10	4.2090	-0.5607	-0.0523	0.00847	0.002222	-0.0004290	-0.0000257
20	4.3636	-0.5515	-0.0522	0.00906	0.001999	-0.0005350	-0.0000025
50	4.5365	-0.5408	-0.0521	0.00963	0.001780	-0.0006422	0.0000219
100	4.6516	-0.5336	-0.0517	0.00995	0.001577	-0.0006993	0.0000382

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	109.	139.	174.	194.	221.	256.	282.
0.100	102.	130.	163.	182.	208.	240.	265.
0.167	83.6	107.	136.	152.	175.	203.	225.
0.333	61.4	79.3	103.	116.	134.	157.	175.
0.500	50.2	65.0	84.9	96.6	112.	132.	148.
1.000	33.9	44.2	58.7	67.3	78.6	93.4	105.
2.000	21.9	28.6	38.6	44.6	52.4	62.8	70.8
3.000	16.6	21.9	29.8	34.6	40.8	49.1	55.6
6.000	10.4	13.7	18.9	22.2	26.3	31.9	36.3
12.000	6.60	8.75	12.2	14.3	17.0	20.7	23.6
24.000	4.34	5.74	7.94	9.31	11.1	13.4	15.3
48.000	2.87	3.76	5.12	5.96	7.01	8.47	9.61
72.000	2.16	2.84	3.82	4.43	5.19	6.27	7.12

RAW DATA 44.75, 8.72, 2.62, 94.20, 20.80, 6.19, 0.010, 2161

Fig. D5.04.9

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 34.825 S 150.375 E * NEAR.. CALYMEA CK BRIDGE

* ENSURE THE COORDINATES ARE THOSE REQUIRED,
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

IFD DATA 39.44, 8.74, 2.60, 84.36, 19.00, 6.31, 0.090, 1HG1

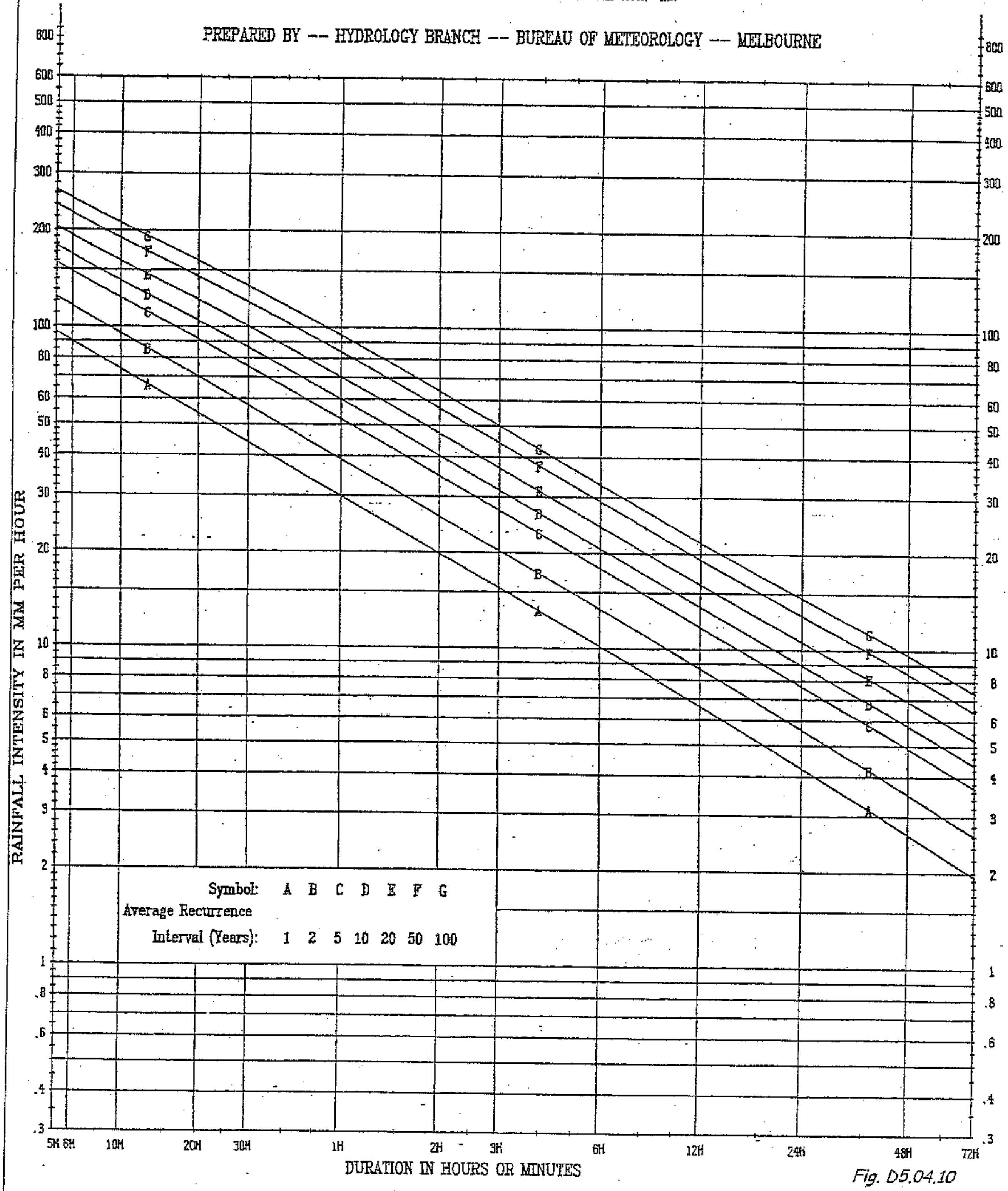


Fig. D5.04.10

LOCATION 34.825 S 150.375 E * NEAR.. CALYMEA CK BRIDGE ISSUED 2ND MARCH 1989 REF.–FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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 MILLIMETRES PER HOUR

T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.3999	-0.5775	-0.0267	0.00841	-0.000319	-0.0003460	0.0000283
2	3.6674	-0.5738	-0.0298	0.00855	0.000038	-0.0003645	0.0000211
5	3.9545	-0.5616	-0.0356	0.00813	0.000726	-0.0003305	-0.0000008
10	4.0963	-0.5553	-0.0392	0.00805	0.001140	-0.0003381	-0.0000099
20	4.2545	-0.5498	-0.0422	0.00768	0.001531	-0.0002999	-0.0000266
50	4.4337	-0.5441	-0.0453	0.00755	0.001898	-0,0002927	-0.0000373
100	4.5530	-0.5399	-0.0475	0.00734	0.002176	-0.0002751	-0.0000471

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	96.3	124.	158.	178.	205.	240.	267.
0.100	90.2	116.	149.	168.	193.	226.	251.
0.167	74.0	95.5	123.	139.	161.	189.	211.
0.333	54.1	70.2	91.8	105.	122.	144.	161.
0.500	44.0	57.3	75.5	86.5	101.	120.	135.
1.000	29.9	39.2	52.2	60.1	70.4	84.2	94.9
2.000	19.9	26.0	34.8	40.3	47.3	56.7	64.0
3.000	15.5	20.3	27.3	31.5	37.0	44.4	50.1
6.000	10.2	13.3	17.8	20.6	24.2	29.1	32.8
12.000	6.62	8.68	11.7	13.6	16.0	19.2	21.7
24.000	4.26	5.61	7.66	8.94	10.6	12.8	14.5
48.000	2.65	3.52	4.91	5.78	6.91	8.44	9.66
72.000	1.95	2.61	3.67	4.35	5.22	6.40	7.36

RAW DATA 39.44, 8.74, 2.60, 84.36, 19.00, 6.31, 0.040, 1163

Fig. D5.04.11

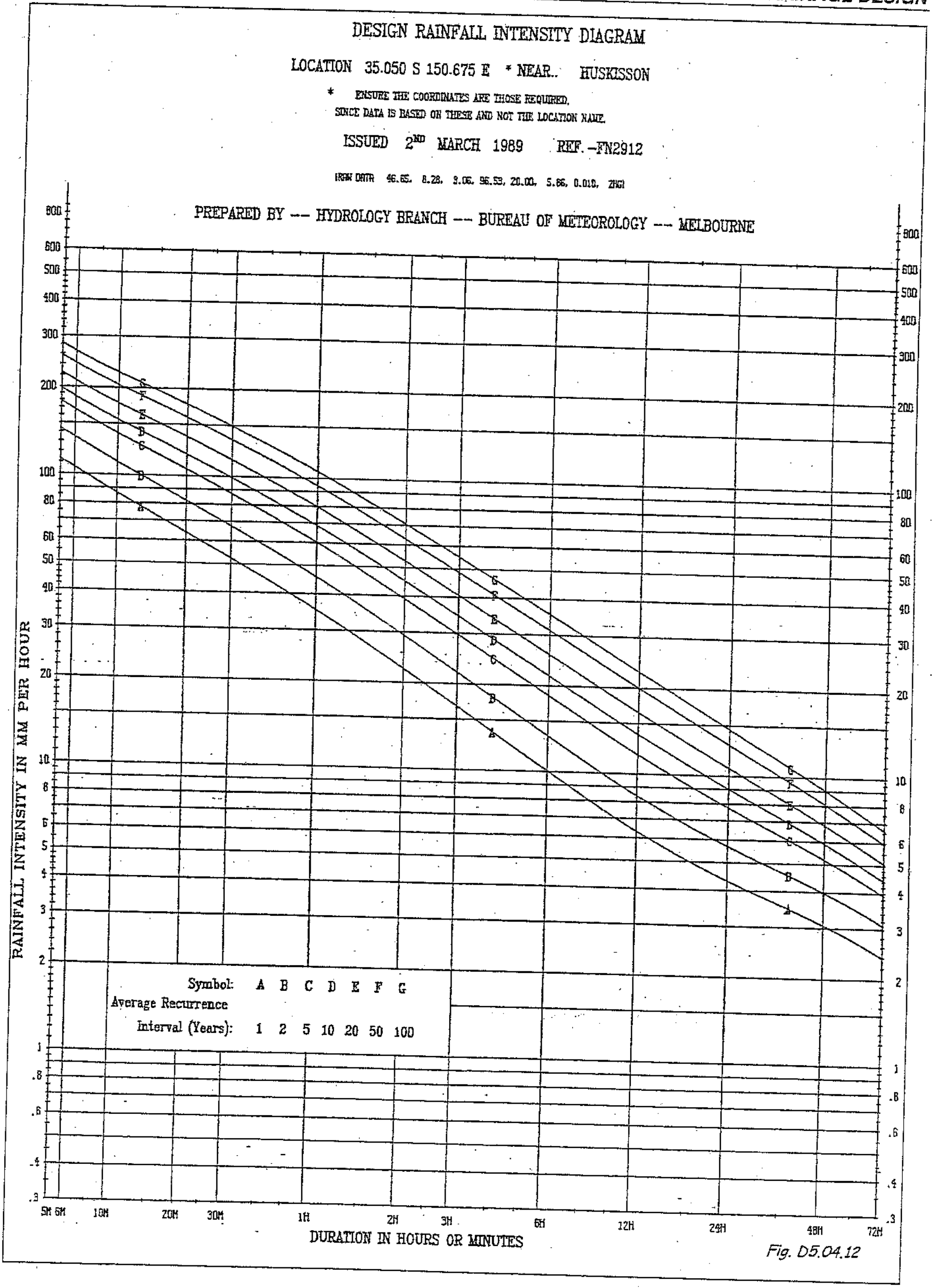

DESIGN RAINFALL INTENSITY DIAGRAM
LOCATION 35.050 S 150.675 E * NEAR.. HUSKISSON
* ENSURE THE COORDINATES ARE THOSE REQUIRED,
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.
ISSUED 2ND MARCH 1989 REF.-FN2912
PREPARED BY --- HYDROLOGY BRANCH --- BUREAU OF METEOROLOGY --- MELBOURNE
RAINFALL INTENSITY IN MM PER HOUR
DURATION IN HOURS OR MINUTES
Symbol: A B C D E F G
Average Recurrence
Interval (Years): 1 2 5 10 20 50 100

Fig. D5.04.12

LOCATION 35.050 S 150.675 E * NEAR. HUSKISSON ISSUED 2ND MARCH 1989 REF.–FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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

I = INTENSITY IN MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.5680	–0.6182	–0.0738	0.00357	0.006232	0.0004081	–0.0002702
2	3.8277	–0.6092	–0.0717	0.00445	0.005679	0.0002526	–0.0002283
5	4.1001	–0.5854	–0.0673	0.00669	0.004421	–0.0001273	–0.0001340
10	4.2330	–0.5729	–0.0650	0.00788	0.003736	–0.0003328	–0.0000820
20	4.3820	–0.5627	–0.0628	0.00882	0.003146	–0.0004937	–0.0000396
50	4.5496	–0.5507	–0.0605	0.00975	0.002530	–0.0006575	0.0000041
100	4.6625	–0.5429	–0.0592	0.01041	0.002137	–0.0007768	0.0000342

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	113.	144.	179.	198.	225.	258.	284.
0.100	106.	135.	168.	186.	211.	243.	267.
0.167	87.0	111.	139.	156.	177.	205.	227.
0.333	64.2	82.4	106.	119.	137.	160.	177.
0.500	52.5	67.7	87.6	99.2	114.	134.	150.
1.000	35.4	46.0	60.3	68.9	80.0	94.6	106.
2.000	22.3	29.2	39.1	45.1	52.7	63.0	70.9
3.000	16.7	21.9	29.7	34.5	40.6	48.8	55.2
6.000	10.0	13.3	18.4	21.6	25.7	31.2	35.5
12.000	6.32	8.38	11.7	13.7	16.4	19.9	22.7
24.000	4.32	5.68	7.74	9.00	10.6	12.8	14.5
48.000	3.07	3.97	5.18	5.88	6.81	8.06	9.00
72.000	2.39	3.07	3.92	4.41	5.06	5.94	6.59

RAW DATA 46.65, 6.28, 3.06, 96.59, 20.00, 5.86, 0.010, [illegible]

Fig. D5.04.13

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 35.075 S 150.525 E * NEAR.. WANDANDIAN

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

(RAW DATA 44.31, 10.15, 3.68, 95.55, 23.91, 7.99, 0.020, 1HG)

PREPARED BY -- HYDROLOGY BRANCH -- BUREAU OF METEOROLOGY -- MELBOURNE

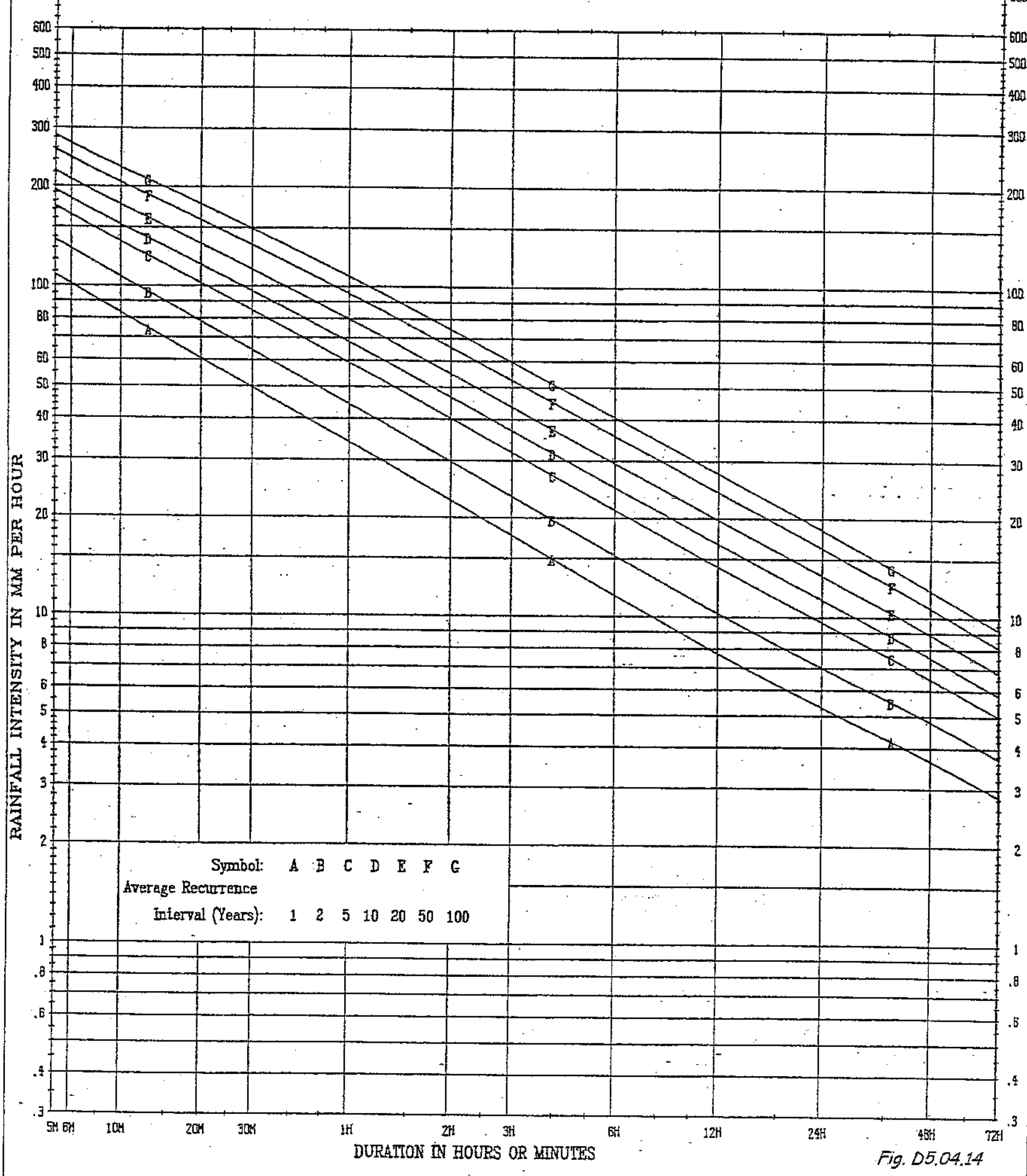


Fig. D5.04.14

LOCATION 35.075 S 150.525 E * NEAR.. WANDANDIAN ISSUED 2ND MARCH 1989 REF.–FN2912

LIST ØF CØEFFICIENTS TØ EQUATIØNS ØF THE FØRM

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

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

I = INTENSITY IN MILLIMETRES PER HØUR

T = TIME IN HØURS

RETURN PERIØD (YEARS)	a	b	c	d	e	f	g
1	3.5209	-0.5683	-0.0320	0.00517	0.001529	0.0001610	-0.0001000
2	3.7878	-0.5600	-0.0325	0.00512	0.001499	0.0001352	-0.0000960
5	4.0777	-0.5406	-0.0343	0.00652	0.001213	-0.0001147	-0.0000470
10	4.2191	-0.5302	-0.0351	0.00712	0.001065	-0.0002277	-0.0000242
20	4.3768	-0.5215	-0.0359	0.00747	0.000970	-0.0003047	-0.0000090
50	4.5549	-0.5119	-0.0367	0.00813	0.000838	-0.0004207	0.0000131
100	4.6736	-0.5049	-0.0372	0.00824	0.000769	-0.0004678	0.0000232

RAINFALL INTENSITY IN MM/HR FØR VARIØUS DURATIØNS AND RETURN PERIØDS

DURATIØN (HØURS)	RETURN PERIØD 1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
0.083	107.	137.	173.	194.	221.	257.	285.
0.100	101.	129.	163.	183.	209.	243.	269.
0.167	82.8	106.	136.	153.	176.	205.	228.
0.333	60.4	78.2	102.	116.	134.	158.	177.
0.500	49.3	64.0	84.3	96.3	112.	133.	149.
1.000	33.8	44.2	59.0	68.0	79.6	95.1	107.
2.000	22.5	29.5	40.0	46.4	54.6	65.7	74.3
3.000	17.6	23.2	31.6	36.8	43.4	52.4	59.5
6.000	11.5	15.2	21.0	24.6	29.2	35.4	40.3
12.000	7.69	10.2	14.1	16.6	19.7	24.0	27.3
24.000	5.27	6.97	9.59	11.2	13.3	16.1	18.3
48.000	3.62	4.76	6.45	7.48	8.82	10.6	12.0
72.000	2.83	3.69	4.96	5.74	6.74	8.08	9.11

(RAW DATA 44.91, 10.15, 3.68, 95.55, 23.91, 7.99, 0.020, 1HS)

Fig. D5.04.15

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 35.100 S 150.275 E * NEAR. CLYDE WATERSHED

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

IFD DATA 38.79, 10.19, 3.77, 79.91, 20.95, 7.53, 0.050, 1980

PREPARED BY -- HYDROLOGY BRANCH -- BUREAU OF METEOROLOGY -- MELBOURNE

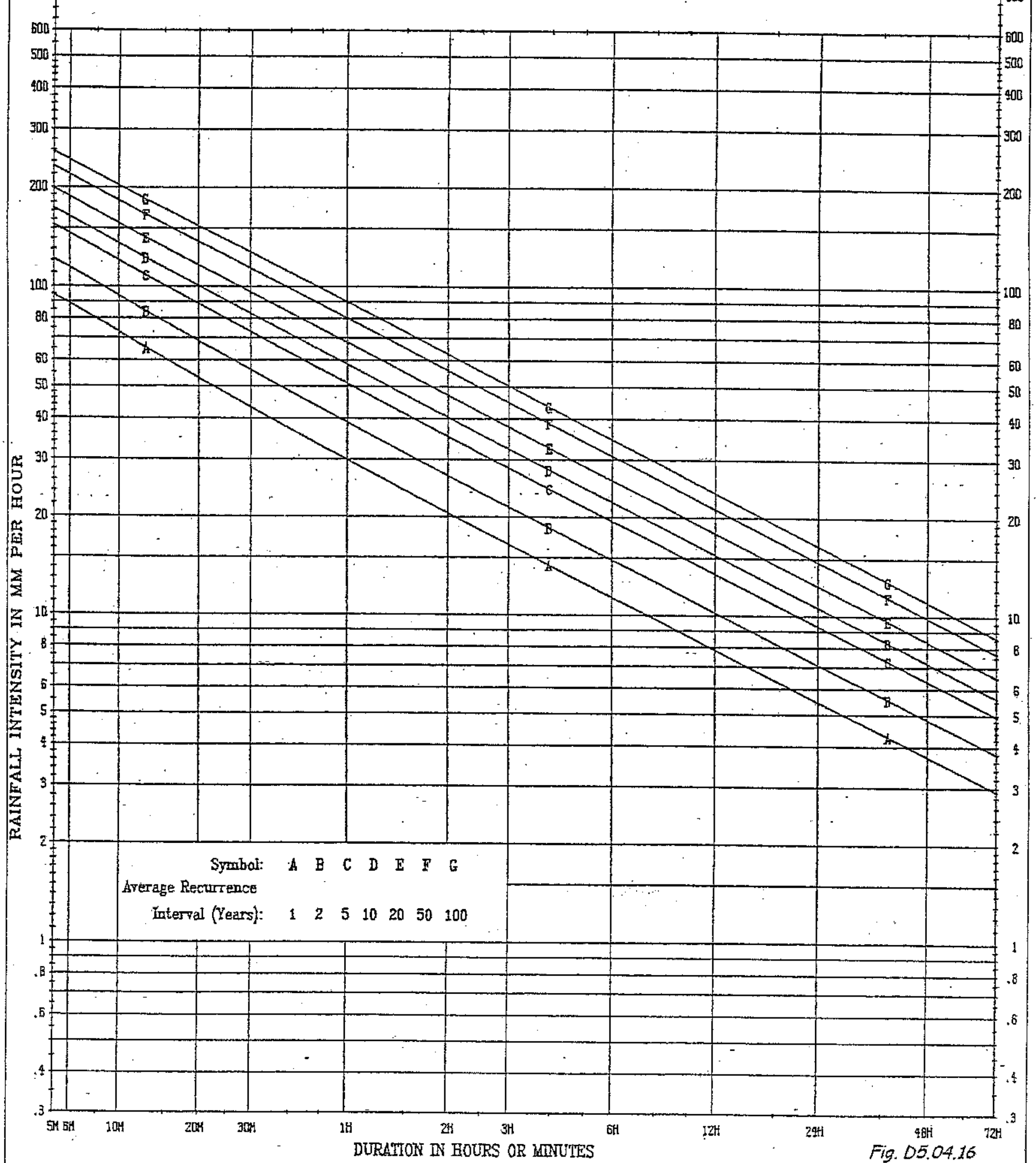


Fig. D5.04.16

LOCATION 35.100 S 150.275 E * NEAR.. CLYDE WATERSHED ISSUED 2ND MARCH 1989 REF.–FN2912

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

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

I = INTENSITY IN MILLIMETRES PER HOUR

T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.4000	–0.5396	–0.0095	0.00544	–0.000737	0.0001127	–0.0000306
2	3.6617	–0.5366	–0.0111	0.00556	–0.000668	0.0000834	–0.0000269
5	3.9338	–0.5291	–0.0159	0.00602	–0.000345	–0.0000074	–0.0000214
10	4.0677	–0.5247	–0.0185	0.00617	–0.000168	–0.0000563	–0.0000180
20	4.2193	–0.5209	–0.0203	0.00616	–0.000050	–0.0000721	–0.0000187
50	4.3914	–0.5175	–0.0226	0.00647	0.000081	–0.0001249	–0.0000139
100	4.5065	–0.5153	–0.0241	0.00679	0.000146	–0.0001745	–0.0000074

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	94.9	122.	155.	174.	199.	233.	258.
0.100	89.4	115.	146.	164.	188.	220.	244.
0.167	73.3	94.3	121.	136.	156.	183.	204.
0.333	53.1	68.7	88.9	101.	117.	138.	154.
0.500	43.3	56.1	73.0	83.1	96.4	114.	128.
1.000	30.0	38.9	51.1	58.4	68.0	80.8	90.6
2.000	20.6	26.7	35.2	40.3	47.0	55.9	62.8
3.000	16.5	21.4	28.2	32.4	37.7	44.9	50.4
6.000	11.3	14.7	19.4	22.2	25.9	30.8	34.6
12.000	7.84	10.2	13.4	15.3	17.8	21.2	23.8
24.000	5.44	7.06	9.25	10.6	12.3	14.6	16.4
48.000	3.72	4.82	6.30	7.17	8.33	9.86	11.0
72.000	2.92	3.78	4.92	5.59	6.48	7.65	8.57

RAW DATA 38.79, 10.1[illegible], 3.77, 79.[illegible], 20.95, 7.53, 0.050, [illegible]

Fig. D5.04.17

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 35.175 S 150.600 E * NEAR.. SUSSEX INLET

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.–FN2912

(RAW DATA 47.22, 9.26, 3.13, 97.54, 21.43, 6.86, 0.020, .2HG)

PREPARED BY — HYDROLOGY BRANCH — BUREAU OF METEOROLOGY — MELBOURNE

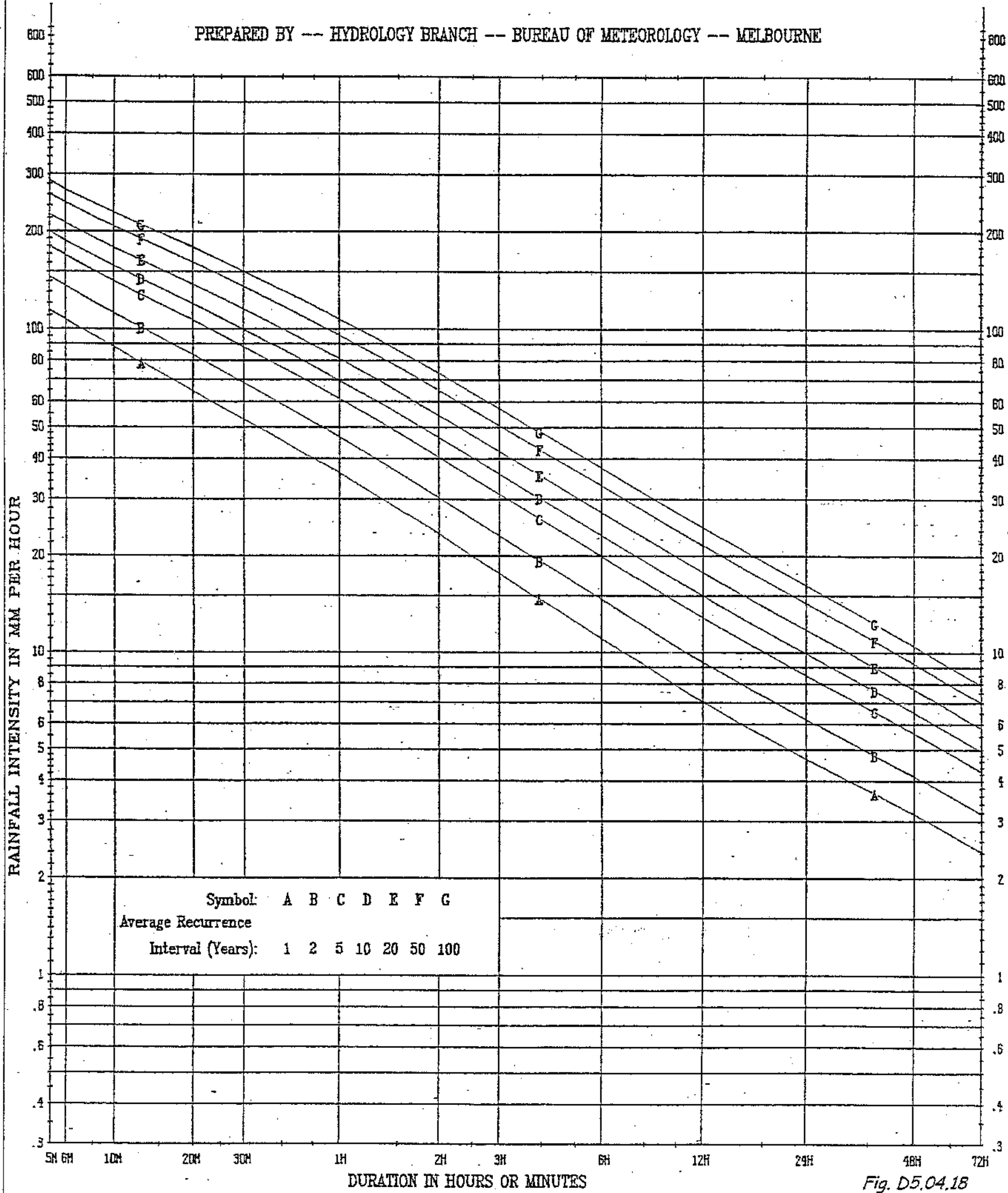


Fig. D5.04.18

LOCATION 35.175 S 150.600 E * NEAR.. SUSSEX INLET ISSUED 2ND MARCH 1989 REF.-FN2912

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

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 MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.5819	-0.5983	-0.0546	0.00555	0.003530	0.0000813	-0.0001446
2	3.8432	-0.5915	-0.0544	0.00672	0.003210	-0.0000743	-0.0001090
5	4.1147	-0.5687	-0.0550	0.00734	0.002965	-0.0002320	-0.0000759
10	4.2477	-0.5570	-0.0552	0.00760	0.002830	-0.0003008	-0.0000616
20	4.3978	-0.5474	-0.0554	0.00810	0.002687	-0.0003950	-0.0000421
50	4.5679	-0.5366	-0.0555	0.00857	0.002520	-0.0004892	-0.0000225
100	4.6806	-0.5291	-0.0556	0.00881	0.002415	-0.0005436	-0.0000101

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	114.	145.	180.	200.	226.	261.	287.
0.100	107.	136.	169.	188.	213.	246.	270.
0.167	88.0	112.	141.	157.	179.	208.	229.
0.333	64.8	83.4	106.	120.	138.	161.	179.
0.500	52.9	68.4	88.3	100.	115.	136.	151.
1.000	35.9	46.7	61.2	69.9	81.3	96.3	108.
2.000	23.2	30.3	40.3	46.4	54.3	64.9	73.0
3.000	17.7	23.1	31.1	36.0	42.2	50.7	57.2
6.000	11.0	14.5	19.8	23.1	27.2	32.9	37.3
12.000	7.04	9.30	12.6	14.9	17.7	21.5	24.4
24.000	4.69	6.18	8.47	9.90	11.7	14.2	16.1
48.000	3.15	4.13	5.61	6.51	7.67	9.22	10.4
72.000	2.39	3.14	4.25	4.91	5.78	6.92	7.69

RAW DATA 47.22, 9.26, 3.13, 97.54, 21.43, 6.88, 0.029, [illegible]

Fig. D5.04.19

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 35.325 S 150.425 E * NEAR. MILTON

* ENSURE THE COORDINATES ARE THOSE REQUIRED, SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

(RAW DATA 44.74, 9.61, 3.06, 98.14, 21.93, 6.82, 0.040, 2HG)

PREPARED BY -- HYDROLOGY BRANCH -- BUREAU OF METEOROLOGY -- MELBOURNE

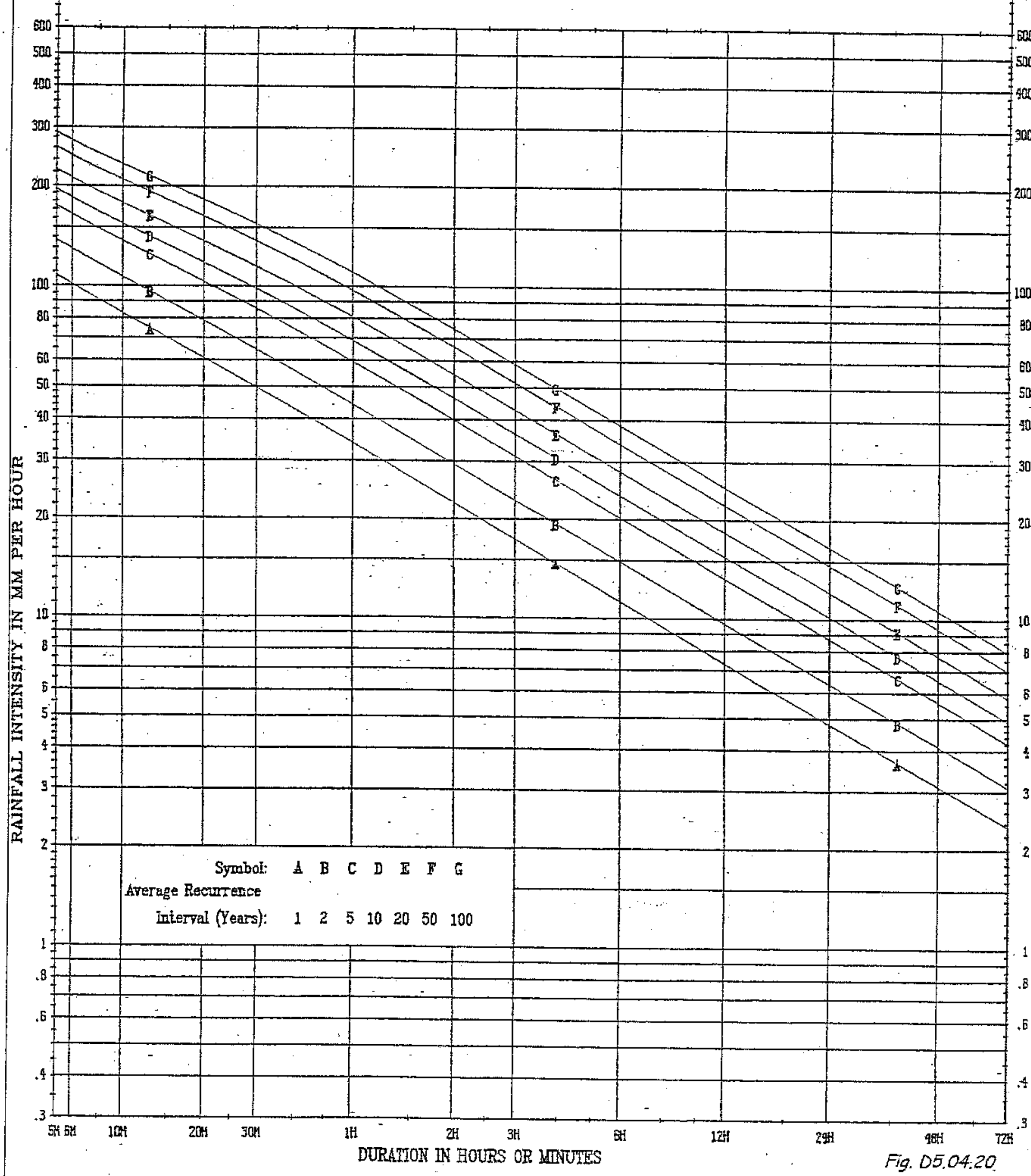


Fig. D5.04.20

LOCATION 35.325 S 150.425 E * NEAR.. MILTON

ISSUED 2ND MARCH 1989 REF.–FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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

I = INTENSITY IN MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.5208	–0.5812	–0.0366	0.00729	0.001195	–0.0001752	–0.0000386
2	3.7913	–0.5746	–0.0386	0.00746	0.001305	–0.0002069	–0.0000375
5	4.0882	–0.5576	–0.0445	0.00810	0.001614	–0.0003597	–0.0000198
10	4.2331	–0.5487	–0.0472	0.00854	0.001714	–0.0004423	–0.0000081
20	4.3957	–0.5412	–0.0499	0.00879	0.001886	–0.0005029	–0.0000039
50	4.5768	–0.5328	–0.0529	0.00923	0.002031	–0.0005889	0.0000070
100	4.7009	–0.5268	–0.0548	0.00922	0.002171	–0.0006109	0.0000071

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	108.	138.	175.	196.	225.	262.	291.
0.100	101.	129.	164.	185.	212.	247.	274.
0.167	82.8	107.	137.	155.	178.	209.	233.
0.333	60.8	78.9	103.	118.	137.	163.	182.
0.500	49.6	64.7	85.7	98.3	115.	137.	154.
1.000	33.8	44.3	59.6	68.9	81.1	97.4	110.
2.000	22.3	29.3	39.8	46.2	54.6	65.9	74.7
3.000	17.3	22.8	31.0	36.1	42.7	51.6	58.6
6.000	11.2	14.7	20.1	23.5	27.8	33.7	38.3
12.000	7.26	9.59	13.1	15.3	18.2	22.1	25.1
24.000	4.77	6.30	8.62	10.1	11.9	14.5	16.4
48.000	3.10	4.06	5.56	6.49	7.67	9.28	10.6
72.000	2.34	3.07	4.17	4.87	5.73	6.93	7.87

RAW DATA 44.74, 9.61, 3.06, 98.14, 21.93, 6.82, 0.050, [illegible]

Fig. D5.04.21

DESIGN RAINFALL INTENSITY DIAGRAM

LOCATION 35.525 S 150.375 E * NEAR BAWLEY POINT

* ENSURE THE COORDINATES ARE THOSE REQUIRED.
SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

ISSUED 2ND MARCH 1989 REF.-FN2912

(RAW DATA 42.10, 6.80, 2.73, 94.74, 19.52, 6.11, 0.050, 2HG)

PREPARED BY — HYDROLOGY BRANCH — BUREAU OF METEOROLOGY — MELBOURNE

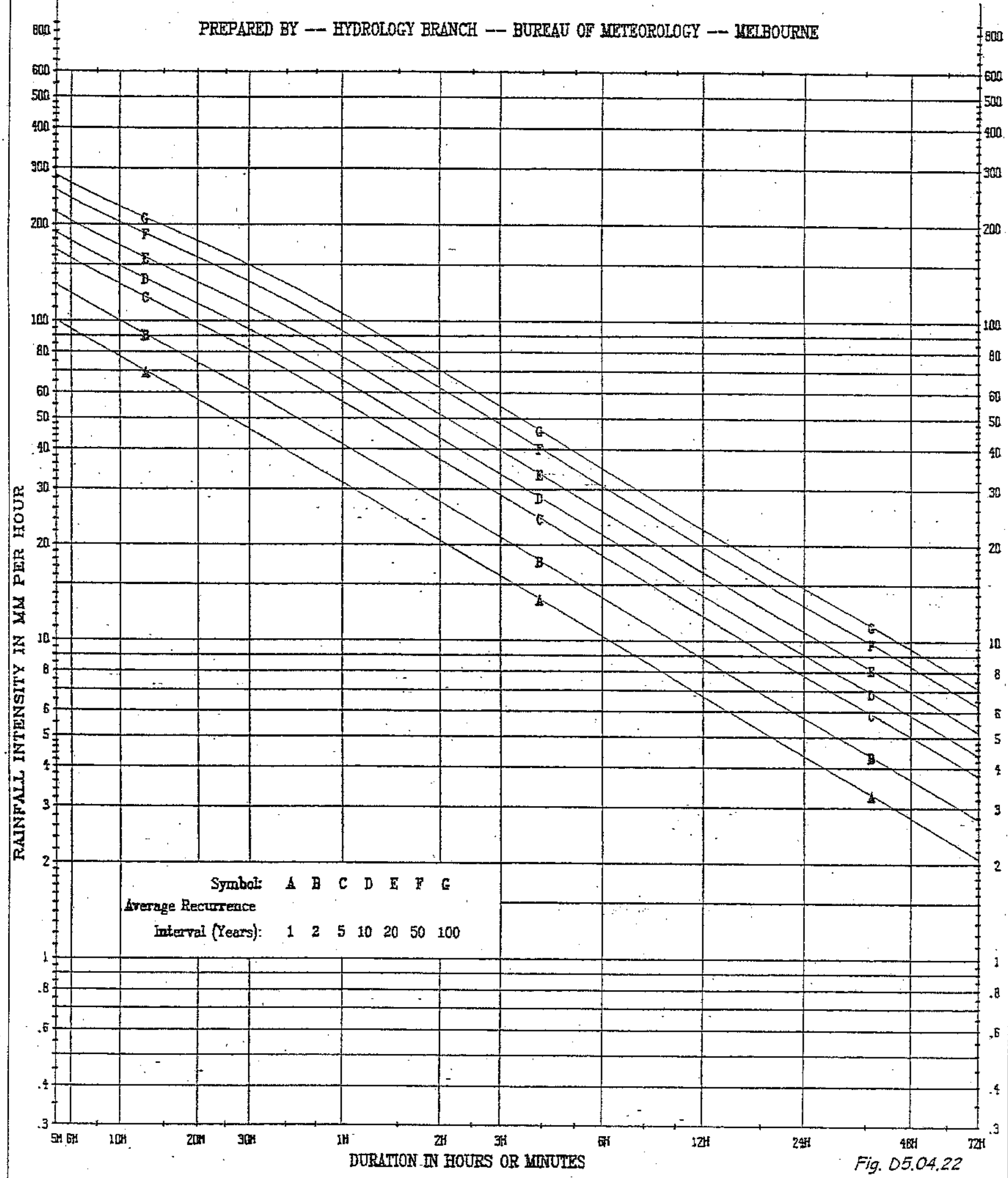


Fig. D5.04.22

LOCATION 35.525 S 150.375 E * NEAR. BAWLEY POINT

ISSUED 2^{ND} MARCH 1989 REF.-FN2912

* ENSURE THE COORDINATES ARE THOSE REQUIRED. SINCE DATA IS BASED ON THESE AND NOT THE LOCATION NAME.

LIST OF COEFFICIENTS TO EQUATIONS OF THE FORM

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

I = INTENSITY IN MILLIMETRES PER HOUR
T = TIME IN HOURS

RETURN PERIOD (YEARS)	a	b	c	d	e	f	g
1	3.4534	-0.5866	-0.0379	0.00792	0.001152	-0.0002529	-0.0000269
2	3.7276	-0.5819	-0.0406	0.00782	0.001349	-0.0002565	-0.0000304
5	4.0351	-0.5695	-0.0500	0.00803	0.002161	-0.0003387	-0.0000382
10	4.1873	-0.5634	-0.0547	0.00843	0.002515	-0.0004158	-0.0000352
20	4.3541	-0.5578	-0.0587	0.00841	0.002878	-0.0004361	-0.0000417
50	4.5434	-0.5518	-0.0631	0.00852	0.003241	-0.0004728	-0.0000456
100	4.6698	-0.5478	-0.0659	0.00873	0.003445	-0.0005166	-0.0000430

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	101.	131.	168.	190.	219.	258.	287.
0.100	94.6	122.	158.	178.	206.	242.	270.
0.167	77.7	101.	131.	149.	173.	205.	229.
0.333	57.0	74.4	98.8	114.	133.	159.	179.
0.500	46.5	60.9	81.8	94.6	111.	133.	151.
1.000	31.6	41.6	56.6	65.8	77.8	94.0	107.
2.000	20.7	27.3	37.3	43.5	51.6	62.4	71.0
3.000	16.0	21.1	28.9	33.7	39.8	48.2	54.8
6.000	10.3	13.6	18.4	21.5	25.4	30.7	34.9
12.000	6.67	8.77	11.9	13.8	16.3	19.7	22.4
24.000	4.34	5.71	7.76	9.03	10.7	12.9	14.6
48.000	2.78	3.66	5.00	5.82	6.89	8.33	9.47
72.000	2.07	2.74	3.73	4.34	5.14	6.21	7.07

Fig. D5.04.23

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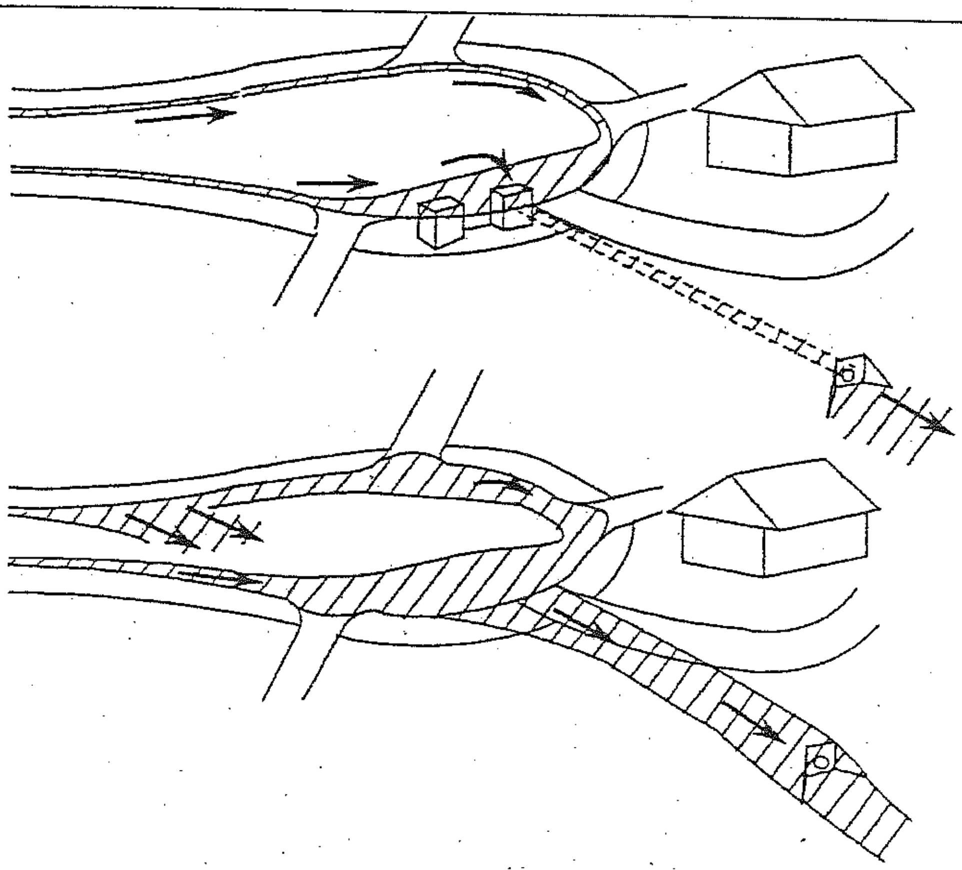
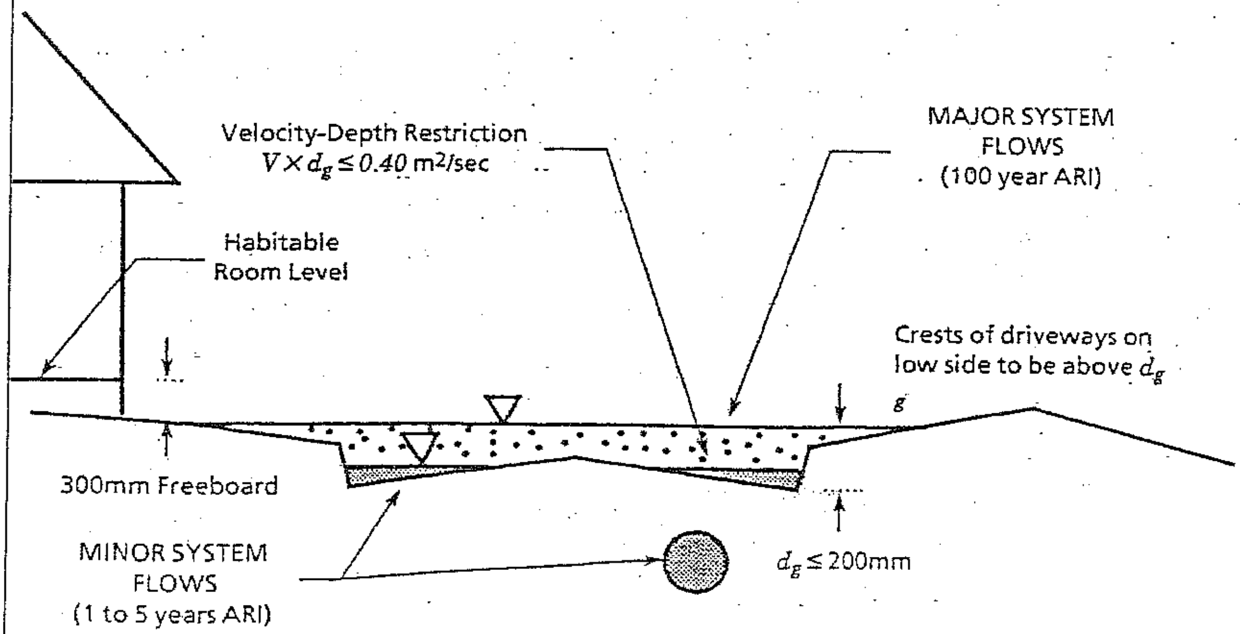


DRAINAGE SYSTEM BEHAVIOUR DURING MINOR AND MAJOR STORMS



MAJOR / MINOR DESIGN STANDARDS

THE MAJOR / MINOR CONCEPT OF DRAINAGE DESIGN

Roads and Traffic Authority N.S.W.

Fig. D5.07.3

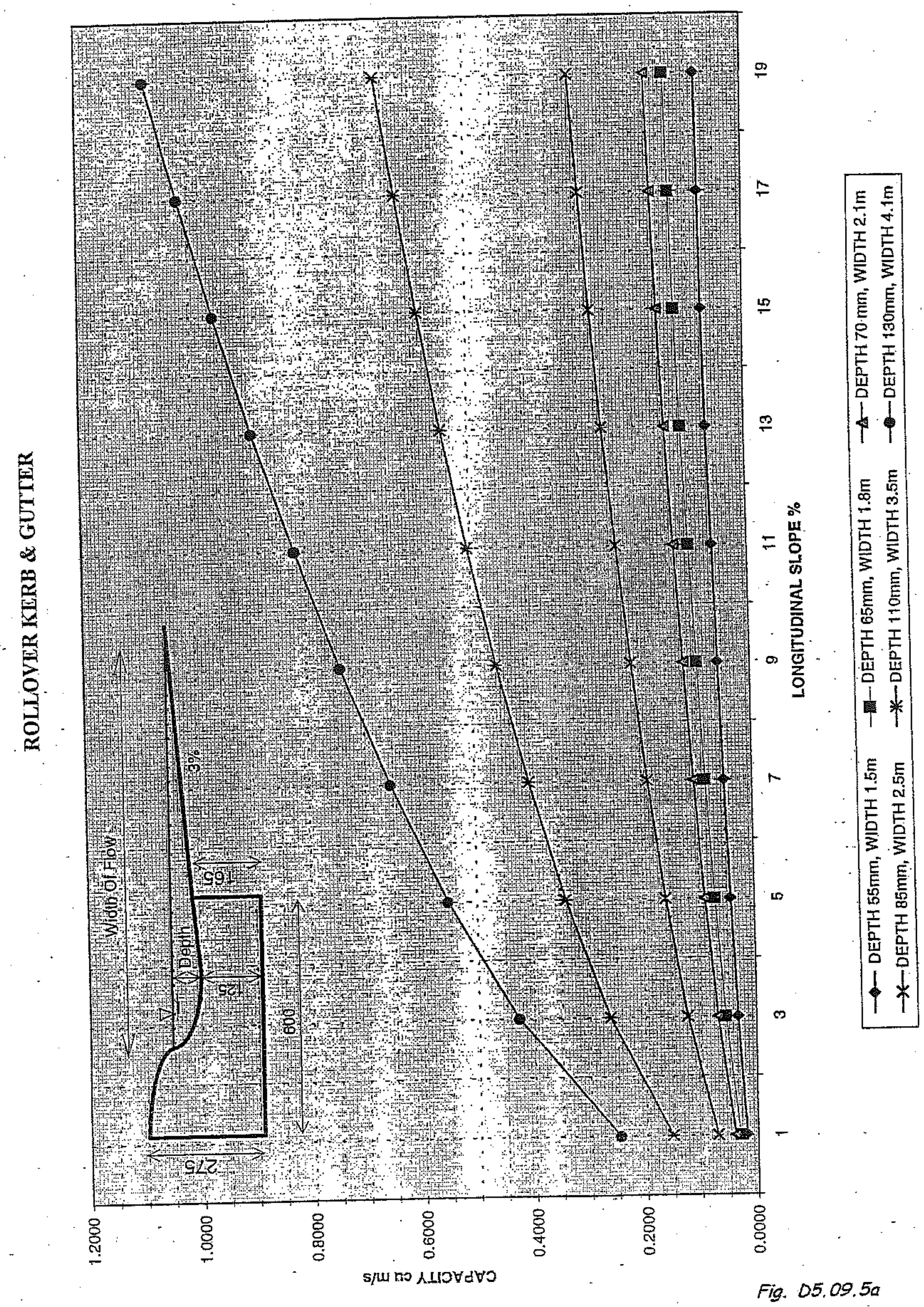

ROLLOVER KERB & GUTTER
Width Of Flow
Depth
3%
165
125
600
275
CAPACITY cu m/s
1.2000
1.0000
0.8000
0.6000
0.4000
0.2000
0.0000
LONGITUDINAL SLOPE %
1
3
5
7
9
11
13
15
17
19
DEPTH 55mm, WIDTH 1.5m
DEPTH 65mm, WIDTH 1.8m
DEPTH 70 mm, WIDTH 2.1m
DEPTH 85mm, WIDTH 2.5m
DEPTH 110mm, WIDTH 3.5m
DEPTH 130mm, WIDTH 4.1m

Fig. D5.09.5a

150mm UPRIGHT KERB AND GUTTER

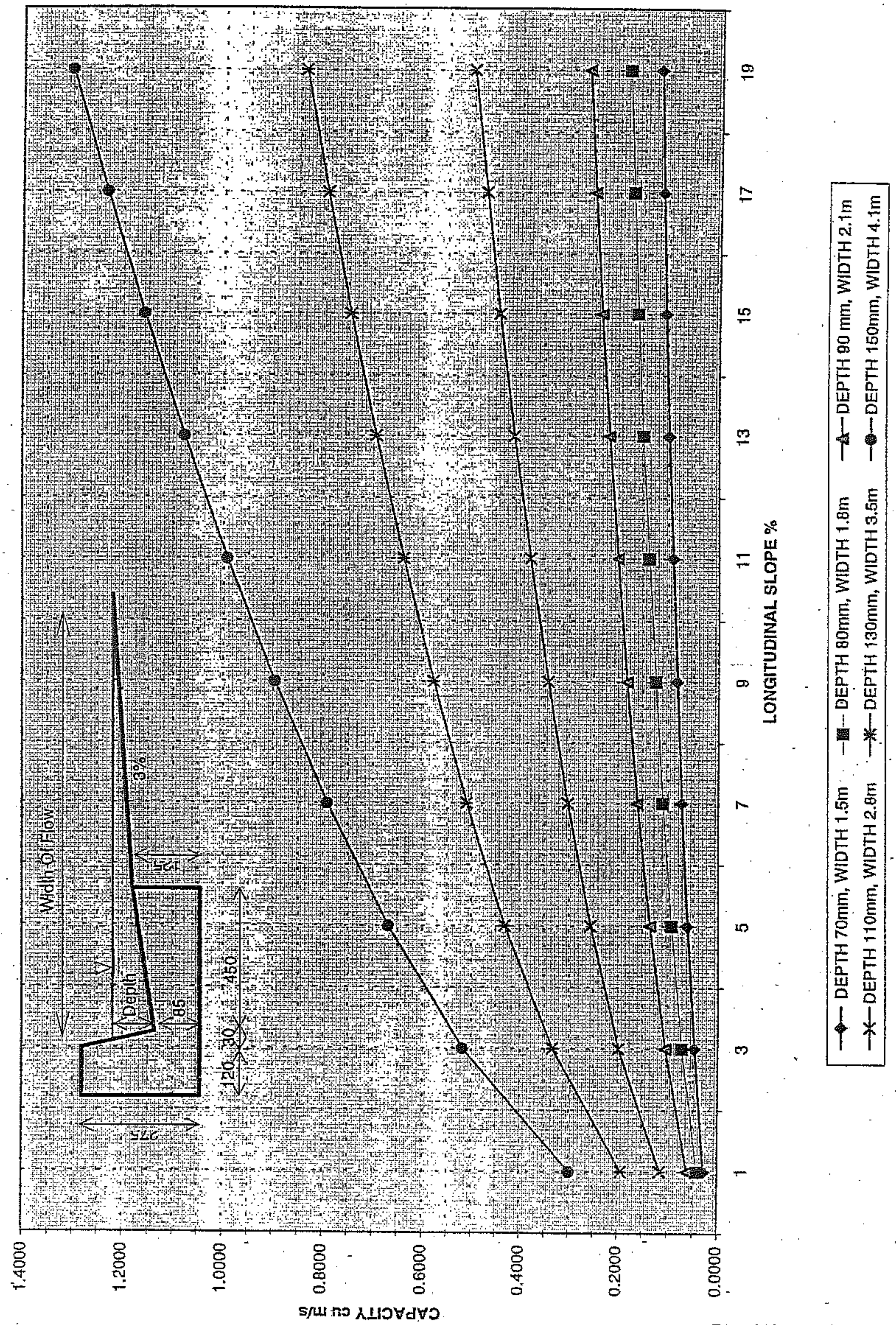


Fig. D5.09.5b

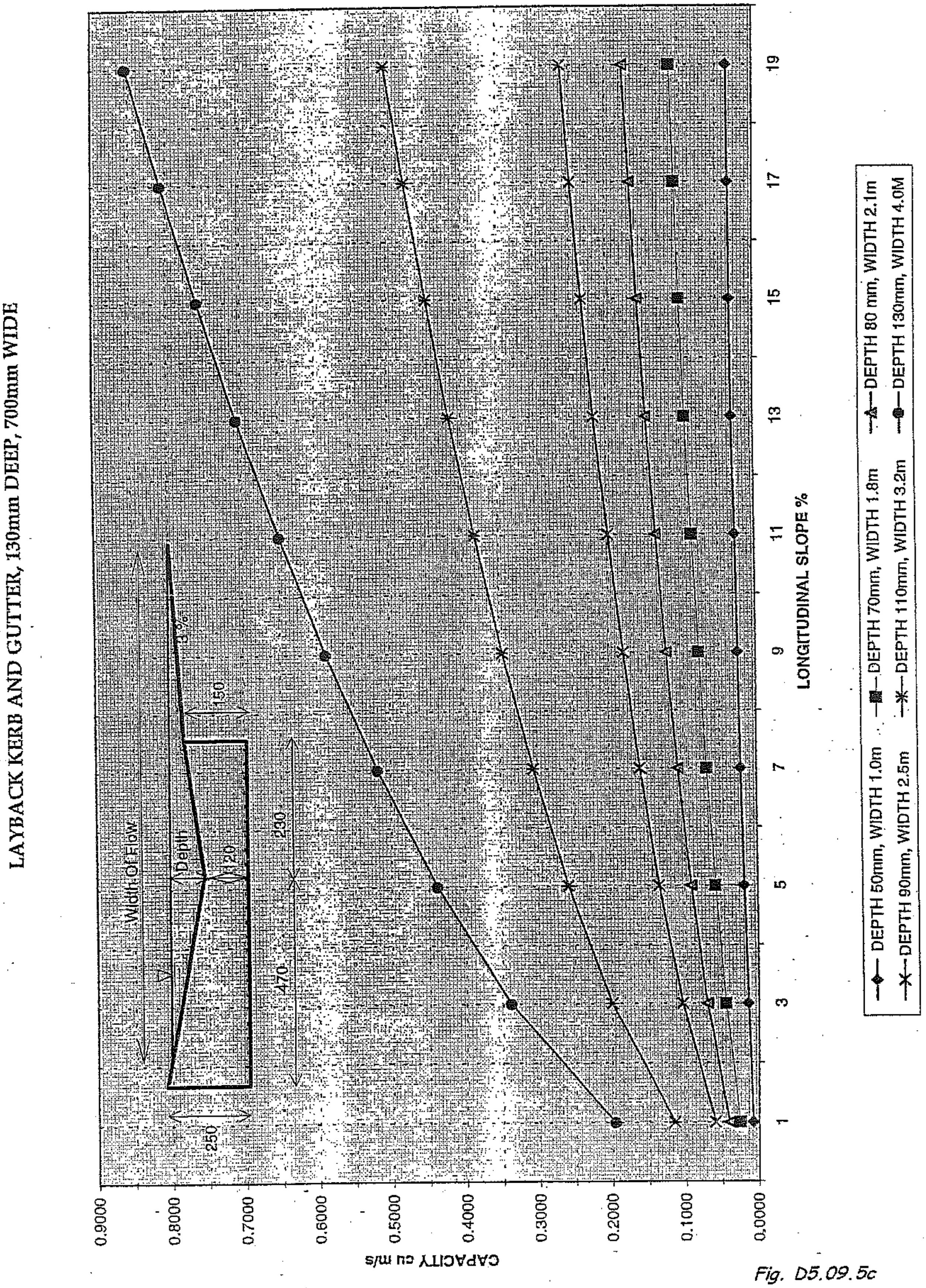
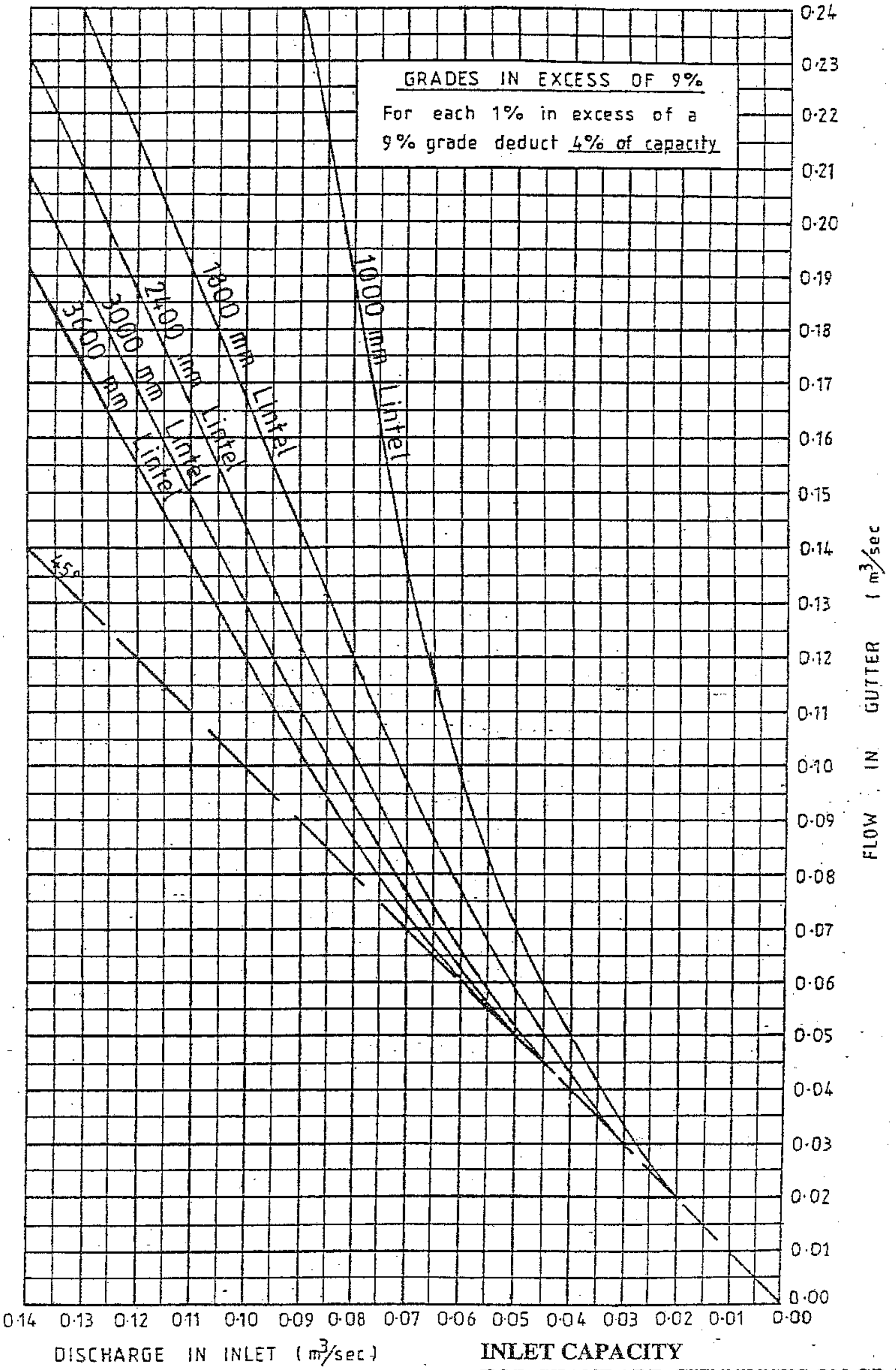

LAYBACK KERB AND GUTTER, 130mm DEEP, 700mm WIDE
CAPACITY cu m/s
0.9000
0.8000
0.7000
0.6000
0.5000
0.4000
0.3000
0.2000
0.1000
0.0000
LONGITUDINAL SLOPE %
1
3
5
7
9
11
13
15
17
19
Width Of Flow
Depth
3%
150
120
230
470
250
DEPTH 50mm, WIDTH 1.0m
DEPTH 70mm, WIDTH 1.8m
DEPTH 80 mm, WIDTH 2.1m
DEPTH 90mm, WIDTH 2.5m
DEPTH 110mm, WIDTH 3.2m
DEPTH 130mm, WIDTH 4.0M

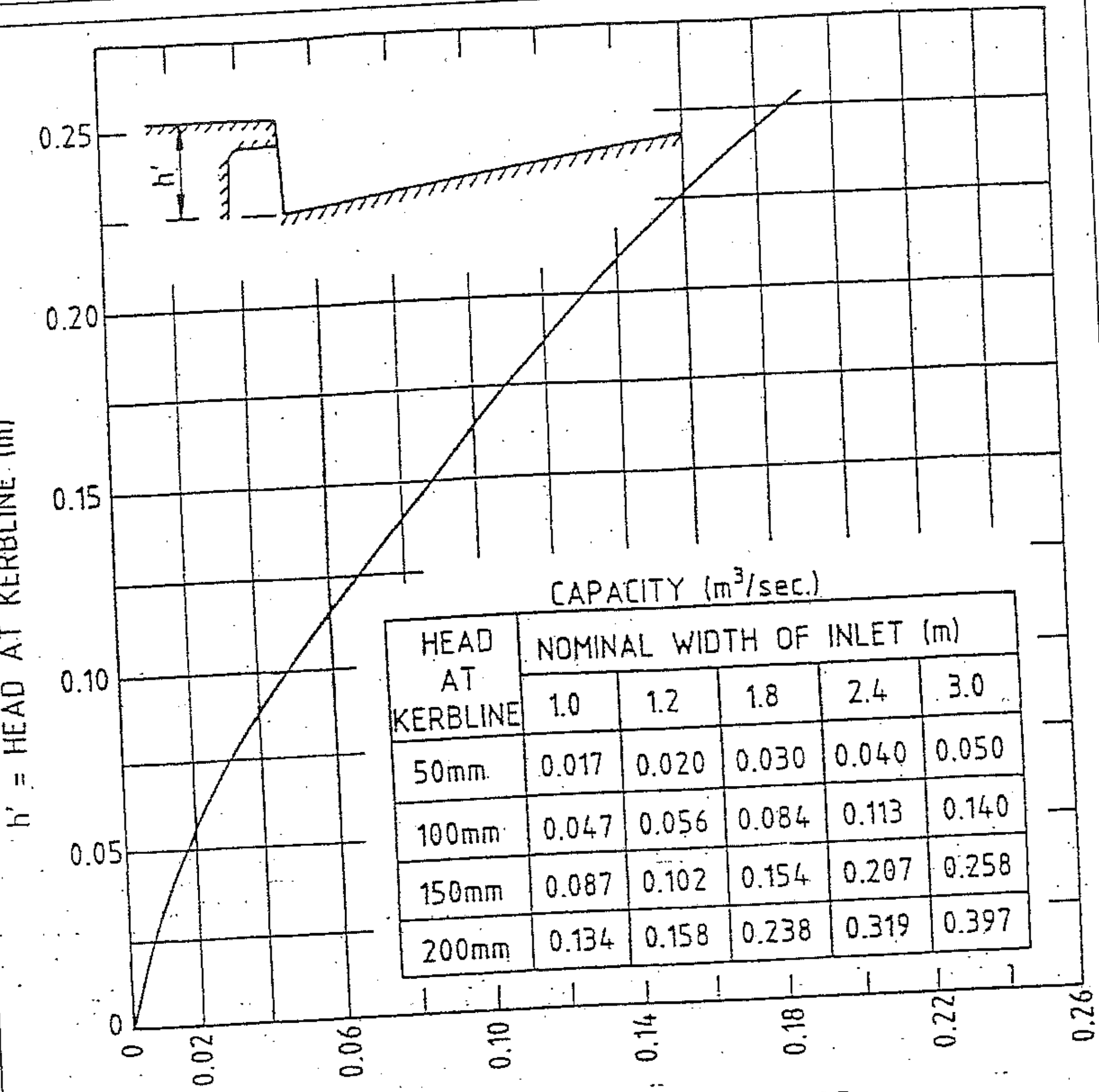
Fig. D5.09.5c



INLET CAPACITY
FOR STANDARD GULLY PITS ON GRADE

LINTEL SIZE REFERS TO OPENING SIZE
ROAD CROSS FALL 3 %

Fig. D5.09.6a



CAPACITY (m^3/sec.)

HEAD AT KERBLINE	NOMINAL WIDTH OF INLET (m)				
	1.0	1.2	1.8	2.4	3.0
50mm	0.017	0.020	0.030	0.040	0.050
100mm	0.047	0.056	0.084	0.113	0.140
150mm	0.087	0.102	0.154	0.207	0.258
200mm	0.134	0.158	0.238	0.319	0.397

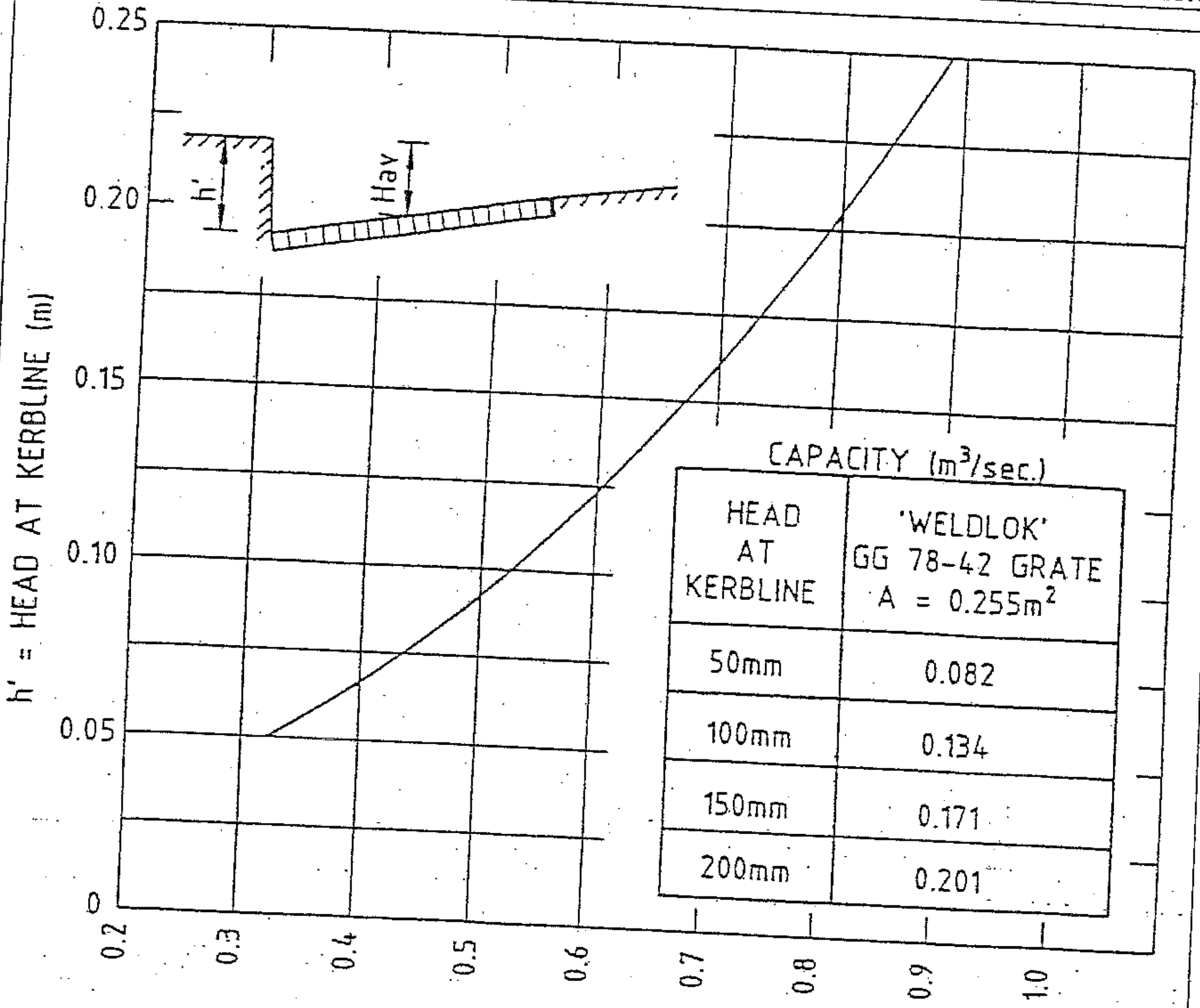
Q: INLET CAPACITY IN m^3/sec. PER METRE LENGTH OF OPENING.
(INCLUDING 10% REDUCTION FOR CLOGGING)

NOTE: $Q = 1.66 \, Lh'^{3/2} \times C$

WHERE: L = Width Of Inlet In Metres
h' = Head At Kerbline In Metres
C = Clogging Factor = 0.9 (10% Reduction For Clogging)

INLET CAPACITY CHART
KERB INLET IN SAG
(NO GRATE)

Fig. D5.09.6b



CAPACITY (m^3/sec.)

HEAD AT KERBLINE	'WELDLOK' GG 78-42 GRATE A = 0.255m^2
50mm	0.082
100mm	0.134
150mm	0.171
200mm	0.201

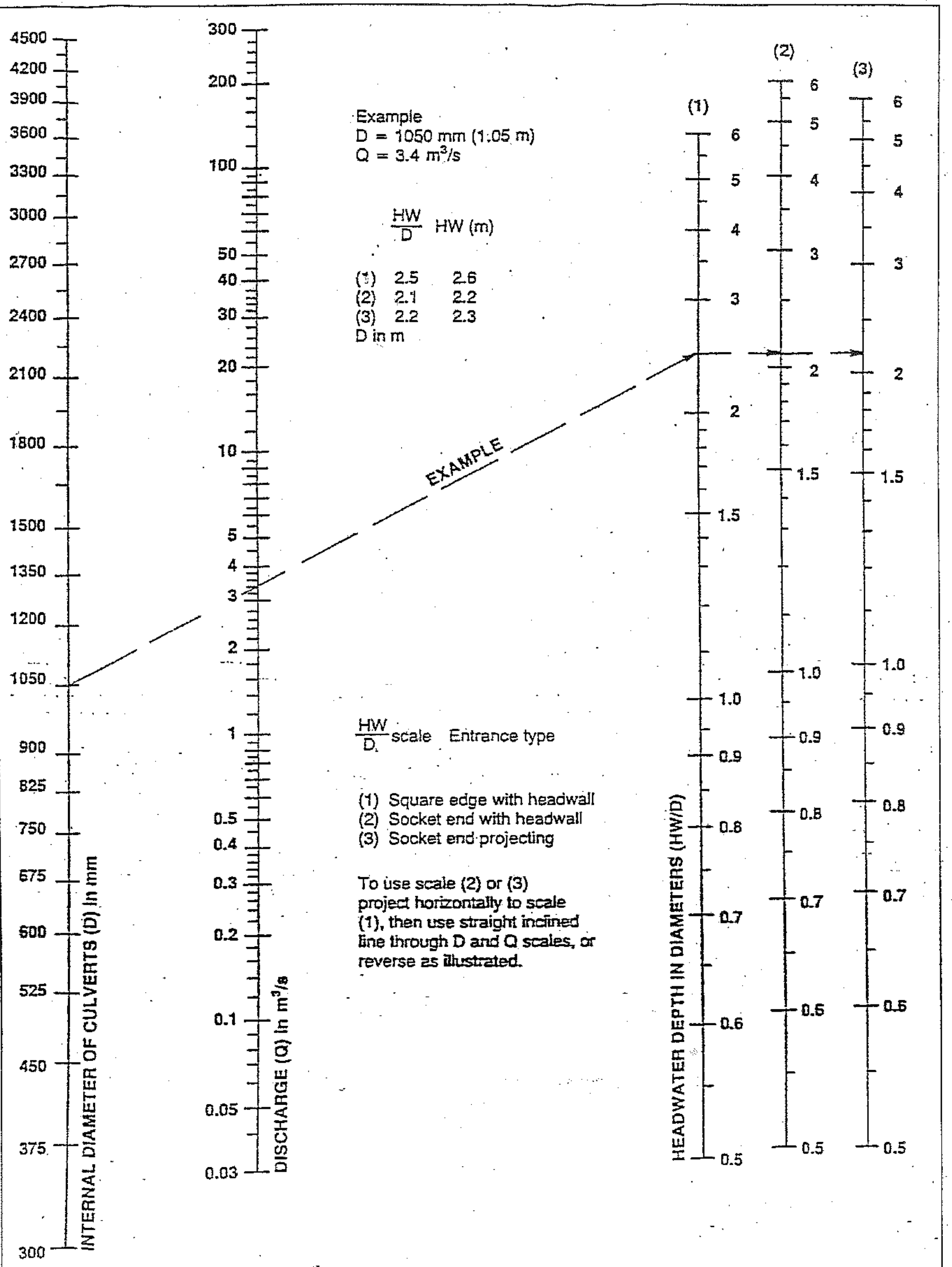
Q: INLET CAPACITY IN m^3/sec. PER SQUARE METRE OF CLEAR OPENING (INCLUDING 30% REDUCTION FOR CLOGGING)

NOTE: $Q = 0.6\ AC\sqrt{2gh}$ m^3/sec.

WHERE: A = Area Of Clear Opening In m^2
C = Clogging Factor = 0.7 (30% Reduction For Clogging)
g = Gravitational Acceleration
h' = Head At Kerbline In Metres
Hav = h' - 0.020m
h = Hav

INLET CAPACITY CHART
GRATE IN SAG

Fig. D5.09.6c



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Fig. D5.09.22.1

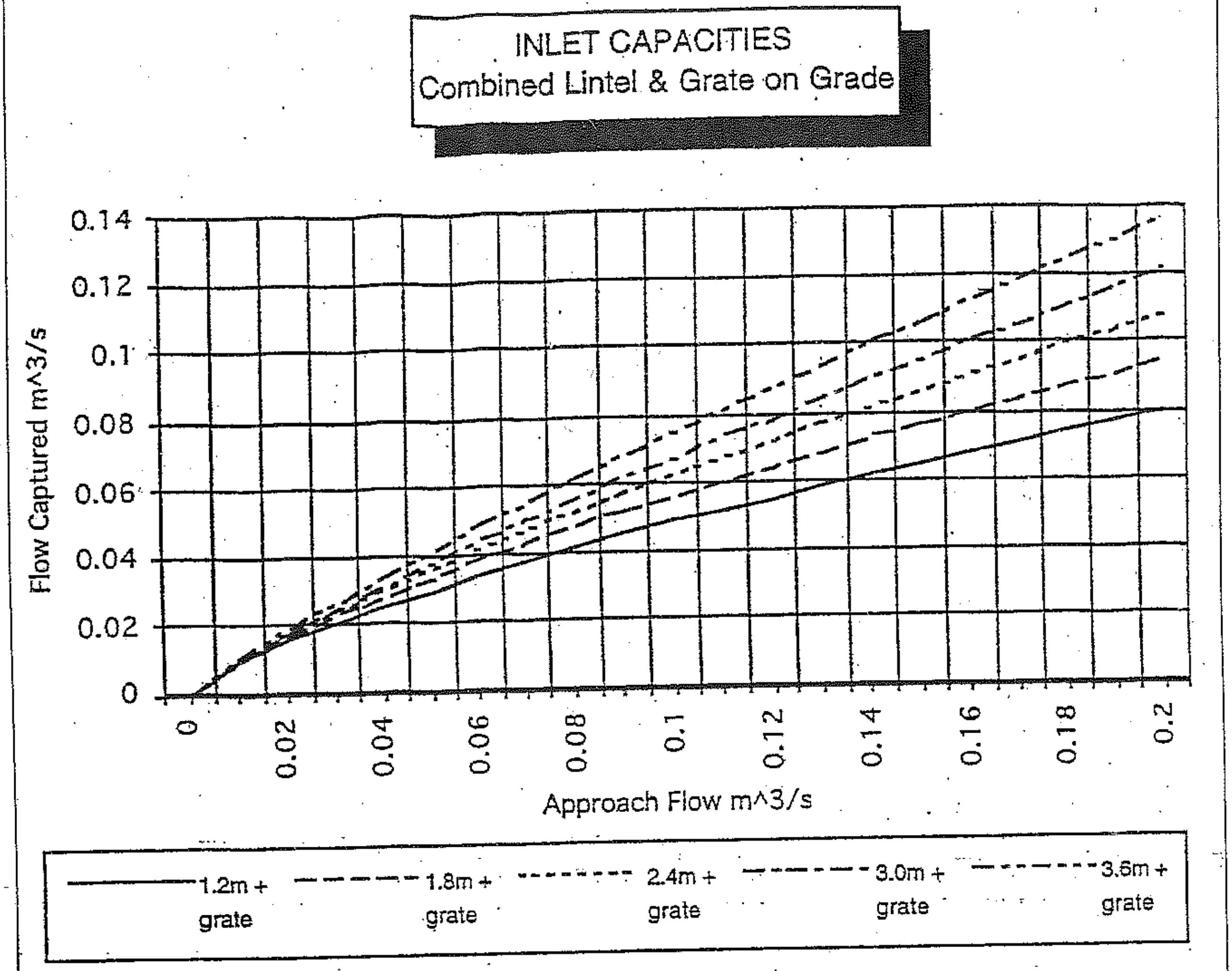
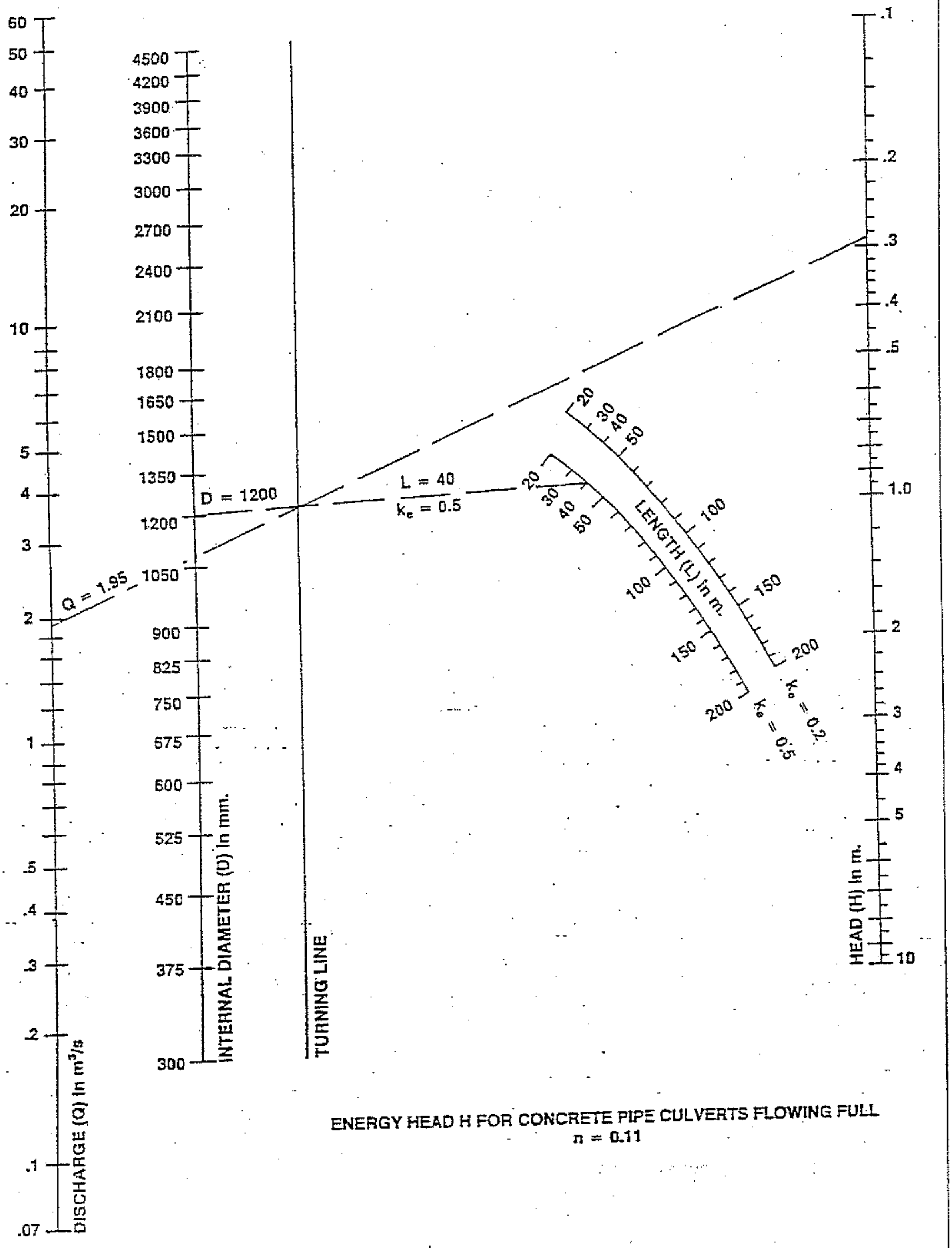

INLET CAPACITIES
Combined Lintel & Grate on Grade
Flow Captured m^3/s
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
0.16
0.18
0.2
Approach Flow m^3/s
1.2m + grate
1.8m + grate
2.4m + grate
3.0m + grate
3.6m + grate

Fig. D5.09.6d



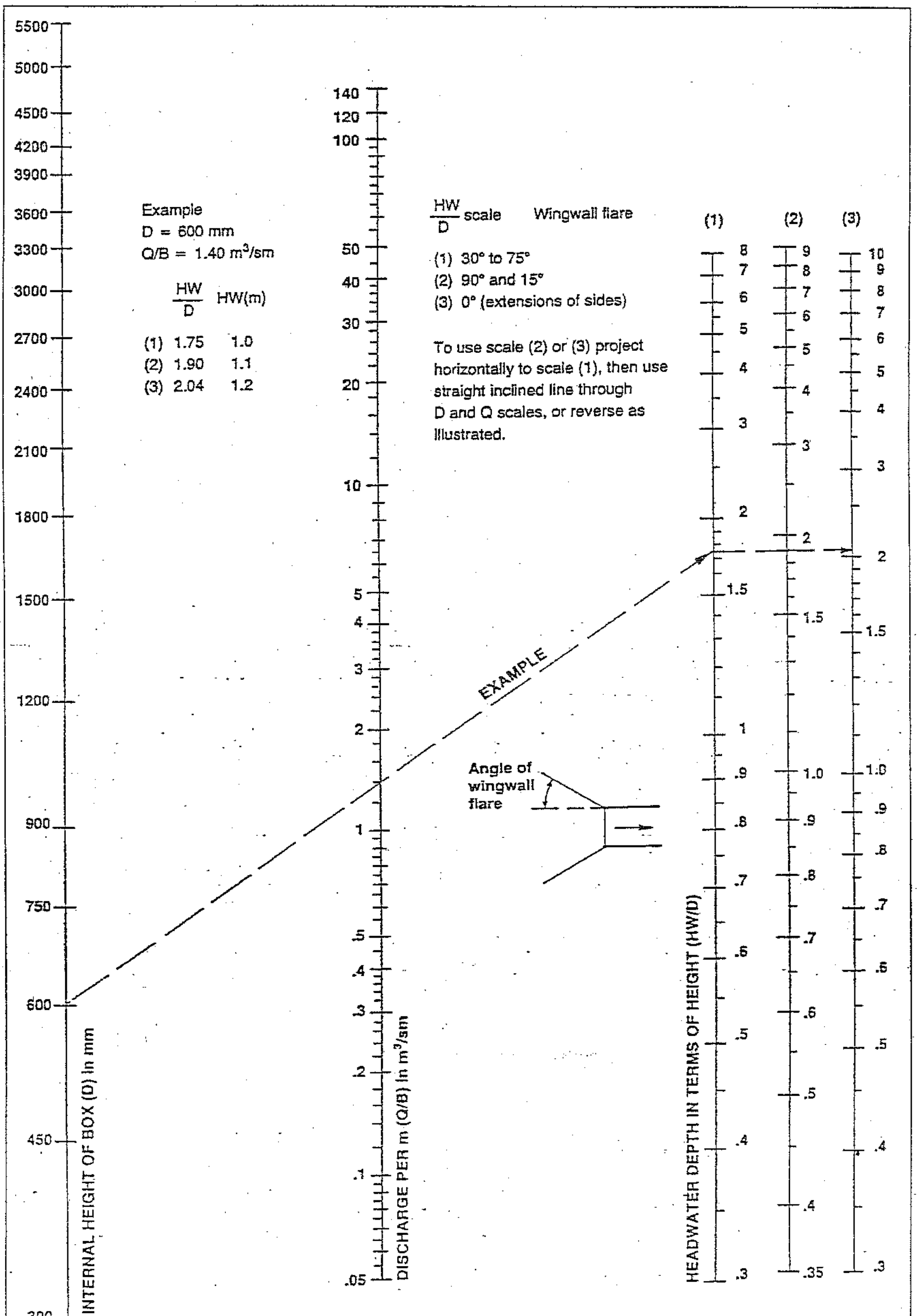
ENERGY HEAD H FOR CONCRETE PIPE CULVERTS FLOWING FULL
$n = 0.11$

NOTE:

(a) For a different value of n, use the length scale shown with an adjusted length $L_1 = L(n_1/n)^2$

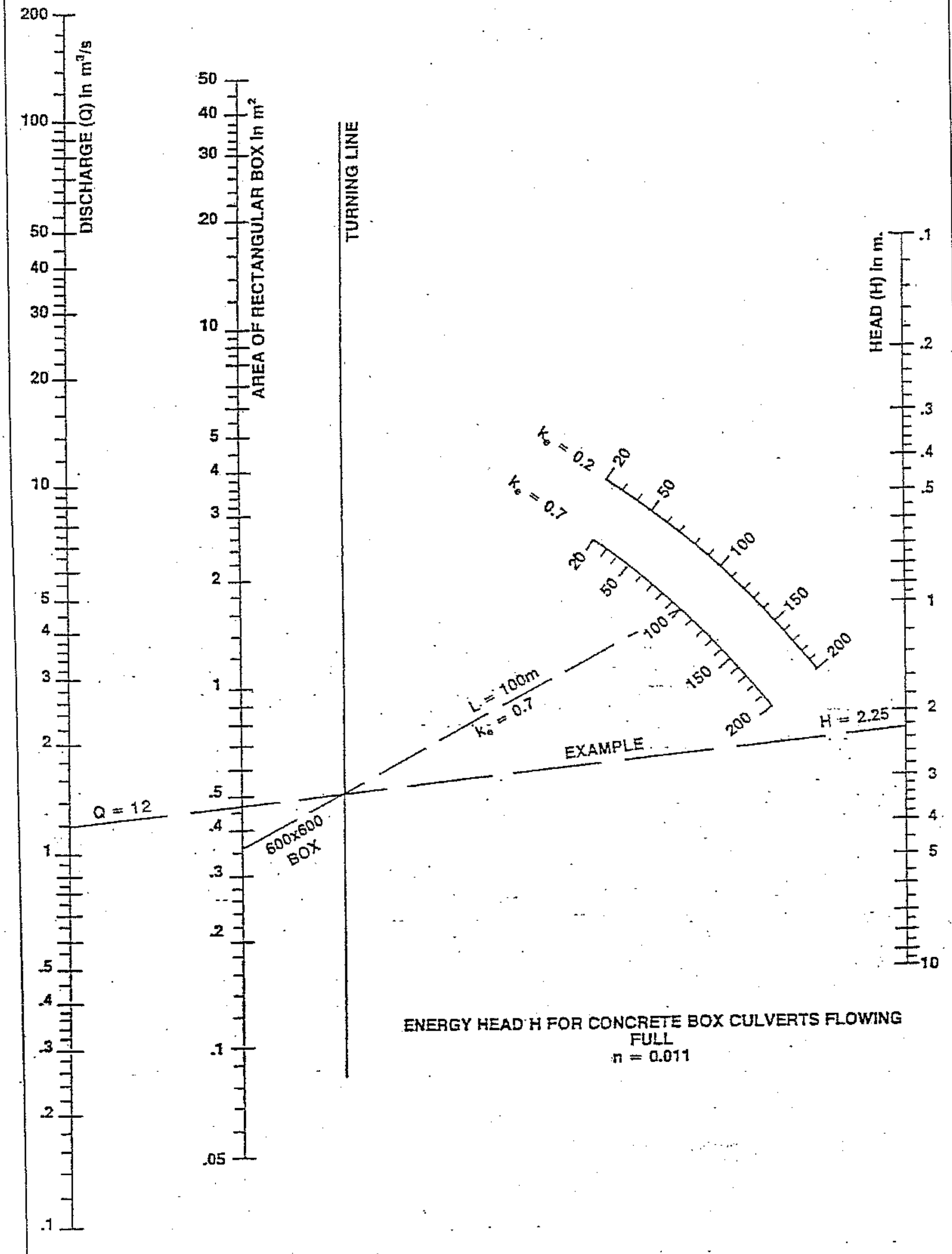
(b) For a different value of k_e connect the given length on adjacent scales by a straight line and select a point on this line spaced from the two chart scales in proportion to the k_e values.

Fig. D5.09.22.2



HEADWATER DEPTH FOR BOX CULVERTS
WITH INLET CONTROL

Fig. D5.09.22.3



ENERGY HEAD H FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.011

NOTE:
(a) For a different value of n, use the length scale shown with an adjusted length $L_1 = L(n_1/n)^2$
(b) For a different value of k_e connect the given length on adjacent scales by a straight line and select a point on this line spaced from the two chart scales in proportion to the k_e values.

Fig. D5.09.22.4

LIST OF SURFACES WITH ROUGHNESS k = 0.15mm

GOOD examples of	Rusty wrought iron Uncoated cast iron
NORMAL examples of	Galvanised iron Coated cast iron Monolith concrete construction against oiled steel form with no surface irregularities, smooth-faced precast pipe lines with no shoulders or depressions at joints Smooth-surface precast concrete pipe lines in units of 2m or over, with spigot an socket joints. Glazed vitrifie clay under 600mm dia. in 1m units or under 300mm in 0,6 units.
POOR examples of.	Wrought iron Coated steel

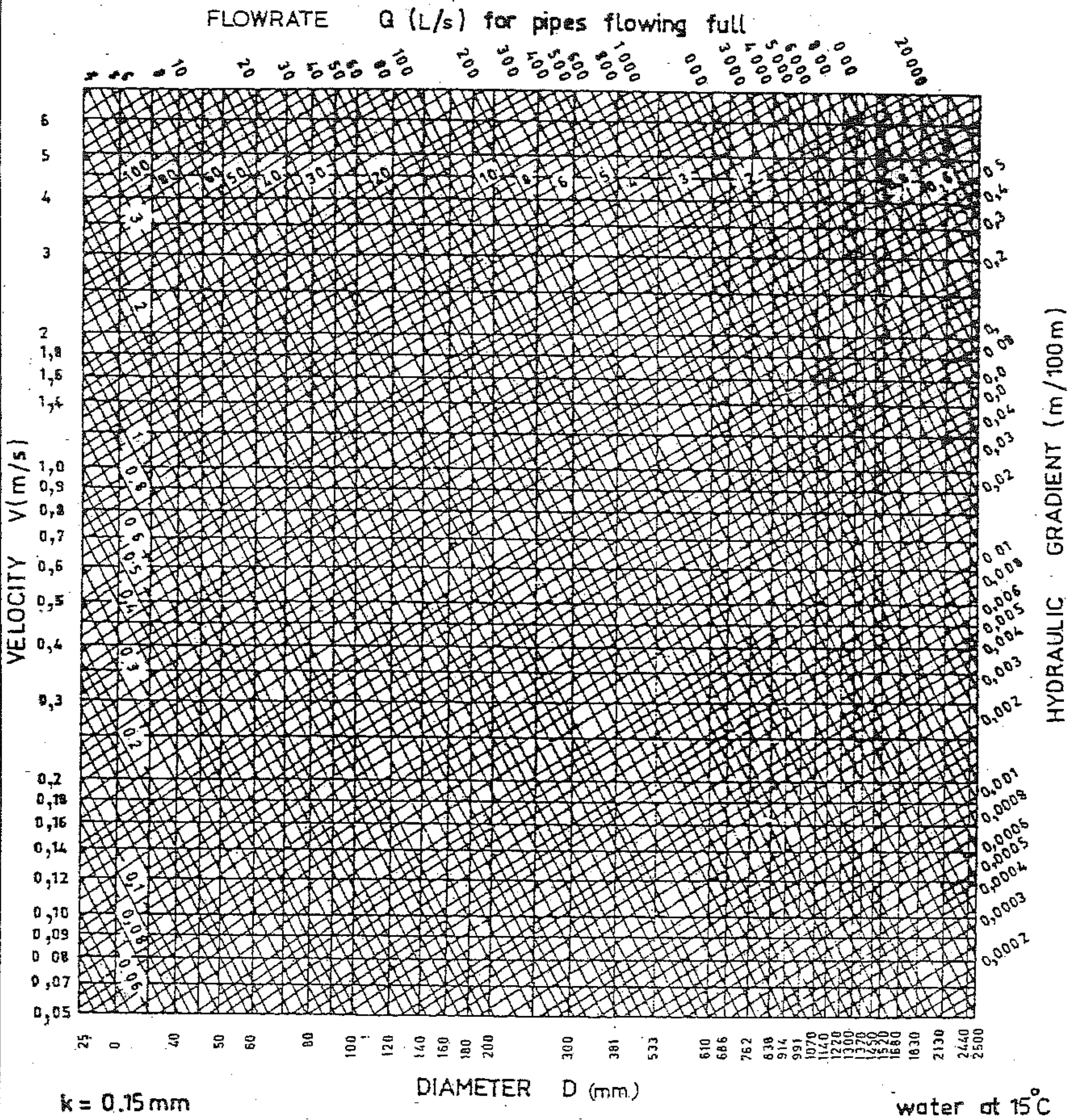


Fig. D5.10.1

Ackers Charts of Colebrook-White equation from *'Hydraulic Research Paper No. 2– Charts for the Hydraulic Design of Channels and Pipes – Hydraulics Research Station Wallingforth, Berks.'.*

UNDERGROUND STORMWATER DRAINAGE BY THE METHOD OF THE ENGINEERING EXPERIMENTAL STATION, UNIVERSITY OF MISSOURI

1. GENERAL

The design of urban underground drainage systems has been handicapped by the lack of reliable data on the losses in manholes and gully pits. Experiments have now been carried out by the University of Missouri with the object of measuring the pressure changes at manholes and other junctions and deriving coefficients to allow designs to estimate pressure changes attributable to such structures. The experiments showed that losses in pits were much greater than expected. The results allow more accurate and reliable design of underground drainage.

Methods of design of drainage systems were evolved and presented in a book titled 'Pressure Changes at Storm Drain Junctions'. The experiments were carried out only on pipes running full : hence the method does not apply where pipes do not run full.

This note attempts to set out briefly how to carry out the necessary computations to design an underground drainage system by this method. It is intended that an inexperienced person can use these notes to quickly learn how to prepare designs. However, for a deeper understanding of the method one should study the original publication which is available form the University of New South Wales Water Research Laboratory.

2. HOW TO PREPARE A DESIGN

The following notes are a precis of the design method working from the upstream end.

The calculations are made with the aim of drawing a 'pressure line' on a longitudinal section of the system, and hence find the water level in the pits.

The pressure line is the height to which water would rise if an open tube were let down into the pipe. The aim of the designer is to ensure that the water level in the pit does not rise 150mm below the top of the pit, as when this occurs the pit will either overflow or will not accept inward flow. The pressure line of the upstream pipe is in most cases equal to the water level in the junction box.

It is necessary to know the dimensions of pits, as these affect the results. Generally, the smaller the pit, the lower the losses.

The first step in a design is to prepare a preliminary layout of pipes and junctions, selecting pipe sizes which could be suitable, then to find the pressure line applicable to this layout.

If the calculated pressure line is too high, then larger pipes will be needed or the system will not work. If the pressure line is excessively low, then smaller pipes may be used for economy. If pipe sizes to be adopted will not run full at design discharge, this method will not apply.

Before any calculations for the pressure line are made it is necessary to calculate the flow (Q) in each pipe. It is very helpful to show these values on a longitudinal section along the system. It is also very helpful to show all pertinent information on the longitudinal section, such as slope, class of pipes, velocity of design flow, and pressure line.

Friction Loss in the pipe is then calculated using the Colebrook White Charts. The discharge is known, and a trial pipe size is selected which gives a velocity between 0.6m/s (siltation) and 7m/s (scouring). For this velocity the frictional head loss in the pipe is determined.

Pit Head Loss for the top pit is then determined from the appropriate chart.

The Total Head loss for the section is then compared with the maximum acceptable water surface levels in the upstream and downstream pits (normally surface level – 0.150m).

Fig. D5.10.3.1

Then Review Design – Ideally, the upstream level minus head losses should equal downstream water level. This would give the most economical design for the pipe reach under consideration.

If the losses exceed the difference in water level, then the pipe can only carry the design flow if the downstream water level is lowered to (upstream level – losses). The result is a lower available head for the next section, or, in an extreme case, a physically difficult/steep/expensive construction. If the water level is below the pipe obvert the pipe will not flow full and the charts are not valid.

If the loses are less then the difference in levels, the pipe is uneconomical in size and unless other factors are involved, should be reduced.

Successive Trials may be necessary to achieve the best compromise.

The nomenclature used is given in each case on the accompanying graph; however, a chart covering all nomenclature used in this method is also included (Chart 1 (Fig. D5.10.3.7)).

The method does not cover all situations that arise in underground design. Drop pits are not covered explicitly nor cases where lateral pipes are at angles other than 0° and 90° to the outfall. In this latter case, however, a rough guide to pressure changes could be taken by interpolating between in-line pressure changes and 90° lateral pressure changes. Generally in cases where the charts do not apply, the designer will have to use other theoretical or empirical methods combined with some engineering judgement.

3. USEFUL INFORMATION

The following are ways of keeping pressure losses to a minimum and should be used in preparing the pipe layout and designing manholes.

1. Smaller manholes are more efficient. The pressure loss decreases as the manhole size decreases relative to the outfall pipe size.

2. Deflecting devices in manholes are very effective in reducing pressure changes involving flow from a lateral.

3. Laterals should not be opposite one another when the velocities are greatly different. In such cases it is better to offset them.

4. For an in-line upstream and outfall main, rounding of the invert, a frequently used practice, is ineffective.

5. For an in-line main with a lateral, rounding of the outfall entrance is effective, particularly if large flows are carried by the lateral.

6. When a small pipe with high velocity flow joins a straight through drainage line, it disrupts the flow in the pit, requiring higher pressure in the pit, i.e. higher pressure losses.

4. GENERAL INSTRUCTIONS FOR USE OF DESIGN CHARTS

Several operations are common to use of the design charts for various types of junctions. Instructions for performing these recurring procedures are consolidated in the following General Instructions. In the detailed instructions for use of the individual charts, references to these General Instructions are made by number (Gen. Instr. 1. etc).

In using the charts, It is possible to work from either end, i.e., upstream or downstream. The following General Instructions are based on working from DOWNSTREAM end.

The general instructions are as follows:

Fig. D5.10.3.2

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or inlet centre. The elevation is obtained by adding the pipe friction loss to the elevation of the pressure line at the preceding structure downstream.

2. Calculate the mean velocity head of the flow in the outfall pipe

   $$\frac{V_0^2}{2g}$$

3. Calculate the required flow rate and size ratios. Examples:

   Q_U/Q_O, Q_L/Q_O, Q_G/Q_O etc.

   D_U/D_O, D_L/D_O, B/D_O, etc.

4. Estimate the dept of water, d, in a manhole with flow into the manhole from a top inlet, either alone or combining with flow from an upstream pipe.

   d = total depth of water, in metres
   = (outfall pressure line elevation minus inlet bottom elevation) + (K) $V_O^2/2g$

   K = the pressure change coefficient for the inlet waster depth. (This is estimated as detailed for each type of manhole. Such estimates are not necessary for manholes with in-line or offset opposed laterals.)

5. Use the coefficients K from the charts for manholes with square-edged entrance to the outfall pipe (entrance flush with box side, with square edges)

6. Use reduced coefficients K, where applicable, for a rounded entrance to the outfall pipe (rounded on ¼ circle arc of approximate radius $^1/_8$ D_O) or for an entrance formed by the socket end of a standard tongue-and-groove concrete pipe.

   Chart 2 – insignificant effect, make no reduction

   Chart 3 – read directly from chart

   Chart 4 – reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5

   Chart 5 – reduce K_U and K_1

   Chart 6 – insignificant effect, make no reduction

   Chart 7 – insignificant effect, make no reduction

   Chart 8 – see specific instructions for each case

7. Calculate pressure change.

   To calculate the change of pressure at a manhole, working upstream form the outfall pipe to an upstream pipe, the design chart applying to the type of junction involved is selected. The pressure change coefficient for a specific upstream pipe is read from the chart for the particular flow rate and size ratios already calculated. The pressure change is calculated from:

   $$h = K \times \frac{Vo^2}{2g}$$

   The coefficient is a dimensionless number, and therefore the change of pressure will be in metres.

8. Apply the pressure change.

   The pressure change, in metres, for each upstream pipe is added to the outfall pipe pressure line elevation at the branch point to obtain the elevation of each pressure line for further calculations upstream along that pipe. In some cases the upstream pressure line at the branch point will be at a lower elevation than the downstream pressure line. Where this less common-

Fig. D5.10.3.3

situation may occur with a particular type of junction, it is mentioned in the instructions for use of the specific chart.

9. Determine the elevation of the water surface.

The elevation of the water surface in a manhole (with or without inlet flow) receiving flow from a pipe or pipes will correspond to that of the upstream in-line pipe pressure line. At a junction with offset opposed laterals, the water surface will correspond to the elevation of the far lateral pipe pressure line. At a junction with in-line opposed laterals, the water surface will correspond to the elevation of the pressure line of the higher-velocity lateral pipe.

Verify that the water surface is above the crown elevation of all pipe connections to the structures that are being analysed. Small pipes, such as laterals to inlets, which carry a small portion of the total flow, may reasonably be considered to effect a manhole in the same way as inlet flow from the ground surface.

Fig. D5.10.3.4

The various cases are summarised below:

Case	Chart
Catch basin with inlet flow only	2
Flow straight through any manhole	3
Rectangular manhole, through pipe and Inlet flow	4
Rectangular Manhole with in-line upstream main and 90° lateral pipe (with or without inlet flow)	5
Rectangular manhole with in-line opposed lateral pipes each at 90° to outfall (with or without inlet flow).	6
Rectangular manhole with offset opposed lateral pipes each at 90° to outfall (with or without inlet flow)	7
Square manhole at 90° deflection	8
Round manhole at 90° deflection	8
Square manhole on through pipeline at junction of a 90° lateral pipe (large size laterals $^{D}L/^{D}O > 0.6$)	8 and 9
Round manhole on through pipeline at junction of a 90° lateral pipe (large size lateral $^{D}L/^{D}O > 0.6$)	9
Square or round manhole on through pipeline at junction of a 90° lateral pipe (smaller size laterals: $^{D}L/^{D}O < 0.6$)	10

The cases where the lateral makes an angle other than 90° with the outfall is not covered by the method. However, it appears that an approximation of the pressure changes could be found by interpolating between the results for straight flow through a junction (chart 3) and 90° deflection (chart 8) as follows:

$K_u = \sin \Theta$ (K for 90° bend − K for straight flow)
+ K for straight flow

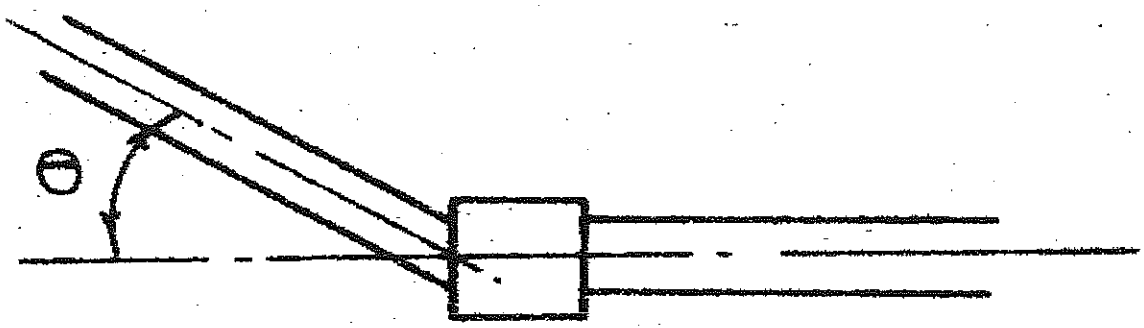
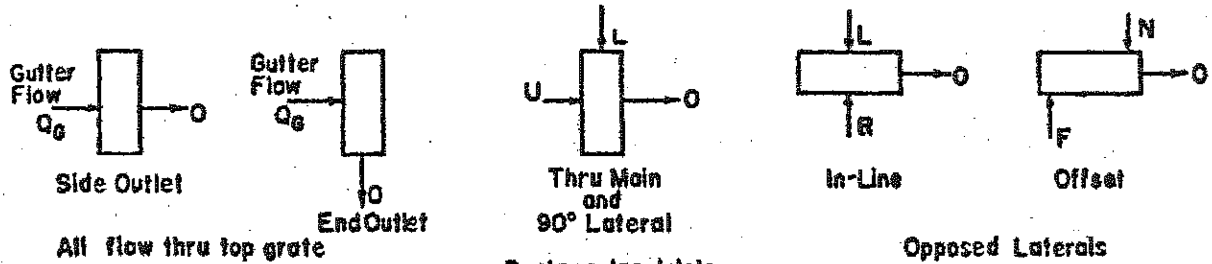
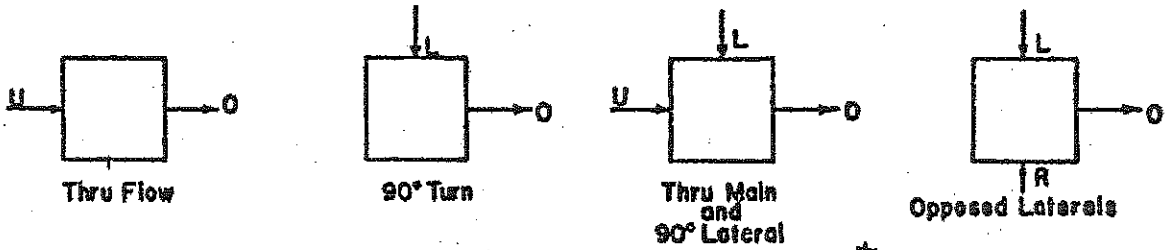


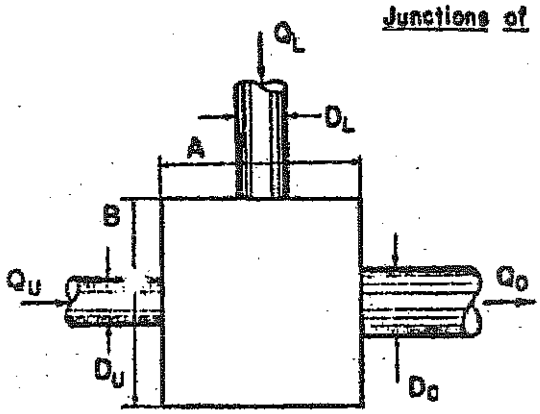
Fig. D5.10.3.5



Rectangular Inlets



Junctions of Any Shape



Junction Dimensions

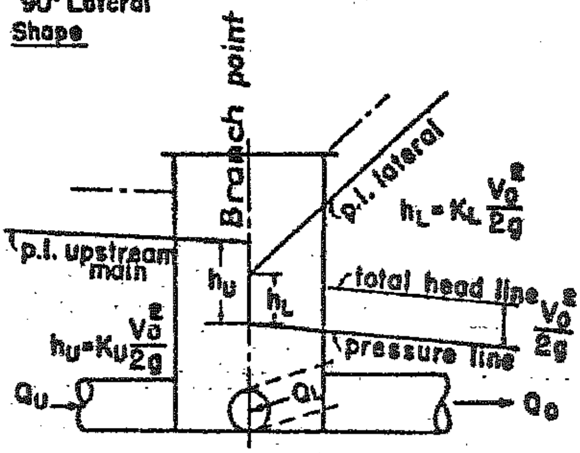


Diagram of Pressure Line Elevations at Junction of A Lateral with A Thru Main

Nomenclature

- Q rate of flow
- D diameter of pipe
- A dimension of junction in direction of outfall pipe
- B dimension of junction at right angles to outfall pipe
- d depth of water in inlet
- S slope of pipe
- S_f friction slope
- Q_G flow into inlet thru top grate
- D_O Q_O dia. and flow in outfall
- D_U Q_U dia. and flow in upstream main
- D_L Q_L dia. and flow in left lateral
- D_R Q_R dia. and flow in right lateral
- D_N Q_N dia. and flow in near lateral } of opposed laterals; designation with reference
- D_F Q_F dia. and flow in far lateral } to position relative to outfall end of junction.
- $D_{h.v.}$ $Q_{h.v.}$ dia. and flow in lateral with higher-velocity flow } for in-line opposed laterals only
- $D_{l.v.}$ $Q_{l.v.}$ dia. and flow in lateral with lower-velocity flow }

Pressure change coefficients for inlet water depth and an upstream pipe pressure relative to the outfall pipe pressure.

- K_G water depth with all flow thru grate
- K_U upstream main pressure
- K_R or K_L lateral pipe pressure
- K_N near lateral pipe pressure } offset opposed
- K_F far lateral pipe pressure } laterals
- $\overline{K_U}$, $\overline{K_L}$ pressure coefficient at $Q_L = Q_O$
- M_U, M_L multipliers for $\overline{K_U}$ or $\overline{K_L}$ to obtain K_U or K_L

Chart 1. Junction types and nomenclature.

Fig. D5.10.3.6

RECTANGULAR INLET WITH GRATE FLOW ONLY

From the attached graph the elevation of the water surface in a pit may be determined, where all inflow enters the top of the pit.

The graph gives values only for the type of box and outlet shown on the graph. Accurate results should not be expected if the pit is a different shape.

It is assumed that the outfall pressure line level has been calculated previously.

1. Estimate K_G as follows:

   a) Pressure line level to bottom not over 2 pipe diameters) 6.0 for end outlet) 4.0 for side outlet

   b) For higher pressure line levels) 4.0 for end outlet) 3.0 for side outlet

2. Use this value of K_G to estimate d

   $d = K_G\ (est) \times V_O{}^2/2g$ + outlet pressure line RL – invert level, outlet

3. Calculate d/Do and use this value to find K_G from the chart.

4. Calculate $h_g = K_G \times V_O{}^2/2g$ where V_O is the outlet velocity, calculated from Q/Area of pipe.

5. Add h_g to the outfall pressure line level to obtain the depth of water in the pit (d).

NOTES:

The result given for d will be approximate only. For a more accurate value of d, steps 3, 4 and 5 should be recalculated using the value between that estimated and that obtained from step 5.

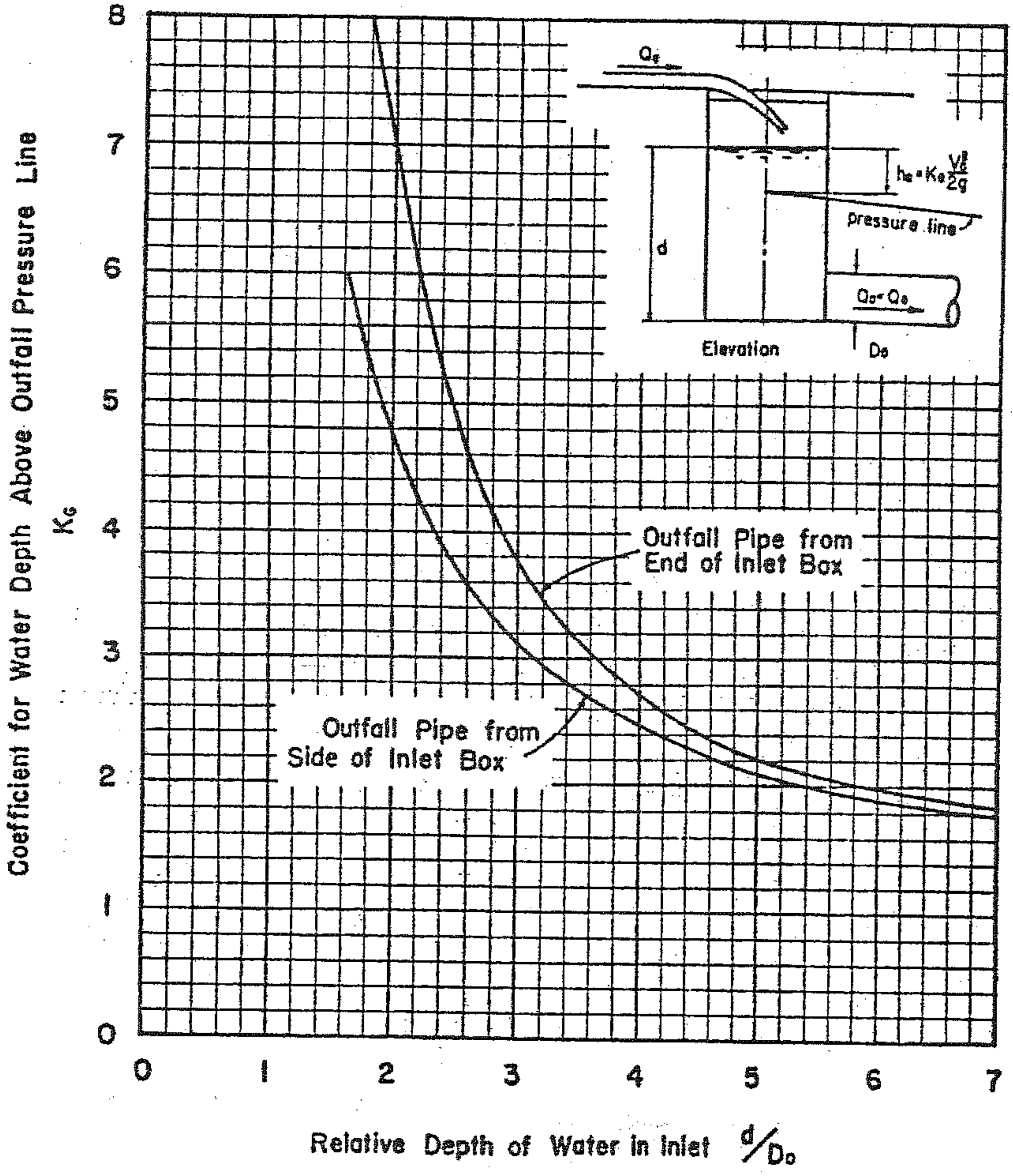
Fig. D5.10.3.7

Case

Chart

Catch basin with inlet flow only

2



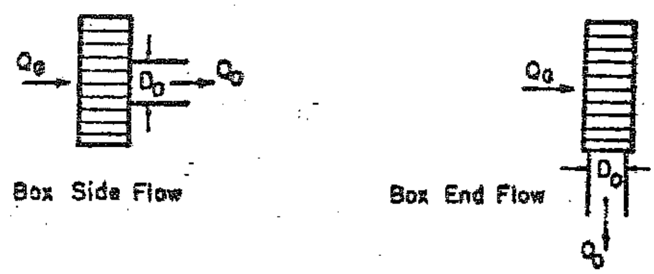


Chart 2. Rectangular inlet with grate flow only.

Fig. D5.10.3.8

Chart 3

STRAIGHT FLOW THROUGH ANY JUNCTION

Pipe centrelines must be parallel and not offset more than would permit the area of the smaller pipe to fall entirely within the projection of the larger.

It is assumed that the outfall pressure line level has been calculated previously.

1. Calculate D_U/D_O and A/D_U

2. Find K_U from chart 3

3. Calculate pressure change (h_u)

   $h_u = K_u \times V_O{}^2/2g$ where V_O is the velocity in the outfall pipe.

NOTES:

1. h_u will be lower if the entrance is rounded, and the graph shows a dotted line to cover this case.

2. Frequently, h_u is negative. In this case, the pressure line level upstream is lower than the pressure line level downstream and h_u should be subtracted.

Fig. D5.10.3.9

Case	Chart
Flow straight through any manhole	3

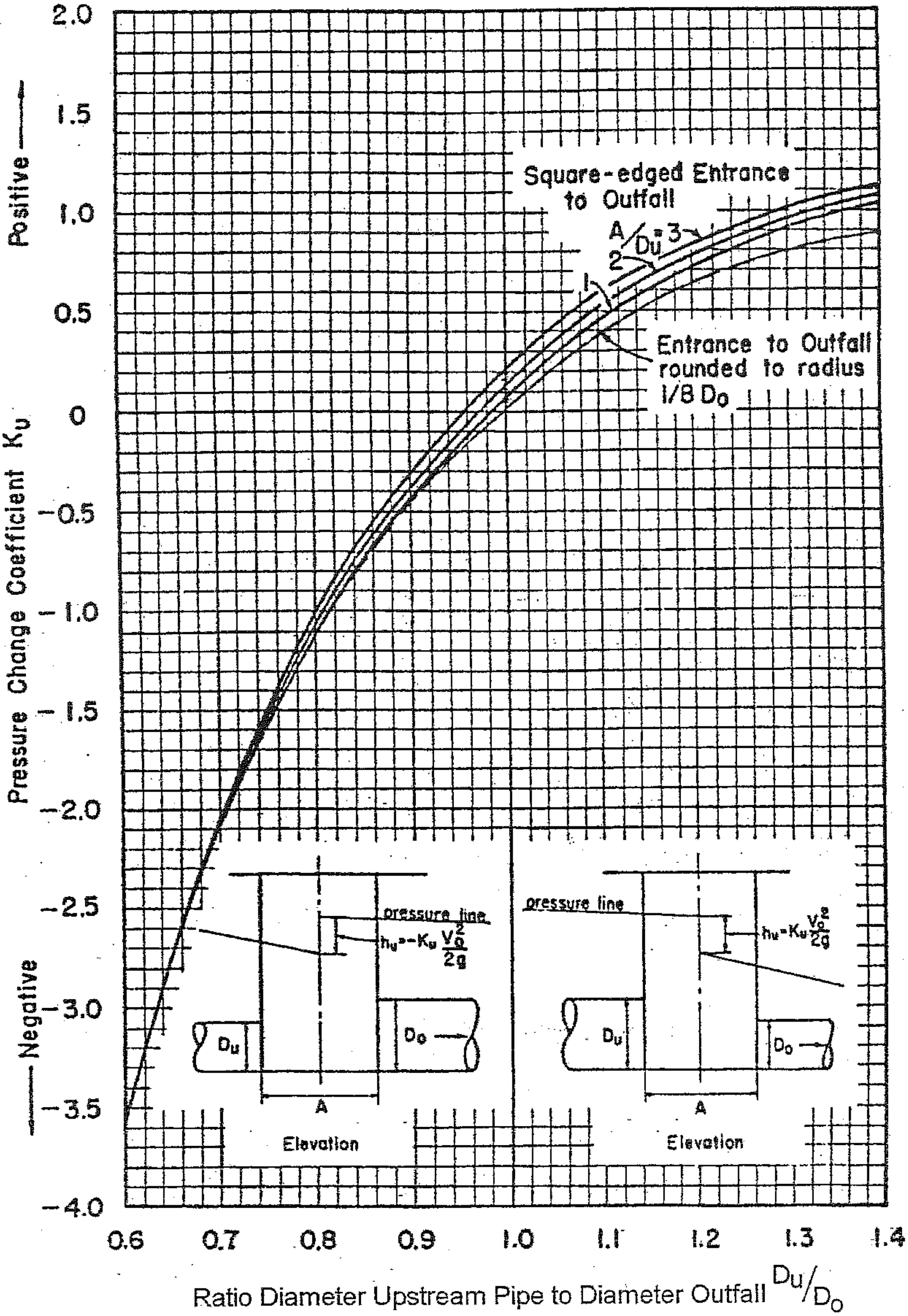


Fig. D5.10.3.10

Chart 3. Flow straight through any junction.

RECTANGULAR INLET WITH THROUGH PIPELINE AND GRATE FLOW

Pipe centrelines must be parallel and not offset more than would permit the area of the smaller pipe to fall entirely within the projection of the larger.

It is assumed that the outfall pressure line level has been calculated previously.

1. Calculate, D_U/D_O, Q_U/Q_O, Q_G/Q_O

2. Estimate K by taking

   $K = 3Q_G/Q_O$

3. Obtain an approximate value of d, using this value of K thus:

   $d = K\,V_O{}^2/2g$ + (pressure line level – invert level of outlet)

4. Calculate d/D_O

5. Obtain K_U from chart 4

6. Calculate h_u from

   $h_u = K_u V_O{}^2/2g$ and obtain the value of d

7. Check this value of d against the assumed value of d. If there is a substantial difference, recalculate h_u using the new value.

8. By additions the upstream pressure line level may now be found. The water level in the pit will be the same.

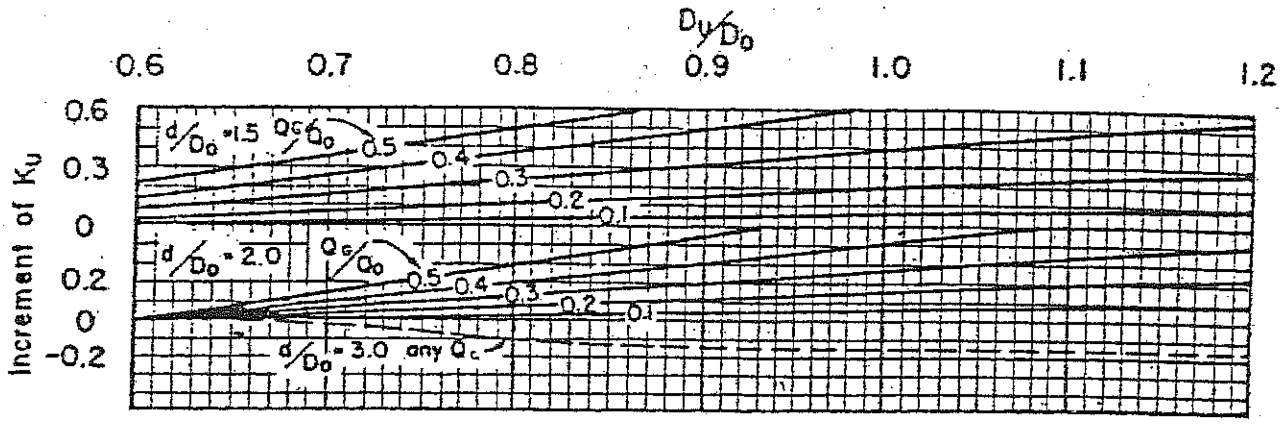
NOTE:

1. Where d/D_O is other than 2.5, the value of K_u obtained in step 5 must be increased by adding an increment which is obtained from the supplementary chart.

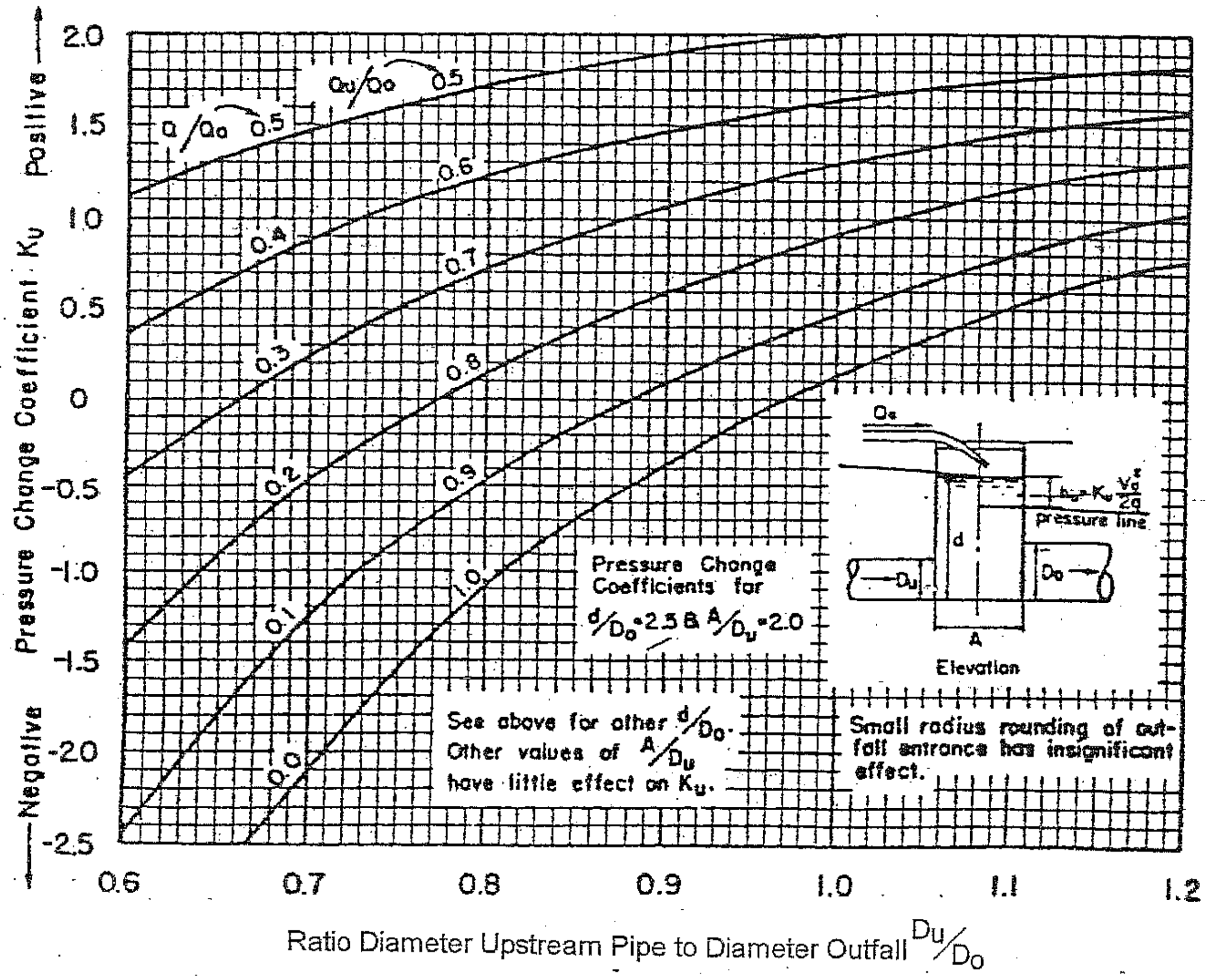
2. Frequently h_u is negative. If so, the pressure line level upstream is lower than the pressure line level downstream and h_u should be subtracted.

Fig. D5.10.3.11

Case | Chart
Rectangular manhole, through pipe and inlet flow | 4



Supplementary Chart for Modification of K_u for Depth in Inlet other than 2.5 D_o



Ratio Diameter Upstream Pipe to Diameter Outfall D_u/D_o

Chart 4. Rectangular inlet with through pipeline and grate flow.

Fig. D5.10.3.12

Chart 5

RECTANGULAR INLET WITH IN-LINE UPSTREAM MAIN AND 90^0 LATERAL PIPE (WITH OR WITHOUT GRATE FLOW)

It is assumed that the outfall pressure line level is known.

1. Calculate the ratios D_U/D_O, Q_U/Q_O and Q_G/Q_O if there is grate flow

2. Use chart 5 to find K_U using D_U/D_O and Q_U/Q_O

3. For grate flow, calculate Q_G/Q_O, estimate

   K = 1.5, and find the corresponding value of d from:

   $d = K\, V_O\, ^2/_2g +$ (outlet pressure line level – outlet invert level)

4. Calculate d/D_O and use the upper charts to find increments of K_u. Add these to the value in step 2.

5. Calculate h_u from

   $h_u = K_u\, V_O\, ^2/_2g$

   and obtain the more precise value of d. If there is a substantial difference, repeat the calculation.

6. By additions, the upstream pressure line level may be found. The upstream pressure line in the in-line and lateral pipes and the water level in the pit will all be the same.

NOTE:

Where there is no grate flow, the value of h_u and hence the pressure line levels may be found from the value of K_u found in step 2, i.e.

$h_u = K_u\, V_O\, ^2/_2g$

Fig. D5.10.3.13

Case	Chart
Rectangular manhole with in-line upstream main and 90° lateral pipe (with or without inlet flow)	5

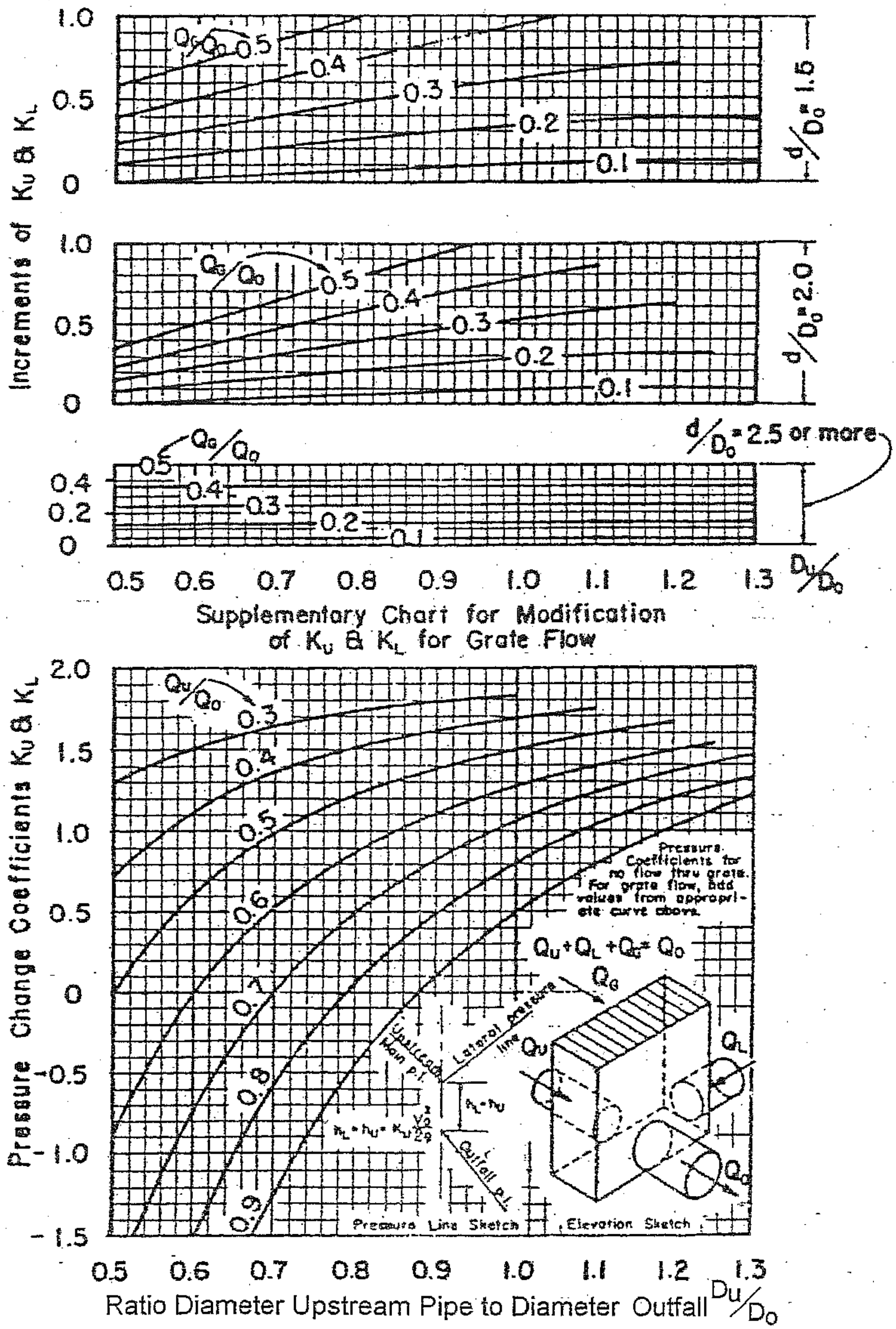


Chart 5. Rectangular inlet with in-line upstream main and 90° lateral pipe (with or without grate flow).

Fig. D5.10.3.14

RECTANGULAR INLET WITH IN-LINE OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT GRATE FLOW)

It is assumed that the outfall pressure line level is known.

1. Calculate the velocities in each lateral to determine which is the higher velocity and which the lower velocity lateral.
2. Calculate the values Q_{hv}/Q_o, D_{hv}/D_o, D_{hv}/D_{lv};
3. Find H from the left hand graph on chart 6.
4. Calculate the ratios Q_{lv}/Q_o and D_{lv}/D_o
5. Find L from the right hand graph on chart 6
6. Calculate K_{lv} from

   K_{lv} = H – L (with grate flow)

   or

   K_{lv} = (H – L) – 0.2 (without grate flow)
7. K_{hv} always equals 1.6 with grate flow and 1.6 without grate flow.
8. Calculate $h_{lv} = K_{lv} \times V_O{}^2/2g$ and $h_{hv} = K_{hv} \times V_O{}^2/2g$.
9. By additions, find the pressure line levels for both laterals.
10. The water surface level in the pit is equal to the higher velocity lateral.

NOTE:

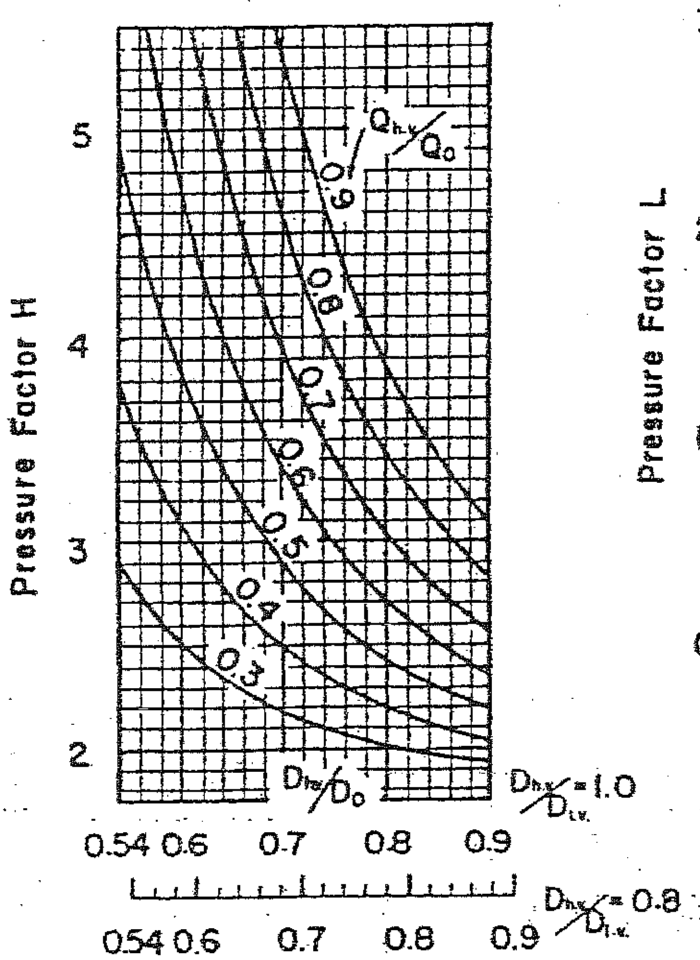
Where there is only one lateral, assume ..

K = 1.8 with grate flow; and

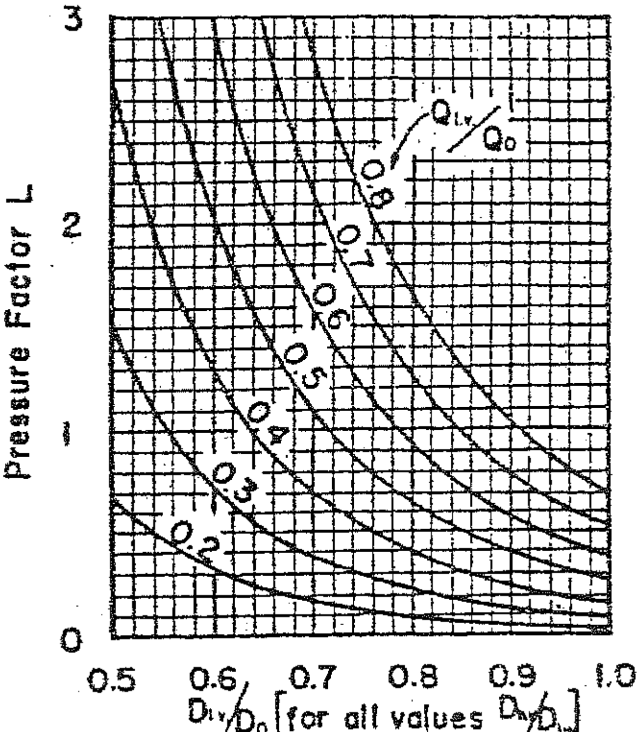
K = 1.6 without grate flow.

Fig. D5.10.3.15

Case	Chart
Rectangular manhole with in-line opposed lateral pipes each at 90° to outfall (with or without inlet flow)	6



Ratio of diameter of higher velocity lateral to diameter of outfall



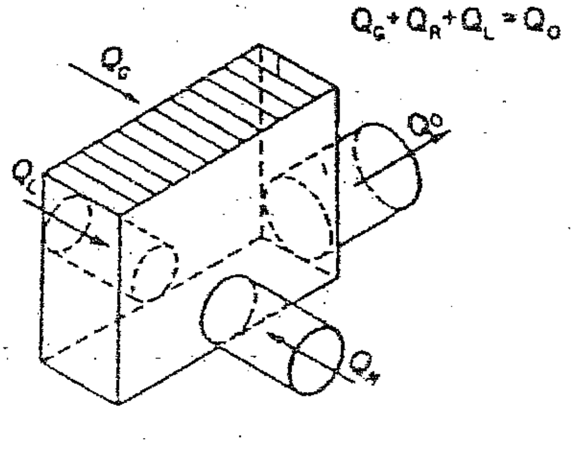
Ratio of diameter of lesser velocity lateral to diameter of outfall

$D_{h.v.}$ = diameter of lateral with higher-velocity flow.

$Q_{h.v.}$ = rate of flow in lateral with higher-velocity flow.

$D_{l.v.}$ = diameter of lateral with lower-velocity flow.

$Q_{l.v.}$ = rate of flow in lateral with lower-velocity flow.



Elevation Sketch

To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q, then: K_R (or K_L) = H − L

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8

$$h_L = K_L \frac{V_o^2}{2g} \qquad h_R = K_R \frac{V_o^2}{2g}$$

Chart 6. Rectangular inlet with in-line opposed lateral pipes each at 90° to outfall (with or without grate flow).

Fig. D5.10.3.16

Chart 7

RECTANGULAR INLET WITH OFFSET OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT GRATE FLOW)

The lateral pipes must be horizontally offset a distance not less than the sum of the two lateral pipe diameters.

It is assumed that the outfall pressure line level is known.

1. Calculate the ratios Q_F/Q_O, Q_N/Q_O, D_F/D_O, D_N/D_O.

2. Calculate the factors

   $Q_F/Q_O \times D_O/D_F$ and $Q_N/Q_O \times D_O/D_N$

3. Find K_F from right hand graph, chart 7

4. Find K_N from left hand graph, chart 7

5. Calculate h_f and h_n from

   $h_f = K_F \times V_O{}^2/2g$ and $h_n = K_N \times V_O{}^2/2g$.

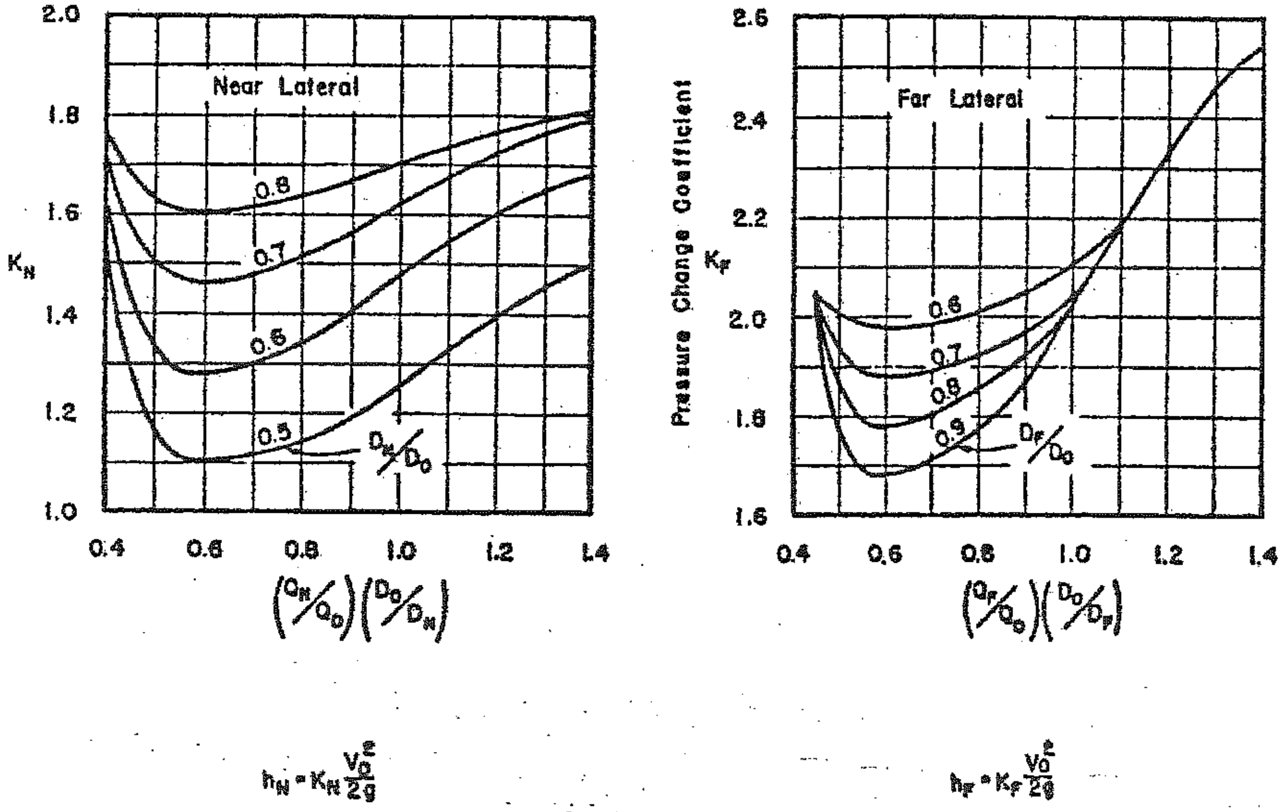
   This solution applies if there is no grate flow if there is grate flow, K_F and K_N should be reduced in the above formula by 0.2.

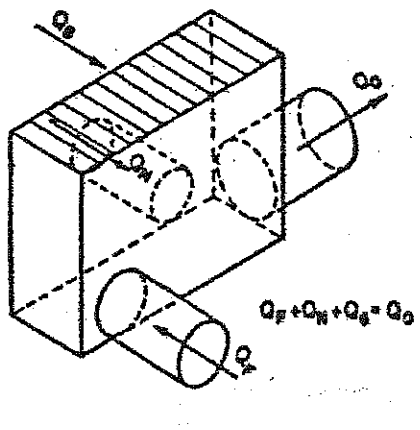
6. By additions, find the pressure line levels of both laterals.

7. The water surface level will be equal to the pressure line level of the higher velocity lateral.

Fig. D5.10.3.17

Case	Chart
Rectangular manhole with offset opposed lateral pipes each at 90° to outfall (with or without inlet flow)	7





Elevation Sketch

Chart 7. Rectangular inlet with offset opposed lateral pipes each at 90° to outfall (with or without grate flow).

Fig. D5.10.3.18

Chart 8
Lower Graph
& Chart 9

SQUARE MANHOLE AT 90° DEFLECTION

It is assumed that the outfall pressure line level is known.

For Lateral Pipe

1. Calculate the ratios D_L/D_O and B/D_O
2. From chart 8, read $\overline{K_L}$, $K_L = \overline{K_L} \times M_L$
3. Calculate pressure change h_L from

   $h_L = K_L \times V_O{}^2/2g$
4. By additions, find the pressure line level for the lateral.

For Water Level in Manhole

The water surface elevation in the manhole will be above the lateral pipe pressure line and may be found as follows:

5. Find $\overline{K_U}$ from chart 9.
6. Calculate Q_L/Q_O; D_L/D_O and $Q_L/Q_O \times D_O/D_L$
7. Obtain M_U from the upper graph of Chart 9.
8. Calculate K_U from $K_U = \overline{K_U} \times M_U$
9. Calculate H_U from $h_U = K_U \times V_O{}^2/2g$
10. By additions, find the water surface level, which will equal the outfall pressure line level, plus h.

NOTES:

This method may be used for round manholes by reducing in accord with the following table:

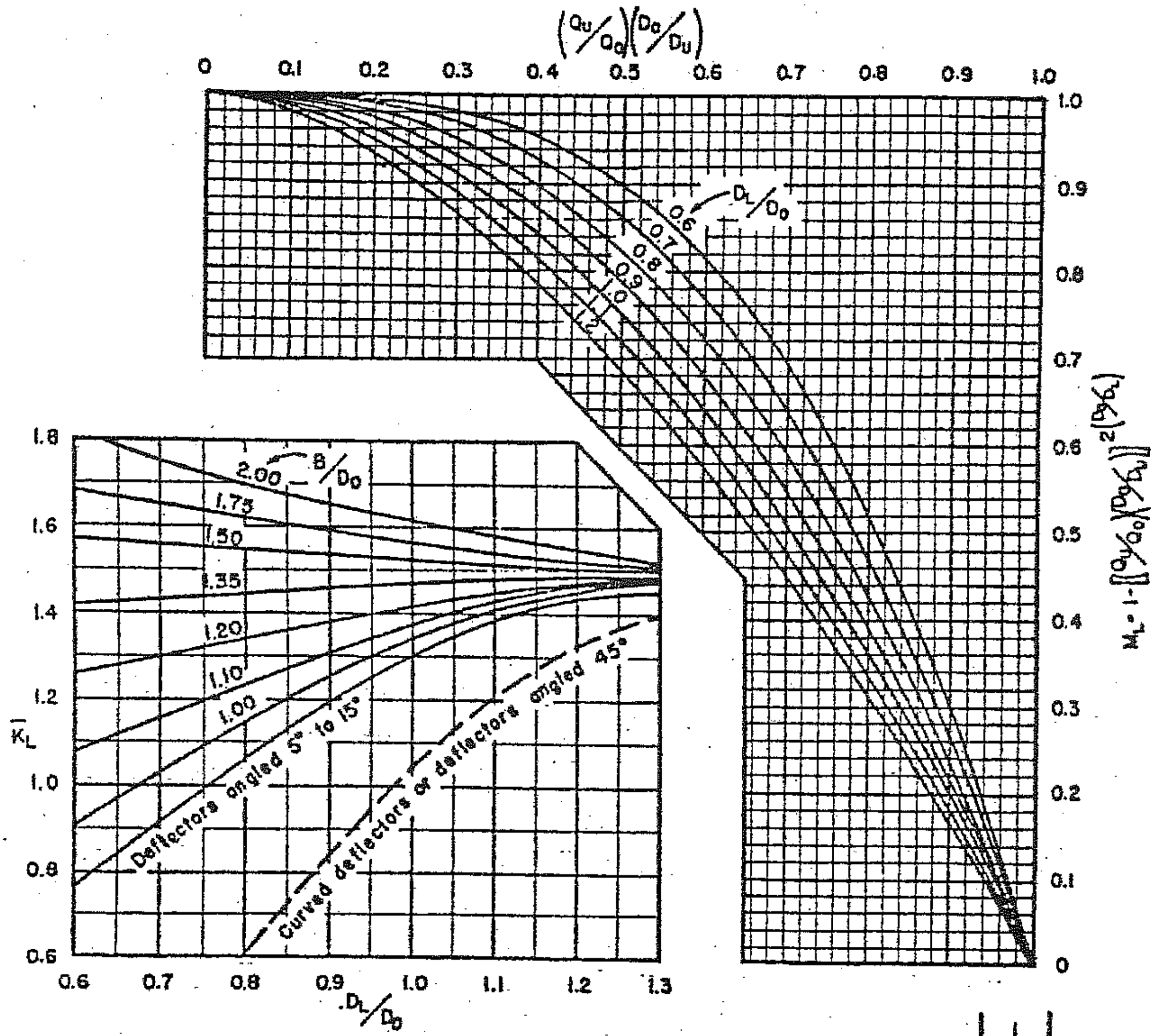
Reductions of K_L for D_L/D_O =

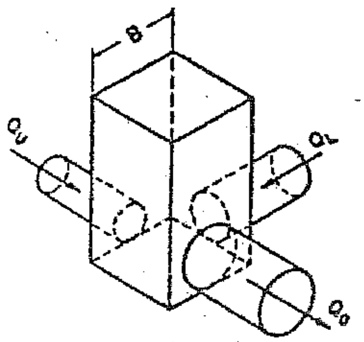
B/Do	0.8	1.0	1.2
1.75	0.3	0.2	0.0
1.33	0.2	0.1	0.0
1.10	0.1	0.0	0.0

These values apply for a sharp edged outfall entrance condition. For a well rounded entrance the above table may be dispensed with and a reduction for K_L of 0.3 for all cases made.

Fig. D5.10.3.19

Case	Chart
Square manhole at 90^0 deflection Round manhole at 90^0 deflection	8





Elevation Sketch

To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L. Then

$$K_L = \bar{K}_L \times M_L$$

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

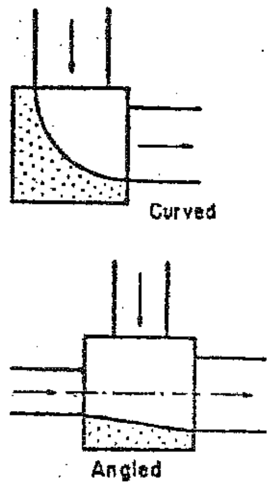
Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

For $Q_u/Q_o \times D_o/D_u > 1$ use Chart 10.

For $D_L/D_o < 0.6$ use Chart 10.

$$h_L = K_L \frac{V_o^2}{2g}$$



Plan of Deflectors

Chart 8. Square or round manhole at 90^0 deflection or on through pipeline at junction of 90^0 lateral pipe (lateral coefficient).

Fig. D5.10.3.20

SQUARE MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90^{O} LATERAL PIPE – CHARTS 8 & 9 (LARGER SIZE LATERALS: D_L/D_O>0.6)

In this case the pressure line level, in both upstream mains, and the water level in the pit, are the same.

Differing velocities have a significant effect on the pressure changes, and this is allowed for in the calculations by multiplying K by a factor of M_U, found from an additional graph, using the formula:

$\overline{K_U} = K_U \times M_U$

It is assumed that the outfall pressure line elevation is known.

For lateral pipe

1. Calculate the ratios D_L/D_O and B/D_O
2. Obtain $\overline{K_L}$ from Chart 8
3. Calculate the ratios Q_U/Q_O, D_U/D_O and $\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}$

   (Note that if this is greater than 1.00, Chart 10 should be used instead of Charts 8, 9)
4. Obtain M_L from the upper graph of Chart 8.
5. Calculate K_L from

   $K_L = M_L \times \overline{K_L}$
6. Calculate h_L from

   $h_L = K_L \times V_O{}^2/2g$
7. By additions find the pressure line level.

For upstream in-line main

8. Obtain K_U from Chart 9
9. Obtain M_U from the upper graph of Chart 9 and calculate

   K_U from

   $K_U = \overline{K_U} \times M_U$
10. Calculate h_u from

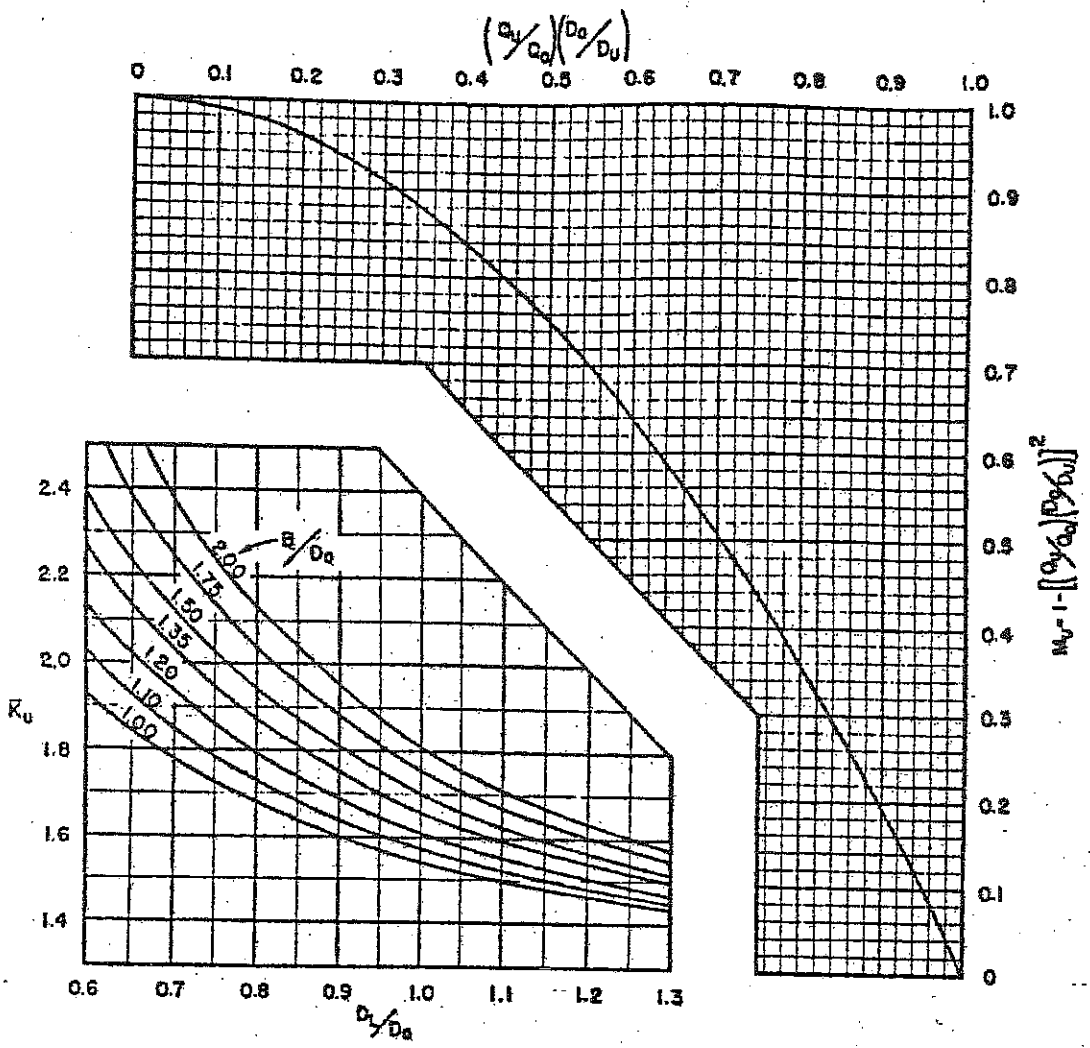
    $h_u = K_U \times V_O{}^2/2g$
11. By additions find the pressure line level.
12. The water surface elevation in the manhole will be equal to the upstream in-line pressure line.

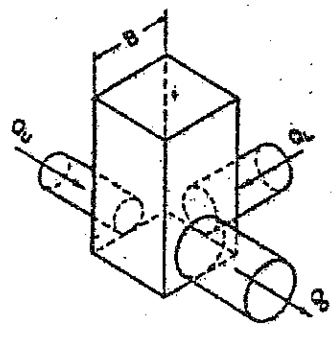
Note:

For both lateral and in-line upstream mains hydraulic conditions can be improved by rounding the entrance to the outfall. This may be allowed for in the calculations by reducing $\overline{K_U}$ and $\overline{K_L}$ each by 0.2.

Fig. D5.10.3.21

Case	Chart
Square manhole on through pipeline at junction of a 90^0 lateral pipe (large size laterals D_L/D_O>0.6)	9
Round manhole on through pipeline at junction of a 90^0 lateral pipe (large size lateral D_L/D_O>0.6)	9





Elevation Sketch

To find K_U for the upstream main, first read $\bar{K}_U$ from the lower graph. Next determine M_U. Then

$$K_U = \bar{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read $\bar{K}_U$ on curve for $B/D_O = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on Chart 8.

For $Q_U/Q_O \times D_O/D_U > 1$ use Chart 10

For $D_L/D_O < 0.6$ use Chart 10

$$h_U = K_U \frac{V_O^2}{2g}$$

Chart 9. Square or round manhole on through pipeline at junction of a 90^0 lateral pipe (in-line pipe coefficient).

Fig. D5.10.3.22

Chart 10

SQUARE OR ROUND MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90^0 LATERAL (SMALLER SIZE LATERALS $D_L/D_O \leq 0.6$)

It is assumed that the outfall pressure line is known.

1. Calculate the ratios D_U/D_O and Q_U/Q_O
2. Obtain K_U from Chart 10. Note K_U may be negative. This will occur when $Q_U/Q_O \times D_O/D_U$ is greater than 1.00.
3. Calculate $h_U = K_U \times V_O^2/2g = h_L$
4. By additions, find the upstream pressure line levels. This will be the same for both upstream pipes. The water surface elevation will also be the same.

NOTES:

If the outfall entrance is rounded, use the reduced values on the chart.

If $Q_U/Q_O < 0.7$, Chart 10 should not be used if other solutions are possible.

Fig. D5.10.3.23

Case	Chart
Square or round manhole on through pipeline at junction of 2 90° lateral pipe (smaller side laterals $D_L/D_O < 0.6$)	10

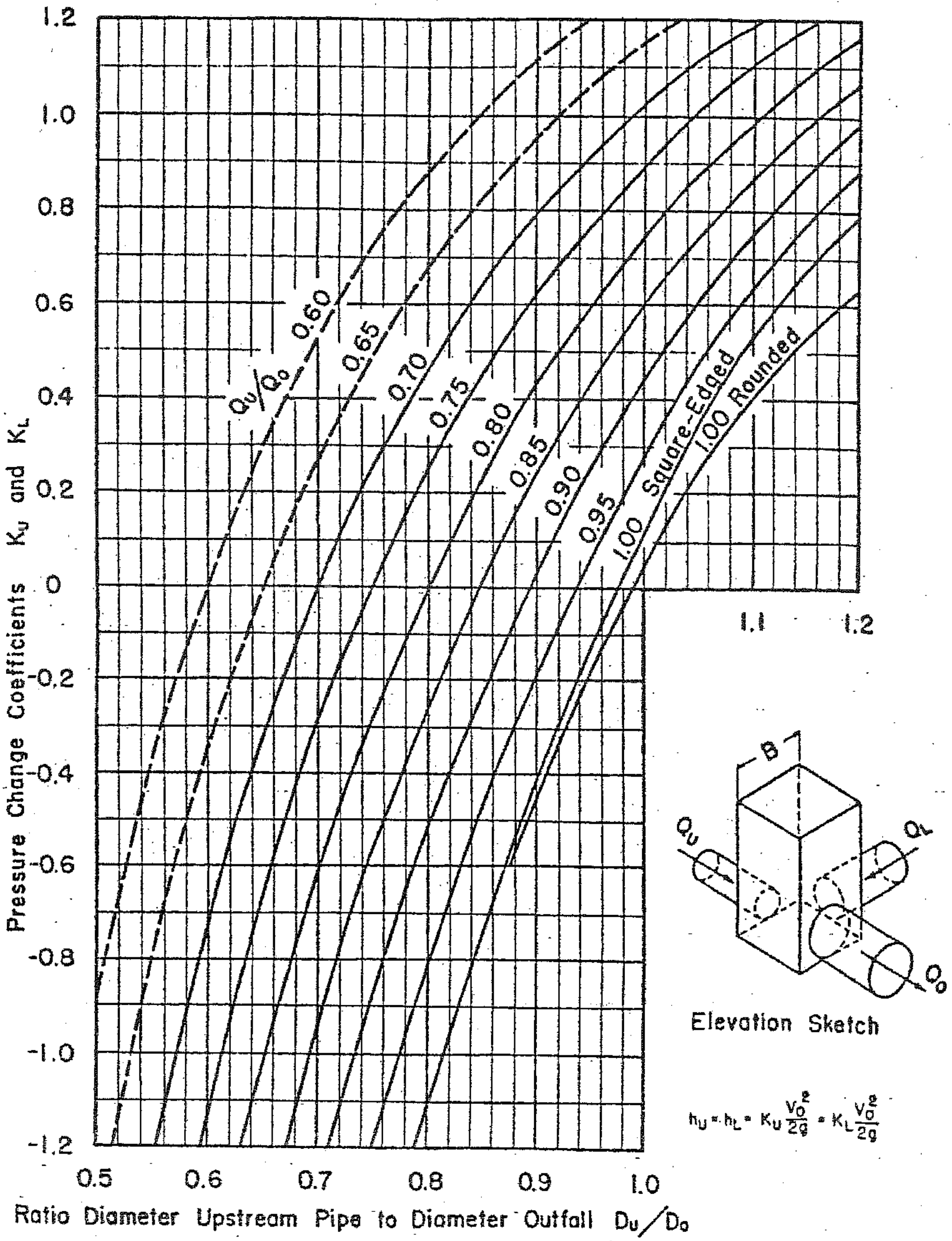


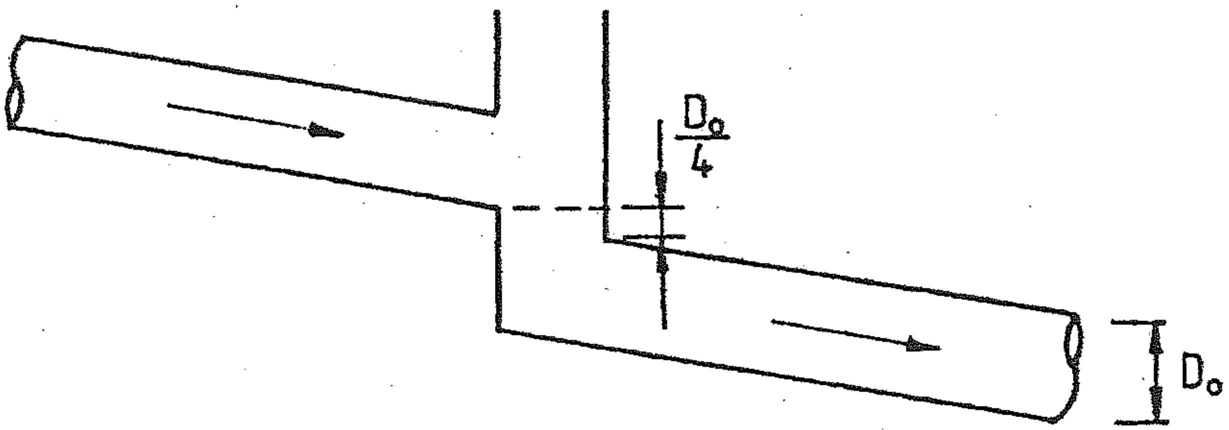
Chart 10. Square or round manhole on through pipeline at junction of a 90° lateral pipe (for conditions outside range of Charts 8 and 9).

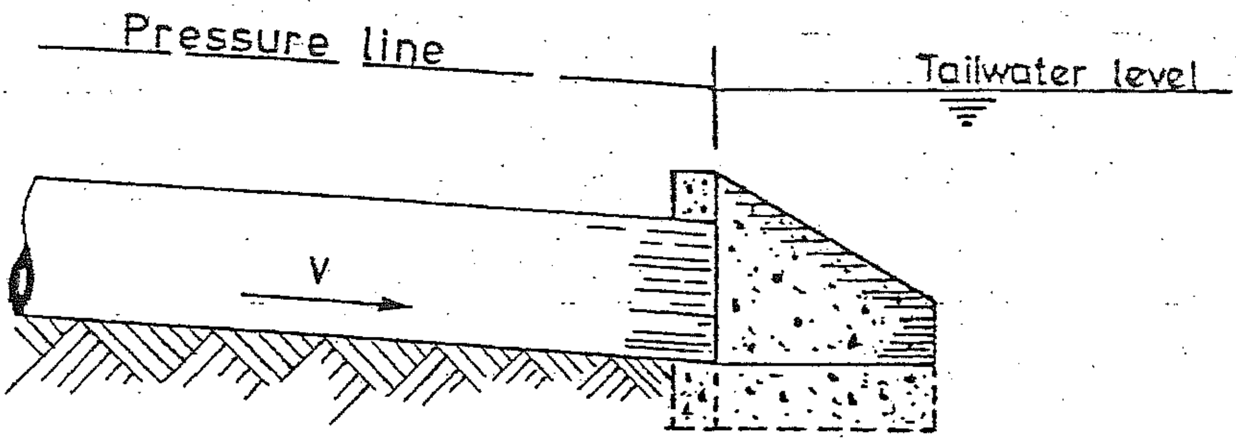
Fig. D5.10.3.24

DROP PITS

The University of Missouri method may be used to calculate head losses at drop pits (i.e. when the projection of the upstream pipe does not fall within the downstream pipe).

When the obvert of the outlet pipe is at or below $D_U/4$ below the invert of the upstream pipe, the inflow shall be regarded as grate flow for the purpose of calculating pressure losses. When the projection of the upstream pipe falls within the downstream pipe, the chart for the appropriate standard case should be used. Intermediate cases may be determined by linear interpolation of "K" values.





1. Find tailwater level. This will be the water level if discharge is into an open channel or natural waterway, and can be determined from observed flood levels or calculation using Manning's equation or other suitable formulae.

   If the outlet is into a large stormwater drain, the tailwater level will be the pressure line level of this drain, which must be found separately.

2. The pressure line just inside the pipe is the same as the tailwater level.

NOTE:

There is a loss of energy at the outlet of the pipe of $V^2/2g$. The total energy line, which is normally $V^2/2g$ above the pressure line, drops at the culvert outlet to the tailwater level, assuming the tailwater velocity is negligible. This does not affect calculations for the pressure line.

Fig. D5.10.3.25

PIPES NOT RUNNING FULL

Frequently an underground network involves pipe which does not run full.

The following design procedure is applicable to pipes flowing with a free water surface. Open channel flow is usually not economical when circular conduits are involved. Although it is theoretically possible for a pipe flowing 95% full to carry more than at 100% full for the same slope, it is from a practical standpoint impossible since any trash accumulation, junction, or other impediment would cause it to flow full.

Based on the assumption that Manning's n varies with depth, the capacity of a circular conduit at a given grade is the same at 91% and 100% ratios of d/D. Since it is impractical to design for the theoretical range where capacity exceeds that for the full conduit, open channel flow should only be assumed below d/D – 90%.

Figure D5.10.3.30 may be used to find flow conditions for pipes flowing part full. The design procedure in the following paragraphs is based upon the same assumption of straight water surfaces. Where sizes of conduits are sufficiently large, or other needs for a higher degree of accuracy exist, backwater or drawn down curves should be calculated.

The energy grade line may be defined at the total energy line of the water, i.e. for pipes running part full the water surface level plus the velocity head.

(a) The basic approach to design of open channel flow in pipes should be to calculate the energy grade line along the system. The assumption is made that the energy grade line is parallel to the pipe grade, and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

(b) Once the discharge has been determined and a pipe size and slope assumed for a given section, the d/D and v/V full, ratios can be determined from a graph of Hydraulic Elements for Circular Conduits, such as Figure D5.10.3.30. Figure D5.10.3.30 applies only to circular cross sections.

(c) The next step is to calculate the energy grade line.

$$H = Z + d + (V^2/2g)$$

where H = level of energy grade line
Z = invert level of pipe
D = depth of flow.

At each pit the energy grade lines of all pipes should coincide, allowing for reasonable values of head loss to the junction. Under certain conditions, this would indicate an upstream invert lower than the downstream invert. Inverts should be set at the same elevations under such circumstances.

(d) The usual method of stating head losses at manholes is in terms of a constant K times the velocity head of the conduit in question, $h_e = K\,V^2/2g$. A difficulty in design of systems is the determination of the value of K.

(e) Simple Transitions in Pipe Size. Simple transitions in conduit size in a manhole with straight through flow may be analysed by the following equation:

$$h_e = K\,\Delta\,(V^2/2g)$$

Where h_e = energy or head loss at junction

The term $\Delta\,(V^2/2g)$ refers to the change in velocity head in the upstream and downstream conduits. The value of K varies from 0.1 for increasing velocity to 0.2 for decreasing velocity transitions if flow is sub-critical. For super-critical flow, greater values of K are probable, but have not been determined.

(f) Bends. Reliable head loss coefficients through bends in open channel lacking. Reasonable assumptions may be made by utilising existing informa bends in pipes running full.

Fig. D5.10.3.26

(g) Junctions. Values for head loss coefficients at junctions on pipes flowing as open channels are not readily available. Complicated methods for calculating head loss at certain types of junctions are available and are justified for certain situations. A chapter in 'Street and Highway Drainage', Volume 2, Design Charts, (Ref. 2) is devoted to Hydraulic Analysis of Junctions. Additional information is contained in references 3 and 4. The designer will have to use engineering judgement and design conservatively.

(h) Storm Water Inlets. The design methods for culverts acting under inlet control presented in the Federal Highway Administration Charts are applicable when designing pit water depths where the connector pipe to the sewer is flowing partly full. An allowance should be made for approach velocity where applicable. Alternatively, the head loss may be calculated by the University of Missouri method by assuming that the downstream pressure line is at the obvert of the outlet pipe.

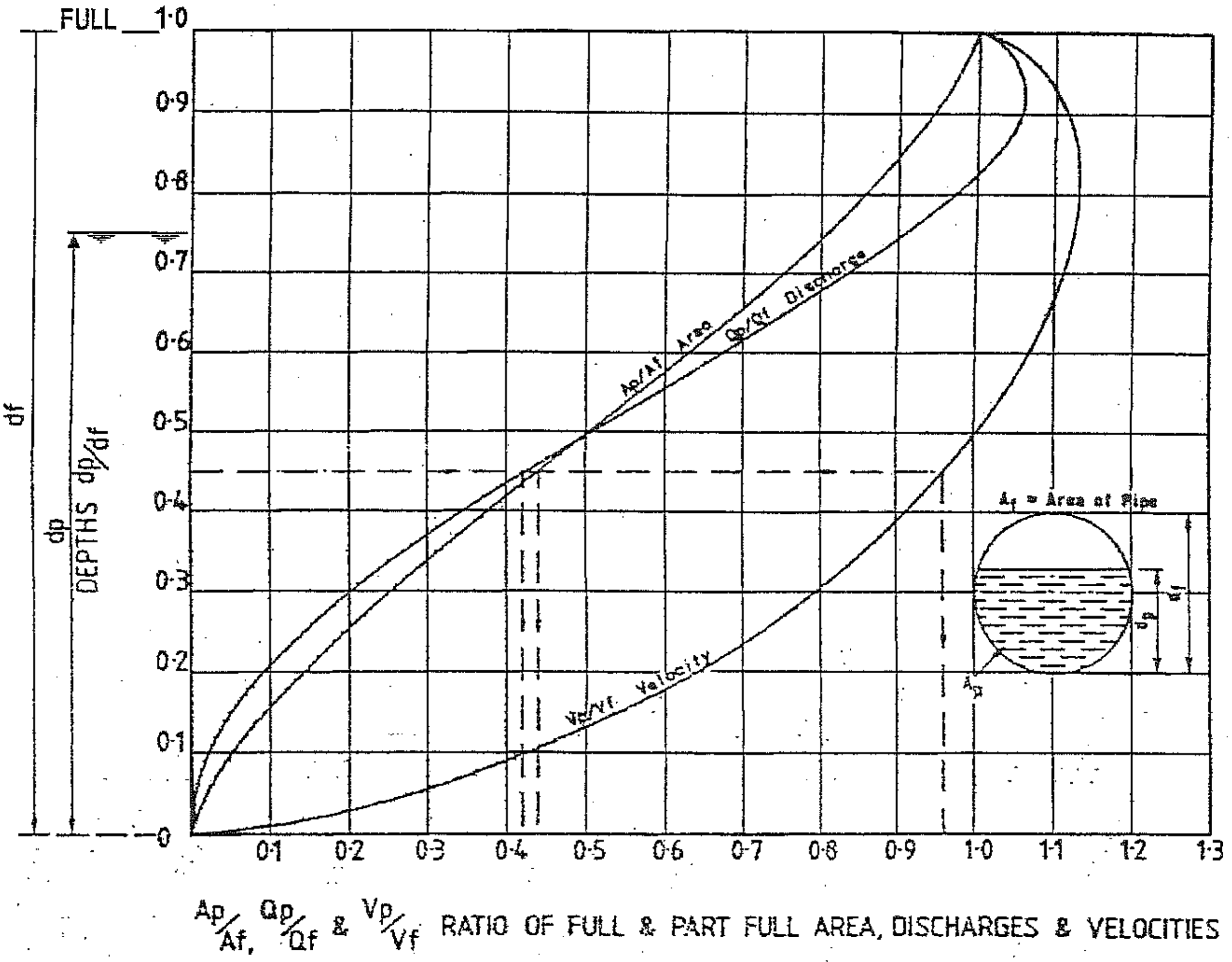
Care should be exercised to verify that the required head depth to attain the design flow in the conduit does not come above the gutter elevation.

Fig. D5.10.3.27

FROM CRB
ROAD DESIGN MANUAL

JANUARY, 1962

PROPORTIONAL VELOCITY AND DISCHARGE IN PART FULL CIRCULAR SECTION



NOTE:
- A_f = Area of pipe
- A_p = Area of flow when pipe is partly full
- Q_f = Discharge when pipe is full
- Q_p = Discharge when pipe is partly full
- V_f = Velocity of flow when pipe is full
- V_p = Velocity of flow when pipe is partly full
- d_f = diameter of pipe or depth of full flow
- d_p = depth of flow when pipe is partly full

EXAMPLE:

d_p/d_f = 0.45

A_p/A_f = 0.44

Q_p/Q_f = 0.42

V_p/V_f = 0.96

From CRB Road Design Manual

Fig. D5.10.3.28

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
A. CLOSED CONDUITS FLOWING PARTLY FULL			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n (*continued*)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

Fig. D5.11.1a

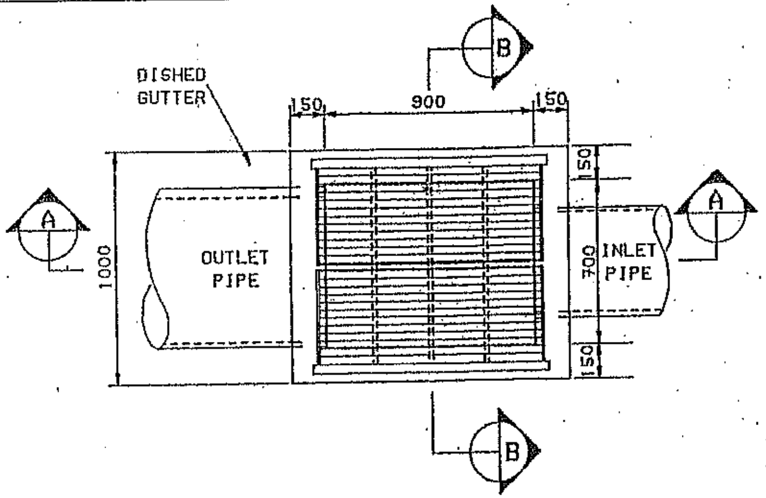
TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

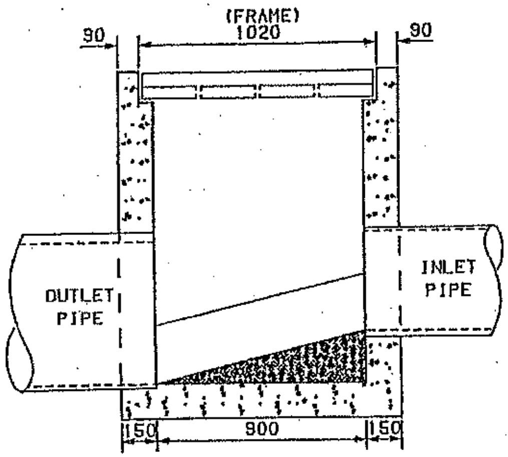
TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3 Major streams (top width at flood stage >100 ft). The *n* value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

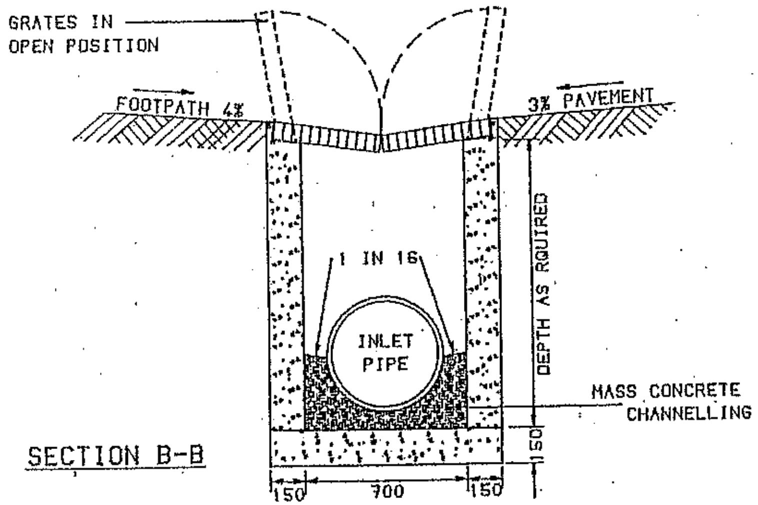
Fig. D5.11.1b



PLAN



SECTION A-A



SECTION B-B

NOTES:

1. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.
2. CONCRETE TO BE OF 20 MPa COMPRESSIVE STRENGTH (Fc) AT 28 DAYS.
3. GRATES TO BE HINGED 'WELDLOCK' DISHED CROSSING G.G.7862 SET PARALLEL TO EDGE OF GUTTER.
4. PIPE TRENCHES BACKFILLED WITH SAND SHALL HAVE 3 m OF SUBSOIL DRAIN PIPE IN THE BOTTOM OF THE TRENCH AND DRAINING INTO THE PIT.

Fig. D5.Misc.1

N. SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD DISHED GUTTER INLET PIT	ENGINEERING WORKS MANAGER	DRAWN	C. FIESELER
				CHECKED	J. BLOM
				PLAN REF.	263701
				DATE	04. MAY 1994

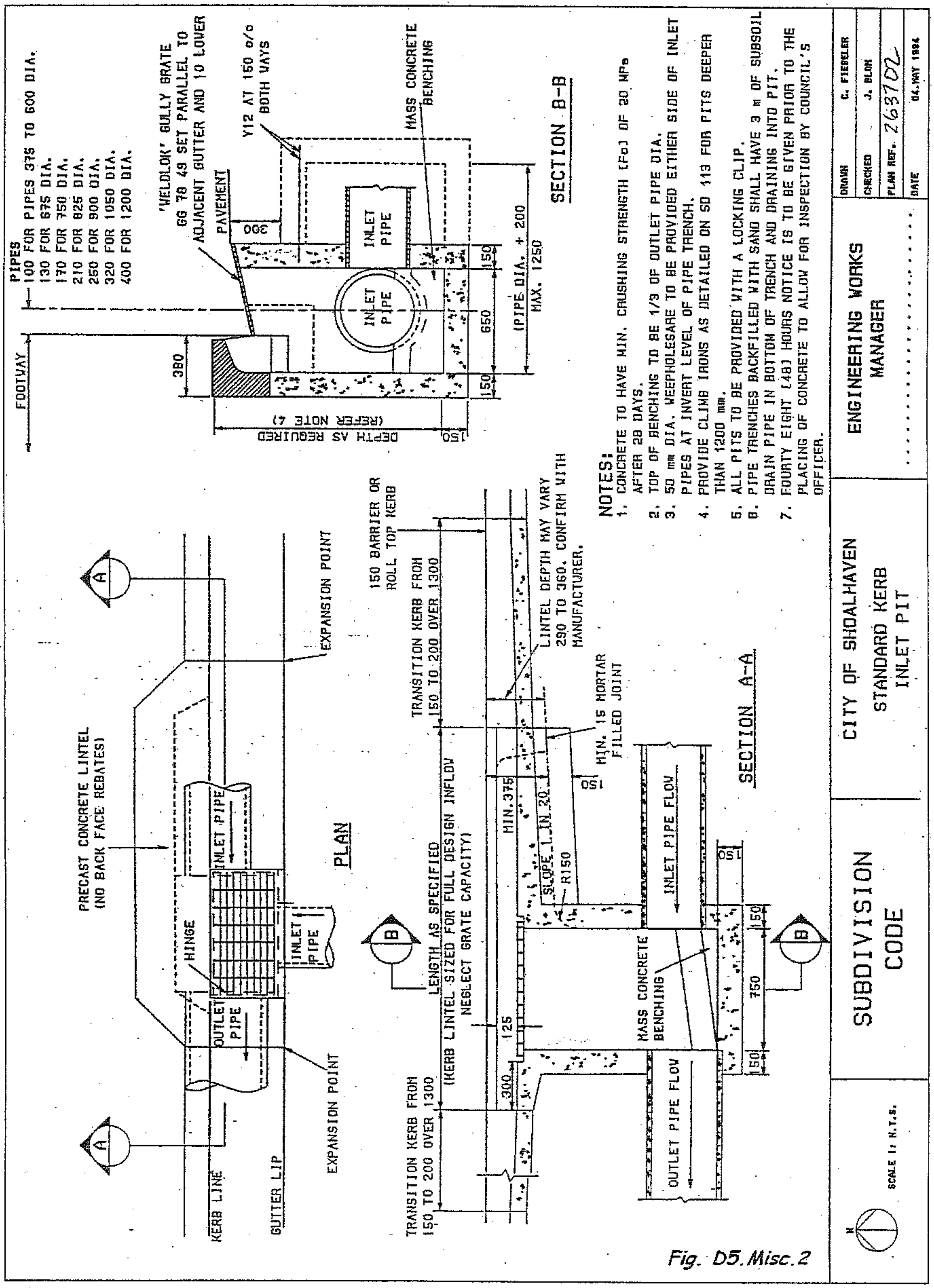

PRECAST CONCRETE LINTEL (NO BACK FACE REBATES)
HINGE
KERB LINE
OUTLET PIPE
INLET PIPE
GUTTER LIP
INLET PIPE
EXPANSION POINT
EXPANSION POINT
PLAN
150 BARRIER OR ROLL TOP KERB
TRANSITION KERB FROM 150 TO 200 OVER 1300
LENGTH AS SPECIFIED (KERB LINTEL SIZED FOR FULL DESIGN INFLOW NEGLECT GRATE CAPACITY)
TRANSITION KERB FROM 150 TO 200 OVER 1300
300
125
MIN.375
SLOPE 1 IN 20
R150
LINTEL DEPTH MAY VARY 290 TO 360. CONFIRM WITH MANUFACTURER.
150
MIN. 15 MORTAR FILLED JOINT
MASS CONCRETE BENCHING
OUTLET PIPE FLOW
INLET PIPE FLOW
150
150
750
150
SECTION A-A
FOOTWAY
PIPES
100 FOR PIPES 375 TO 600 DIA.
130 FOR 675 DIA.
170 FOR 750 DIA.
210 FOR 825 DIA.
250 FOR 900 DIA.
320 FOR 1050 DIA.
400 FOR 1200 DIA.
380
'WELDLOK' GULLY GRATE GG 78 49 SET PARALLEL TO ADJACENT GUTTER AND 10 LOWER
PAVEMENT
300
Y12 AT 150 c/c BOTH WAYS
DEPTH AS REQUIRED (REFER NOTE 4)
INLET PIPE
INLET PIPE
MASS CONCRETE BENCHING
150
150
650
150
(PIPE DIA. + 200 MAX. 1250
SECTION B-B
NOTES:
1. CONCRETE TO HAVE MIN. CRUSHING STRENGTH [Fc] OF 20 MPa AFTER 28 DAYS.
2. TOP OF BENCHING TO BE 1/3 OF OUTLET PIPE DIA.
3. 50 mm DIA. WEEPHOLESARE TO BE PROVIDED EITHER SIDE OF INLET PIPES AT INVERT LEVEL OF PIPE TRENCH.
4. PROVIDE CLIMB IRONS AS DETAILED ON SD 113 FOR PITS DEEPER THAN 1200 mm.
5. ALL PITS TO BE PROVIDED WITH A LOCKING CLIP.
6. PIPE TRENCHES BACKFILLED WITH SAND SHALL HAVE 3 m OF SUBSOIL DRAIN PIPE IN BOTTOM OF TRENCH AND DRAINING INTO PIT.
7. FOURTY EIGHT [48] HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.
N
SCALE 1: N.T.S.
SUBDIVISION CODE
CITY OF SHOALHAVEN
STANDARD KERB INLET PIT
ENGINEERING WORKS MANAGER
DRAWN C. FIEBELER
CHECKED J. BLOM
PLAN REF. 263702
DATE 04.MAY 1994

Fig. D5.Misc.2

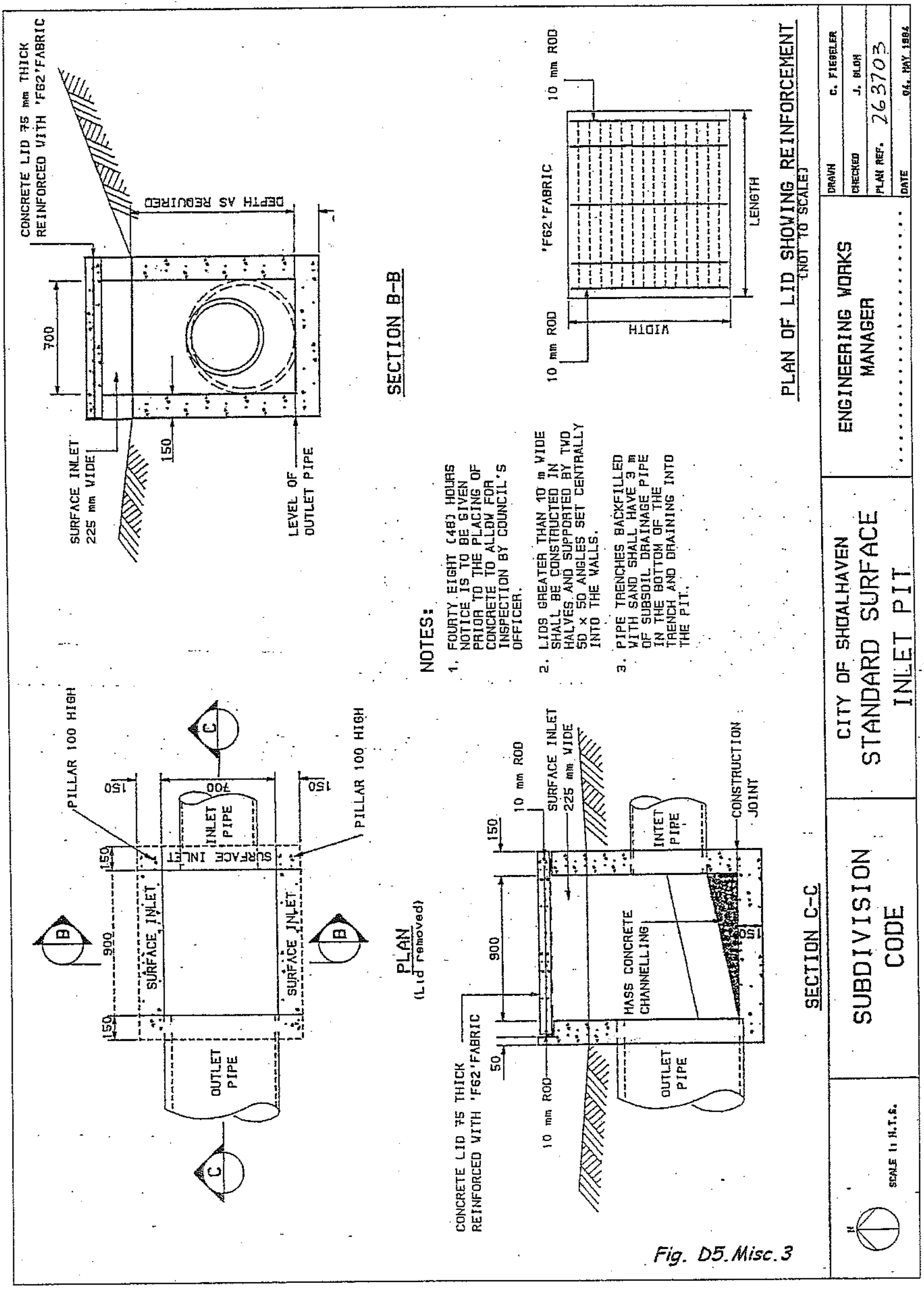

CONCRETE LID 75 mm THICK REINFORCED WITH 'F62'FABRIC
DEPTH AS REQUIRED
700
150
SURFACE INLET 225 mm WIDE
LEVEL OF OUTLET PIPE
SECTION B-B
10 mm ROD
'F62'FABRIC
LENGTH
WIDTH
PLAN OF LID SHOWING REINFORCEMENT (NOT TO SCALE)
NOTES:
1. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.
2. LIDS GREATER THAN 10 m WIDE SHALL BE CONSTRUCTED IN HALVES AND SUPPORTED BY TWO 50 x 50 ANGLES SET CENTRALLY INTO THE WALLS.
3. PIPE TRENCHES BACKFILLED WITH SAND SHALL HAVE 3 m OF SUBSOIL DRAINAGE PIPE IN THE BOTTOM OF THE TRENCH AND DRAINING INTO THE PIT.
PILLAR 100 HIGH
900
INLET PIPE
SURFACE INLET
OUTLET PIPE
PLAN (Lid removed)
B
C
SURFACE INLET 225 mm WIDE
CONSTRUCTION JOINT
INTET PIPE
MASS CONCRETE CHANNELLING
50
SECTION C-C
ENGINEERING WORKS MANAGER
DRAWN C. FIEGLER
CHECKED J. BLOH
PLAN REF. 263703
DATE 04. MAY 1984
CITY OF SHOALHAVEN
STANDARD SURFACE INLET PIT
SUBDIVISION CODE
SCALE 1: N.T.S.
N

Fig. D5.Misc.3

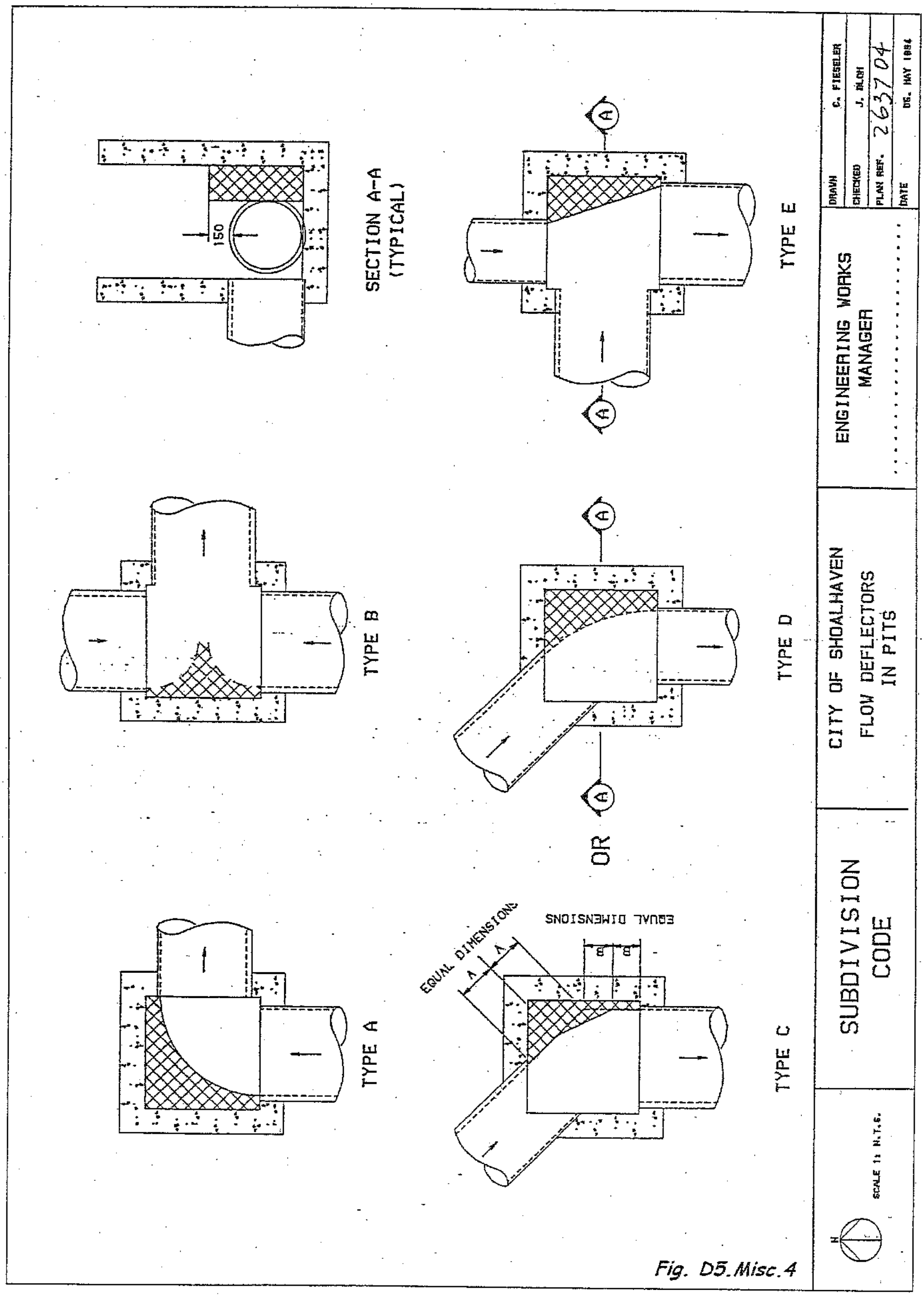
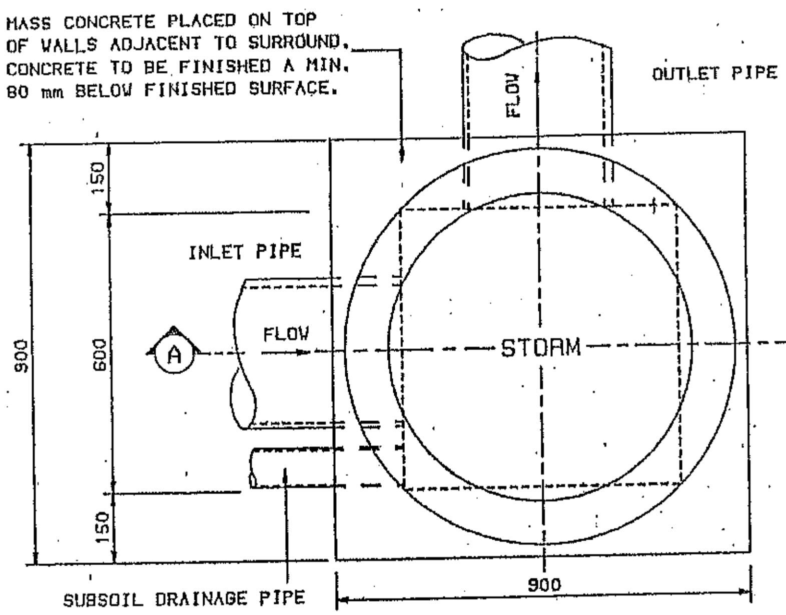
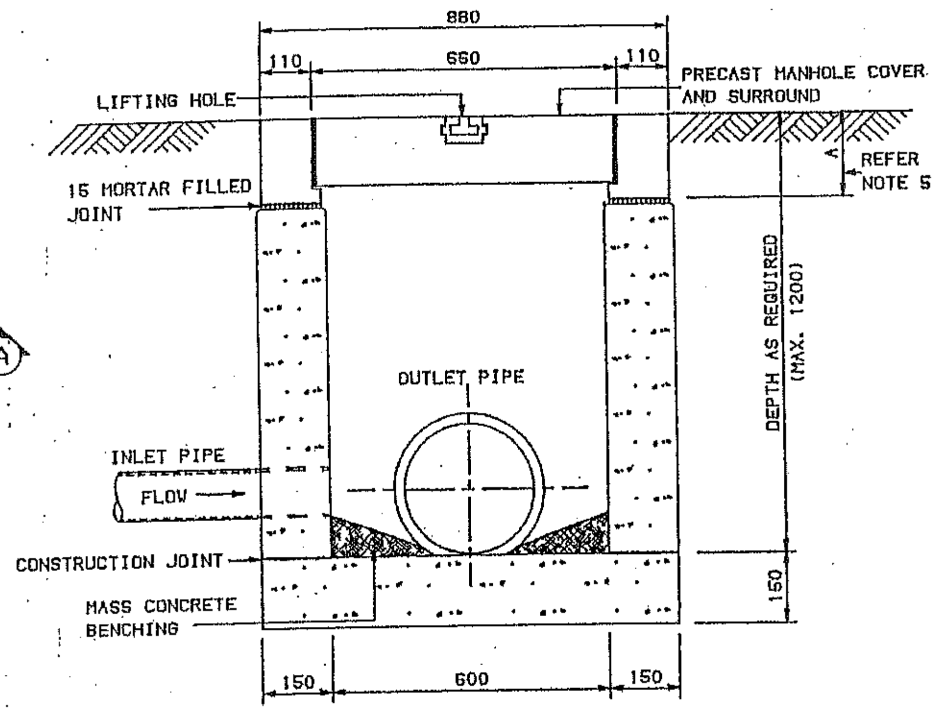

SECTION A-A
(TYPICAL)
150
TYPE A
TYPE B
TYPE C
TYPE D
TYPE E
OR
EQUAL DIMENSIONS
A
B
ENGINEERING WORKS
MANAGER
CITY OF SHOALHAVEN
FLOW DEFLECTORS
IN PITS
SUBDIVISION
CODE
SCALE 1: N.T.S.
DRAWN
C. FIESELER
CHECKED
PLAN REF.
263704
DATE

Fig. D5.Misc.4



PLAN



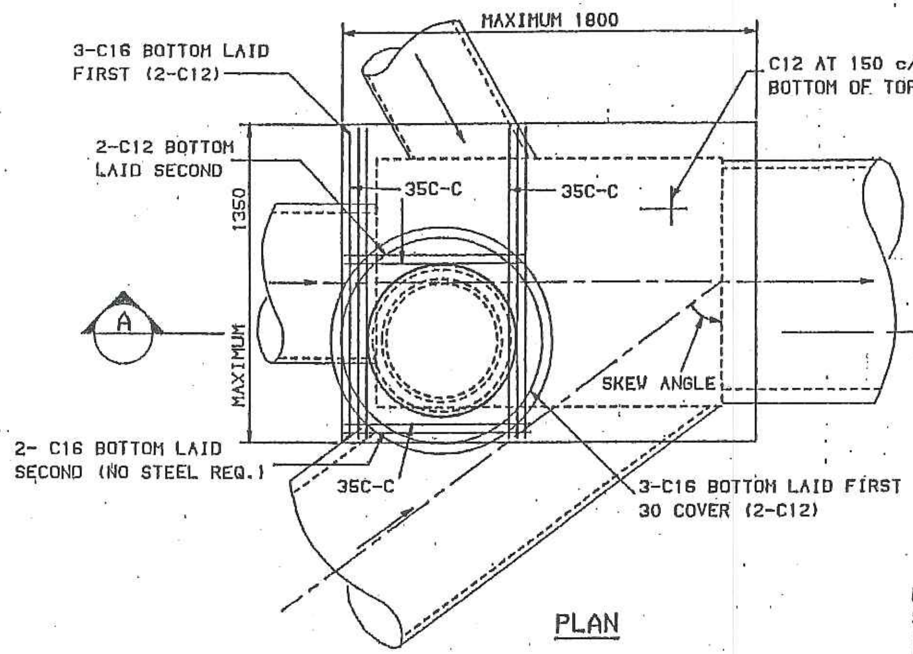
SECTION A-A

NOTES:

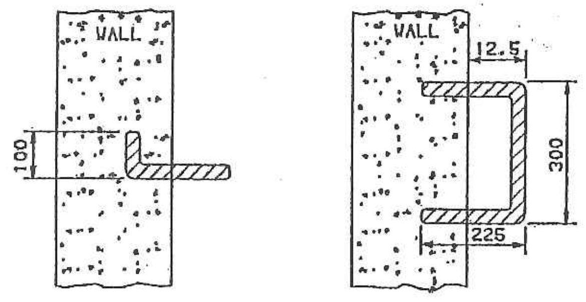
1. ALL PRECAST COMPONENTS TO BE FACTORY PRODUCED.
2. MINIMUM CONCRETE COMPRESSIVE STRENGTH 25 MPa AT 28 DAYS.
3. MINIMUM PIPE DIAMETRE 150 mm.
4. PROVIDE 3 m OF 100 mm DIAMETRE SUBSOIL DRAINAGE PIPE WRAPPED IN FABRIC SOCK ADJACENT TO INLET PIPE.
5. A - 150 mm LIGHT DUTY PIPE
   - 200 mm HEAVY DUTY PIPE
6. ENSURE FACE OF PIPE IN PIT WALL ARE SMOOTHLY GROUTED
7. LIDS TO BE MARKED 'STORM' TO DISTINGUISH FROM UNMARKED SEWER LIDS.
8. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.

Fig. D5. Misc.5

N SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD INTERALLOTMENT DRAINAGE JUNCTION PIT	ENGINEERING WORKS MANAGER	DRAWN	C. FIEGELER
				CHECKED	J. BLOH
				PLAN REF.	263705
				DATE	05.MAY 1994



PLAN

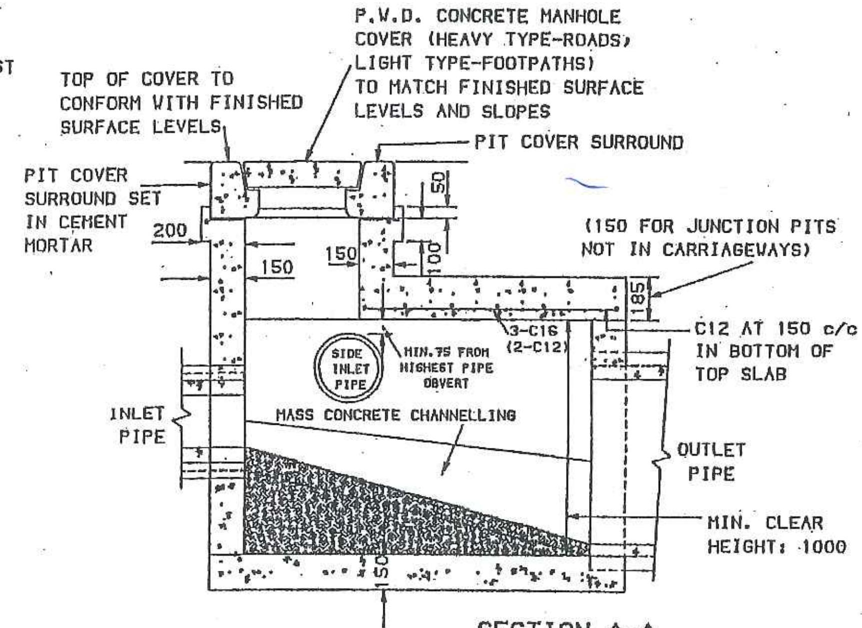


ELEVATION

PLAN

DETAIL OF CLIMB IRON

CLIMB IRON OF 20 GALVANISED STEEL MADE TO ABOVE SHAPE AND DIMENSIONS SHALL BE PROVIDED AT 950 c/c UNDER LID WHERE PIT IS DEEPER THAN 1200.



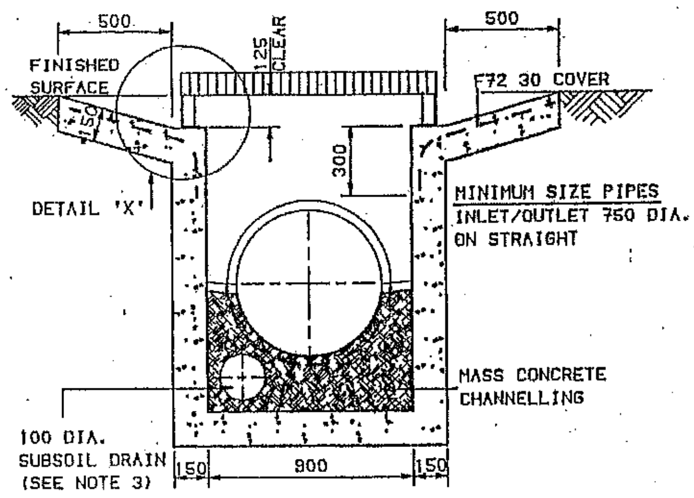
SECTION A-A

NOTES:

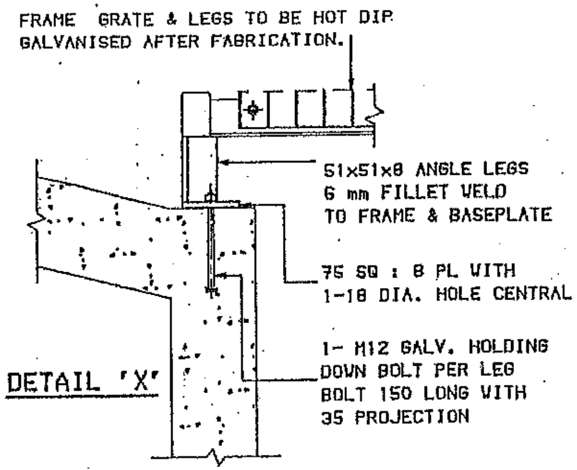
1. MAXIMUM OUTLET PIPE ON STRAIGHT 900 DIAMETRE.
2. MAXIMUM OUTLET PIPE ON SKEW 825 DIAMETRE.
3. MAXIMUM SIDE ENTRY PIPE 825 DIAMETRE AT APPROX. 40° SKEW
4. MINIMUM INTERNAL DIMENSIONS; LENGTH 900
   WIDTH 700
   HEIGHT 1000
5. CONCRETE TO BE OF 20 MPa COMPRESSIVE STRENGTH (Fc) AT 28 DAYS.
6. PIPE TRENCHES BACKFILLED WITH SAND SHALL HAVE 3 m OF SUBSOIL DRAIN PIPE IN THE BOTTOM OF THE TRENCH AND DRAINING INTO THE PIT.
7. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.
8. FOR STANDARD PITS IN LOCATIONS OTHER THAN CARRIAGEWAYS DIMENSIONS AND REINFORCING SHOWN IN BRACKETS APPLY.

Fig. D5.Misc.6

N SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD JUNCTION PIT	ENGINEERING WORKS MANAGER	DRAWN C. FIEBELER CHECKED J. BLOH PLAN REF. 263706 DATE 06 MAY 1986



SECTION A-A
SURCHARGE PIT TYPE A



DETAIL 'X'

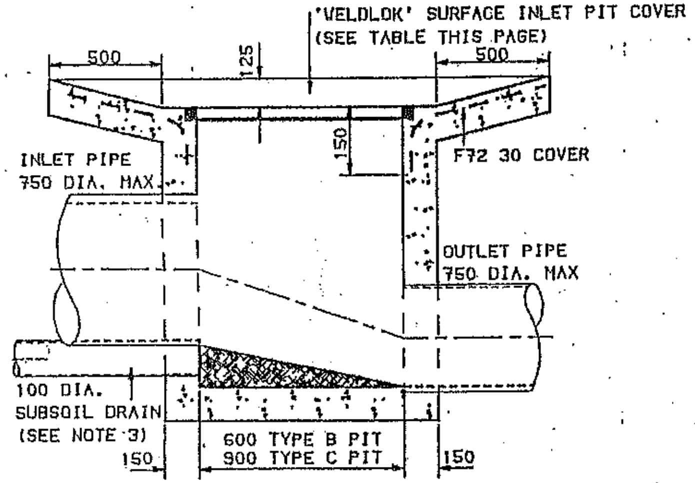
SURCHARGE PIT SIZES TABLE

PIT	OPENING SIZE	GRATE TYPE
TYPE A	900 x 900	PC 9090 B *
TYPE B	600 x 900	PC 9090 A
TYPE C	900 x 900	PC 9090 B

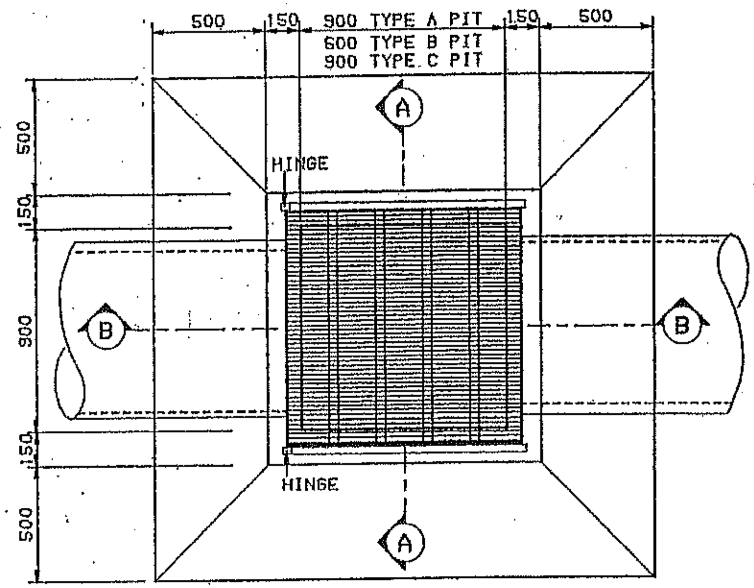
* GRATE FITTED WITH LEGS (DETAIL'X')

NOTES:

1. COMPRESSIVE STRENGTH OF CONCRETE Fc AT 28 DAYS TO BE 20 MPa
2. TOP OF BENCHING TO BE 1/2 OF OUTLET PIPE DIA.
3. 100 DIA. SUBSOIL DRAINAGE PIPE 3000 LONG WRAPPED IN FABRIC SOCK TO BE PROVIDED IN PIPE TRENCHES ADJACENT TO INLET PIPES.
4. PROVIDE CLIMB IRONS WHERE PIT IS DEEPER THAN 1200.
5. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.



SECTION B-B
SURCHARGE PIT TYPES B & C



PLAN SURCHARGE PITS TYPE A, B, & C

Fig. D5.Misc.7

N SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD SURCHARGE PITS	ENGINEERING WORKS MANAGER	DRAWN	C. FIEGELER
				CHECKED	J. BLOM
				PLAN REF.	26 3707
				DATE	05 MAY 1994

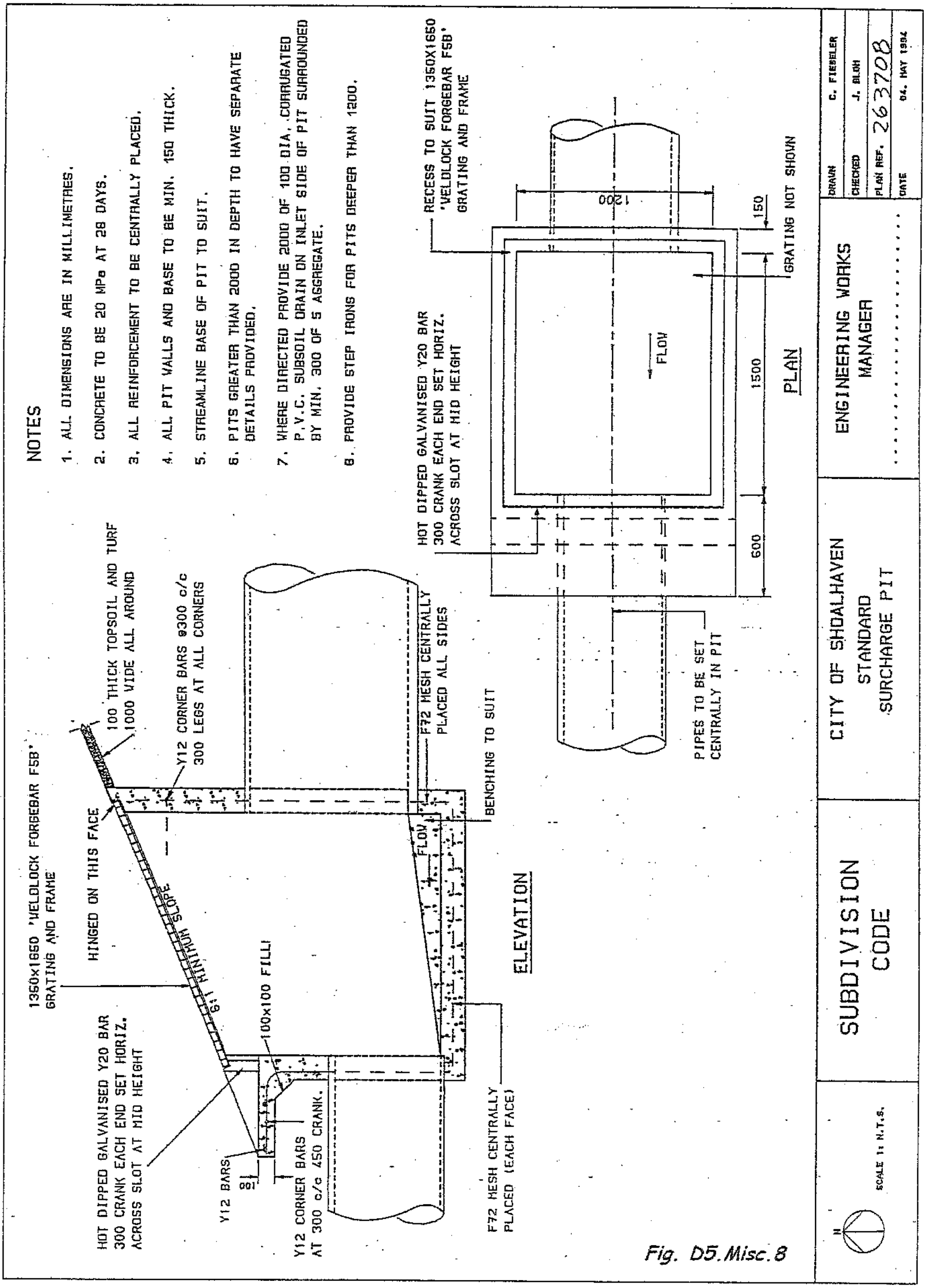

NOTES
1. ALL DIMENSIONS ARE IN MILLIMETRES.
2. CONCRETE TO BE 20 MPa AT 28 DAYS.
3. ALL REINFORCEMENT TO BE CENTRALLY PLACED.
4. ALL PIT WALLS AND BASE TO BE MIN. 150 THICK.
5. STREAMLINE BASE OF PIT TO SUIT.
6. PITS GREATER THAN 2000 IN DEPTH TO HAVE SEPARATE DETAILS PROVIDED.
7. WHERE DIRECTED PROVIDE 2000 OF 100 DIA. CORRUGATED P.V.C. SUBSOIL DRAIN ON INLET SIDE OF PIT SURROUNDED BY MIN. 300 OF 5 AGGREGATE.
8. PROVIDE STEP IRONS FOR PITS DEEPER THAN 1200.
RECESS TO SUIT 1350X1650 'WELDLOCK FORGEBAR F5B' GRATING AND FRAME
1200
150
GRATING NOT SHOWN
FLOW
1500
PLAN
HOT DIPPED GALVANISED Y20 BAR 300 CRANK EACH END SET HORIZ. ACROSS SLOT AT MID HEIGHT
600
PIPES TO BE SET CENTRALLY IN PIT
1350x1650 'WELDLOCK FORGEBAR F5B' GRATING AND FRAME
HINGED ON THIS FACE
100 THICK TOPSOIL AND TURF 1000 WIDE ALL AROUND
Y12 CORNER BARS @300 c/c 300 LEGS AT ALL CORNERS
F72 MESH CENTRALLY PLACED ALL SIDES
BENCHING TO SUIT
6:1 MINIMUM SLOPE
FLOW
100x100 FILLET
ELEVATION
HOT DIPPED GALVANISED Y20 BAR 300 CRANK EACH END SET HORIZ. ACROSS SLOT AT MID HEIGHT
Y12 BARS
Y12 CORNER BARS AT 300 c/c 450 CRANK.
F72 MESH CENTRALLY PLACED (EACH FACE)
SUBDIVISION CODE
CITY OF SHOALHAVEN STANDARD SURCHARGE PIT
ENGINEERING WORKS MANAGER
DRAWN C. FIEBELER
CHECKED J. BLOM
PLAN REF. 263708
DATE 04. MAY 1994
SCALE 1: N.T.S.
N

Fig. D5.Misc.8

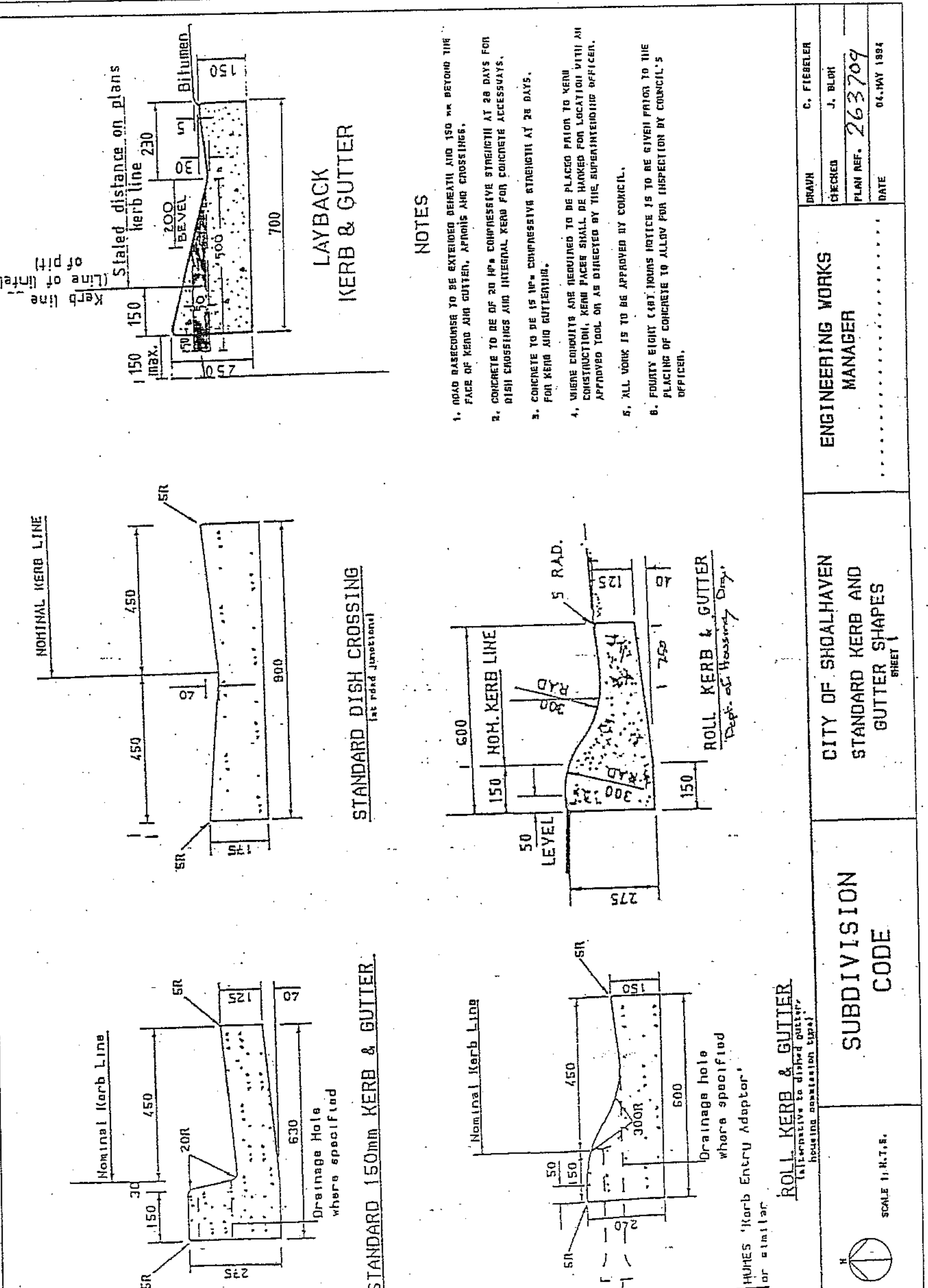

LAYBACK KERB & GUTTER
STANDARD DISH CROSSING
ROLL KERB & GUTTER
STANDARD 150mm KERB & GUTTER
NOTES
CITY OF SHOALHAVEN
STANDARD KERB AND GUTTER SHAPES
SUBDIVISION CODE
ENGINEERING WORKS MANAGER
DRAWN C. FIEBELER
CHECKED J. BLOM
PLAN REF. 263709
DATE 04.MAY 1994

Fig. D5.Misc.9

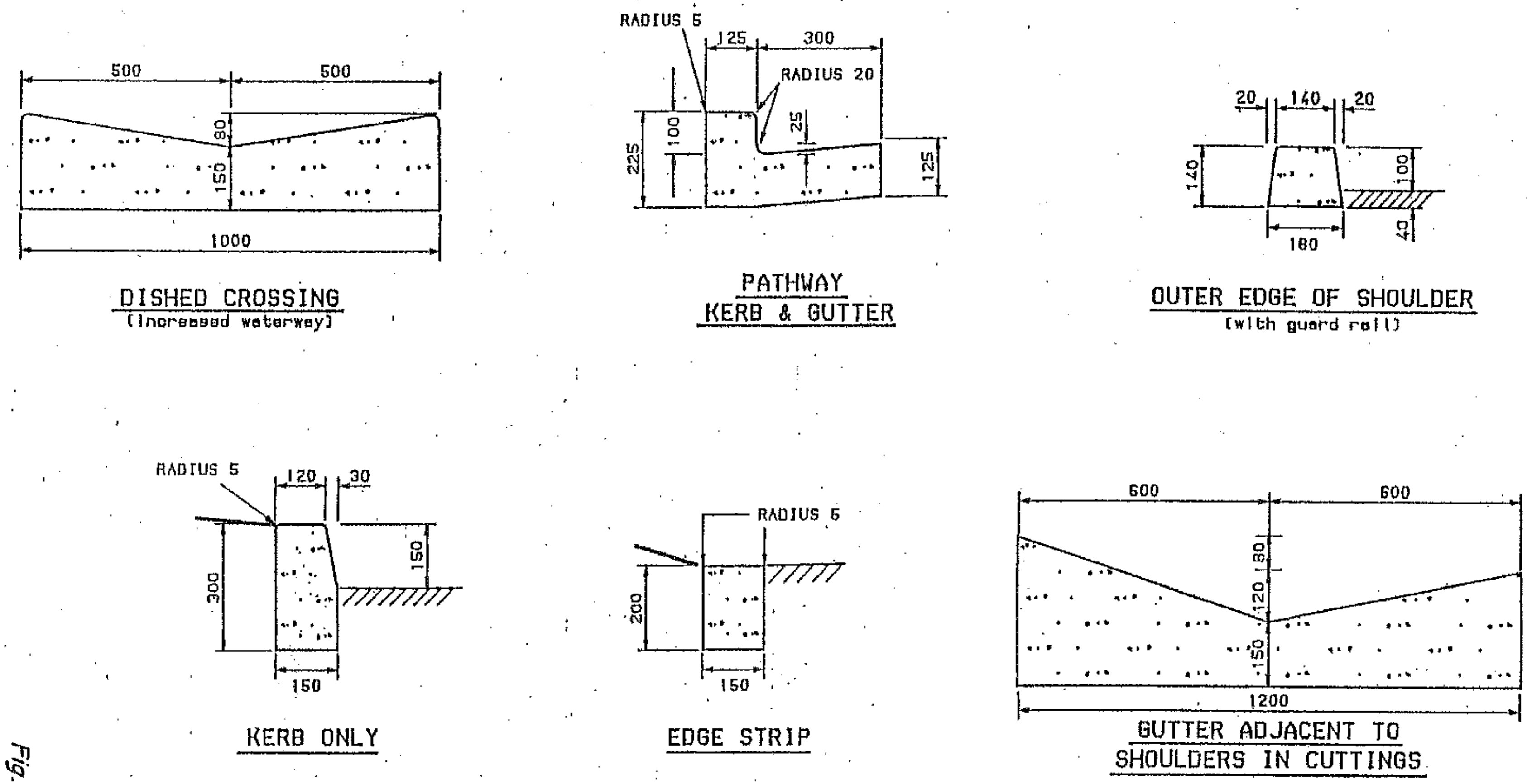


Fig. D5.Misc.10

NOTES:

1. ROAD BASECOURSE TO BE EXTENDED BENEATH AND 150 mm BEYOUND THE FACE OF KERB AND GUTTER, APRONS AND CROSSINGS.
2. CONCRETE TO BE OF 15 MPa COMPRESSIVE STRENGTH AT 28 DAYS FOR KERBING, GUTTERING AND EDGE STRIPS.
3. ALL WORK IS TO BE APPROVED BY COUNCIL.
4. FOURTY EIGHT (48) HOURS NOTICE IS TO BE GIVEN PRIOR TO THE PLACING OF CONCRETE TO ALLOW FOR INSPECTION BY COUNCIL'S OFFICER.

N SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD KERB AND GUTTER SHAPES SHEET 2	ENGINEERING WORKS MANAGER	DRAWN C. FIESELER CHECKED J. BLOM PLAN REF. 263710 04 MAY 1994

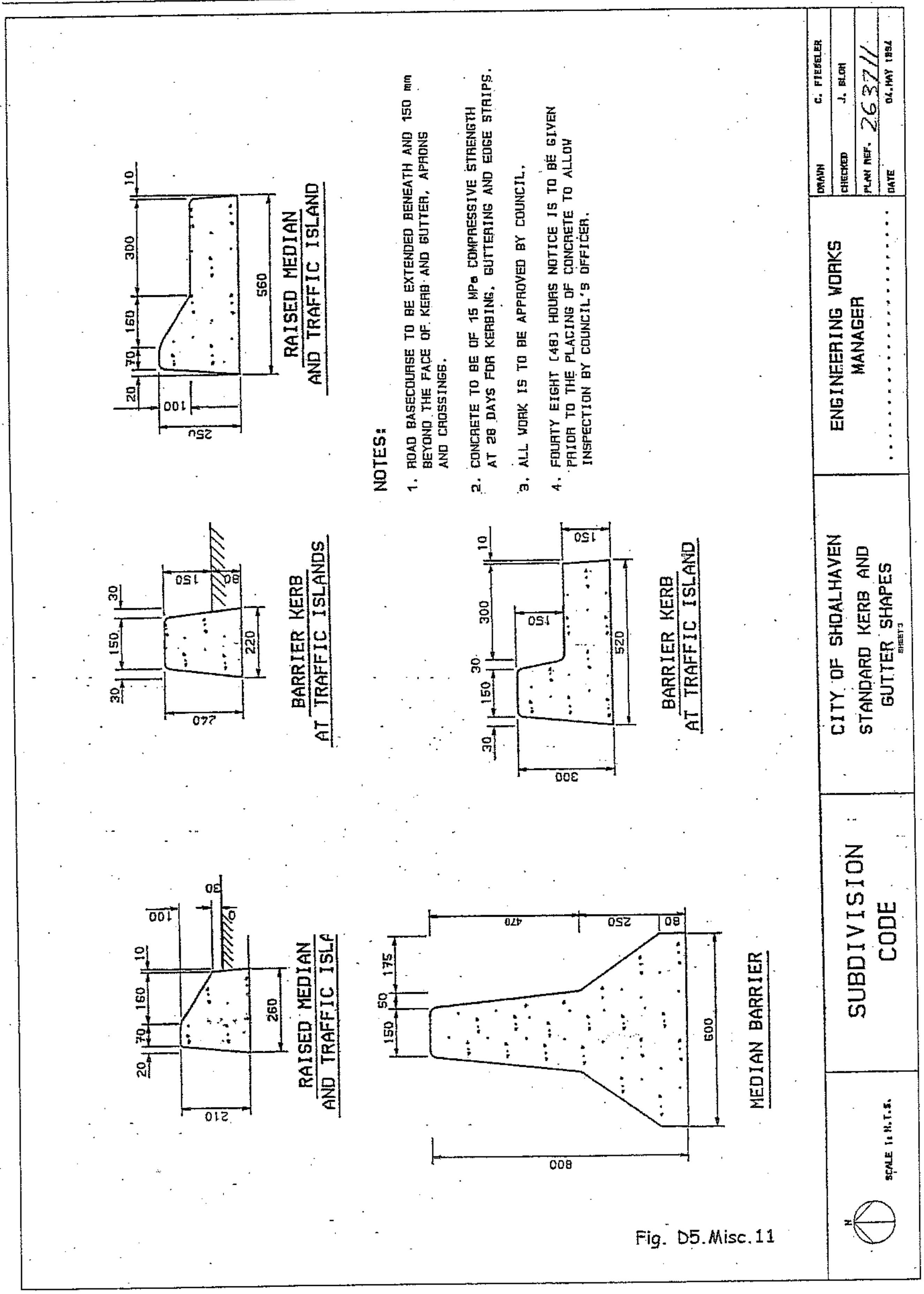


Fig. D5.Misc.11

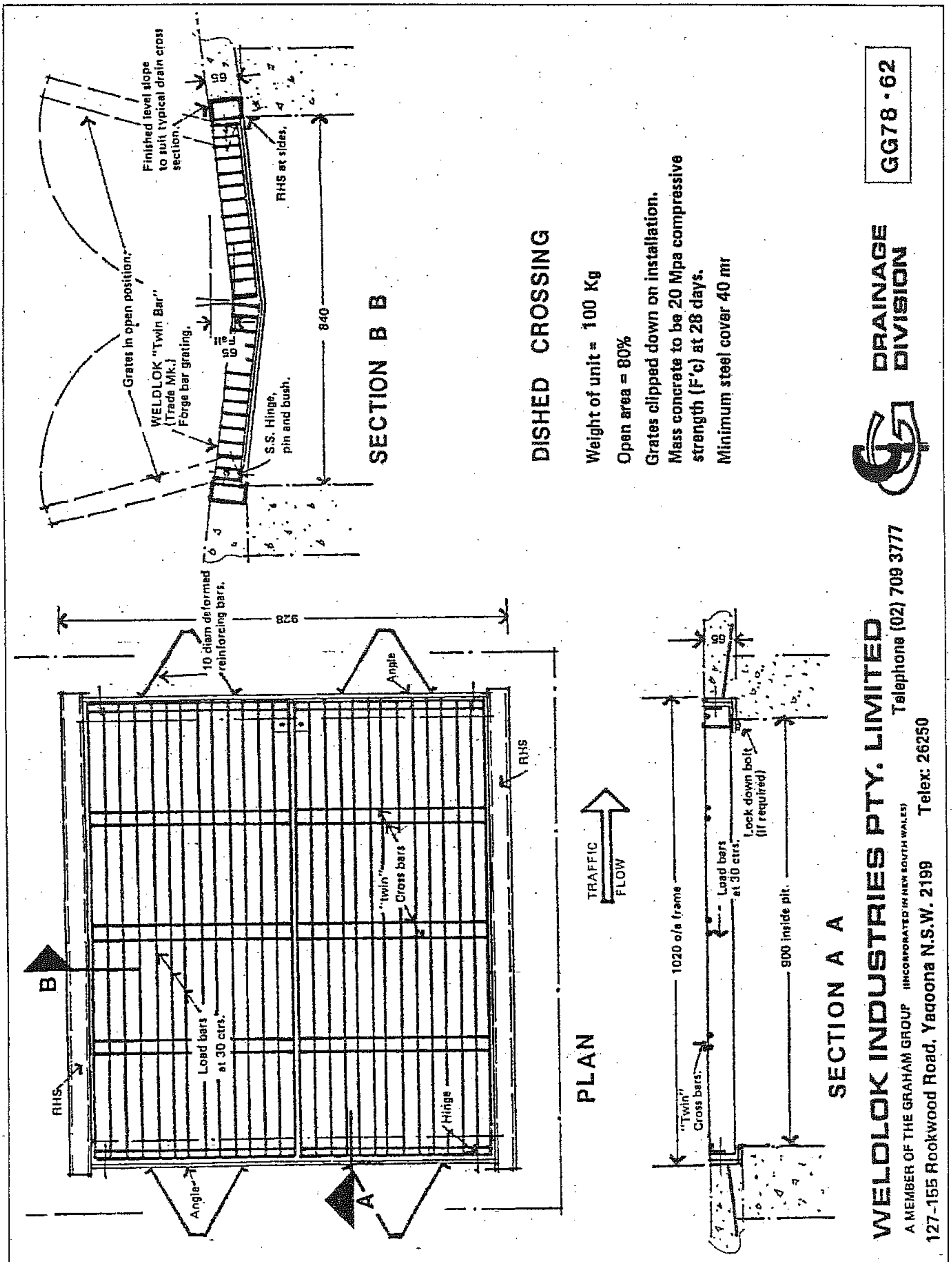
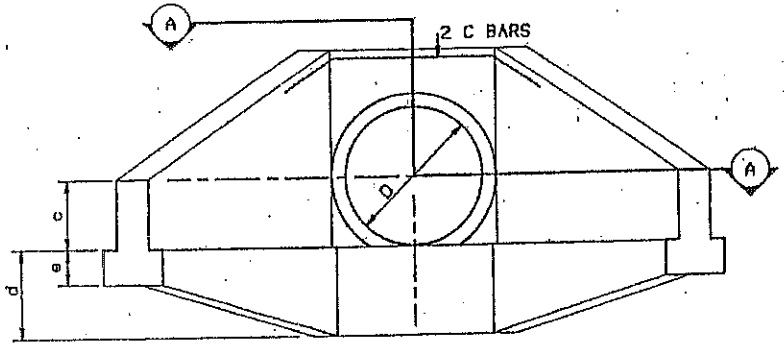
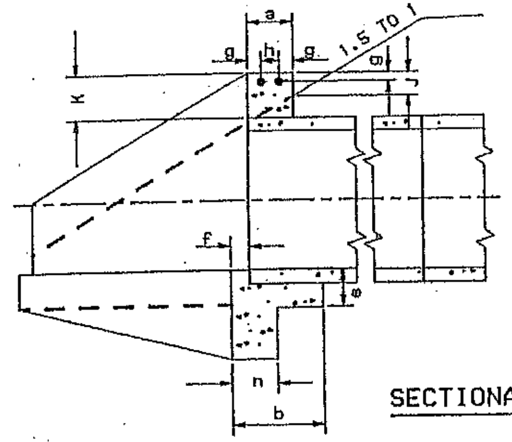


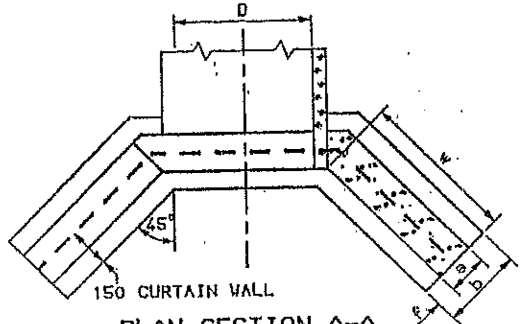
Fig. D5.Misc.12



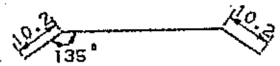
ELEVATION



SECTIONAL ELEVATION



PLAN SECTION A-A



C10 OR C12 BARS

2 REQUIRED EACH HEADWALL

PIPE DIAMETER D	300	375	450	525	600	675	750	825	900
a	150	150	150	150	180	180	205	205	230
b	300	300	300	300	450	450	450	450	450
c	300	300	300	300	380	380	380	380	380
d	380	380	380	380	530	530	530	530	530
e	150	150	150	150	180	180	205	205	230
f	75	75	75	75	100	100	100	100	100
g	40	40	40	40	50	50	50	50	50
h	70	70	70	70	80	80	105	105	130
J	100	100	100	100	100	100	100	100	100
k	200	200	200	200	250	250	250	250	250
n	150	150	150	150	150	150	150	150	150
W	690	690	840	990	1120	1300	1450	1600	1780
L	800	840	915	990	1100	1200	1250	1350	1400
REINF. DIA.	10	10	10	10	12	12	12	12	12
REINF. LENGTH	1600	1680	1830	1980	2200	2400	2500	2700	2800
REINF., kg MASS	0.986	1.035	1.127	1.220	1.954	2.131	2.220	2.398	2.486
CONC. VOLUME m	0.25	0.27	0.33	0.38	0.67	0.85	1.02	1.22	1.40

NOTES:

1. ALL EXPOSED SURFACES TO HAVE 25 CHAMFER.
2. REINFORCING BARS TO BE COLD WORKED DEFORMED.
3. CONCRETE COMPRESSIVE STRENGTH (F'c) 20 MPa AT 28 DAYS.
4. PRECAST CONCRETE HEADWALLS OF SIZES 375 TO 750 (INCL.) MAY BE SUBSTITUTED FOR THE EQUIVALENT SIZE IN-SITU HEADWALL.

Fig. D5.Misc.13

N SCALE 1: N.T.S.	SUBDIVISION CODE	CITY OF SHOALHAVEN STANDARD CONCRETE HEADWALLS FOR 300 TO 900 DIA.PIPES	ENGINEERING WORKS MANAGER	DRAWN C. FIESELER CHECKED J. BLOM PLAN REF. 263714

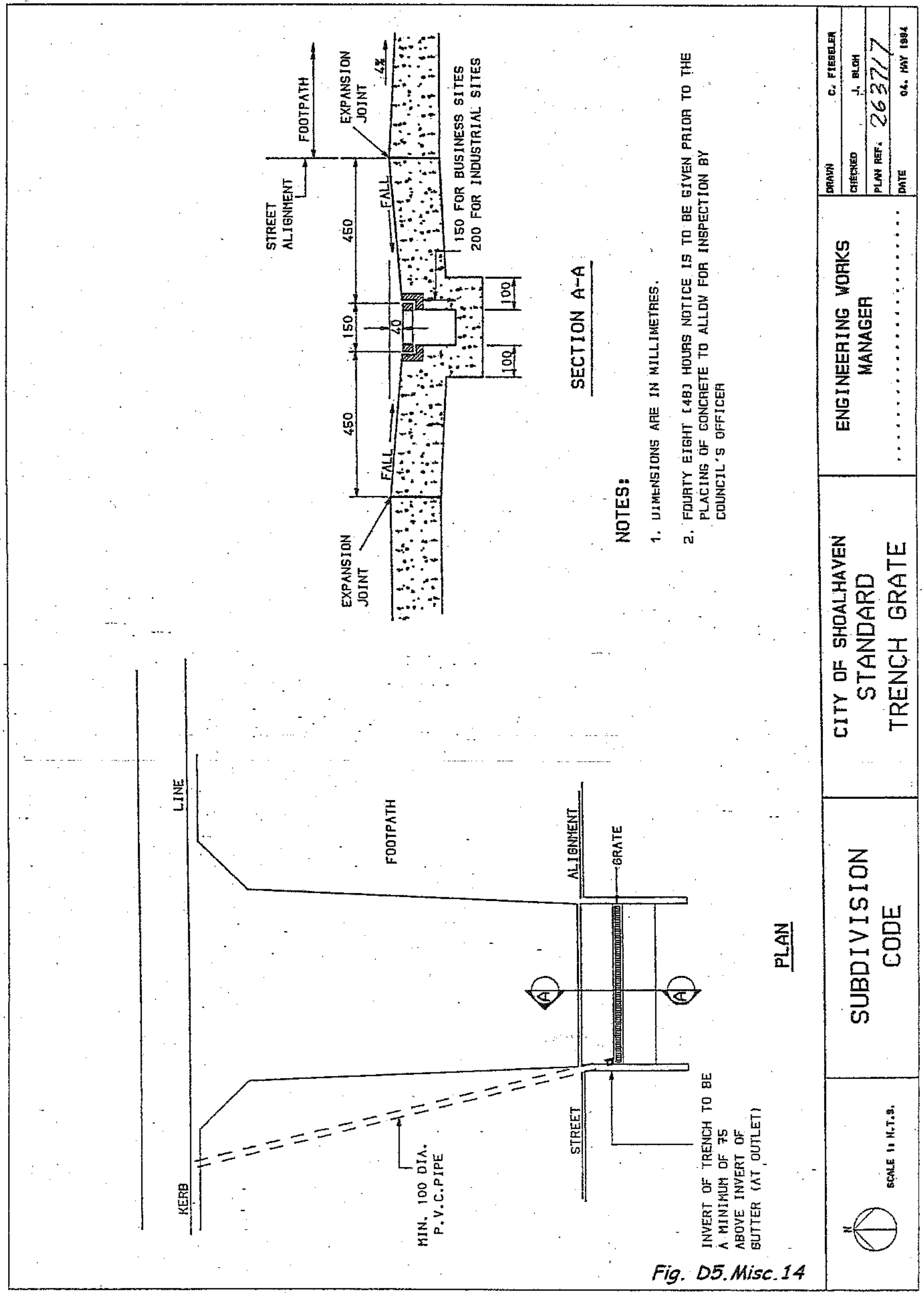


Fig. D5.Misc.14

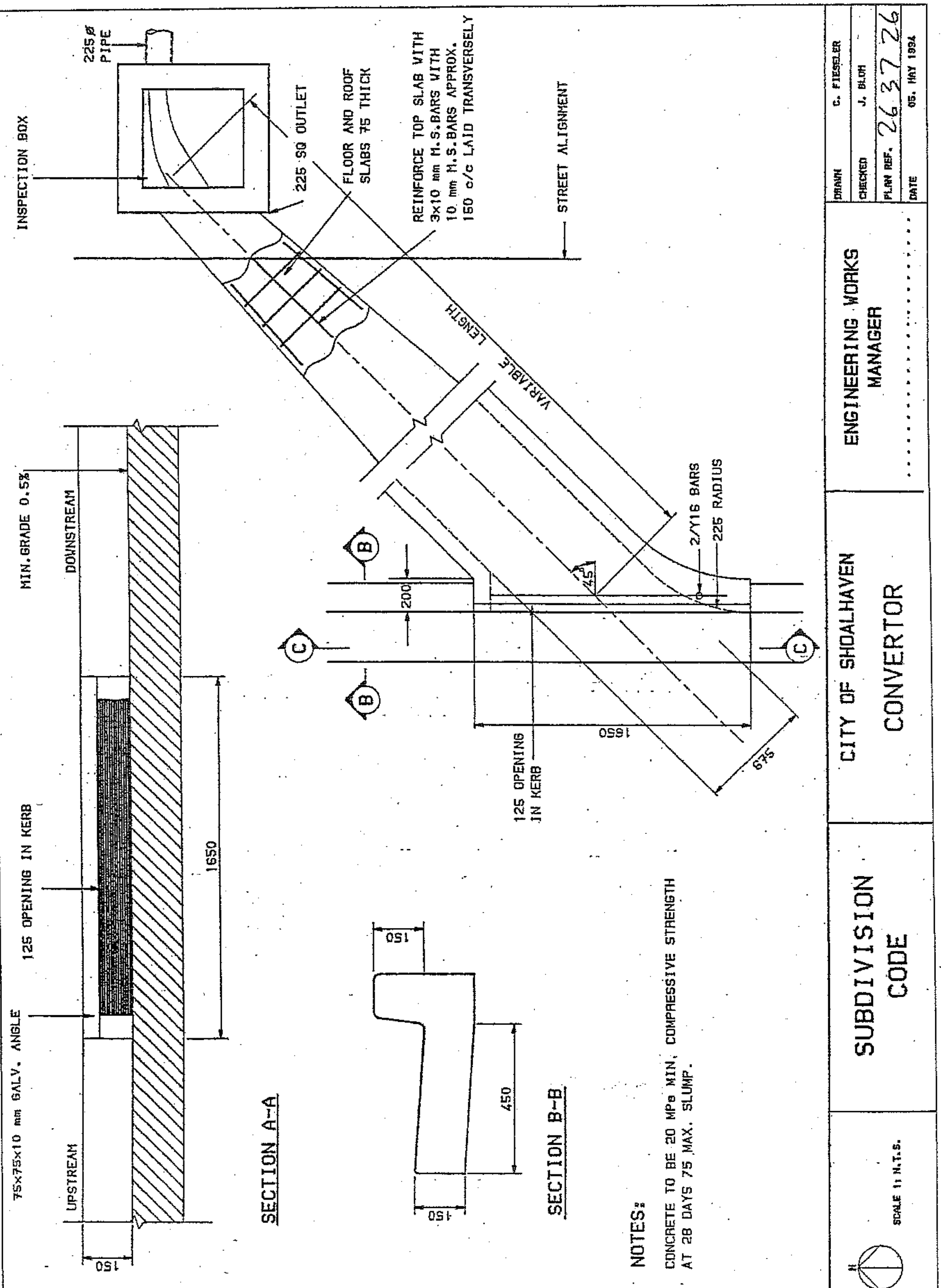


Fig. D5.Misc.15

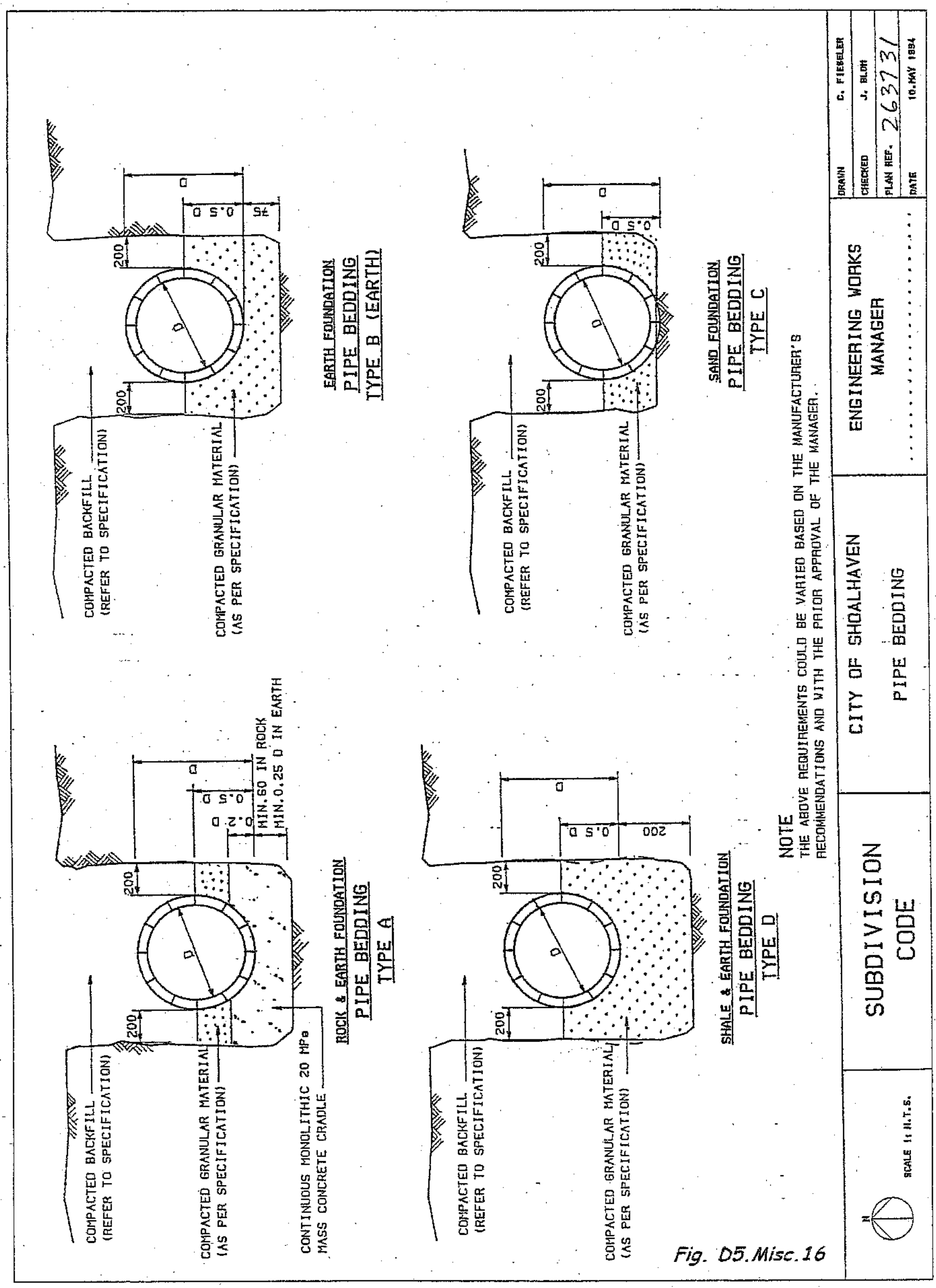


Fig. D5.Misc.16