HANDBOOK ON DRAINAGE DESIGN GUIDELINES

December 2013

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1. Introduction

This guideline must be read in conjunction with the following Council documents:

- Lake Macquarie Development Contol Plan 2013 (DCP 2013),
- Section 0074 Stormwater Drainage (Design) of Part 1 of the Engineering Guidelines,
- Part 3 of the Engineering Guidelines (Stormwater Quality Improvement Device Guidelines), and
- The drainage drawings of Part 6 of the Engineering Guidelines (Engineering Standard Drawings).

The handbook provide the requirements and guidance for the drainage of public assets or infrastructure that will become a public asset. For private developments, Section 2 provides detailed guidance to supplement the information in DCP 2013 whilst the tables and graphs in the appendices listed in Section 4 provide Lake Macquarie City Council specific data that must be used in the analysis.

2. Design Guidelines for On-site Stormwater Management

The following guidelines are to be adopted for Residential Development, Industrial and Commercial Developments. Where works are proposed to become public assets and handed over to Council, the works shall be designed in accordance with Part 1 of Council's Engineering Guidelines (Section 0074 Stormwater Drainage (Design)) for subdivision and Development. These guidelines are to be used in conjunction with DCP 2013. Reference is made to the Hunter and Central Coast Regional Environmental Strategy (HCCREMS) practice notes 1-11 (2007).

DCP 2013 prescribes a step-wise approach to the application of on-site storm water management based on the number of dwellings (on equivalent tenements).

These plans may be prepared, based on guidelines and practice notes, by a competent designer.

For larger developments, a detailed design is to be prepared and certified by an engineer suitably experienced in the relevant field and who has or is eligible for NPER registration with Engineers Australia. Certain specialist in-puts may also be required depending on the solutions proposed, such as a Geotechnical Engineer, Landscape Architect, etc.

2.1 Site Discharge Index

Ref HCCREMS Practice Note 11.

Site Discharge Index (SDI) is defined as the ratio of the impermeable area that drains directly to a drainage system (DC) to the total site area (S).

$$SDI = DC / S$$

DCP 2013 sets a performance criteria of 0.1 (10%) for the maximum allowable SDI (Practice Note 11 explains how to determine the SDI in some detail).

Therefore, 90% of stormwater runoff from any site must be managed through suitably designed stormwater source controls. (Note that a stormwater detention system is not to be regarded as a "source control" when calculating SDI). These guidelines assist in the design of these controls, porous paving, infiltration devices, water tanks and associated water re-use. The guidelines supplement HCCREMS Practice Notes 1-11.

2.2 General Principles

All designs should:

- Consider the local constraints, eg. catchment lie of the land, soil types, geotechnical conditions, adjoining development, available drainage capacity and easements and vegetation.
- Manage stormwater so that there is no nuisance flows to adjoining properties.
- Cater for runoff from outside the site and from adjoining properties, ensuring that outside runoff bypasses the onsite detention storage (if applicable). The design shall ensure that no works such as; filling, retaining walls etc. will impede the flow from upstream properties.
- Conveyance of the 100 year ARI runoff onto and from the site must be considered, with particular regard to the effect on site floor levels and any off site structures downstream.
- Consultation with Council's Development Assessment and Compliance Department should

be made to determine design criteria for individual drainage systems and to establish whether Council has carried out investigations and designs in the vicinity, or has a local management plan.

2.3 General Requirements

1. Roof water and site drainage is to be sized in accordance with the Building Code of Australia and AS/NZS 3500.1:2003.

2. If water supply is supplemented by interconnection with a reticulated system operated by a water supply authority, backflow prevention is to be provided in accordance with AS/NZS 3500.1:2003.

3. Noise emissions from any pumps shall not exceed 5dB(A) above ambient background noise level measured at the allotment boundary.

4. Habitable floor levels shall be shown to be 300 mm clear of the storage level of any above ground detention basins.

5. Drainage Pits for the collection of surface runoff are to be constructed with gratings or controlled entries to prevent foreign material from entering pipes.

6. The laying of stormwater pipes beneath buildings will not be permitted unless it can be demonstrated that no alternative exists. Any such pipes shall be rubber ring RCP or sewer grade PVC, have no bends, and have an inspection pit at each end.

7. Pipes under trafficable areas are to be **sewer grade**.

8. Council will not permit the discharge of stormwater directly into kerbing and guttering or table trains for any development other than that of a minor nature. NB. Council does allow up to **2x100mm** diameter pipes for **20** metres of frontage, to discharge to kerb. The velocity of the discharge at the kerb shall not exceed 1.5 metres/sec.

9. Where necessary, the developer shall extend a piped system from a development to a piped or open Council drain, a natural watercourse or an inter-allotment drain to dispose of stormwater from the site.

10. It is the responsibility of the developer to investigate the available points of connection to Council drains and points of discharge.

11. Full design calculations and plans are to be submitted to Council for drainage works within any existing or proposed Council drains or natural watercourses and for onsite management systems (for sites larger than 2 equivalent tenements).

12. Drainage lines from pits collecting runoff from parking and accessway areas are to be a diameter of not less than 150mm. The minimum grate and pit dimensions are to be in accordance with Table 2 below.

13. Pipe sizes may be determined from Table 1 – Pipe Chart – as being the size of the pipe required to drain an impervious area indicated in square metres assuming that the pipes are laid at a minimum grade of 1%.

14. Pipes laid at less than the 1% grade stipulated above are to be accompanied by design calculations and should be designed for ease of flushing.

15. Pipes are to be laid with 450mm minimum cover under areas used by vehicular traffic and

300mm minimum cover elsewhere within the site. Minimim cover should also comply with AS 3500:2003.

Table 1 – Pipe Chart

Impervious Area Served (m^2)	Pipe Size (mm) (Approx. only)
0 – 150	100 PVC-U
150 – 500	(150 PVC-U)
500 – 1,000	(225 PVC-U)
1,000 – 2,000	300 RCP (250 PVC-U)
2,000 +	By Calculation

Table 2 - Minimum Internal Dimensions for Stormwater Pits (Table 8.2 AS3500.3:2003)

Depth to Invert of Outlet (mm)		Pit Minimum Internal Dimensions (mm)		
		Rectangular Pits		Circular Pits
		Width	Length	Diameter
	< 600	450	450	600
> 600		600	600	900
> 900		600	900	1000
> 1200*		900	900	1000
* Step Irons shall be provided for pits with depths exceeding 1200 mm				

2.4 Device Specific Requirements

2.4.1 On-site Stormwater Detention (OSD)

For residential developments larger than a dual occupancy and for all commercial and industrial developments, Council may require on site detention of stormwater.

Detention Systems

A number of on-site detention systems will be considered by Council. These may include in-ground tanks, aboveground tanks, above ground depressions in driveways or landscaped areas etc. The following list details some items that should be considered in design solutions.

In-Ground Tanks

- Ease of access for maintenance and visual inspection shall be a design consideration. Access shall be available via a lockable grate (min 600 x 600mm). For tanks deeper than 1.2 metres step irons shall be provided.
- Tanks shall incorporate at least a grate at each end of the tank for ventilation and ease of cleaning. The preferred location for the grates would be above the inlet and outlets pipes.
- Any orifice for the control of discharge from the tank shall be screened to prevent blockage of the orifice.
- A high-level relief overflow pipe shall be provided to cater for flows in the event that the main outlet from the tank becomes blocked.
- Tanks are not to store water permanently unless specifically designed for water reuse applications, and having a dedicated detention volume.

Above Ground Storages in Driveway & Parking Areas

- Shall not exceed 200mm in depth. Care should be taken to avoid inundating disabled parking spaces.

Above Ground Storages in Landscaped Areas

- Generally should not exceed 300mm for storage depth. For detention basins deeper than 300mm, or basins that also store permanent water, personal safety measures (fencing, low batter slopes etc.) shall be incorporated. Reference should be made to Part 5 of Engineering Guidelines – “Batter and Fencing Guidelines for SQIDs and Detention Basins”.
- Shall be located in areas of high visibility to ensure regular maintenance
- Shall be designed to include measures to discourage unauthorised future modifications. Regardless of this the volume shall be increased by **20%** over the design volume to cover losses caused by vegetation growth, topsoiling etc.

Rainwater Tanks

- Rainwater tanks and the reuse of site water is encouraged in residential development. These systems will be assessed by Council on a site-specific basis. OSD capacity may be accommodated by a dedicated detention volume.

- Where a development contains rainwater tanks that have been sized to comply with BASIX, a credit of 20% of the rainwater tank(s) volume may be given so as to reduce the size of the dedicated OSD facility.

Design Guidelines for OSD Systems

The following guidelines are acceptable design procedures for the computation of OSD systems with storage volumes less than **$50m^3$.** If this method produces a storage volume of greater than **$50m^3$,** a recognised routing method must be used.

1. Inflow

Inflow shall be based on runoff from the developed site.

A **1 in 20** year average recurrence interval storm shall be adopted for inflow calculations. (Q^{20}_{dev})

2. Outflow

The maximum outflow from the basin shall be computed on a 1 in 5 year average recurrence interval storm, based on runoff from the undeveloped site. (Q^{5}_{und}). The undeveloped site is the site in its *natural* state. ie 0% impervious.

3. Time of concentration

The time of concentration (t^c) is the the time needed for water to flow from the most remote point in a watershed to the watershed outlet. It is used in predicting the response of a watershed to a given rain event.

Detention Volume Calculation.

The following formula can be used to estimate the detention volume.

$$\textbf{Detention Volume } (m^3) = (Q^{20}_{dev} - Q^{5}_{und}) \times t^{c}_{20\ dev} \times 0.06 \qquad \text{(eq 1)}$$

where:- Q^{20}_{dev} (litres/sec), Q^{5}_{und} (litres/sec) and $t^{c}_{20\ dev}$ (minutes)

NB. The flow from the site for the Q^{100}_{dev} shall be checked to ensure that it does not exceed the Q^{100}_{und}. The following formula may be used:-

$$Q^{100}_{dev} - Q^{20}_{dev} + \text{Detention outflow (normally} = Q^{5}_{und}) \leq Q^{100}_{und} \quad \text{(eq 2)}$$

If the above equation is not satisfied then generally the detention outflow may need to be further constricted with a corresponding increase in detained volume. The first check would be to assess the volume difference between the Q^{100}_{dev} & Q^{100}_{und}.

$$\text{ie Vol} = (Q^{100}_{dev} - Q^{100}_{und}) * 0.06 * \times t^{c}_{100\ dev} \qquad \text{(eq 2a)}$$

Restraints

I. Minimum surface fall shall be 1%;

II. Charged drainage systems will not be accepted in most instances (see “Section 2.5 – Alternate Discharge Systems” below);

III. Non-habitable floor levels shall be a minimum of 150mm above the stored water level or 100mm above the overflow level, which ever is greater;

IV. Notwithstanding I, II and III the standards outlined in AR & R 1987 shall be adopted, in regard to detention systems.

Overflow

All storages shall be provided with an approved overflow facility to cater for controlled discharge of waters in excess of the design event. Overflows shall be designed to cater for flows up to the 1 in 100 year average recurrence interval storm. Any overflows shall not exceed the 100 year undeveloped flow from the site.

2.4.2 Rainwater Tanks – Roofwater

(Ref: HCCREMS Practice Note 4).

The tank capacity should be designed to suit the catchment and reuse demand. Council is investigating suitable design models, and sourcing local data for input. As a guide a typical house will require a tank capacity between 3,000 to 6,000 litres.

If rainwater is to be reused for household non-potable in-house use such as toilets and hot water systems, the following requirements apply:

Supply source

- Rainwater is sourced only from roof surfaces.
- If supply is supplemented by interconnection with a reticulated system operated by a water supply authority, backflow prevention is provided in accordance with Australian Standard AS 3500.1:2003.

Treatment and use

- The collection system incorporates an effective first flush device for removing roof surface contamination. This should cater for the first 1mm of rainfall.
- The tank system is connected to toilet, laundry, external tap etc in accordance with the BASIX requirements for the development.
- There is no connection to other indoor fixtures unless the supply is treated to potable standard by an approved purification system.

Noise

- Noise emissions from any pumps do not exceed 5dB(A) above ambient background noise level measured at the allotment boundary.

Recommended sources for design and best practice

- Refer to HCCREMS Water Smart Practice Note No. 4.

2.4.3 Stormwater Tanks

Water tanks designed for collection of surface runoff must satisfy the following requirements:

Supply source

- Rainwater is sourced from a combination of roofs, driveways, paved surfaces or grassed areas.

- There is no interconnection with a reticulated system operated by a water supply authority.

Treatment and use

- The collection system incorporates suitable treatment measures, such as a first flush pit or a sand/gravel filter.

- The tank system is connected to irrigation or other outdoor fixtures, but is not connected to indoor water fixtures.

- All fixtures connected to the supply system are marked 'NOT SUITABLE FOR DRINKING'.

Noise

- Noise emissions from any pumps do not exceed 5dB(A) above ambient background noise level measured at the allotment boundary.

2.4.4 Infiltration

Infiltration devices must be properly designed to perform satisfactorily and to avoid adverse impacts on adjoining development.

For properties that drain to the rear (and/or over adjacent properties) infiltration systems may be considered if it can been shown that other stormwater disposal methods have been considered and are not possible or practical i.e. the preferred disposal method would be for a drainage easement to be obtained over downstream properties.

Where conditions are otherwise suitable (Refer Tables 3 and 4) and the site falls towards adjoining properties, the device is to be a minimum of 5m from the boundary (normal to the contours) and located to comply with the minimum distances to footings in accordance with Table 4. A typical layout is shown in the Engineering Guidelines Standard Drawings (EGSD-407).

The following requirements apply:

- Table 3 sets out unsuitable conditions for infiltration devices.

- The infiltration trench shall be designed by a competent Engineer.

- Geotechnical site testing shall be undertaken by a Geotechnical Engineer to establish the permeability of the soil and whether the site is suitable for infiltration in regard to slope stability.

- The trench system shall be designed to infiltrate the runoff from the site for all storm durations up to and including the **20 year** ARI with no surcharge onto neighbouring propoerties. A suitable time-area computer model shall be used such as ILSAX or Drains.

- The trench system shall be designed so as to accommodate a site impervious percentage of at least 80% to allow for a future, more intensive use of the site.

- The trench system shall be designed so as to avoid clogging by sediment and leaf litter and allow for cleaning and inspection. (see the attached typical diagrams). Regardless or any

systems used to mitigate clogging of the system, the system shall be designed to be al least 20% oversize.

- Should be located an adequate distance from existing, or possible future, foundation locations and adjacent property boundaries. Refer to Table 4.

Table 3 Unsuitable conditions for infiltration trenches
Loose sands
Heavy clays
Shallow soil over rock or shale
Steep terrain (slopes > 10%)
High water table (depth <1 metre blow surface)
Contaminated sites

Table 4 Minimum separation between infiltration trenches and building		
Soil Type	**Hydraulic conductivity**	**Distance to footings**
Sand	> 180 mm/hr	1 metre
Sandy clay	180 – 36 mm/hr	2 metres
Medium clay	36 – 3.6 mm/hr	4 metres
Reactive clay	3.6 – 0.036 mm/hr	5 metres

2.4.5 Porous Paving

Part 3 of the Engineering Guidelines provides guidelines for the design of Stormwater Quality Improvement Devices (SQIDs). These guidelines prohibit the use of porous pavers in assets intended for handing over to Council.

For other applications, porous paving should satisfy acceptable standards for site suitability, installation, maintenance and protection from material likely to hinder performance.

Site selection and protection from sediment

- The area to be paved does not receive high vehicular traffic volumes or regular use by heavy vehicles.
- The area to be paved is not located immediately downstream from areas likely to contribute significant amounts of sediment, debris or windblown material.
- Sediment traps, vegetated filter strips or specially designed gutter systems are installed

upstream of porous paving so as to reduce sediment inputs and minimise likelihood of clogging, particularly during the construction phase.

- The maximum grade for porous paving area is approximately 5% (refer to manufacturers technical information).

Recommended sources for design and best practice

- Refer to HCCREMS Water Smart Practice Note No. 6.

2.4.6 Runoff Controls for Streets and Carparks

Runoff from streets, car parks or other extensive paved areas is adequately treated before discharge to an infiltration device, piped drainage system or natural waterway.

Treatment measures

- A treatment system is designed and installed that incorporates one or more of the following measures:
    - Gross pollutant trap
    - Sand/gravel filter
    - Grassed swales
    - Vegetated filter strip
    - Constructed wetland

Recommended sources for design and best practice

- Refer to HCCREMS Water Smart Practice Note No. 7.

2.5 Alternate Discharge Options

2.5.1 Interallotment Drainage Easements

Interallotment Drainage Easements

An interallotment drainage easement is required to be created wherever it is necessary to convey stormwater across lands, other than the development site, to gain access to the public drainage system or a natural watercourse.

- Council will not approve a Development Consent where an easement is required for drainage until:-
    - Evidence of a written agreement from the relevant landowners has been provided to Council
    - A preliminary stormwater plan has been assessed by Council
- Council will not approve the corresponding Construction Certificate until:-
    - Evidence has been provided to Council that the easement has been registered with NSW Land and Property
- Where an easement has been created on a downstream property for a development that is required to have on site detention, the pipe in the easement shall convey the 100 year discharge from the development site. This would typically be the 100 year developed flow less the detained flow.

$$\textbf{Flow in easement pipe} = (Q^{100}_{dev} - Q^{20}_{dev} + Q^{5}_{und}) \qquad \text{(eq 2b)}$$

2.5.2 Pump Systems

The use of pump systems is discouraged by Council, however Council may permit pump systems under the following circumstances:-

- Pump Systems design shall be in accordance with AS3500.3:2003 Section 9 except for those matters below.
- Evidence shall be provided to Council that all avenues to secure easements through downstream properties has failed. Copies of letters to and from adjacent landowners, that include details of offers of financial compensation, would be the minimum evidence required.
- The detention (storm) storage required (ie the size of the storage tank) shall be the runoff volume from the site in the 10 Year ARI 2 hour storm (for LMCC = 32.25 mm/h).
- This required Storm Storage Volume (SSV) may be reduced by the volume that the associated pump system can discharge from the storage tank in a 30 minute period (Note that the pump discharge rate shall be limited to a maximum of the 5 year undeveloped flow from the site).
- The reduced volume, however, cannot be less than 25% of the initial calculated Storm Storage Volume so as to allow for some storage volume in case of pump failure.
- The discharge from the pump line at the kerb shall not exceed the **5** year ARI undeveloped

flow from the site.

- The pump line shall terminate within the property at a stilling pit from which drainage is by gravity (where possible) to the kerb. This is to reduce flows to acceptable velocities at the kerb (max 1.5 m/s).
- Pumped stormwater disposal systems shall be designed using **2** pumps operating alternatively.
- An Inspection and Maintenance schedule shall be incorporated into the Strata By-Laws or Management Statement (if Community Title).
- The design of the system shall be such that if overflow does occur, the overflow would be spread to mimic natural sheet flow.

2.5.3 Charged Systems

- Charged systems are generally not acceptable for developments larger than one single dwelling.
- The system must be completely sealed with an inspection/cleaning eye at the lowest point.
- The roof gutter must be no less than 2 metres above the corresponding kerb.
- The charged system can only convey roof stormwater, therefore approved alternate disposal would need to be provided for driveway and other hardstand areas.

3. Recommended Sources for Design and Best Practices

Item No	Reference
1	Lake Macqaurie City Council. Stormwater Quality Improvement Device Guidelines – Part 3 to the Engineering Guidelines for DCP 2013.
2	Lake Macquarie City Council. Water Cycle Management Guidelines – Guidelines for DCP 2013.
3	The Australian Government (1995). AMCORD Practice Notes. PND 15 – Landscape Guidelines for Water Conservation. AGPS, Canberra.
4	NSW Environment and Heritage - Managing urban stormwater: soils and construction http://www.environment.nsw.gov.au/stormwater/publications.htm
5	NSW Environment Protection Authority (19987). Managing Urban Stormwater: Treatment Techniques. EPA, Sydney. http://www.environment.nsw.gov.au/resources/stormwater/usp/treattech.pdf
6	Geary Phillip Milton, Stafford David John, Whitehead Joseph Henry, (2005) On-Site Domestic Wastewater Treatment and Reuse, BEDP Environment Design Guide 1-12
7	Australian Standard AS 3500.1:2003.
8	Water Smart Practice Note No. 2 – Site Planning. HCCREMS 2007.
9	Water Smart Practice Note No. 3 – Drainage Design. HCCREMS 2007.
10	Water Smart Practice Note No. 4 – Rainwater Tanks. HCCREMS 2007.
11	Water Smart Practice Note No. 5 – Infiltration Devices. HCCREMS 2007.
12	Water Smart Practice Note No. 6 – Paving. HCCREMS 2007.
13	Water Smart Practice Note No. 7 – Landscape Measures. HCCREMS 2007.
14	Water Smart Practice Note No. 8 - Landscape Practices. HCCREMS 2007.
15	Water Smart Practice Note No. 11 – Site Discharge Index. HCCREMS 2007.

4. Appendices

1. Overland Flow Table
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4.1 Overland Flow Time – Table

Based on the Kinematic Wave equation

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

where:

t = overland travel time (min)

L = overland sheet flow path length (m)

n* = surface roughness/retardance coefficient

I = rainfall intensity (mm/hr)

S = slope of surface (m/m)

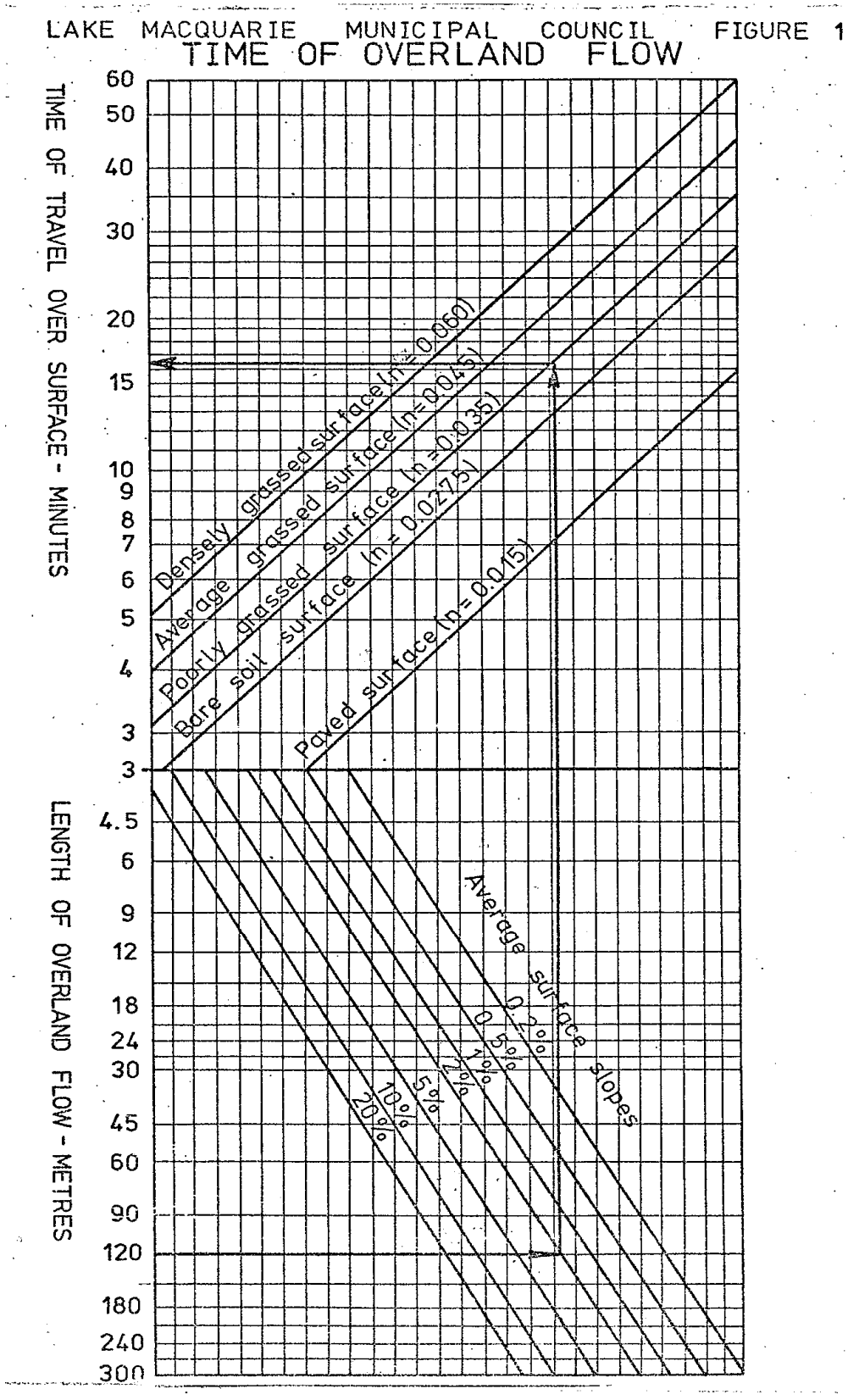
Typical values for *n** are: (As derived from ARR (1998))

Surface Type Horton's Roughness Coefficient *n**	
Concrete or Asphalt	0.010 – 0.013
Bare Sand	0.010 – 0.016
Gravelled Surface	0.012 – 0.030
Bare Clay-Loam Soil (eroded)	0.012 – 0.033
Sparse Vegetation	0.053 – 0.130
Short Grass Paddock	0.100 – 0.200
Lawns	0.170 – 0.480

1) Calculate $t\, I^{0.4} = (6.94(L \times n^*)^{0.6} / S^{0.3}$
2) Read overland travel time from table below
3) Note that is overland travel time occurs over several different segments, it is incorrect to add travel times for each segment. A more complex approach is required. You should refer to Australian Rainfall & Runoff.

Overland Flow Time	Value calculated from (1) above								
	1 yr	2 yr	5 yr	10 yr	20 yr	50 yr	100 yr	200 yr	500 yr
5 min	29.65	32.77	36.17	37.93	40.09	42.67	44.49	46.22	48.41
6 min	34.65	38.30	42.27	44.34	46.87	49.89	52.02	54.05	56.62
7 min	39.50	43.65	48.19	50.55	53.44	56.89	59.32	61.64	64.57
8 min	44.20	48.86	53.95	56.59	59.83	63.70	66.42	69.02	72.30
9 min	48.79	53.94	59.56	62.48	66.06	70.34	73.35	76.22	79.85
10 min	53.28	58.89	65.05	68.24	72.15	76.83	80.12	83.26	87.23
12 min	61.96	68.50	75.68	79.40	83.96	89.42	93.26	96.92	101.55
14 min	70.33	77.76	85.92	90.16	95.35	101.55	105.92	110.08	115.35
16 min	78.43	86.72	95.84	100.57	106.37	113.30	118.18	122.83	128.72
18 min	86.28	95.41	105.46	110.69	117.07	124.72	130.09	135.22	141.71
20 min	93.93	103.87	114.83	120.53	127.50	135.83	141.69	147.28	154.36
22 min	101.39	112.12	123.97	130.14	137.67	146.67	153.01	159.06	166.71
24 min	108.68	120.19	132.91	139.53	147.61	157.28	164.08	170.57	178.79
26 min	115.81	128.09	141.66	148.73	157.35	167.66	174.92	181.85	190.62
28 min	122.81	135.83	150.24	157.74	166.90	177.85	185.55	192.91	202.22
30 min	129.67	143.43	158.67	166.59	176.27	187.84	195.99	203.77	213.61
40 min	162.36	179.62	198.78	208.76	220.93	235.49	245.74	255.52	267.90

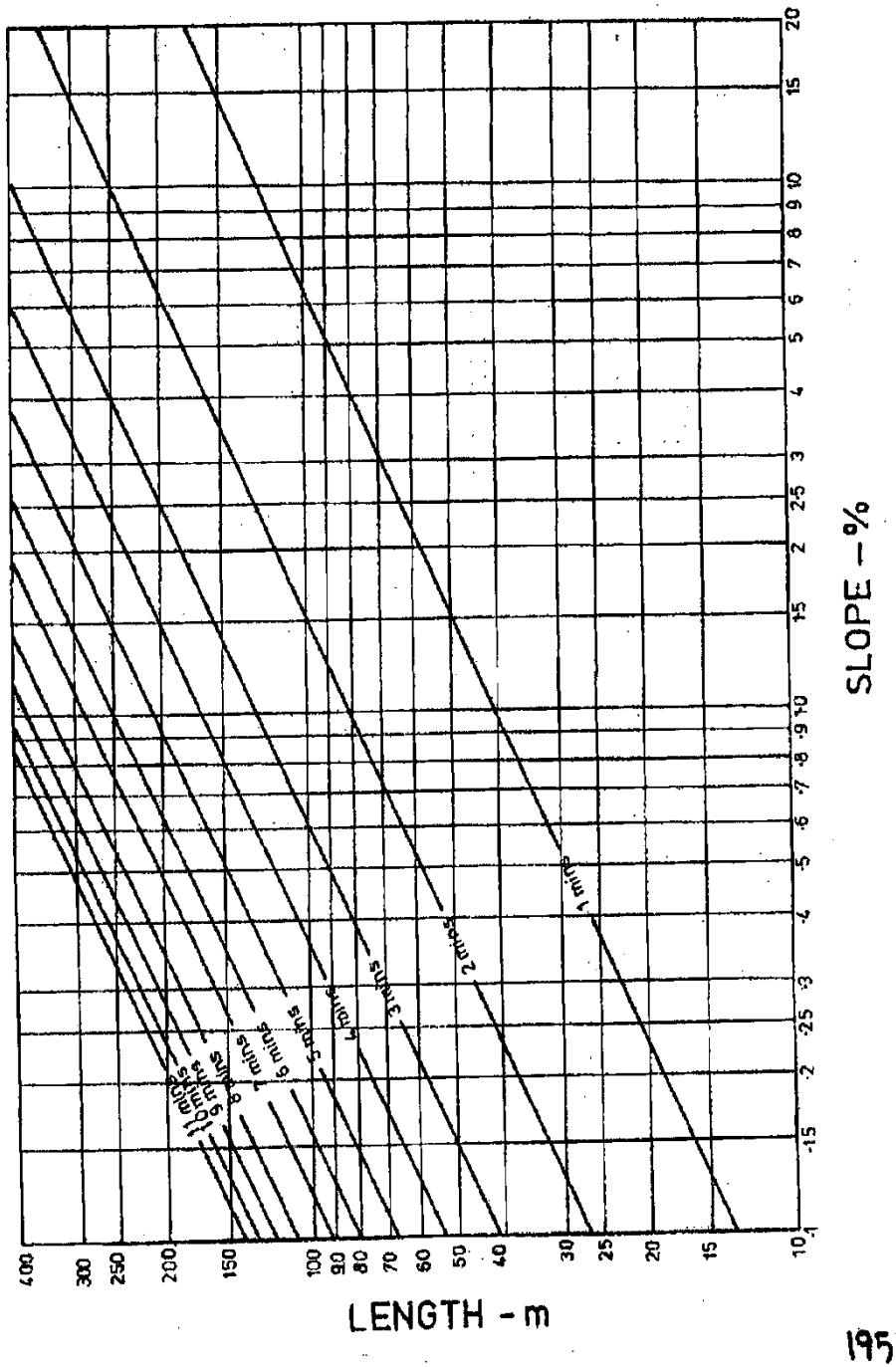
4.2 Overland Flow Time – Graph



4.3 Gutter Flow Time

FIGURE 2

GUTTER FLOW TIME



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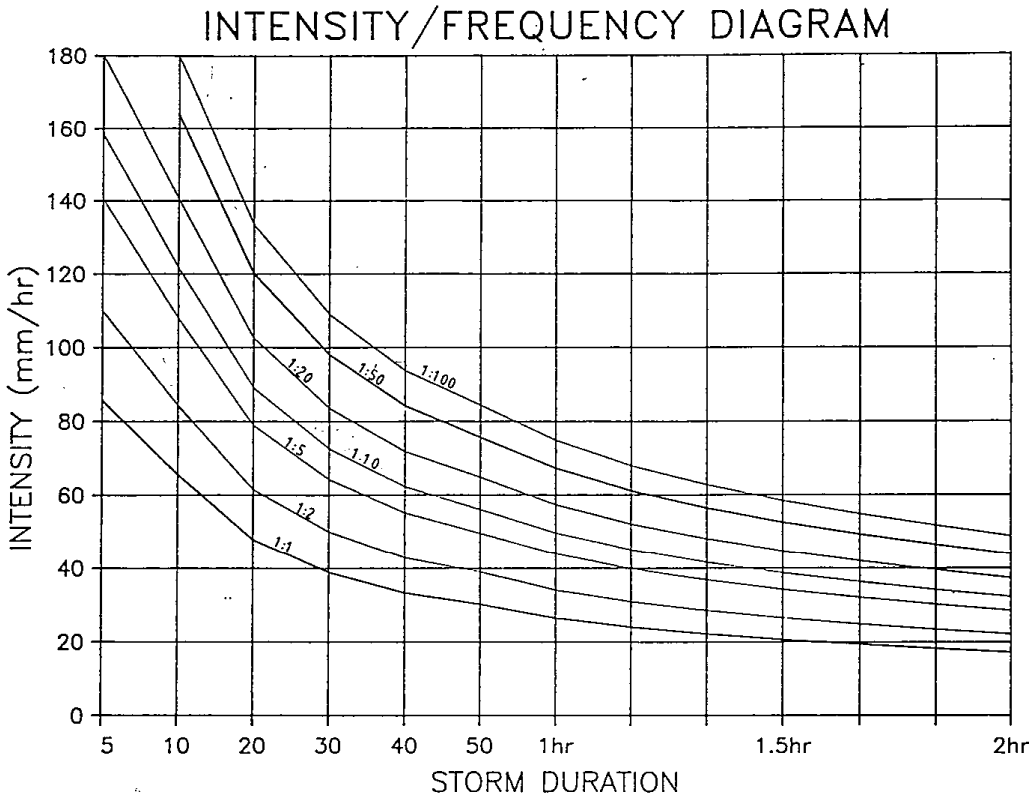
4.4 Intensity/Frequency/Duration Diagram



INTENSITY/FREQUENCY TABLE (AR&R 1987)

Duration	1 yr	2 yr	5 yr	10 yr	20 yr	50 yr	100 yr	200 yr	500 yr
5 min	85.60	109.90	140.63	158.37	181.91	212.65	236.01	259.66	291.55
6 min	80.20	102.99	131.85	148.52	170.63	199.51	221.46	243.69	273.66
10 min	65.59	84.27	108.04	121.79	140.01	163.83	181.94	200.29	225.05
12 min	60.63	77.91	99.94	112.70	129.59	151.68	168.48	185.50	208.47
15 min	54.79	70.44	90.42	102.00	117.33	137.37	152.63	168.08	188.95
18 min	50.25	64.61	83.00	93.66	107.76	126.21	140.26	154.49	173.71
20 min	47.73	61.38	78.87	89.02	102.44	120.00	133.37	146.92	165.22
24 min	43.55	56.02	72.03	81.33	93.62	109.70	121.95	134.37	151.14
30 min	38.77	49.89	64.20	72.52	83.51	97.90	108.86	119.97	134.99
45 min	31.07	40.00	51.57	58.30	67.18	78.81	87.68	96.68	108.85
1.0 hr	26.38	33.98	43.85	49.61	57.19	67.14	74.72	82.42	92.83
1.5 hr	20.51	26.43	34.14	38.64	44.57	52.35	58.28	64.30	72.45
2.0 hr	17.10	22.04	28.49	32.25	37.21	43.72	48.68	53.72	60.55
3.0 hr	13.19	17.00	22.00	24.92	28.77	33.82	37.67	41.58	46.88
4.5 hr	10.16	13.10	16.97	19.24	22.22	26.12	29.11	32.14	36.25
6.0 hr	8.44	10.89	14.12	16.01	18.50	21.76	24.25	26.78	30.22
9.0 hr	6.51	8.41	10.91	12.37	14.30	16.83	18.76	20.73	23.39
12.0 hr	5.42	7.00	9.08	10.31	11.92	14.03	15.65	17.29	19.51
18.0 hr	4.23	5.47	7.12	8.08	9.35	11.02	12.30	13.60	15.36
24.0 hr	3.55	4.58	5.97	6.79	7.86	9.27	10.35	11.44	12.93
30.0 hr	3.08	3.99	5.20	5.91	6.85	8.08	9.02	9.99	11.29
36.0 hr	2.74	3.55	4.63	5.27	6.11	7.21	8.05	8.91	10.08
48.0 hr	2.26	2.93	3.83	4.37	5.06	5.98	6.68	7.40	8.37

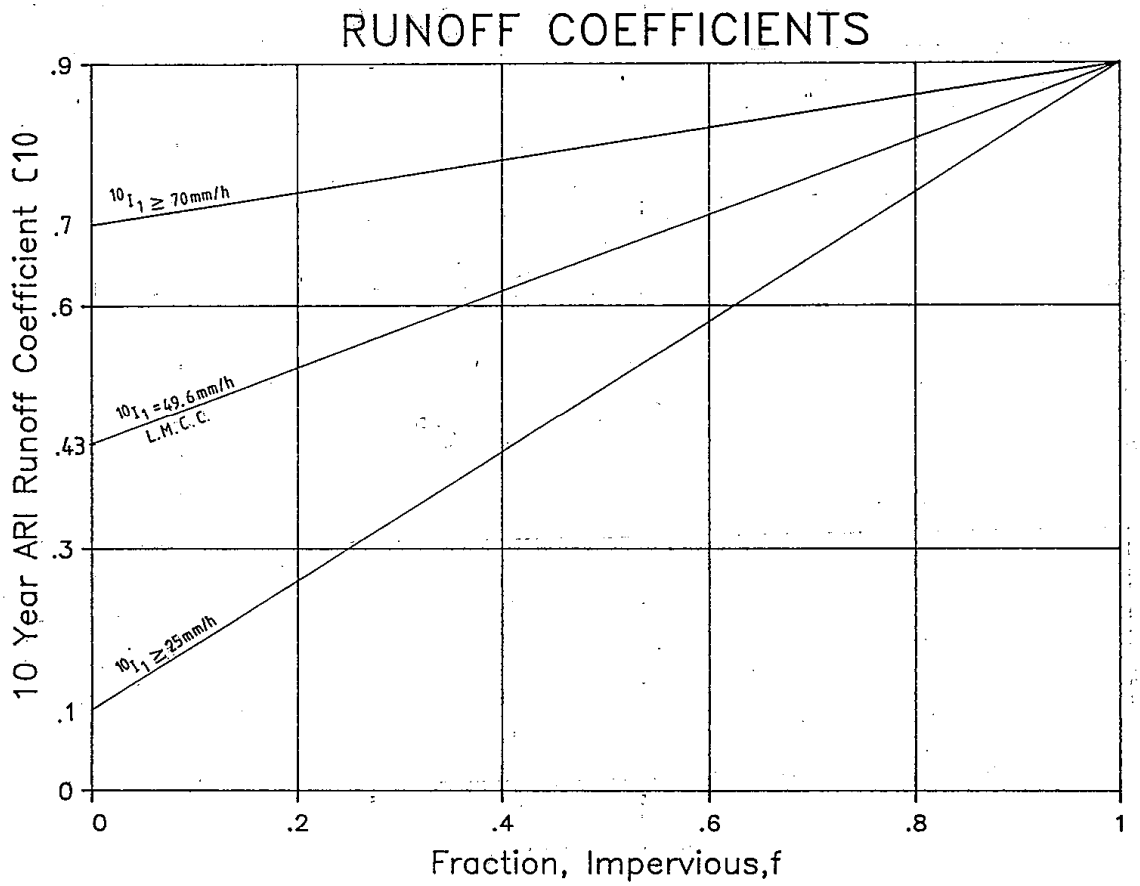
IFD Polynomial:

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

where duration D is in hrs and average intensity I is in mm/hr

ARI	a	b	c	d	e	f	g
1	3.2682154	-.5942423	-.0383643	.0074596	.0013082	-.0001793	-.0000427
2	3.5212629	-.5930203	-.0385677	.0074275	.0013389	-.0001770	-.0000439
5	3.7761122	-.5897489	-.0391105	.0073409	.0014211	-.0001706	-.0000469
10	3.8993162	-.5880349	-.0393949	.0072956	.0014641	-.0001673	-.0000485
20	4.0415185	-.5866177	-.0396301	.0072581	.0014997	-.0001645	-.0000498
50	4.2017359	-.5850211	-.0398950	.0072158	.0015398	-.0001614	-.0000512
100	4.3086945	-.5839552	-.0400719	.0071876	.0015666	-.0001594	-.0000522
200	4.4066745	-.5829788	-.0402340	.0071618	.0015911	-.0001575	-.0000531
500	4.5255373	-.5817943	-.0404305	.0071304	.0016209	-.0001552	-.0000542

4.5 Runoff Coefficients



RUNOFF COEFFICIENTS (AR&R 1987)

(1) Runoff coefficients are read from the above graph for the 10 year recurrence interval (C_{10}).

Note: Open space parkland f = 0.0
Roads and footpaths f = 0.85
Residential areas (ultimate) f = 0.35

(2) Read frequency factor (Fy) from table below.

A.R.I. (years)	Frequency Factor (Fy)
1	0.8
2	0.85
5	0.95
10	1.0
20	1.05
50	1.15
100	1.2

(3) Calculate runoff coefficient $Cy = Fy \times C_{10}$

4.6 Hydrological Design Sheet

Source – Department of Housing

PIPED URBAN STORMWATER DRAINAGE — HYDROLOGICAL DESIGN SHEET — SHEET ____ OF ____

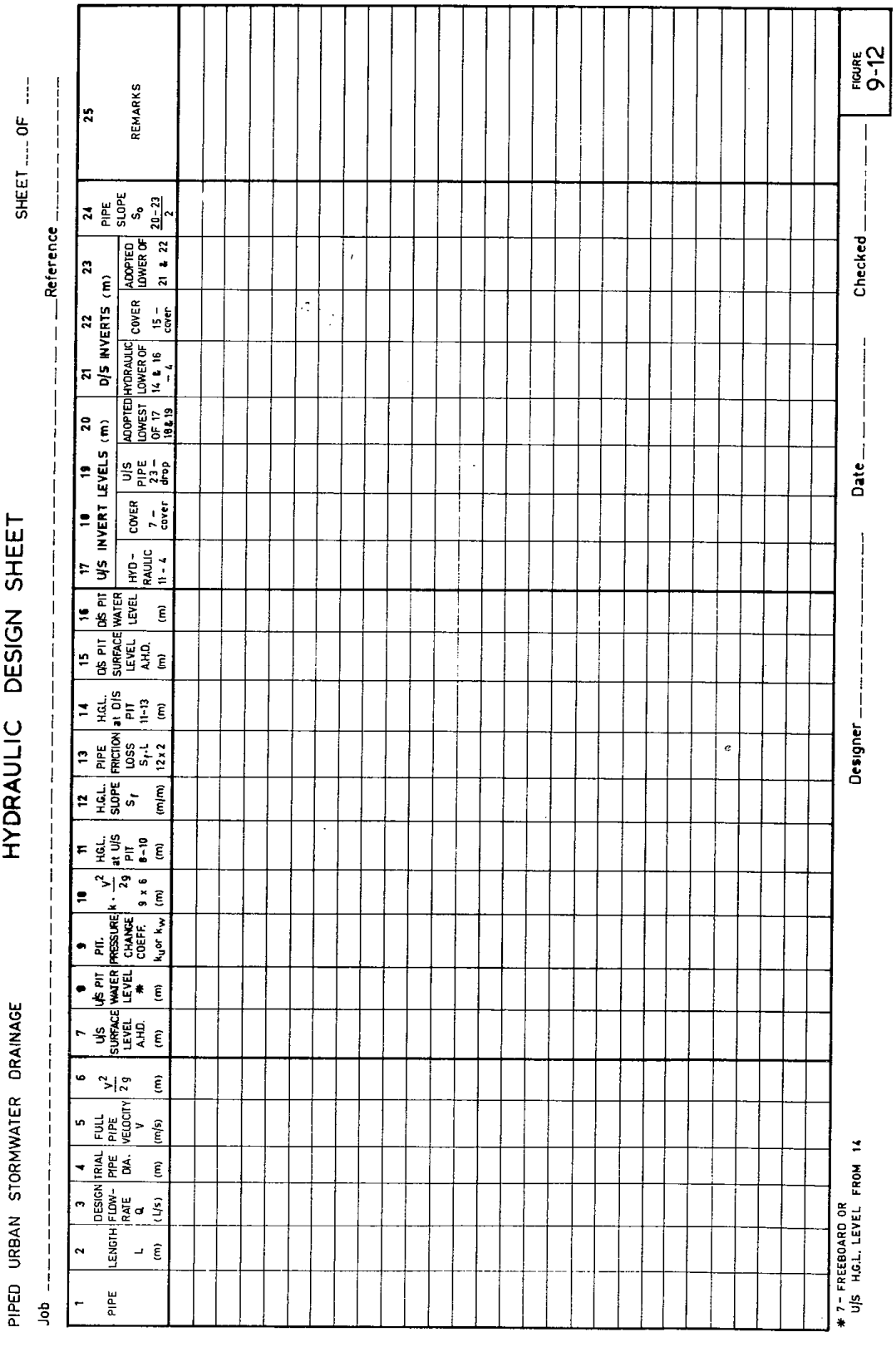
Job _____ Reference _____

1	2	3	4		5	6		7	8	9	10	11	12	13	14	15	16	17		18	19	20	21
PIT	LAND-USE TYPE	FLOW LENGTH (m)	FLOW SLOPE (m/m)		"n"	TIMES TIME (min)		TOTAL TIME (min)	INTENSITY I (mm/h)	RUNOFF COEFF C	AREA A (ha)	C.A. (ha) 9 x 10	Σ C·A (ha)	Q = C.I.A. (L/s) 8 x 12 /0.36	BYPASS (L/s) from Pit( )	ADOPTED FLOW RATE (L/s) 13+14	GUTTER SLOPE (m/m)	PIT FLOW WIDTH (m)	DEPTH (m)	INLET TYPE	INFLOW (L/s)	BYPASS FLOW (L/s) to Pit( )	REMARKS

Designer _____ Date _____ Checked _____

FIGURE 9-11

4.7 Hydraulic Design Sheet



PIPED URBAN STORMWATER DRAINAGE — HYDRAULIC DESIGN SHEET — SHEET ____ OF ____

Job _____ Reference _____

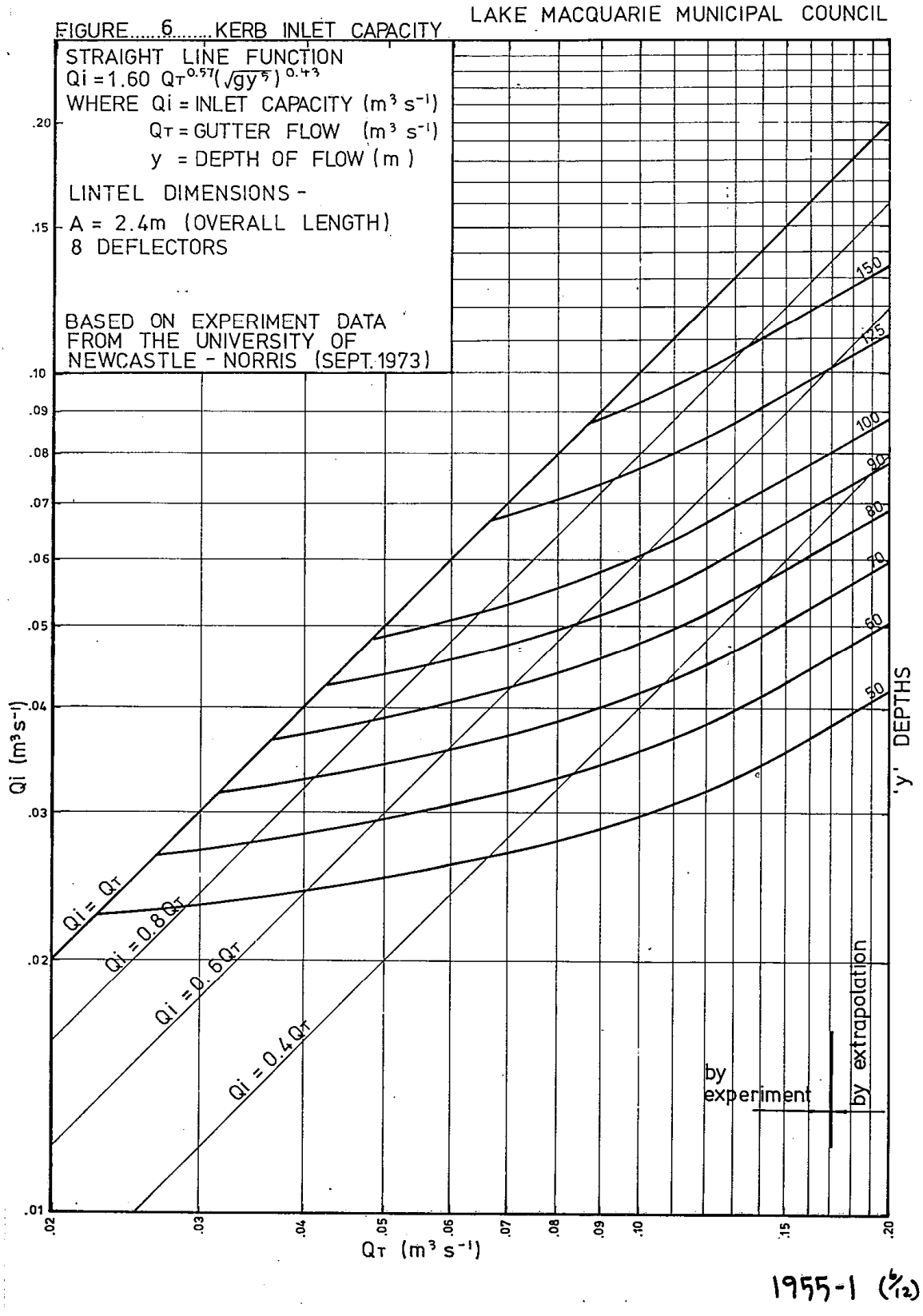
1 PIPE	2 LENGTH L (m)	3 DESIGN FLOW-RATE Q (L/s)	4 TRIAL PIPE DIA. (m)	5 FULL PIPE VELOCITY V (m/s)	6 $\frac{v^2}{2g}$ (m)	7 U/S SURFACE LEVEL A.H.D. (m)	8 U/S PIT WATER LEVEL * (m)	9 PIT PRESSURE CHANGE COEFF. k_u or k_w	10 $k \cdot \frac{v^2}{2g}$ 9 x 6 (m)	11 H.G.L. at U/S PIT 8-10 (m)	12 H.G.L. SLOPE S_f (m/m)	13 PIPE FRICTION LOSS $S_f \cdot L$ 12 x 2	14 H.G.L. at D/S PIT 11-13 (m)	15 D/S PIT SURFACE LEVEL A.H.D. (m)	16 D/S PIT WATER LEVEL (m)	17 U/S INVERT LEVELS (m) HYD-RAULIC 11 - 4	18 COVER 7 - cover	19 U/S PIPE 23 - drop	20 ADOPTED LOWEST OF 17 18 & 19	21 D/S INVERTS (m) HYDRAULIC LOWER OF 14 & 16 - 4	22 COVER 15 - cover	23 ADOPTED LOWER OF 21 & 22	24 PIPE SLOPE S_o $\frac{20-23}{2}$	25 REMARKS

Designer _____ Date _____ Checked _____

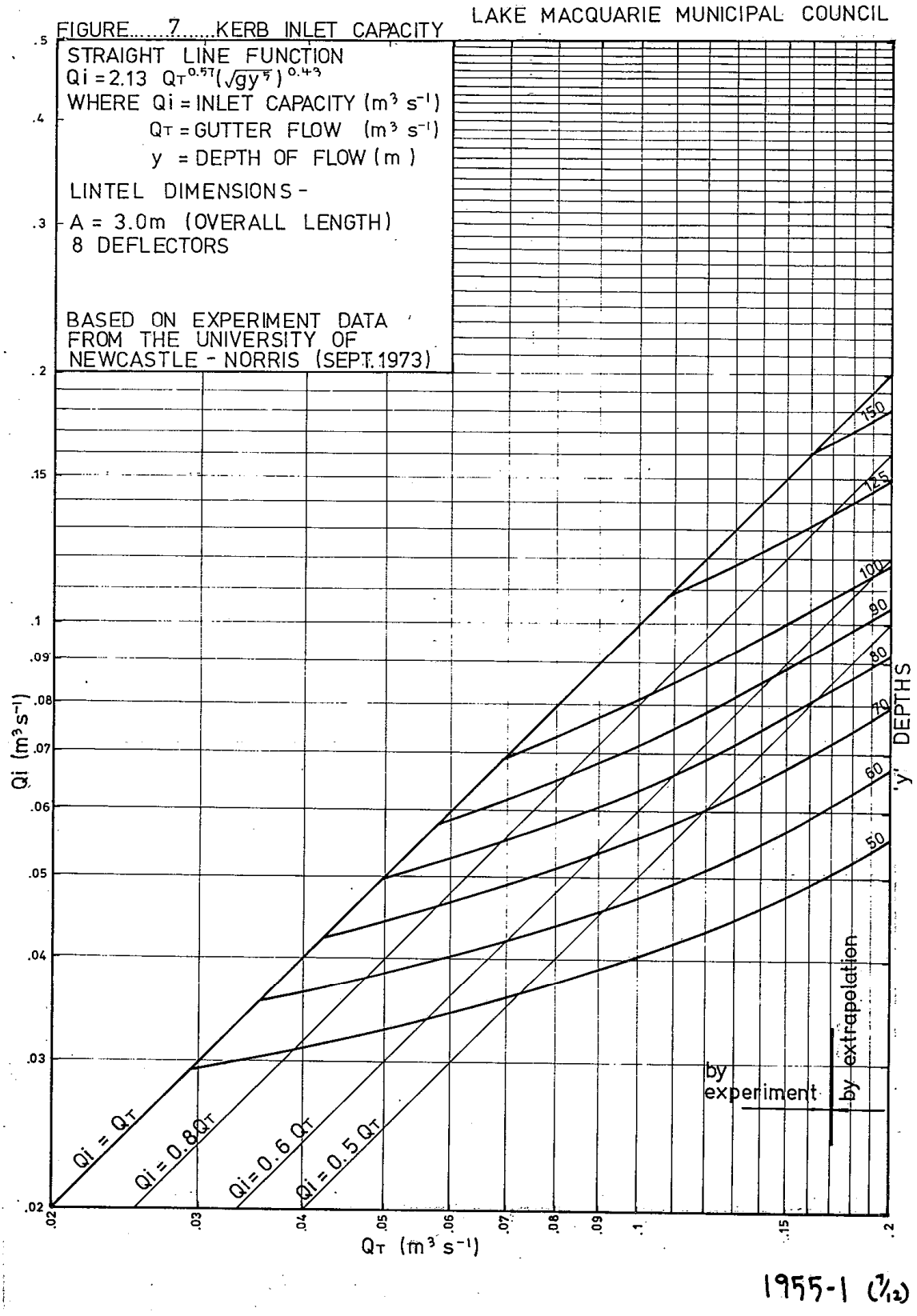
FIGURE 9-12

* 7 - FREEBOARD OR U/S H.G.L. LEVEL FROM 14

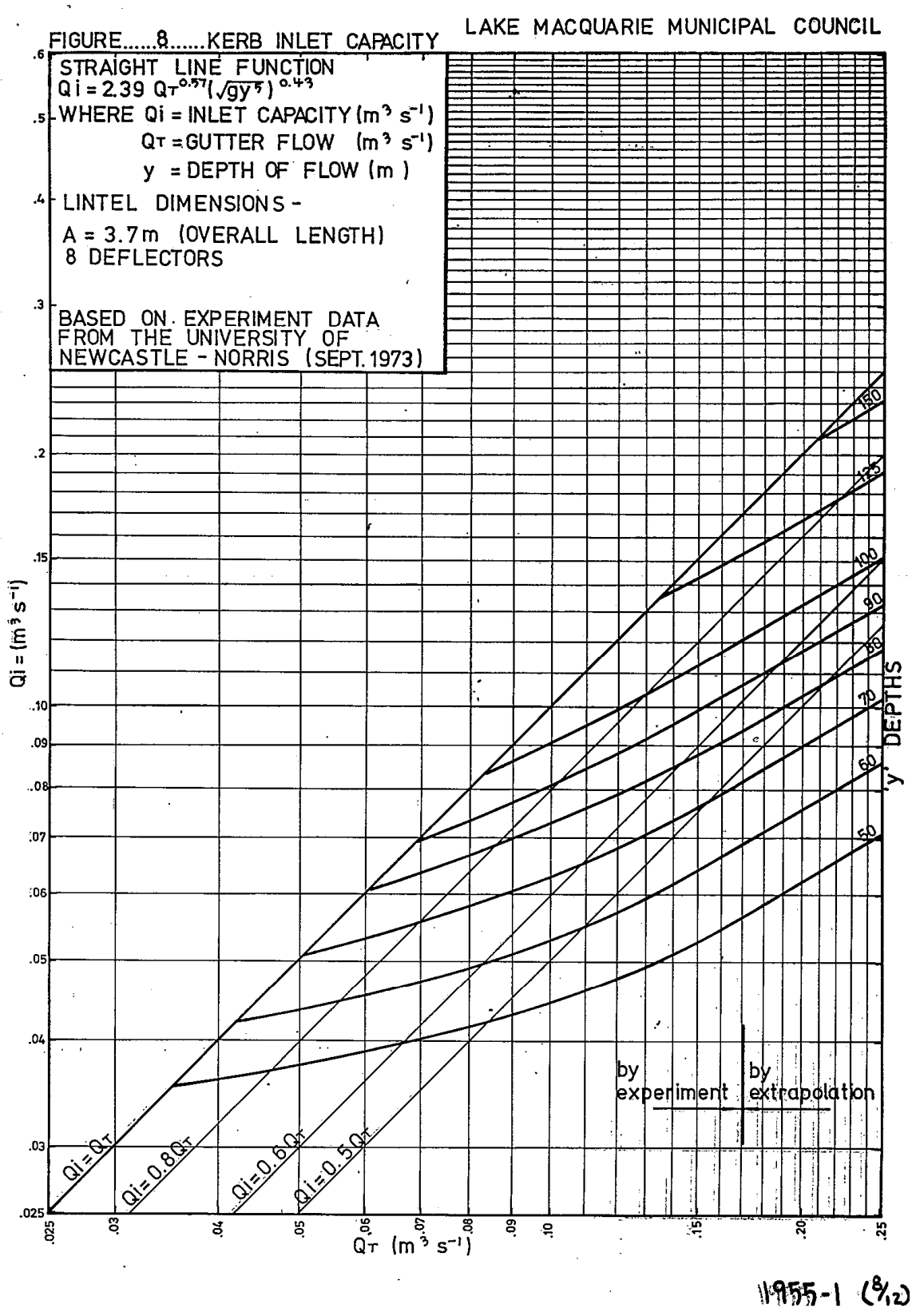
4.8 Kerb Inlet Capacity 2.4m Lintel



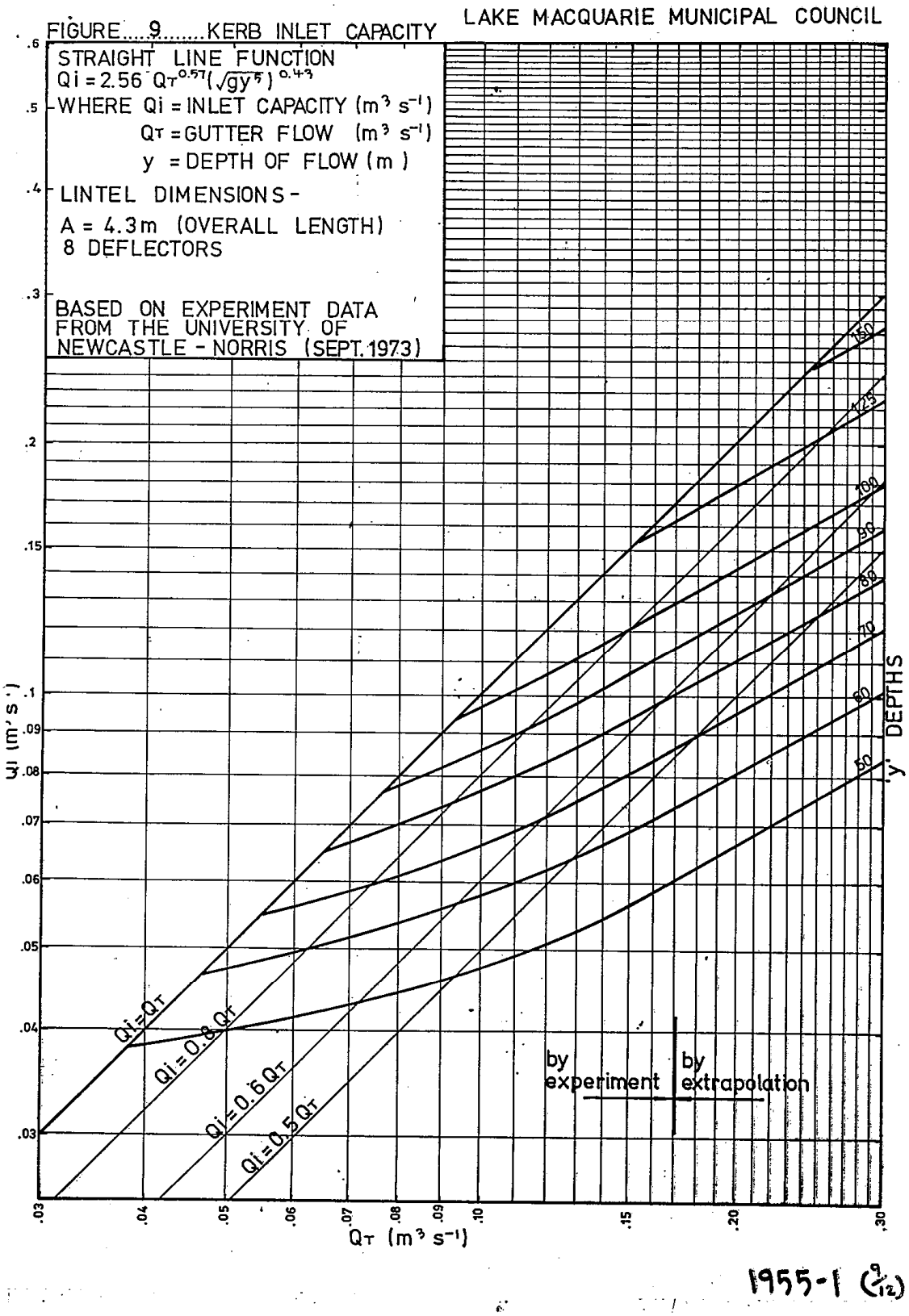
4.9 Kerb Inlet Capacity 3.0m Lintel



4.10 Kerb Inlet Capacity 3.7m Lintel



4.11 Kerb Inlet Capacity 4.3m Lintel



4.12 Sag Inlet Capacity - Crossfall

LAKE MACQUARIE MUNICIPAL COUNCIL

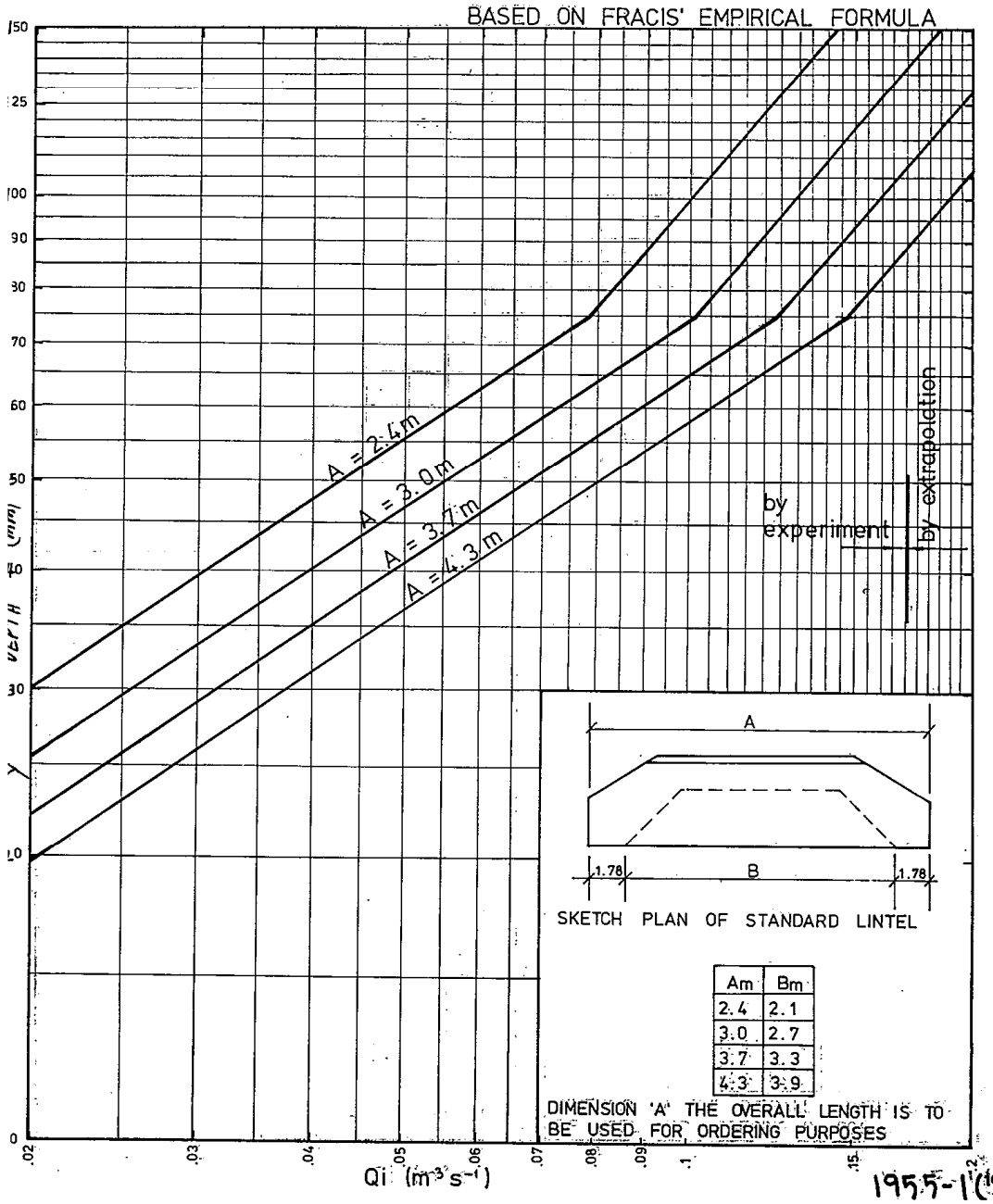
FIGURE 10 SAG INLET CAPACITIES CROSSFALL 3 %

$Qi = \frac{2}{3} Cd\sqrt{2g} (B - 0.1n H_2) H_2^{3/2}$ FOR $H_2 \leq 0.075m$

$Qi = \frac{2}{3} Cd\sqrt{2g} B (H_2^{3/2} - H_1^{3/2})$ FOR $H_2 \leq 0.075m$

WHERE Qi = INLET CAPACITY ($m^3 s^{-1}$)
B = LINTEL OPENING (m)
n = No. OF END CONTRACTIONS = 2
H_2 = y DEPTH (m)
H_1 = $[H_2 - 0.075]$ (m)
Cd = 0.62

BASED ON FRACIS' EMPIRICAL FORMULA



SKETCH PLAN OF STANDARD LINTEL

Am	Bm
2.4	2.1
3.0	2.7
3.7	3.3
4.3	3.9

DIMENSION 'A' THE OVERALL LENGTH IS TO BE USED FOR ORDERING PURPOSES

1955-1(10/12)