7.08 Pipe and materials standards

7.08.1 Local authority requirements

The following provisions are included for general guidance. Specific advice should be obtained from the relevant local authority on what material types and other special requirements are applicable.

The following requirements are applicable to the trunk or local authority drainage system. Detailed requirements in respect of pipe work and appurtenances for the Roof and Allotment Drainage System is provided in Section 7.13.

7.08.2 Standards

Materials used for the construction of stormwater systems should comply with the following Australian Standards and other Standards as applicable.

AS 1254	PVC Pipes and Fittings for Storm or Surface Water Applications
AS 1260	PVC-U Pipes and Fittings for Drain, Waste and Vent Applications
AS 1273	Unplasticized PVC (UPVC) Downpipe and Fittings for Rainwater
AS 1597	Precast Reinforced Concrete Box Culverts
AS 1646	Elastomeric Seals for Waterworks Purposes
AS 1761	Helical Lock-Seam Corrugated Steel Pipes
AS 1762	Helical Lock-Seam Corrugated Steel Pipes – Design and Installation
AS 2032	Code of Practice for Installation of UPVC Pipe Systems
AS 2041	Buried Corrugated Metal Structures
AS 2042	Corrugated Steel Pipes, Pipe-Arches and Arches – Design and Installation
AS 2566.1	Buried Flexible Pipelines – Structural Design
AS 2566.2	Buried Flexible Pipelines – Installation
AS 3500.3	National Plumbing and Drainage Code – Part 3: Stormwater Drainage

AS 3500.5	National Plumbing and Drainage Code – Part 5: Domestic Installations
AS 3571	Glass Filament Reinforced Thermosetting Plastics (GRP) Pipes – Polyester Based – Water Supply, Sewerage and Drainage Applications
AS 3600	Concrete Structures
AS 3725	Loads on Buried Concrete Pipes.
AS 3735	Concrete Structures Retaining Liquids
AS 3996	Access Covers and Grates
AS 4058	Precast Concrete Pipes (Pressure and Non-Pressure)
AS 4139	Fibre Reinforced Concrete Pipes and Fittings
AS 4799	Installation of Underground Utility Services and Pipelines within Railway Boundaries
AS 5100	Bridge Design CD-ROM (AustRoads)
MRS 11.24	Manufacture of Precast Concrete Culverts (Main Roads Department, Queensland)
MRS 11.25	Manufacture of Precast Concrete Pipes (Main Roads Department, Queensland)
MRS 11.26	Manufacture of Fibre Reinforced Concrete Drainage Pipes (Main Roads Department, Queensland)
AS/NZS 5065	Polyethylene and Polypropylene Pipes and Fittings for Drainage and Sewerage Application.
AUSTROADS	Guide to Bridge Technology (2005) and Waterway Design – A Guide to the Hydraulic Design of Bridges (1994)

Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

Designers should also note the structural design and cover conditions outlined in Section 7.09.

7.08.3 Pipes and pipelaying

It is recommended that jointing for pipes comply with Table 7.08.1.

Table 7.08.1 Jointing requirements for pipes – normal conditions

Pipe Size (mm)	Joint Type
Up to 600	Spigot and Socket, Rubber Ring Joint
675 and above	Flush Jointed, External Rubber Band or Approved Equivalent

Notwithstanding the requirements of Table 7.08.1 rubber ringed spigot and socket joints should generally be used for all sizes of pipe in unstable ground, when pipes are laid in sand, or where pipe movement is possible, such as on the side of fills or at transitions from cut to fill.

Rubber ringed spigot and socket joints should also be used where the normal groundwater level is above the pipe obvert or where the design H.G.L. is significantly (1.5m or greater) above obvert level.

(a) Minimum pipe size

The minimum diameter of any pipe in a local government drainage system should be 375mm, except that a gully connection from a single gully, the connection between twin spaced gullies, the connection from a sag gully provided purely to prevent ponding after a storm may, subject to hydraulic analysis, be 300mm diameter. Recommendations in respect of pipe sizes for roof and allotment drainage are given in Section 7.13.

(b) Lateral spacing of pipes

Where multiple pipes are used they should be spaced sufficiently to allow adequate compaction of the fill between the pipes. The clearance between the outer face of the walls of multiple pipes should generally be in accordance with Table 7.08.2. The local government may permit lesser spacing in special circumstances to reduce structure costs, where easement width is limited, or for relief drainage works.

Table 7.08.2 Recommended minimum spacing of multiple pipes

Diameter of Pipes (mm)	Recommended Minimum Clear Spacing (mm)
Up to 600	300
675 to 1800	600

Notes:

1. The above minimum spacings may need modification to satisfy structural considerations especially when laid at depth, under traffic loads or for pipes greater than 1800mm in diameter.
2. Where lean mix concrete vibrated in place or cement stabilised sand is used for backfill, the clear spacing may be reduced to 300mm for all diameters, subject to structural considerations.

Pipe laying shall be carried out in accordance with the specification of the relevant local authority, or other specification acceptable to the local authority.

(c) Pipe trench compaction

Construction supervisors and stormwater managers are warned about the potential damaging effects of compacting trenches with wheel roller attachments that can impart significant live loads on the pipe. The choice of pipe material and structural grade will depend on the chosen method of installation.

Recommendations on the compaction of earth around concrete pipes may be obtained from the Concrete Pipe Association's web site or *Concrete Pipe Selection* software.

7.08.4 Box sections

Box culverts may be used where available depth to invert is restricted or to provide maximum waterway area and minimum obstruction to flow.

The minimum waterway dimension of any box section should normally be 300mm (or 375mm for cross drainage road culverts). However in the case of a connection from a single gully pit, other than in a sag, the minimum vertical dimension may be 225mm.

The minimum cover over a box section should normally be 400mm. This may be reduced to 100mm in conjunction with a concrete or asphaltic concrete full road depth surfacing subject to structural considerations. The maximum depth of fill for box sections is normally limited to 10m, again subject to structural considerations.

Where box culverts are constructed on a skew, special precautions may need to be taken to resist unbalanced earth pressures.

7.08.5 Access chambers and structures

All structural concrete work should be executed in accordance with the current edition of:

AS 3600 – SAA Concrete Structures Code.

AS 3610 – Formwork for Concrete.

AS 1302 – Steel Reinforcing Bars for Concrete.

Concrete finishes shall be in accordance with Table 3.3.1 of AS 3610, as follows:

(i)	Normally exposed to view e.g. faces of wingwalls, etc.	Class 3
(ii)	Not normally exposed to view e.g. inside of access chamber, etc.	Class 4
(iii)	Base slabs for box culverts, floors and benching of pits, aprons and channel inverts.	Dense, wood float finish of uniform texture.

The minimum concrete class for stormwater drainage works should be as follows:

(i)	Major endwalls and other major structures	32 MPa
(ii)	Access chambers, kerb inlets, minor endwalls and other minor structures	25 MPa

Requirements relating to the durability of concrete in aggressive groundwater and salt-water conditions are presented in Section 7.09. Designers should also note the structural design and cover conditions outlined in Section 7.09.

Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

7.09 Structural design of pipelines and access chambers

Loads on buried pipelines include:

(a) Fill over the pipe, which is a function of:

- Height of fill
- Type of fill material
- Installation conditions (e.g. "trench" or "embankment")

(b) Normal traffic loads

(c) Construction traffic loads

(d) Other or abnormal load conditions

The load bearing capacity of a pipeline is a function of:

(a) Pipe strength class

(b) Type of bedding and backfill material

(c) Pipe diameter

In the case of culverts, the invert level is generally fixed by the bed level of the adjacent watercourse. The design problem is thus to select a suitable class of pipe and type of bedding to suit the pipe diameter, height of fill over the pipe, type of fill material, installation condition and traffic load.

In urban drainage design the depth of the pipeline is usually not a constraint. In this case the design exercise is to select the most economic combination of pipe depth, strength class and bedding type.

The structural design of pipelines should be carried out in accordance with AS 3725 Loads on Buried Concrete Pipes, CPAA Pipe Class v1.1 Concrete Pipe Selection Software, the latest version of Austroads Bridge Design Code, and AS 2566.2 Buried Flexible Pipelines – Installation

The absolute minimum cover over any pipe, irrespective of location, class and bedding, should be 300mm, unless special protection is provided, such as a structural concrete slab. Table 7.10.1 details recommended minimum cover.

All pipes, box sections and access chambers in road reserves, whether under the road pavement or within the footpath area, and all pipes within Industrial and Commercial allotments, should be designed for a W7 wheel loading in accordance with Austroads (2005) where applicable standard drawings are not available from the local authority. Note that the W7 loading should be modified for impact effects in accordance with the buried structures provisions and distributed in accordance with Austroads (2005).

The minimum strength class for concrete drainage pipes should be Class 2. To achieve uniform pavement compaction, pipes under the road pavement should be laid prior to placing of the pavement material. Accordingly, such pipes should have adequate cover between the top of the pipe and the subgrade level, to support loads imposed by construction plant. In general, such loads may be taken as being equivalent to Standard W7 loading unless unusual conditions prevail.

Where pipelines, whether located under road pavements or otherwise, are laid prior to completion of bulk earthworks, the possibility of them being subjected to heavy construction traffic should be considered and extra cover provided, a stronger class of pipe used, or the pipes otherwise protected.

Where aggressive ground conditions exist, or where the system might be exposed to salt water, it may be necessary to provide additional concrete cover to reinforcement or protective coating to exposed surfaces. The supply and proper installation of high-quality impermeable concrete is the most effective means of corrosion prevention.

This can be achieved by designing a dense concrete mix with water:cement ratio less than 0.5 and cement content of at least 330 kg/m^3 and ensuring that placement is properly supervised. Designers should refer to Technical Note TN57 (C.&C.A. 1989) for more detailed recommendations. Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

7.10 Minimum cover over pipes

The minimum cover over pipes to be adopted for pipe grading purposes should be:

Table 7.10.1 Recommended minimum cover over pipes

Location	Minimum Cover (mm)	
	Rigid Type Pipes e.g. Concrete, F.R.C.	Flexible Type Pipes e.g. Plastic or Thin Metal
Residential private property, and parks not subject to traffic	300	450
Private property and parks subject to occasional traffic	450	450
Footpaths	450	600
Road pavements and under kerb and channel	600	600

Notes:

1. For special cases, and with the agreement of the local authority, cover can be reduced by using a higher-class pipe, special bedding, concrete protection or a combination of these.
2. Where pipes are to be laid under the footpath consideration should be given to the possibility of future road widening, both in respect of the reduced cover that might result from the widening and vehicle loading.

7.11 Flow velocity limits

The velocity of stormwater in pipes and box sections should be maintained within acceptable limits to ensure that:

(i) self cleaning of the pipe or box section is maintained;

(ii) scouring and erosion of the conduit, (particularly the invert) does not occur.

The range of acceptable flow velocities are as detailed in Table 7.11.1.

Table 7.11.1 Acceptable flow velocities for pipes and box sections

Flow Condition	Absolute Minimum [1] (m/s)	Desirable Minimum [1] (m/s)	Desirable Maximum [2] (m/s)	Absolute Maximum [2] (m/s)
Partially full	0.7	1.2	4.7	7.0
Full	0.6	1.0	4.0	6.0

Notes:

[1] Minimum flow velocities apply to 1 in 1 year ARI design storm, and apply to all pipe materials.

[2] Maximum flow velocities apply to concrete pipes. For other pipe materials, refer to manufacturer's advice.

Part-full flow characteristics of pipes may be determined from the appropriate Design Chart contained in Volume 2.

In steep terrain the velocity of flow should not be greater than the absolute maximum velocity of 6.0 m/s under "pipe full" conditions. To achieve this requirement, it may be necessary to construct access chambers with drops to dissipate some of the kinetic energy of the flow, or to limit the pipe diameter.

Reference should be made to Tables 9.05.1 and 9.05.3 for details of velocity limits for vegetated and grassed/unlined channels.

Notwithstanding the above suggested velocity limits, hydraulic considerations may require the velocity be controlled to well below the "Desirable Maximum" and/or the pipe size increased to minimise structure losses and the slope of the hydraulic grade line.

7.12 Pipe grade limits

To conform with the requirements of Section 7.11, and construction limitations the following maximum and minimum grades are recommended for design purposes:

Table 7.12.1 Acceptable pipe grades for pipes flowing full

Pipe Diameter (mm)	Maximum Grade (%)	Minimum Grade (%)
300	20.0	0.50
375	15.0	0.40
450	11.0	0.30
525	9.0	0.25
600	7.5	0.20
675	6.5	0.18
750	5.5	0.15
900	4.5	0.12
1050	3.5	0.10
1200	3.0	0.10
1350	2.5	0.10
1500	2.2	0.10
1650	2.0	0.10
1800	1.7	0.10
1950	1.5	0.10
2100	1.4	0.10
2250	1.3	0.10
2400	1.2	0.10

Notes:

1. Based on maximum velocity for pipe flowing full of 6.0m/s.
2. Based on minimum velocity for pipe flowing full of 1.0m/s except where Note 4 is applicable.
3. Manning's n = 0.013 for all cases (concrete pipes).
4. The minimum grade of 0.10% (1:1000) is based on construction tolerance requirements.
5. The **Maximum Grade** requirement applies to both the pipe grade and the hydraulic grade.
6. The **Minimum Grades** apply to the pipe grade only.
7. Where a pipe is flowing less than half full for the design flow being considered, it is permissible to exceed the above maximum grades provided that the velocity limits specified in Table 7.11.1 are not exceeded.

7.13 Roof and allotment drainage

7.13.1 General

To incorporate the principles of Water Sensitive Urban Design (WSUD), roof and allotment drainage systems should be designed to minimise the direct connection of impervious areas to the trunk drainage network. Methods for designing drainage systems in accordance with these principles are provided in the various publications referenced in Section 11.07 of this Manual.

This Manual presents information on the design of roof and allotment drainage systems where the WSUD approach is impractical, due to site constraints or concerns over public health or amenity.

Five levels of roof and allotment drainage are considered in this Manual.

The reasons for selecting one of the following levels over another may be based on land use (e.g. commercial or residential), density of development, community standards, or the requirement for a given level of protection from flooding by storm runoff. In certain developments a combination of these systems may be required.

The applicable levels to be adopted within a particular development shall be determined by the relevant local government.

Design and construction of roof and allotment drainage systems and appurtenances should comply with AS 2180 and AS 3500.3.

7.13.2 Roof drainage

The design of gutters and downpipes for roof drainage should be undertaken in accordance with NSB 151, NSB 152 and NSB 153 (C.S.I.R.O.) and AS 2180 to adequately convey the runoff from the design storm detailed in Table 7.13.1.

Table 7.13.1 Design of roof gutters and downpipes

Design Storm	ARI = 20 years, Duration = 5 minutes [1]
Check Storm [2]	ARI = 100 years, Duration = 5 minutes [1]

Notes:

[1] The critical storm duration of 5 minutes should be adopted unless special circumstances justify a longer duration.

[2] A design check should be undertaken to determine the effect of the “check storm” where the consequences of hydraulic failure are significant or where the system contains vulnerable components such as internal box gutters.

7.13.3 Roof and allotment drainage – general

Outside the requirements for WSUD, the drainage system provided within allotments for the disposal of roof and allotment drainage depends upon the topography, the importance of the development, and the consequences of failure. The local government may determine that the provision of a piped allotment drainage system to receive roof and allotment runoff is necessary in the following circumstances:

(a) Where allotments fall away from the street.

(b) Where the proportion of impervious area within a development is such that the frequency and volume of surface runoff is likely to be intolerably high, e.g. industrial and multi-unit residential allotments.

(c) Where zoning may permit construction of buildings up to side or rear boundaries thus blocking or concentrating natural flow paths.

(d) Where there is significant catchment draining into the rear of the property.

7.13.4 Level of roof and allotment drainage system

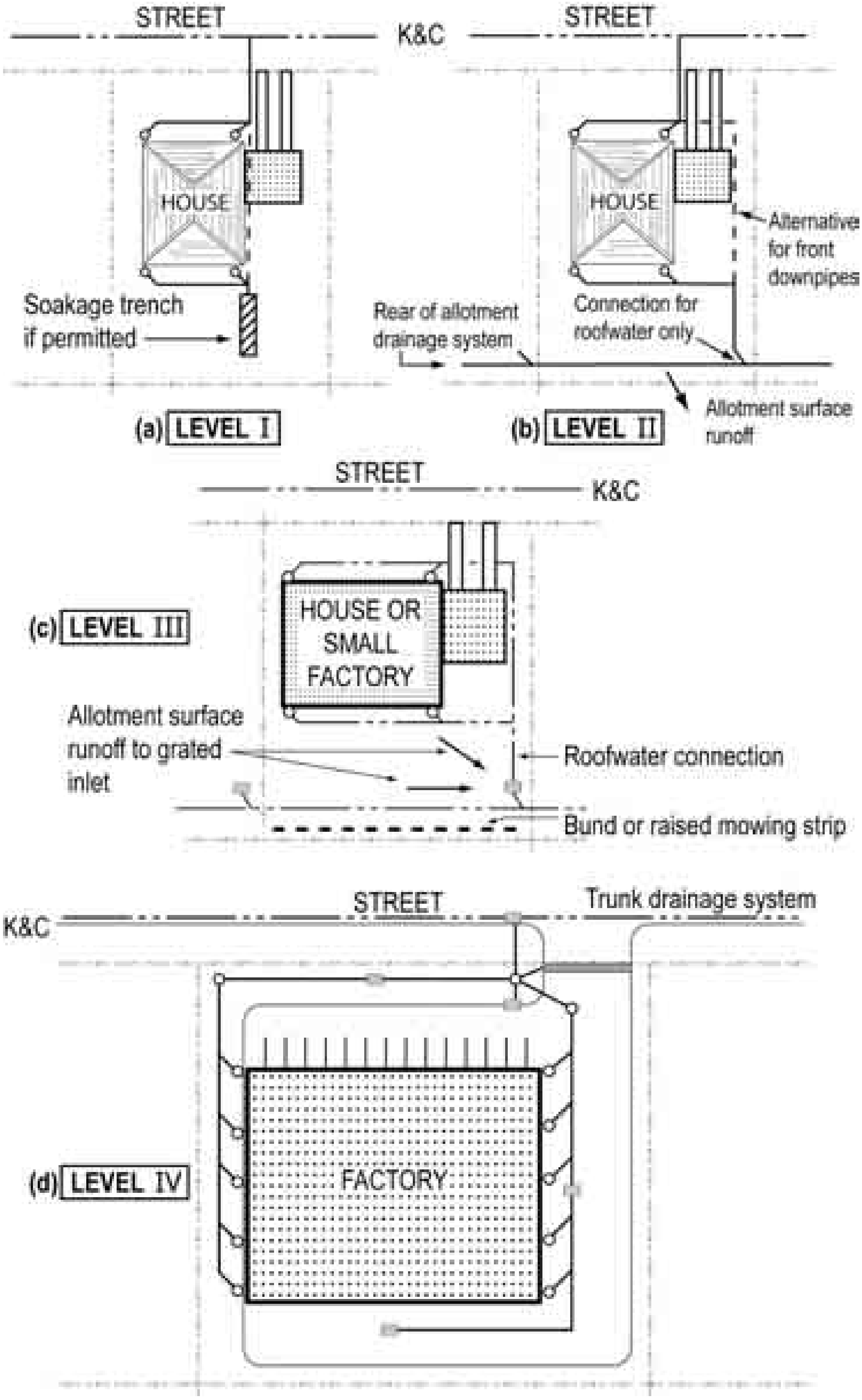
The level of roof and allotment drainage system provided within a development is differentiated by the components making up the system and the sophistication necessary in the design of these components.

Depending upon the size or importance of a development or the consequences of failure of the roof and allotment drainage system, the local government may nominate the level of system to be provided. Figure 7.13.1 indicates the types of developments to which the various levels may be applicable. Table 7.13.2 details the various components and Table 7.13.3 indicates the level of system to which these are applicable.

Each of the examples provided in Figure 7.13.1 may be appropriately modified to incorporate the use of rainwater tanks and/or on-site detention systems to the discretion of the local government.

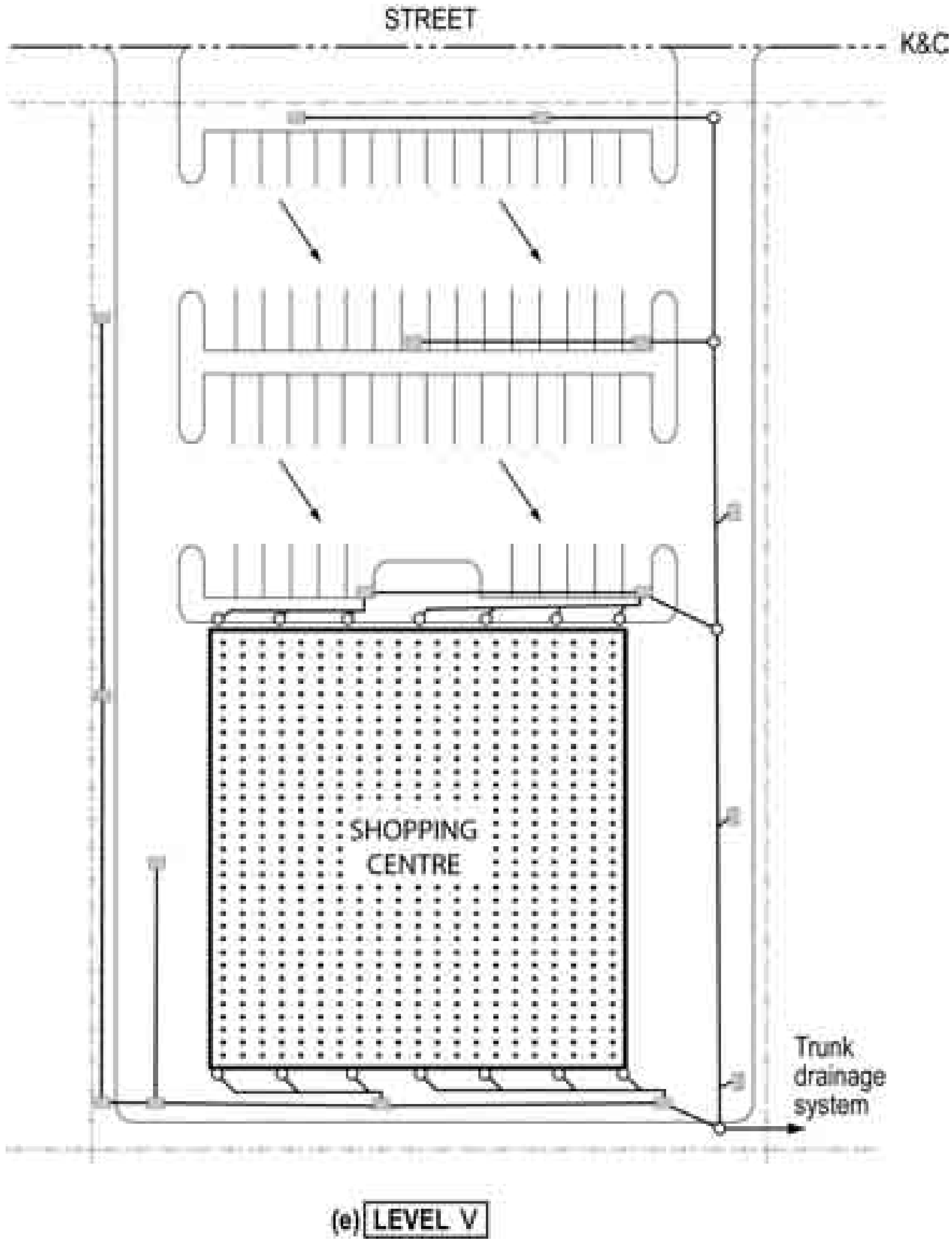
The following sections permit the design of underground allotment and rear of allotment drainage pipes in some cases to an ARI less than that detailed in Table 7.13.1. This implies that surcharge may occur from the underground system.

The sections of underground pipe leading from the downpipes to the points where surcharge can occur should be sized to prevent a constriction of flow in the downpipe system. Beyond those points the provisions of Table 7.13.4 are applicable.



Levels of roof and allotment drainage system
(see also Figure 7.13.1 (e))

Figure 7.13.1 (a) to (d)



Levels of roof and allotment drainage system
Figure 7.13.1 (e)

Note: Each of the examples provided in Figure 7.13.1(a) to (e) may be appropriately modified to incorporate the use of rainwater tanks and/or on-site detention systems to the discretion of the local government.

Table 7.13.2 Roof and allotment drainage components

Description of Component	Identifier
Guttering	(a)
Downpipes	(b)
Rainwater tanks	(c)
Minor pipes in allotment.	(d)
Connection to kerb and channel.	(e)
Seepage trenches or rubble pits (where permitted).	(f)
Connection to a kerb inlet or trunk drainage system in the street.	(g)
Connection to rear of allotment drainage system.	(h)
Rear of allotment drainage system designed to receive roof-water from one or more allotments and with a connection point to receive roof-water only at each allotment.	(i)
Rear of allotment drainage system designed to receive both roof-water and allotment surface runoff from one or more allotments and with a connection point to receive roof-water and a grated kerb inlet to receive surface runoff at each allotment.	(j)
Allotment drainage system designed to receive both roof-water and allotment surface runoff from one allotment or complex and comprising kerb inlets, junction pits or access chambers and underground pipe system etc. and discharging to a rear of allotment drainage system, kerb inlet or trunk drainage system.	(k)
As for (j) but discharging normally only to a trunk drainage system or other nominated lawful point of discharge.	(l)

Table 7.13.3 Levels of roof and allotment drainage

Level	Components (as applicable)	Design Complexity	Where Normally Applicable (Refer to Table 7.13.2)
I	(a), (b), (c), (d), (e) and (f) where permitted	N.S.B. And nominal pipe sizes underground.	Low density Urban Residential, corner stores and other minor developments.
II (Roofwater Only)	(a), (b), (c), (d), (e), (h) and (i)	N.S.B. Rational Method and pipe flow nomograph, or nominal pipe sizes. See Table 7.13.5.	Low density Urban Residential and other minor developments as nominated by the local government.
III (Roof and Allotment Runoff)	(a), (b), (c), (d), (e), (h) and (j). (f) where nominated	N.S.B., Rational Method and pipe flow nomograph, or nominal pipe sizes. See Table 7.13.6.	Where nominated by local government.
IV	(a), (b), (c) and (k). (d) where permitted	N.S.B. Rational Method, full hydraulic analysis or pipe flow nomograph with allowance for structure losses.	Commercial, Industrial, high density Urban Residential and other developments as nominated by the local government.
V	(a), (b), (c) and (l)	N.S.B. Rational Method and full hydraulic calculations including structure losses and determination of H.G.L.	Central Business and large Commercial, Industrial and high density Urban Residential Developments or where nominated by the local government.

Abbreviations (Table 7.13.3 and 7.13.4):

FRC = fibre reinforced cement (pipe)

NSB = Notes on the Science of Building (C.S.I.R.O.)

RCP = reinforced concrete pipe

RRJ = rubber ring jointed

S & S = spigot and socket

UPVC = unplasticised polyvinyl chloride (pipe)

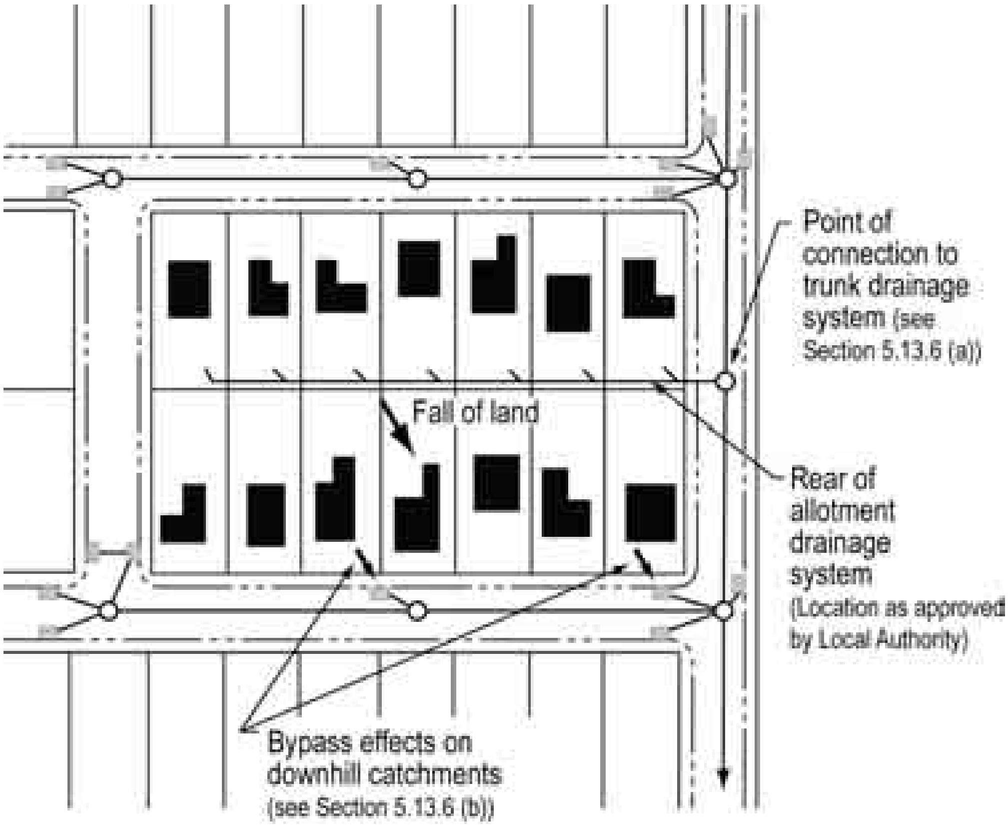
7.13.5 The rear of allotment drainage system

The rear of allotment drainage system is provided for the collection of storm runoff from allotments falling away from the street or from other allotments which are impeded from discharging runoff from the whole of the allotment to the trunk drainage system in the street. These systems are normally constructed by the developer and may or may not become part of the trunk drainage system owned and maintained by the local government.

The rear of allotment drainage system is sometimes referred to as "inter allotment drainage".

The system should be designed to receive the peak runoff as determined from the guidelines set out in Table 7.13.4. This table also contains certain recommendations in respect of construction requirements etc.

The location of the rear of allotment drainage system and boundary clearance should be as directed by the local government.



Effects on trunk drainage network
Figure 7.13.2

Table 7.13.4 Design recommendations for the rear of allotment drainage system (See table 7.13.3 for abbreviations)

Item	Level Applicable				
	I	II	III	IV	V
Minimum Pipe Size	N.A.	150mm	225mm	375mm [1]	
Minimum Stub Size	–	150mm	150mm	To be designed	
Pipe Material	–	UPVC	UPVC, RCP, FRC	RCP, FRC	
Jointing System	–	RRJ, S&S	RRJ, S&S	RRJ, S&S	
Flow Calculation	–	10 L/s per allotment	See Table 7.13.6	Rational Method or runoff model	
ARI for Design	N.A.	See Table 7.13.5	See Tables 7.13.5 & 7.13.1	20 years [2]	
Pipe System Design	N.A.	See Table 7.13.5	See Table 7.13.6	Full hydraulic analysis or pipe nomograph plus structure losses	Full hydraulic analysis with determination of H.G.L.
Major Design Storm overland flow check	Ensure the land development and its drainage system does not unlawfully concentrate flows onto, or aggravate flooding within, neighbouring properties. The overland flow path is to be identified within the system design. Also refer to Tables 7.13.7 and 7.13.8.				
				Refer to Note [2] below	

Note:

[1] Subject to hydraulic analysis the connection from a single kerb inlet may be 300mm diameter.

[2] For Level IV and V systems the underground drainage system should be designed to convey discharge for the Major System ARI storm from trapped sags and other locations where an acceptable overland flow path is unavailable.

Table 7.13.5 Recommended design criteria for level II rear of allotment drainage system

<table>
<tr><th colspan="2">Item</th><th colspan="7">Recommendation</th></tr>
<tr><td colspan="2">Maximum No. of Allotments Served</td><td colspan="7">20</td></tr>
<tr><td colspan="2">Flow Applicable</td><td colspan="7">10 L/s per allotment [4]</td></tr>
<tr><td colspan="2">Minimum Pipe Grade</td><td colspan="7">0.35%</td></tr>
<tr><td colspan="2">Minimum Pipe Cover (mm)</td><td colspan="7">500</td></tr>
<tr><td colspan="2">Pit Dimensions For Depth to Invert
(a) ≤ 750
(b) > 750</td><td colspan="7">(a) 600 x 600
(b) 600 x 900</td></tr>
<tr><td rowspan="3">Nominal Pipe Diameter (mm)</td><td colspan="8">Flow (L/s) [1]</td></tr>
<tr><td colspan="8">Pipe Gradient (%) [2]</td></tr>
<tr><td>0.5</td><td>1.0</td><td>1.5</td><td>2.0</td><td>2.5</td><td>3.0</td><td>4.0</td><td>5.0</td></tr>
<tr><td>150</td><td>[3]</td><td>18</td><td>23</td><td>26</td><td>30</td><td>33</td><td>38</td><td>42</td></tr>
<tr><td>225</td><td>38</td><td>56</td><td>67</td><td>78</td><td>87</td><td>96</td><td>110</td><td>125</td></tr>
<tr><td>300</td><td>84</td><td>120</td><td>146</td><td>170</td><td>190</td><td>210</td><td>N.A.</td><td>N.A.</td></tr>
</table>

Notes:

[1] Based on Manning's n = 0.011 and the likely use of UPVC for smaller pipes.

[2] Where the pipe gradient is in excess of 5% a more detailed hydraulic analysis should be undertaken including the assessment of structure losses, where appropriate.

[3] Minimum grade 1% for 150mm diameter pipe to comply with AS 3500.3.

[4] Based on roof areas of 180 m^2 and ARI = 20 years for S.E. Queensland.

Table 7.13.6 Recommended design criteria for level III rear of allotment drainage system

Item	Recommendation
Maximum No. of Allotments Served	20
Flow Applicable Allotment ≤ 750m^2 Allotment > 750m^2	 – Rational Flow with Pipe Size from Table Below. [5] – Rational Flow< Use Pipe Nomograph
ARI for Design (yrs)	Minor System ARI as per Table 7.02.1
Minimum Pipe Grade	0.35%
Minimum Pipe Cover	500mm

Number of Allotments	Recommended Pipe Diameter (mm) [2 & 5]							
	Pipe Gradient (%) [1]							
	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
1	225	225	225	225	225	225	225	225
2	300	300	225	225	225	225	225	225
4	375	300	300	300	300	300	300	225
6	450	375	375	300	300	300	300	300
8	450	450	375	375	375	375	300	300
10	525	450	450	375	375	375	375	375
12	525	450	450	375	375	375	375	375
14	525	450	450	450	375	375	375	375
16	525	525	450	450	450	375	375	375
18	600	525	450	450	450	450	450	375
20	600	525	525	450	450	450	450	450

Notes:

[1] Where the pipe gradient is in excess of 5% a more detailed hydraulic analysis should be undertaken including assessment of structure losses, where appropriate.

[2] Based on Manning's $n = 0.013$.

[3] Grated inlets should be designed with allowance for blockage as detailed in Table 7.05.1.

[4] The gully inlet at each allotment should be located where possible at the lowest point and should be contained in a bund or catch drain to minimise bypass.

[5] The pipe sizes shown have been based on discharge from allotments of average size = 750 m^2, a 5 minute storm duration and ARI = 2 years for Brisbane, i.e. 150 mm/h. This equates to 20 L/s per allotment. **Note:** For other locations and/or allotment densities, pipe sizes should be adjusted accordingly.

7.13.6 Effect of roof and allotment drainage system on trunk drainage network

There are two effects of the roof and allotment drainage system on the design and performance of the nearby trunk drainage network as illustrated in Figure 7.13.2.

(a) Hydraulic effects at point of connection

This relates to hydraulic design of the trunk drainage system and the rear of allotment drainage system at the point of connection to the trunk drainage system. Table 7.13.7 details the manner in which this should be undertaken.

Table 7.13.7 Roof and allotment drainage system design procedure at point of connection

Level	Design Procedure
I	Design for street flows and trunk network with appropriate catchment area. Ignore local effect at connection to kerb and channel.
II	(a) For minor storm ARI – Design for full discharge [1] from rear of allotment drainage system in trunk network downstream of connection. Ignore structure losses at point of connection. (b) For major storm ARI – Ignore rear of allotment drainage system. [2]
III	(a) For minor storm ARI – Design for full discharge [1] from rear of allotment drainage system and associated structure losses at point of connection. [3] (b) For major storm ARI – Ignore rear of allotment drainage system. [2]
IV and V	(a) For minor storm ARI – Design for full discharge from roof and allotment system and associated structure losses at point of connection. [3] (b) For major storm ARI – check ability of trunk network to accept flow at point of connection and design for inflow accordingly including associated structures losses. [3]

Notes:

[1] The full discharge referred to corresponds to 100% of the calculated discharge determined in accordance with Table 7.13.5 or Table 7.13.6.

[2] For level II and III systems it is assumed that the rear of allotment system will be ineffective during the major design storm and that roof and allotment runoff will bypass to the downstream catchments.

[3] Although the roof drainage and pipes connected immediately thereto will be designed for the ARI detailed in Table 7.13.1 the design storm applicable to the roof and allotment drainage system to satisfy the design check required by Table 7.13.7 should be that of the trunk drainage system to which it is connected.

(b) Bypass effect on downhill catchments

Concurrent with the design discharge to the trunk drainage system referred to in (a) above allowance should be made for bypass resulting from possible inefficiency of collection associated with the roof and allotment drainage system. The downhill catchments should be designed to receive the bypass as detailed in Table 7.13.8.

Table 7.13.8 Bypass from roof and allotment drainage system to downhill catchments

Level	Bypass Allowance
I	100% of calculated runoff
II	(a) For minor design storm ARI – 100 % of allotment runoff (i.e. roof runoff not bypassed). (b) For major design storm ARI – 100% of roof and allotment runoff.
III and IV	(a) For minor design storm ARI – Nil. (b) For major design storm ARI – 100% of roof and allotment runoff for major design storm less minor design storm capacity of roof and allotment drainage system.
V	(a) For minor design storm ARI – Nil. (b) For major design storm ARI – 100% of roof and allotment runoff for major design storm less calculated capacity of roof and allotment drainage system during the major design storm.

7.14 Public utilities and other services

7.14.1 General

In urban areas, drainage is only one of many public utility services that must be provided. Appropriate consideration should be given to all services, with priority being given to those services which are grade dependent, e.g. sewer and stormwater. Designers should check for potential conflicts and allow for these in the design.

The following is a list of services commonly encountered:

- Water supply – reticulation and trunk
- Sewerage – reticulation and trunk
- Telecommunication – distribution, coaxial and fibre optic
- Gas – distribution and trunk
- Oil and natural gas pipelines
- Electricity – distribution and mains
- Water service crossings
- Sewer house connections
- Roof-water drainage
- Other stormwater

7.14.2 Clearances to services

Where conflicts exist in the alignment and level of services it will be necessary to ensure that adequate clearance is provided between the outer faces of each service. The nominated clearance should allow for collars and fittings on pipes and special protection if required (e.g. a concrete surround).

In general the minimum clearance between the outer faces of services should be 200mm, or as permitted by the services authority.

Penetrations by services through stormwater pipes should be avoided. Where it is necessary for a service to penetrate a stormwater pipe or access chamber allowance should be made for the hydraulic losses in the system resulting from the penetration. In addition the service should be contained in a pipe or conduit of sufficient strength to resist the forces imposed on it by flow, including debris, in the stormwater system. Unless otherwise agreed by the local authority and/or utility owner, penetrations should be constructed using ductile iron pipe. To assist in the removal of debris collected on service pipes or conduits passing through a drainage system it is recommended that an access chamber be located at the pipe or conduit penetration.

Reference should be made to the utility allocations applicable in the local government area, when designing the stormwater system.

7.15 Discharge calculations

7.15.1 General

The system objectives and design philosophy outlined in Chapter 1 seeks to limit flooding of property and to ensure a reasonable level of pedestrian and vehicular traffic safety and accessibility. These objectives are met by ensuring that major and minor storm flows are managed within specified limits and by designing both major and minor system components in conjunction.

If the major and minor component of the surface system do not have the capacity to carry the difference between the respective design peak flow and the pipe flow then additional inlets and hence larger pipes are required to ensure that the surface system operates within the specified limits.

Where the drainage system contains few or no underground pipe components, it will be necessary for the surface system to perform within the limits detailed in Section 7.04 and Chapter 9 – *Open Channel Hydraulics* as applicable.

7.15.2 General Principles

(a) The drainage system as a whole is provided to mitigate against property flooding and to ensure the safety and convenience of pedestrians and vehicles.

(b) The minor drainage system comprising underground pipes and/or surface flow paths is designed to provide for the safety and convenience of pedestrians and vehicles.

(c) Where flood immunity cannot be provided for property and buildings under major storm conditions via overland flow paths the capacity of the underground pipe system and the inlets leading to it need to be increased in order to reduce surface flows to acceptable levels.

(d) Under normal conditions the capacity of the underground pipe system should not be less than its minor storm flow conditions while the system is operating under major storm conditions. The exceptions would be when tailwater levels downstream have a significant effect on the system's hydraulic gradeline, or the surface gradient is considerably flatter than the pipe gradient, thus causing the H.G.L. to rise above the ground surface.

(e) The underground system should be designed with a suitable allowance for blockage at kerb inlets as described in Section 7.05.2. In this way the full design capacity of the underground system can be taken into account under both major and minor storm conditions.

7.15.3 Design procedure

The design procedure is detailed below and in Figure 7.15.2. Example design calculation sheets are provided in Volume 2.

Note that the procedures described herein do not attempt to ascribe an ARI to the flow conveyed in the pipe system, or even set the type of Minor Drainage System (e.g. pipe or swale). Rather the total system is designed to convey the calculated peak flows during major and minor storm events of selected ARI whilst adhering to public safety and convenience criteria separately applicable under relevant conditions.

Phase A: Layout and topographical assessment

(i) Identify the preferred location of major overland flow paths as discussed in Section 7.01 – *Planning Issues*.

(ii) Decide preliminary road layout and road widths (if not existing). Depending on the results of Phases D and E, this preliminary layout may need to be altered to optimise the stormwater drainage system.

(iii) Assess where trapped sags or other topographical constraints will result in a need for an overland flow path other than along a road. Use this as a basis for locating parks, drainage reserves, etc.

Note: Relief drainage or upgrading works may involve flow through existing private allotments.

Phase B: Water sensitive urban design

(i) Identify opportunities for application of the principles of Water Sensitive Urban Design (Section 11.03).

(ii) Identify opportunities for the retention and/or rehabilitation of natural waterways (Section 9.02(b)) and other natural water features that will be compatible with the urban landscape.

Phase C: Conceptual design of stormwater quality requirements

(i) Identify stormwater quality requirements from an existing Stormwater Quality Management Plan or identified Water Quality Objectives (Sections 2.06 and 11.06).

(ii) Identify those areas of land with topographic features best suited to specific stormwater treatment systems (e.g natural detention areas for wetland placement, and highly porous soils for infiltration systems).

(iii) Using appropriate modelling techniques prepare a preliminary design of the stormwater treatment system.

Phase D: Minor storm initial assessment

(i) Assess critical locations in the street network where roadway flow width is likely to be the limiting criterion under minor storm conditions. Refer to the limitations detailed in Figures 7.03.1 and 7.04.1 and Tables 7.03.1 and 7.04.1, e.g. intersections, sags, bus stops, kerb returns and intermediate locations. This provides an indication of sub-catchment boundaries.

(ii) Determine the area of the critical sub-catchments at the locations determined in (i) and calculate peak discharges for the minor storm event at these locations using standard inlet times (Table 4.06.1), the design average recurrence interval for the minor storm (Table 7.02.1) and weighted coefficients of runoff (Table 4.05.3).

Notes:

(a) Significant bypass will not normally occur at kerb inlets under minor storm conditions. Accordingly the use of standard inlet times will be appropriate when planning the initial layout of the system. If significant bypass does take place the time of concentration at downstream inlets will need to be appropriately adjusted.

(b) Depending upon the local rainfall intensity regime, kerb inlet capacity and assessed sub-catchment coefficient of runoff, the designer can readily determine the approximate maximum size of sub-catchment area that is likely to be acceptable.

e.g. For Q = 0.352 m^3/s (see note below).

t_c = 10 minutes

${}^{10min}I_2$ = 120 mm/h

C_{10} = 0.76

C_2 = 0.65

$$A = \frac{Q}{\left(2.78\,x\,10^{-3}\right).C_y.{}^tI_y}$$

$$= \frac{0.352}{\left(2.78\,x\,10^{-3}\right)x\,0.65\,x\,120}$$

$$= 1.62 \text{ ha}$$

Note: 0.352 m^3/s corresponds to the half road flow capacity of a road measuring 8m invert to invert and with 3% longitudinal slope and 2.5% crossfall.

(iii) For the longitudinal road slope determine the road capacity at critical locations based upon flow width and depth limitations.

(iv) From (ii) and (iii) determine the required inlet capacity and underground pipe capacity (if used) at the critical locations together with surface flows and bypasses for minor storm conditions.

Phase E: Major storm initial assessment

(i) Assess those critical locations in the street and overland flow network where flow capacity is likely to be the limiting criterion under major storm conditions. Refer to Figure 7.03.1 and Tables 7.03.1 and 7.04.1.

(ii) Determine total catchment peak discharge $Q_{T(\,)}$ at the critical locations under major storm conditions.

Notes:

(a) The critical locations under these conditions are likely to require a number of minor storm sub-catchments and significant bypass between sub-catchments will be permitted. Based upon a detailed assessment of overland flow time and channel flow time (Section 4.06.10) the peak discharge from the critical catchments can be determined.

(b) Where there is a significant difference between overland flow travel time and pipe flow time to the location in question, designers should consider the travel time of least duration, otherwise the designer should evaluate the catchment hydrology using an appropriate runoff-routing model.

(iii) Determine the permissible street flow capacity based on major storm criteria at the critical locations, $Q_{LIM(\,)}$.

(iv) Starting at the top of the catchment determine the pipe flow at the upstream end of the sub-catchment under consideration $Q_{PU(\,)}$ (see Figure 7.15.1).

(v) Subtract $Q_{PU(\,)}$ from $Q_{T(\,)}$ to establish the nett surface flow at the critical location under consideration.

(vi) Where the nett surface flow at the critical location is less than the permissible street or overland flow move to the next downstream critical location.

(vii) Where the nett surface flow at the critical location is more than the street or overland flow capacity then:

(a) allow for the provision of increased inlet and underground pipe capacity upstream of that point to accept the excess;

(b) modify the street cross-section;

(c) or otherwise increase the surface flow capacity.

(viii) Check that the calculated pipe capacity at the critical location is not less than that required upstream of that point. A reduction in pipe capacity would not occur unless provision is made for surcharge outflow.

(ix) Adopt trial pipe sizes for the hydraulic analyses to suit the greater of the flows derived during the major and minor storm hydrologic checks in accordance with the Flow Chart in Figures 7.15.2 (a), (b) & (c).

(x) The calculated major storm kerb inlet inflows and pipe flows are used for subsequent hydraulic analysis of the performance of the system under major storm conditions.

This procedure allows the identification of points where underground capacity needs to be increased to cater for the flow requirements of both the major and minor design storms. It ensures the selection of pipe sizes that are capable of conveying both major and minor storm discharges, and largely obviates iterative hydrologic and hydraulic analysis of the pipe system.

Thus, if $Q_{T()}$ = peak discharge from the total catchment at the critical location under consideration, based on Rational Method theory, i.e. it is not the sum of upstream sub-catchment discharges.

$Q_{LIM()}$ = permissible major storm street or overland flow at the critical location under consideration.

$Q_{P()}$ = required pipe discharge capacity at the critical location under consideration, i.e. at the downstream end of the catchment being considered. Thus $Q_{P(A)}$ = required pipe discharge at A.

$Q_{PU()}$ = sum of the pipe discharges at the critical locations immediately upstream of the location now under consideration, i.e. sum of $Q_{P()}$ values upstream.

$Q_{SURF()}$ = nett surface flow at the critical location assuming that no kerb inlets have been provided in the section immediately upstream of the critical location now under consideration.

$Q_{gs()}$ = required kerb inlet capacity of the inlets located in the section upstream of the critical location, i.e. between the location under consideration and the next upstream critical locations.

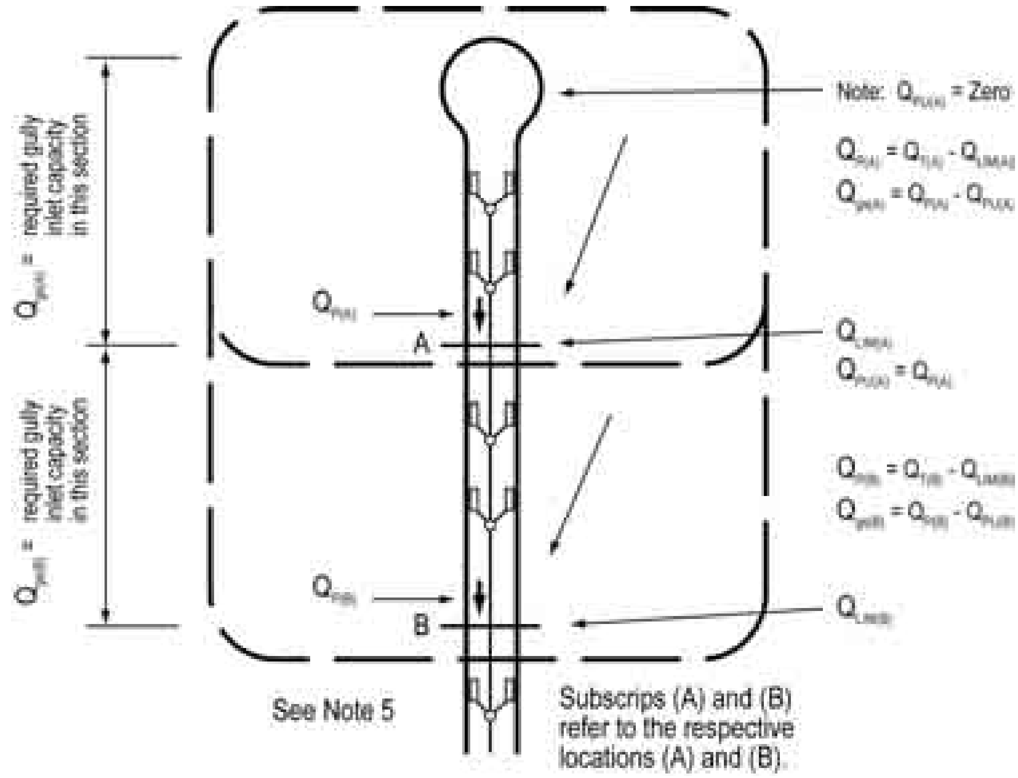
Then $$Q_{SURF()} = Q_{T()} - Q_{PU()} \quad (7.07)$$

$$Q_{P()} = Q_{T()} - Q_{LIM()} \quad (7.08)$$

$$Q_{gs()} = Q_{P()} - Q_{PU()} \quad (7.09)$$

$Q_{P(\,)}$ not less than Q_{PU}, or provide surcharge outflow structure if appropriate (see Note 4)

Figure 7.15.1 explains the above procedure, (example only).



Kerb inlet capacity for major storm
Figure 7.15.1

Notes:

1. The inflow capacity at kerb inlets under major storm conditions is expected to be equal to the inlet capacity under minor storm conditions unless elevated tailwater conditions under major storm conditions result in significantly reduced capacity, or the surface gradient is significantly flatter than the pipe gradient.

2. Where a number of minor storm sub-catchments exist upstream of the location being considered the capacity of the kerb inlet at that location may need to significantly exceed the minor storm inflow, in order to satisfy major storm criteria.

3. It should be assumed that kerb inlets will be designed with provision for blockage as detailed in Table 7.05.1.

   Accordingly there will be no need to further reduce the capacity of the underground drainage system under major storm conditions. This approach differs from that proposed by some authorities e.g. Argue (1986) etc. where reduction to 50% or zero pipe capacity is suggested.

4. Where Equation 7.09 results in a negative value of $Q_{g(\,)}$ the kerb inlet capacity required in that section to satisfy road flow capacity requirements is nil. In this case the method may also indicate a reduced pipe capacity requirement in the lower reach. However the pipe capacity will normally not be reduced unless provision is made for surcharge outflow.

5. Note that at point B, the peak discharge $Q_{T(B)}$ comprises flow from both catchments A and B etc.

Notes to accompany Figures 7.15.2 (a) and 7.15.2 (b)

1. Designers should endeavour to place kerb inlets at locations on grade where the width of spread of roadway flow is at the allowable limit as detailed in Section 7.04 and Figure 7.05.1.

2. The capacity of kerb inlets shall be determined from the "Kerb Inlet Capacity Charts" made available by the relevant local authority and modified to allow for blockage in accordance with Table 7.05.1.

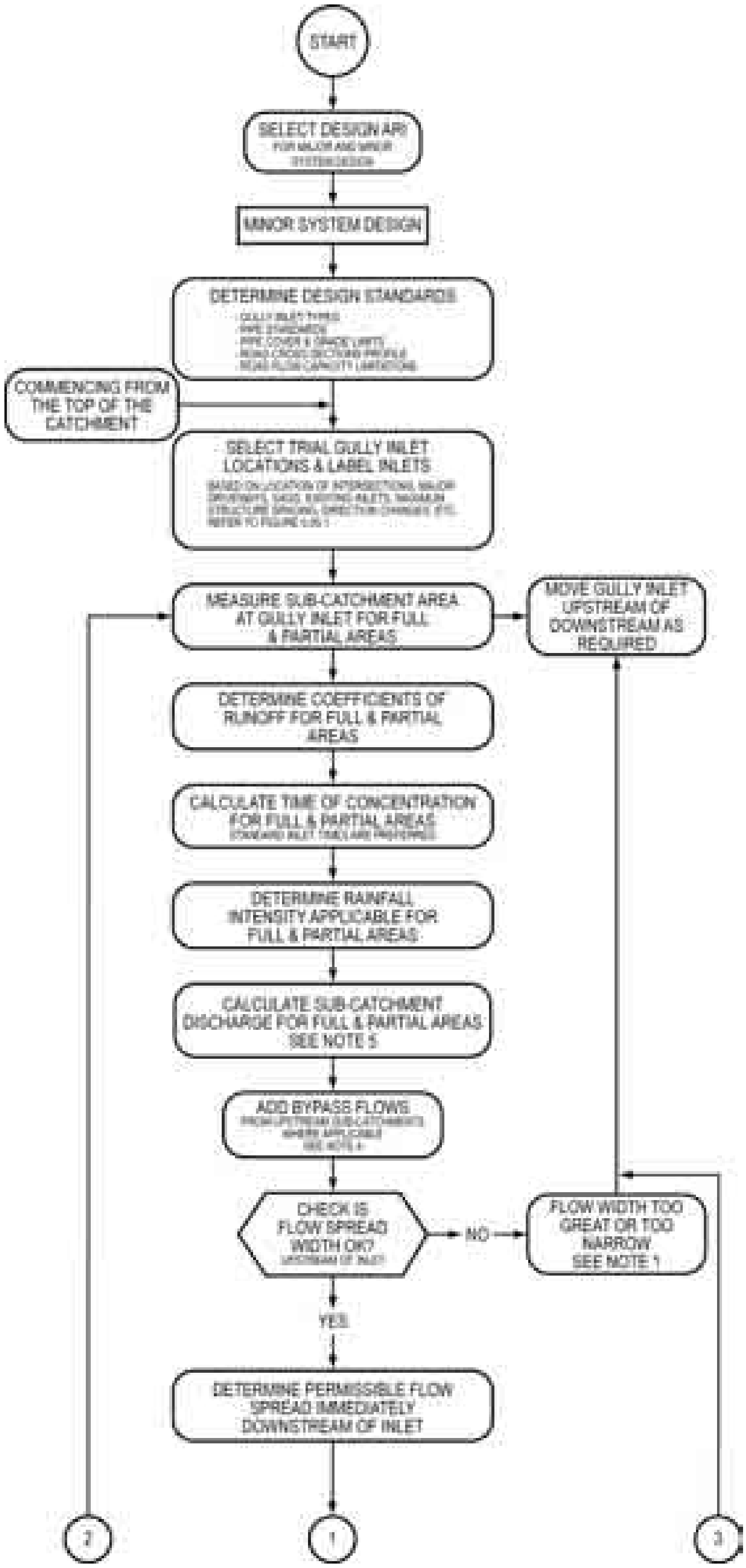
3. Constraints on the levels and gradient for pipe reaches may be caused by:
   (i) existing or future services e.g. sewer, water, gas, electricity;
   (ii) minimum cover under roadways;
   (iii) minimum or maximum depth for kerb inlets.

4. The bypass referred to is from other upstream catchments not from the uppermost of the two under consideration.

5. The peak discharge needs to be assessed for the full or partial area as for the minor storm design.

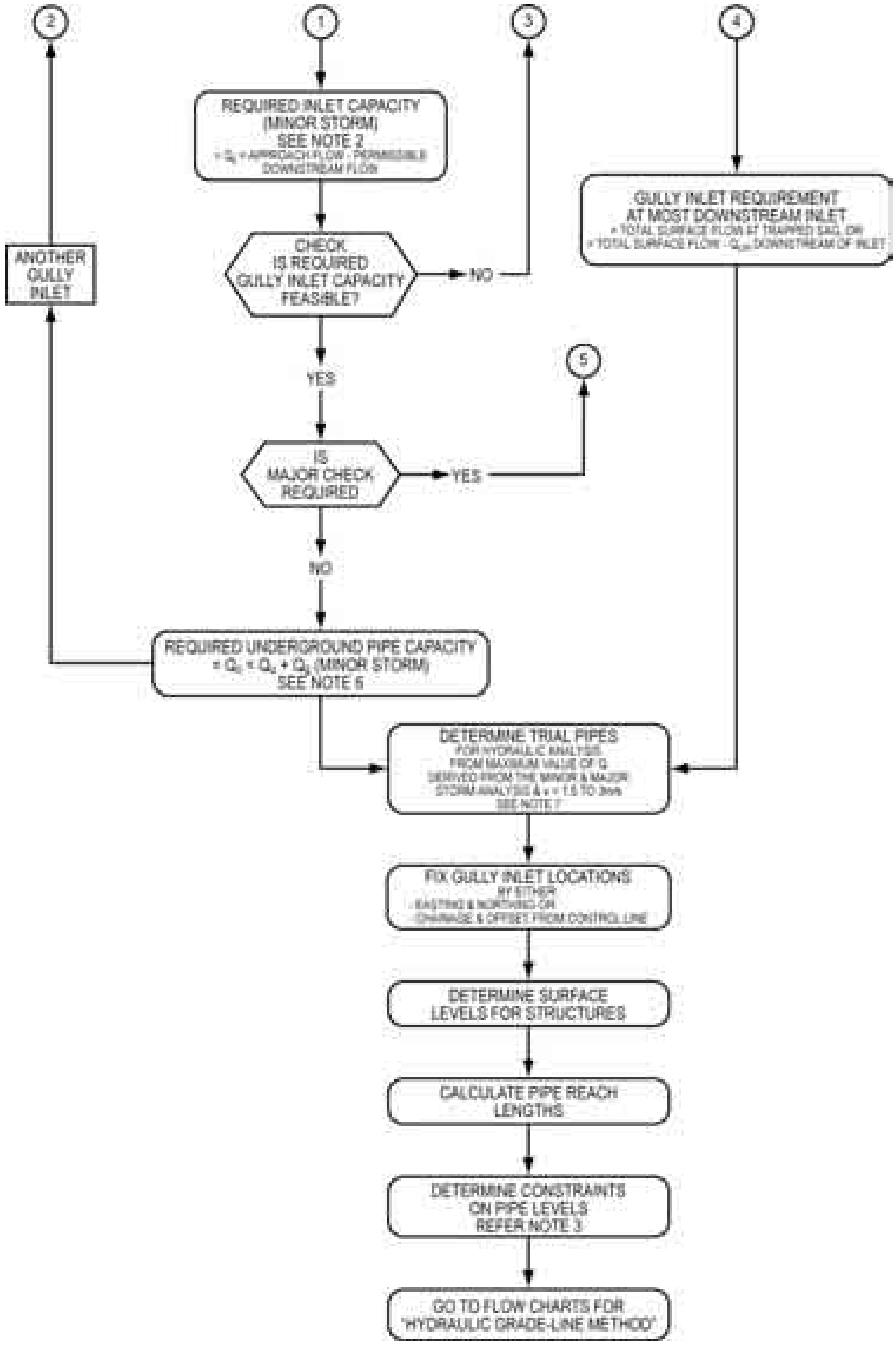
6. Q_u and Q_g are the inflows to the structure in accordance with Rational Method Theory and do not equal the sum of the upstream pipe and kerb inlet flows. Q_u may include lateral inflows.

7. The velocity limits indicated are those that should give optimum hydraulic conditions.



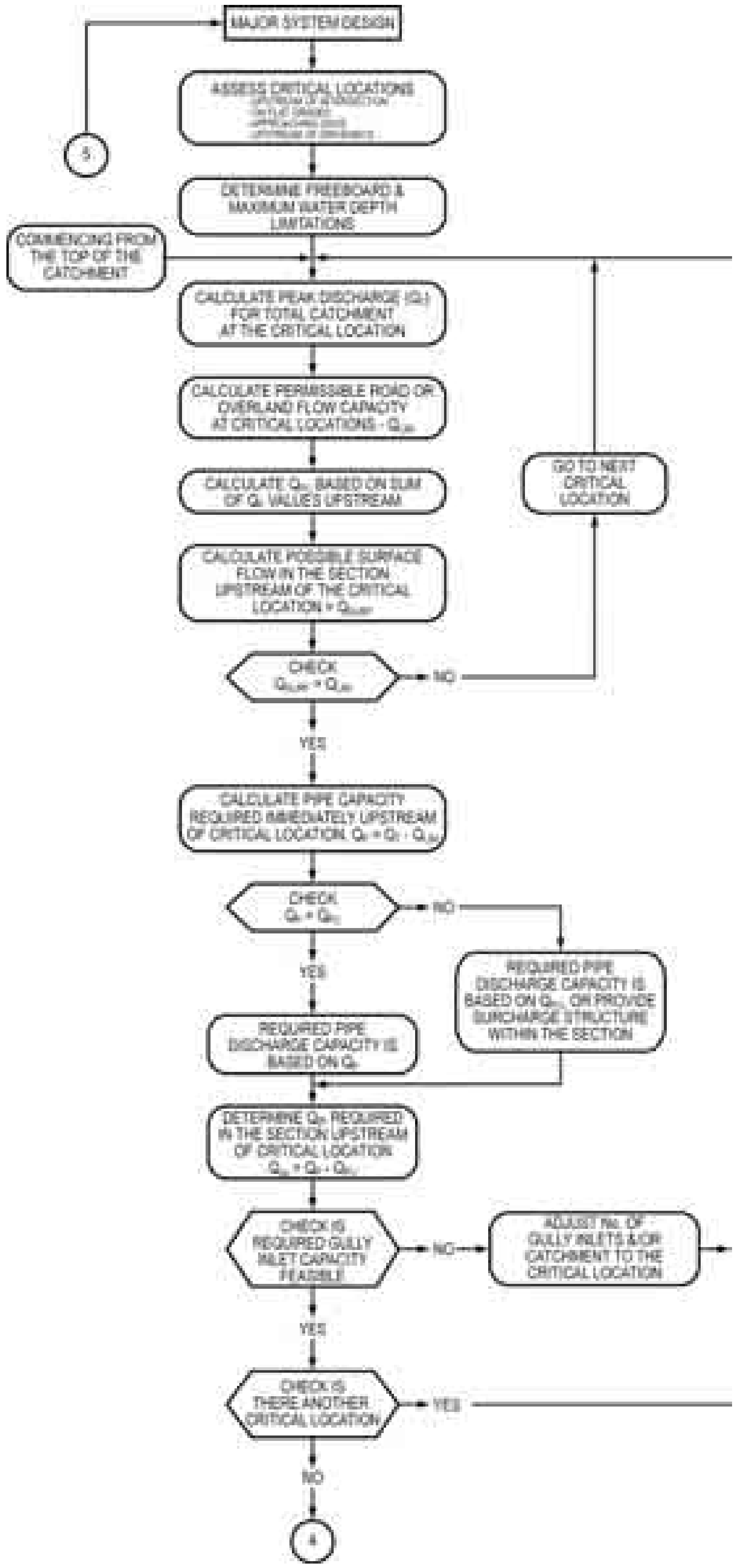
Flow chart for initial design assessment
Figure 7.15.2 (a)

Continued on Figure 7.15.2 (b)



Flow chart for initial design assessment
Figure 7.15.2 (b)

Continued on Figure 7.15.2 (c)



Flow chart for initial design assessment
Figure 7.15.2 (c)