5.00 Detention/retention systems

5.01 General

In the absence of adequate controls, urban development can increase both storm runoff volumes and peak discharge rates. Such increases can aggravate downstream flooding, initiate creek erosion and cause the degradation of downstream environmental values.

One of the main objectives of an urban drainage system is to limit property flooding to acceptable levels. The use of stormwater detention/retention systems is one means of achieving this objective. Another objective is to minimise the degradation of downstream environmental values. Unfortunately the *science* of designing stormwater drainage systems to achieve this latter objective is not as well understood. If inappropriately applied, some detention systems can aggravate creek erosion rather than decrease it.

In the context of this chapter, detention/retention systems include traditional detention basins, on-site detention (OSD), extended detention systems and stormwater retention devices, all of which have the effect of reducing and delaying peak flow rates.

5.02 Planning issues

While helping to reduce many of the impacts of urbanisation, detention and retention systems can also introduce new problems that designers and regulators should be aware of.

Potential problems that may be associated with the use of stormwater detention/retention systems are listed below.

(i) The creation of coincident flood peaks causing increased downstream flooding.

(ii) Cumulative increases in flows downstream of several basins resulting from the overlapping of the extended falling limbs of the various outflow hydrographs.

(iii) Increased potential for accelerated creek erosion downstream of the detention systems.

(iv) Extended periods of inundation of the basin area especially during the more frequent flood events.

(v) Potential salt intrusion of low-lying excavated basins.

(vi) Safety risks associated with both the flooded basin and its outlet structure.

Many of these problems can be avoided through detailed catchment planning. Preference should always be given to the use of total catchment modelling to determine the preferred location and operational requirements of stormwater detention/retention systems. Such modelling would usually be carried out in association with a *Flood Study*, *Stormwater Management Plan*, or *Master Drainage Study*.

If a total catchment model is being used to investigate the design and operation of a stormwater detention/retention system for a single land development, then the following issues need to be considered:

(i) It is inappropriate to consider the impact of a single development in isolation from the cumulative effects of full catchment development.

(ii) The cumulative effects of stormwater detention/retention should be determined by modelling the hydraulic conditions that would exist if all future land developments were conducted in accordance with the current Planning Scheme.

(iii) Consideration also needs to be given to the likely impacts of the development that would occur under existing catchment conditions.

(iv) The potential adverse impacts of waterway flooding needs to be considered over all reaches of a waterway where flood waters are likely to adversely affect either the "value" or "potential use" of the land.

5.03 Functions of detention/retention systems

Stormwater detention and retention systems perform a variety of functions depending on their design. A short description of these functions is provided below, with a summary provided in Table 5.03.1.

5.03.1 Detention systems

(a) Discharge control

On-site detention and regional detention systems may be designed to restrict peak outflows for selected design storms to either pre-development conditions, or to the maximum capacity of the existing downstream drainage network. These outflow restrictions may apply to the hydraulic capacity of the downstream drainage systems, or to safety issues associated with an overland flow path.

(b) Flood control

Detention systems can be used to alleviate flooding concerns resulting from past development activities, or from changing community attitudes to what is an acceptable flood risk.

Traditional detention systems delay stormwater runoff for a few hours, or fractions of an hour, while *Extended Detention Systems* can be used to store and discharge part of the total runoff over a period of 1 to 2 days. Extended detention systems can be effective for the management of new developments located within the lower half, or lower third of a catchment where traditional detention basins may aggravate downstream flooding due to the effects of coincident floodwave peaks.

(c) Erosion control

The operation of stormwater detention systems within a catchment can have both positive and negative impacts on downstream channel erosion. It should be noted that channel erosion within vegetated waterways is not solely governed by the peak discharge of flood events. Instead, it is the frequency and duration of near-bankfull flows that primarily governs channel erosion within these waterways.

Increases in peak flood flows without a significant increase in flood volume may cause a moderate increase the frequency and duration of near-bankfull flows. However, an increase in flood volume is likely to cause a significant increase in the frequency and duration of near-bankfull flows, especially if peak flood flows are restricted to pre-development conditions.

Unlike some retention systems, detention systems generally cannot be used to compensate for changes in runoff volume. Thus, in circumstances where urbanisation has increased the volume of runoff, the use of stormwater detention systems may contribute to an increase in the potential for downstream creek erosion.

Therefore, in most circumstances, detention systems need to operate in coordination with appropriate runoff-reducing stormwater measures (i.e. WSUD) if the objective of reducing the risk of downstream channel erosion is to be realised.

(d) Pollution control

Most detention basins provide little if any measurable water quality benefit, especially if an impervious low-flow drainage system is constructed through or below the open basin. Permanent sedimentation basins, however, can provide both stormwater detention and stormwater quality treatment (i.e. settlement of sediment and particulates).

Extended detention systems can provide water quality benefits through extended sedimentation and solar treatment. In some circumstances, filter basins and sand filters can be designed to operate as extended detention systems, thus providing both stormwater detention and stormwater treatment benefits.

5.03.2 Retention systems

(a) Rainwater harvesting

Household rainwater tanks operate as a stormwater retention system. In some cases the tanks may consist of two zones, one zone for stormwater detention (which freely drains after each storm), and one zone for rainwater harvesting.

Under certain geological conditions, stormwater captured in retention basins may be injected into underground aquifers as a water storage measure. Argue (2004) provides guidelines on such practices.

The use of retention systems for stormwater harvesting and the design of rainwater tanks will not be discussed within this chapter. Designers should refer to the relevant local government guidelines.

(b) Control of runoff volume

Stormwater retention systems can be designed to reduce the total annual runoff volume, and/or reduce the runoff volume from a specified design storm. Reducing the *total annual runoff volume* provides water quality benefits, especially in circumstances where the stormwater ultimately flows to a large, semi-confined water body such as a lake, river, estuary or bay. Reducing the *runoff volume from a specific storm event* can be beneficial for the control of erosion and flooding in minor watercourses such as creeks.

Stormwater retained within these systems may be made available for secondary (non potable) purposes through a stormwater harvesting system, or removed from the system through infiltration and/or evaporation.

(c) Pollution control

Retention systems may incorporate stormwater quality treatment measures, such as a pond or wetland, or they may actually be the treatment measure, such as an infiltration trench or basin.

5.03.3 Summary of functions

A summary of the possible functions of detention and retention systems is provided in Table 5.03.1.

Table 5.03.1 Summary of detention/retention system functions

		Discharge Control	Flood Control	Volume Control	Scour Control	Stormwater Harvesting	Pollution Control
Detention Systems	**On-site Detention**	✔	✔				
	Detention Basins	✔	✔		[1]		[1]
	Extended Detention Basins [2]	✔	✔		[1]		✔
	Filter Basins	[1]	[1]				✔
Retention Systems	**Rainwater Tanks**	[3]		[4]		✔	
	Retention Basins	✔	✔	✔	[1]	✔	✔
	Infiltration Trenches	✔	✔	✔	[1]		✔
	Infiltration Basins	✔	✔	✔	[1]	[1]	✔

Notes:

[1] Not the normal function of this type of system, however, this function may be achieved if modifications are made to the design.

[2] The most commonly used terminology is *Extended Detention Basin*, however, the concept of extended detention may also apply to the design of retention basins.

[3] Generally rainwater tanks cannot be used for on-site discharge control.

[4] When wide spread across a catchment, rainwater tanks can contribute to runoff volume control through activities such as water reuse, garden watering and groundwater infiltration.

5.04 Design standards

5.04.1 General

Design standards depend on the required functions of the detention/retention system. **If the detention/retention system is required to satisfy more than one function, e.g. flood control and the control of creek erosion, then consideration must be given to all specified design requirements.**

In all cases, detention/retention systems must not cause unacceptable increases in flood levels upstream or downstream of the system. An "unacceptable increase in flooding" would include any change in flood characteristics on surrounding properties that could cause damage to, or adversely affect either the *value* or *potential use* of the land, or cause problems resulting from changes in flow velocity or the distribution of flow velocity within that land.

5.04.2 On-site detention systems

There are generally three design standards set by regulating authorities, they are:

(i) a specified minimum site storage requirement (SSR) and permissible site discharge (PSD) relative to either the site area, land use, or the change in impervious area;

(ii) a permissible site discharge for the specified design storm frequency with no minimum storage volume specified;

(iii) a requirement not to exceed pre-development peak discharge rates for a range of design storm frequencies.

The first two design criteria are often adopted by local governments following the development of a regional flood control strategy, Master Drainage Plan, or Stormwater Management Plan.

Most small on-site detention systems incorporate underground tanks. When appropriate soil and groundwater conditions exist, some underground tanks can be converted into infiltration systems. Aboveground tanks are rarely used on single residential properties because of the risk of the tanks being converted to rainwater tanks.

5.04.3 Flood control systems

Traditionally detention basins have been designed to ensure no increase in post development peak discharge immediately downstream of the basin for specified storm events such as 1, 2, 10, 50 and 100 year ARIs. Satisfying this criterion however, will not necessarily guarantee that there will be no adverse impacts on flood levels well downstream of the development. The full impacts of a stormwater detention system can only be assessed by modelling the full catchment, including all flood prone areas downstream of the detention system.

An increase in downstream flooding may occur for one or more of the following reasons:

(i) changes in the speed of the flood wave passing down the catchment and the resulting risk of coincident flooding;

(ii) changes in the volume of stormwater runoff from new land developments and the impact this has on the “shape” of the basin’s discharge hydrograph.

An increase in runoff volume is an inevitable result of traditional urban development. Thus, discharge rates within the rising and/or falling limb of a detention basin outflow hydrograph may be significantly higher than the corresponding pre-development discharge rates. If several detention basins are located within a given catchment, then these increased discharge hydrographs may overlap causing an increase in flood flows and flood levels downstream of the basins.

Significant hydrologic modelling was carried out during the development of this edition of QUDM in order to establish a simple design procedure that would avoid the problems of overlapping discharge hydrographs; however, no procedure could be established.

One of the benefits of adopting Water Sensitive Urban Design (WSUD) is that it reduces the potential for increases in runoff volume thus reducing the potential for increases in downstream flows and flooding.

(a) Greenfield and infill Developments

In cases where the design requirements of a detention/retention system has not been determined from an appropriate total catchment study, the recommended sizing of such a flood control system shall be based on achieving the following minimum requirements:

(i) No increase in flood levels on adjoining land where such an increase would cause damage to, or adversely affect either the “value” or “potential use” of the land.

(ii) No increase in peak discharges immediately downstream of the development for a selected range of storm durations, for a selected range of ARIs up to the “Defined Flood Event”.

Technical Note 5.04.1:

Point (ii) above indicates that the peak discharge for each of the selected storm durations shall not increase even if that storm duration does not produce the highest peak discharge for the given ARI.

It is recommended that the selected storm durations tested should include the 1-hour storm, 3-hour storm and a storm of duration at least three times the critical storm duration of the detention/retention basin.

Exceptions to this rule may be considered by the local government only if a storm of a given duration does not inundate floor levels or adversely affect the potential "use" of land upstream or downstream of the basin.

In cases where the design requirements for detention/retention systems have been determined from an appropriate total catchment study, the recommended modelling of such flood control systems shall be based on achieving the following minimum requirements:

(i) No increase in flood levels on land adjoining a basin where such an increase would cause damage to, or adversely affect either the "value" or "potential use" of the land.

(ii) No increase in peak flood level and/or discharge at any location downstream of any basin where existing land owners/users may be adversely affected by such an increase. This requirement shall apply to a full range of storm duration and frequencies up to the "Defined Flood Event" where such storms result in flooding that either inundates floor levels or adversely affect the potential "use" of land.

Technical Note 5.04.2:

Point (ii) above indicates that the peak flood level or discharge for each of the selected storm durations shall not increase even if that storm duration does not produce the highest peak discharge for the given ARI.

It is recommended that the selected storm durations tested should include the 1-hour storm, 3-hour storm and a storm of duration equal to the critical storm duration for the most downstream flood-affected property.

Exceptions to this rule may be considered by the local government only if a storm of a given duration does not inundate floor levels or adversely affect the potential "use" of land upstream or downstream of the basin.

(b) Control of existing flooding problems

Flood control detention/retention basins constructed to alleviate existing flooding problems should be designed to achieve one or more of the following outcomes:

(i) Maximum flood attenuation benefits from the available land area (i.e. where storage volume is limited by site constraints). This option usually requires the basin's low-flow outlet to be sized to make maximum use of the safe hydraulic capacity of the downstream drainage system. The local authority should be consulted when determining the maximum allowable discharge rate into the downstream drainage system.

(ii) All requirements listed above for greenfield developments.

5.04.4 Control of accelerated channel erosion

If one of the primary objectives of a stormwater system is to minimise the risk of accelerated channel erosion, then consideration must be given to those measures that will minimise changes to:

(i) the frequency and duration of near-bankfull flows; and
(ii) the peak discharge of stream flows greater than or equal to the bankfull flow rate.

This can usually be achieved, in part, by:

(i) adopting the principles of Water Sensitive Urban Design;
(ii) minimising changes in impervious surface area, particularly on highly porous soils;
(iii) decreasing the percentage of directly connected impervious surfaces;
(iv) maximising stormwater infiltration;
(v) using rainwater harvesting to minimise changes to runoff volume;
(vi) adopting stormwater retention rather than detention systems.

In this context, the primary aim is <u>not</u> to reduce changes in the "annual runoff volume", but to reduce changes in the runoff volume of those storms that are likely to contribute to near-bankfull flows. Thus the focus is likely to be on storms with an ARI between 1 in 1 year and 1 in 10 years.

It is noted that this requirement is different from that used in the management of stormwater quality and the protection of instream ecology where the aim is to reduce changes in the "annual runoff volume" and the "total water cycle".

It should also be noted that an increase in the frequency and duration of low flows within a waterway (i.e flows less than the 1 in 1 year ARI) may increase the stress on instream aquatic ecology and habitats. Thus the only way to minimise the risk of both accelerated channel erosion, and a decline in aquatic habitats, is to minimise changes to the natural water cycle, including the frequency, duration, velocity, volume and peak discharge of all runoff events.

No specific design procedures are provided in this Manual because such procedures are currently [2007] being prepared in association with new WSUD guidelines for Queensland and various State planning policies and schemes.

5.05 Flood-routing

5.05.1 Initial sizing

Initial sizing of a detention basin may be undertaken in order to assess the feasibility of a basin as a flood management option, or to determine the order of magnitude of the storage required.

The initial sizing of a basin volume ($\boldsymbol{V_s}$) can be undertaken by a comparison of the following estimation procedures.

(i) $\boldsymbol{V_s} / \boldsymbol{V_i} = r(1+2r)/3$ (5.01)
(After Culp 1948)

(ii) $\boldsymbol{V_s} / \boldsymbol{V_i} = r$ (5.02)
(After Boyd 1989)

(iii) $\boldsymbol{V_s} / \boldsymbol{V_i} = r(3+5r)/8$ (5.03)
(After Carroll 1990)

(iv) $\boldsymbol{V_s} / \boldsymbol{V_i} = r(2+r)/3$ (5.04)
(After Basha 1994)

where r is the reduction ratio calculated as:

$$r = (Q_i - Q_o)/Q_i \qquad (5.05)$$

The above procedures may give widely different answers and thus should be used with care. Typically Basha's equation produces a result closest to an average of the four methods.

If the Rational Method is used for the determination of Q_i, then the initial estimate of the inflow volume ($\boldsymbol{V_i}$) may be determined as: $\boldsymbol{V_i} = 4t_cQ_i/3$.

Technical Note 5.05.1:

The above equations are most appropriate when it is necessary to limit the peak discharge for only the nominated design storm, such as the 1 in 100 year ARI. In those circumstances where it is necessary to control peak discharges for a range of storm frequencies, or where it is essential to ensure that the post development peak discharge for each tested storm duration is not increased, then these equations are likely to significantly underestimate the required detention volume.

5.05.2 Final sizing

The final sizing of the basin should be completed with the aid of a computer model. The selected model must accurately simulate the hydraulic behaviour of the basin outlet, especially when partial full pipe flow or tailwater submergence occurs.

To account for the effects of urbanisation upon the flood hydrograph the procedures contained in Section 4.09 are recommended.

Technical Note 5.05.2:

As an alternative to the use of a computer model, the final sizing can be undertaken by manual flow routing based on a direct solution of the storage equation:

$$(I_1 + I_2) + (2S_1/T - Q_1) = (2S_2/T + Q_2) \tag{5.06}$$

where: I = the inflow rate
S = the volume in storage
Q = the outflow rate
T = the routing time step
$_{1,\,2}$ denote the start, finish of the routing step

Equation 5.06 requires the shape of the inflow hydrograph to be determined. Full details of the procedure are given in Book 5, ARR (1998).

Whichever technique is used for final basin sizing, the routing time-step or increment must be short enough relative to the storm duration to ensure that the peak storage requirements will be accurately determined.

The design of the basin and its outlet structures must be based on a range of storm durations and appropriate temporal patterns in order to identify the critical hydraulic dimensions. If the basin is required to prevent an increase in flooding at a given location downstream of the basin, then the performance of the basin needs be checked for a storm of duration equal to the critical storm duration of this downstream location. If the basin is required to prevent an increase in flooding at all locations downstream of the basin, then the performance of the basin needs be checked for a range of storm durations up to the critical storm duration of the most downstream location.

Note; it is not sufficient to simply determine which storm duration produces the largest peak discharge from the basin. Even though a storm of greater duration than the basin's critical storm duration produces a lower peak discharge, it may require a greater detention volume to prevent an increase in the peak discharge of such a storm.

5.05.3 Temporal patterns

The design of the low-level outlet can normally be based on the average temporal patterns given in the latest version of ARR. Design of the high-level outlet and the embankment crest height should account for the fact that the temporal patterns given in ARR are only the averages of the many storm bursts that can actually occur. It should also be noted that these temporal patterns do not represent full storms, but just the worst burst within a longer storm.

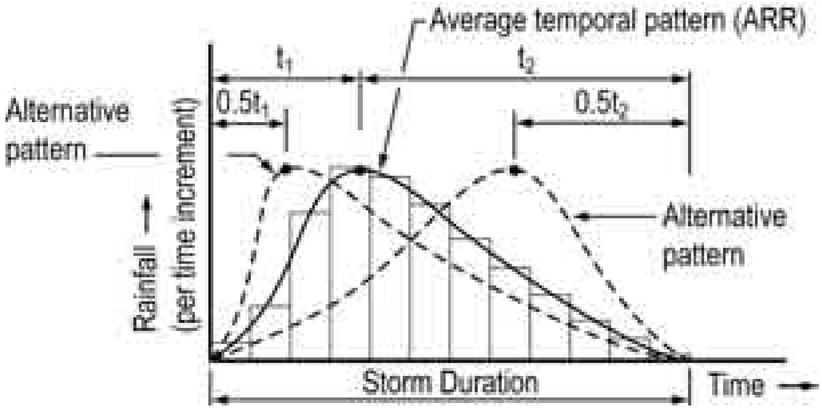
Designers should confirm with the relevant regulating authority the types of temporal patterns to be used. It is recommended that the response of the basin should also be checked using real storms, even if such storms have an ARI significantly different from the design storm.

If data from a real storm with an ARI similar to the specified design storm is not available, then the size of the basin should be checked using the following three alternative temporal patterns, in addition to the average pattern:

(a) A pattern in which the peak intensity is located midway between the start of the storm and the peak of the average pattern.

(b) A pattern in which the peak intensity is located midway between the end of the storm and the peak of the average pattern.

(c) A pattern recorded during a major storm at a rainfall gauging station near the site, if available.



Additional temporal patterns for use in design of embankments and high level outlets

Figure 5.01

5.05.4 Allowance for existing channel storage

When a hydrologic analysis is performed on a detention/retention basin located within a waterway, it is important to ensure that:

(i) The flood mitigation effects of the existing channel storage are not duplicated within both the channel routing component of the model (i.e. routing from node to node) and the detention storage routing (i.e. flood routing through a basin at the downstream node).

(ii) Appropriate consideration is given to the potential effects of lead-up rainfall prior to the storm burst as normally occurs in real storms.

The first issue (i) may be addressed by reducing the modelled basin storage by the measured natural channel storage. Alternatively, a new *node* may be inserted at the upstream influence of the basin (i.e. limit of the basin's backwater effects), with the flood routing coefficient adjusted so that there is no flood attenuation between the upstream and downstream basin nodes (i.e. for Muskingum routing, x = 0.5).

The second issue (ii) may be addressed by modelling the basin using real storm data to assess likely storage levels prior to a storm burst.

5.06 Basin freeboard

Recommendations on the selection of freeboard are provided in Table 5.06.1.

Table 5.06.1 Guidelines for basin freeboard requirements

Situation	ARI (years)	Maximum Depth or Level
Basin formed by road embankment (a)	20	Bottom of pavement box
(b)	50	0.3 m below edge of shoulder
Basin formed by railway embankment	50	Underside of ballast
Large basins with separate high level spillway	100	Embankment crest with Freeboard ≥ 10% of the 100 year ARI storage depth and with minimum freeboard = 0.3 m [1]

Note:

[1] Freeboard must fully contain the potential wave height if the resulting overtopping is likely to represent a safety risk to the embankment or undesirable erosion. U.S. Army Corps of Engineers (1984) provides guidelines on the estimation of wave height.

5.07 Basin floor drainage

Design of the low-flow drainage system through the basin will depend on numerous factors including the required dry-weather function of the basin, the need for water quality treatment of the low flows, and safety and maintenance issues. General guidelines for the design of low-flow channels are provided in Section 9.08 of this Manual.

The design of the basin floor should take appropriate consideration of the following recommendations:

(i) Minimum cross gradient of 1 in 80 for grassed basins to allow efficient surface drainage (this is based on the recommended minimum cross fall for school ovals).

(ii) Minimum cross gradient of 1 in 100 for vegetated basins (i.e. deep-rooted plants such as trees and shrubs). Minimum cross gradient does not apply if the basin floor is a natural drainage surface.

(iii) Minimum invert level of mowable grassed areas at least 300 to 500mm above the invert of an adjacent stream (i.e. on-line basin). The range 300 to 500mm depends on the soil drainage properties and the degree of sedimentation likely to occur within the stream channel.

If field inlets are used to help drain the basin floor, then adequate scour protection needs to be placed around the inlet as discussed in Section 7.05.4(c).

It is noted that safety issues may require an inlet screen of sufficient size to limit flow velocities through the screen to a maximum of 1 m/s. Minimum dimensions of dome inlet safety screens are presented in Section 12.04.8.

5.08 Low-level outlet structures

5.08.1 Outlet types

Low-level outlet structures generally consist of orifice plates, pipes or culverts placed at a low level in the basin to cater for the discharge of normal outflows.

Recommendations for the design of outlet structures are given by the American Society of Civil Engineers (1985). Hydraulic relationships for various outlet structures are provided in the User Manuals for software packages such as DRAINS, RAFTS and RORB. The storage-discharge curve used in the flood-routing analysis must accurately reflect expected hydraulic conditions including allowances for part-full pipe flow, inlet/outlet control where appropriate, partial blockages and the effects of external catchments on the hydraulic grade line.

Low-level outlet structures for small basins (Figure 5.02) will generally consist of a single orifice or pipe. In some cases a pump will be installed with capacity designed to match the outflow limitation only at the ARI at which the high-level outlet just begins to operate. Where a pump is allowed by the local government, a stand by power supply may be required.

Typical outlets for small basins
Figure 5.02

Low-level outlet structures for large detention basins will more often be required to limit the outflows over a range of intermediate ARIs up to the ARI for the Design Flood. In such cases, the low-level outlet structure may comprise either a single-level outlet sometimes preceded by a weir, or a multi-level outlet.

A weir located immediately upstream of a single-level outlet may have an orifice of smaller diameter than the main basin outlet to attenuate the outflows for smaller ARIs and to provide free drainage for the ponded water. During higher inflows the weir will overtop. A multi-level outlet will have a range of pipes or culverts set at different levels, possibly of different sizes to achieve the required attenuation throughout the ARI range.

If the basin outlet is directly connected to a downstream piped drainage network, then this system should be checked for undesirable surcharge. A full HGL analysis may be required by the regulating authority.

5.08.2 Protection of basin outlet

The intake to a detention basin outlet should be protected against expected debris blockages and designed to minimise the safety risk to a person trapped against the outlet structure. The level of protection will vary depending on the consequences of failure caused by blockage of the intake and the potential frequency of blockage.

Consideration should also be given to the consequences of a fully blocked low-level outlet.

Protection can be achieved by the installation of a trash rack, bar screen and/or a fence. These should be designed to shed debris and to assist egress by persons trapped in the basin generally in accordance with the recommendations of Weisman (1989) and Section 12.04 of this Manual. Trash racks comprising inclined vertical bars (inclined in the direction of flow) and spaced horizontal support bars are preferred.

Design criteria for intake structures are given in Table 5.08.1.

Table 5.08.1 Criteria for basin outlet structures

Item	Criterion
Spacing of vertical bars	125 mm (max)
Inclined spacing of horizontal supports	600 mm (max) [1]
Nett clear opening area	≥ 3 times the calculated outlet area [2]
Limiting velocity through trash rack [3]	0.6 m/s (not readily accessible) 1.5 m/s (accessible)

Notes:

[1] The maximum (inclined) spacing of horizontal supports aims to allow a trapped person to climb up the screen to safety.

[2] The calculated outlet area may depend upon the level of the outlet relative to the water surface. Where the outlet is contained in a drop structure, the outlet area used for determination of the nett clear opening for the intake may need to be adjusted to account for the level difference.

[3] The limiting velocity through the trash rack should be related to the accessibility of the intake structure for cleaning purposes.

Detailed procedures for determining the hydraulic losses through trash racks are given in Chow (1959) and U.S. Bureau of Reclamation (1987) otherwise refer to Section 12.04.6 of this Manual.

5.08.3 Pipe protection

Outlet pipes should have spigot and socket rubber-ring joints and lifting holes should be securely sealed. Pipe and culvert bedding should be carefully specified to minimise its permeability. Cut-off walls or seepage collars must be installed where appropriate, to control seepage and prevent piping failure adjacent to the outlet pipe.

Appropriate measures, such as internal sealing of pipe joints and lifting holes, and bolting down of access chamber lids, should be applied to any existing downstream systems which could be pressurised by the discharge from the outlet. Alternatively, surcharge chambers may need to be incorporated into the outlet pipe to limit the internal pressure.

5.08.4 Outfall protection

Where the outlet from a basin is to a free outfall, this should be located, where possible, within a well-defined natural depression or watercourse. The outlet should also be located a suitable distance upstream of the downstream property boundary to ensure that the downstream properties will not be adversely affected by the velocity or the concentration of the outflow. Adequate protection must be provided both downstream and immediately upstream of the outlet, where appropriate, to prevent scour.

5.09 High-level outlet structures

5.09.1 Extreme flood event

The designer may select the storage level at which the high-level outlet will begin to discharge; however, care must be taken to ensure that flooding of upstream properties is not worsened.

The spillway and embankment should be designed both hydraulically and structurally to permit the safe discharge of floods in excess of the Design Flood. The ARI of the *Extreme Flood* for which the performance of the basin should be checked, needs to be determined with appropriate consideration of the likely consequences of failure, and in consultation with the local government. ANCOLD (2000a) provides a basis for determining the ARI of the Extreme Flood based upon consideration of the incremental hazard associated with failure. Designers should refer to ANCOLD (2000a & 2000b).

Table 5.09.1 shows the range of ARIs applicable.

Table 5.09.1 Recommendations for extreme flood [1]

Incremental Flood Hazard Category [2]	Extreme Flood ARI (years)
Extreme	PMF [3,4]
High A	PMP Design Flood [3,5]
High B	10,000 to PMP Design Flood or 1,000,000
High C	10,000 to PMP Design Flood or 100,000
Significant	1,000 to 10,000
Low to very Low	100 to 1,000

Notes:

[1] Sourced from ANCOLD 2000a

[2] Refer to Table 5.09.2.

[3] Pre-flood reservoir level to be taken as the maximum normal operating level of the reservoir.

[4] PMF refers to Probable Maximum Flood

[5] PMP Design Flood refers to flood hydrograph generated by the Probable Maximum Precipitation

Table 5.09.2 Hazard categories [1]

<table>
<tr><th rowspan="2">Population at Risk</th><th colspan="4">Severity of Damage and Loss</th></tr>
<tr><th>Negligible</th><th>Minor</th><th>Medium</th><th>Major</th></tr>
<tr><td>0</td><td>Very Low</td><td>Very Low</td><td>Low</td><td>Significant</td></tr>
<tr><td>1 to 10</td><td>Low [2,5]</td><td>Low [5,6]</td><td>Significant [6]</td><td>High C [7]</td></tr>
<tr><td>11 to 100</td><td rowspan="3">[2]</td><td>Significant [2,5]</td><td>High C [7]</td><td>High B [7]</td></tr>
<tr><td>101 to 1000</td><td rowspan="2">[3]</td><td>High A [7]</td><td>High A [7]</td></tr>
<tr><td>> 1000</td><td>[4]</td><td>Extreme [7]</td></tr>
</table>

Notes:

[1] Sourced from ANCOLD 2000b.

[2] With a population at risk of 5 or more, it is unlikely that the severity of damage and loss will be “Negligible”.

[3] “Minor” damage and loss would be unlikely when the population at risk exceeds 10.

[4] “Medium” damage and loss would be unlikely when the population at risk exceeds 1000.

[5] Change to “Significant” where the potential for one life being lost is recognised.

[6] Change to “High” where there is the potential for one or more lives being lost.

[7] Refer to ANCOLD (2000b) – Section 2.7 and 1.6 for explanation of the range of High Hazard Categories.

5.09.2 Spillway design

The high-level outlet, usually formed by a spillway, must be designed to safely convey extreme outflows from the basin. The design flow should consider the potential for full or partial blockage of the low-flow outlet. Wherever practical, design of the spillway should assume full blockage of the low-flow outlet.

Where possible, the spillway should be cut into virgin ground at the side of the embankment, or otherwise located to minimise the possibility of embankment failure.

In some circumstances the high-level outlet may be constructed as a glory-hole inlet (with bar screen and anti-vortex device as required) leading to a pipe or a culvert through the embankment.

The spillway chute may be protected by riprap, concrete, paving, or other suitable coverings. A grass or reinforced grass cover may be adequate where spillway slopes are flatter than 1 on 6 (1V:6H). Care should be taken to maintain a healthy, continuous grass cover on grass spillways. Trees, shrubs, watering tap outlets, or any other fixed structure that may cause turbulence or eddy-induced erosion must not be located within a grassed spillway chute. Design information for grassed spillways is described by the U.S. Soil Conservation Service (1979).

5.10 Embankments

Detention basins are intermittent water-retaining storages for which the embankments do not need to be as rigorously designed as dams unless they are particularly high or have special soil problems. Retention basins designed to have a permanent or semi-permanent water storage need particular design measures if the retention depth is significant. Nevertheless, the design of the embankment should be undertaken, or at least reviewed by a suitably experienced Geotechnical specialist.

The sides of grassed embankments, including any inner basin grassed slopes, should generally be flatter than 1 on 6 and never steeper than 1 on 4. The top-width should be at least three (3) metres. Steeper slopes may be used on embankments or basins lined with structural facings or low-maintenance ground covers, but steps must be provided at appropriate intervals if the steepness of the slope could impede the egress of a person from the basin during a flood.

5.11 Public safety issues

While detention basins are generally less hazardous than drainage channels because of the slower movement of water, the associated safety hazards are often less obvious to the public. The hazards associated with off-stream basins (i.e. basins not directly connected to a watercourse) are likely to be less obvious than those associated with on-stream basins, thus greater consideration may need to be given to safe egress from off-stream basins.

The side slopes of basins should preferably be 1 on 6 or flatter to allow easy egress up the likely "wet" surface. Areas with slopes steeper than 1 on 4 will require steps and a handrail to assist egress. These recommendations especially apply to basins that incorporate dual use activities such as passive or active recreation.

The provision of exclusion fencing around open water stormwater detention/retention systems should be considered a last resort. Wherever practical, the first preference should be to minimise the safety risk through appropriate design.

Where suitable land is available, designers should aim to restrict basin depths to 1.2m at the 20 year ARI level and, if possible, for a greater recurrence interval. In cases where this is neither practical nor economical, and the provision of a detention basin is considered to be better on safety grounds than other alternatives, greater depths may be acceptable. Not withstanding this, designers are responsible for:

(i) investigating the overall safety risks associated with the basin;
(ii) design of the basin and the surrounding landscape in a manner that minimises these safety risks;
(iii) satisfy any safety requirements specified by the local government.

Suitable safety provisions (such as raised refuge mounds within large basins, fences and warning signs) should be provided for deeper basins.

Depth indicators should be installed within the basin and in the channel downstream of the embankment for basins with a storage depth of greater than one (1) metre. The indicator within the basin should have its zero level relative to the lowest point in the basin floor.

Special attention should be paid to basin outlets to ensure that persons trapped in the basin's water are not drawn into the basin's outlet system. Rails, fences, anti-vortex devices, trash racks or grates should be provided where necessary. Outlet systems should be located well away from the water's edge of the flooded basin such that a person wading along the edge of the basin cannot be "drawn" into the basin's outlet. This usually requires the outlet system to be located well away from the embankments.

5.12 Statutory requirements

Works constructed within a watercourse generally require approval under the *Water Act 2000* and need to satisfy all legal requirements of this Act. Reference should be made to this Act for definition of the term "watercourse", also refer to Department of Natural Resources and Mines (2002).

Under the *Water Act 2000,* and under common law, responsibility for the safety of a dam rests with the dam owner. Dam owners may be liable for loss and damage caused by the failure of a dam or the escape of water from a dam. Consequently, dam owners need to be committed to dam safety and have an effective dam safety management program. A dam safety management program is intended to minimise the risk of a dam failing and to protect life and property from the effects of such a failure should one occur.

In addition the embankment for a detention basin may be a Referable Dam requiring the approval of the Chief Executive Officer of the State agency responsible for administering the Water Act.

A dam is referable if:

(i) a failure impact assessment is required to be carried out under the *Water Act 2000;*

(ii) that assessment states that the dam has or will have a Category 1 or Category 2 failure impact rating;

(iii) the Chief Executive has, under the Water Act 2000, accepted the assessment.

In addition, some dams may be made referable by:

- a regulation made under the *Water Act 2000*, or
- the transitional provisions in the *Water Act 2000*.

A failure impact assessment is required when a dam is or will be:

(i) more than 8 metres in height and have a storage capacity of more than 500 megalitres;

(ii) more than 8 metres in height and have a storage capacity of more than 250 megalitres, and a catchment area that is more than 3 times the surface area of the dam at full supply level.

Additionally, the Chief Executive may give a dam owner a notice to have a dam failure impact assessed (regardless of its size), if the Chief Executive reasonably believes the dam will have, a Category 1 or Category 2 failure impact rating.

Referable dams are classified according to categories which are based on the population at risk if the dam fails, therefore, a failure impact assessment is required for a detention basin to establish if it is a referable dam.

- Dams with a Category 1 failure impact rating have between 2 and 100 people at risk.
- Dams with a Category 2 failure impact rating have over 100 people at risk.

If less than 2 people are at risk by the dam failing then the dam is not referable under the *Water Act 2000*.

5.13 References

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6.00 Computer models

6.01 Introduction

As with all computer software, designers are expected to be familiar with the underlying concepts used, the limitations of those concepts and the capabilities/limitations of the programs themselves. Further guidance on the use of numerical models is provided in *Australian Rainfall and Runoff* (ARR) and *Australian Runoff Quality* (ARQ).

Designers should be aware of the need for model calibration and the limitations which should be placed upon results where such calibration is not available. Sensitivity analysis is recommended so that the sensitivity of the program's performance in any given situation can be measured against variation in uncertain parameters.

Full details of the design assumptions, including copies of input data should be made available to the local authority.

6.02 Computer models

The use of computer modelling for flood assessments and drainage design is now standard industry practice in all but minor drainage systems; however, manual calculation procedures for the estimation of flow and the sizing of drainage components remain an important part of the checking and calibration process.

In broad terms, computer models of relevance to this Manual can be split into three categories, being hydrologic, hydraulic and water quality. The latter is dealt with in detail in the ARQ (Engineers Australia, 2005) and is mentioned here only briefly for completeness.

(a) Hydrologic models

In broad terms, there are two types of hydrologic models, being:

(i) individual rainfall event simulation;

(ii) continuous, long-term simulation of run-off characteristics.

Continuous long-term simulation models are becoming more widely used in understanding the total hydrologic cycle, including effects on volumetric runoff, base-flow in streams and seasonal variability, and the effects of development and infrastructure on the hydrologic cycle. They are also used as part of catchment pollutant yield simulations and associated stormwater management.

Individual rainfall event simulations are aimed primarily at assessing the effects of severe to extreme flood events due to specific rainfall events, usually of durations less than a day, for all but large river systems.

Generally, dynamic analysis—taking account of the shape and volume of the flood hydrograph—is required (except for minor drainage systems) to ensure that the true effects of flooding and development impacts, such as loss of floodplain storage and the timing of the flood wave, are properly understood. Note, this will generally require a combination of hydrologic and hydraulic modelling.

(b) Hydraulic models

With the rapid increase in computational ability of microcomputers, the use of dynamic flow models has become routine and full two-dimensional surface linked to one-dimensional sub-surface models has also become more widespread.

Hydraulic models fall into the following general categories:

(i) peak flow steady state/backwater (both pipe and surface/open channel) one-dimensional (1D);

(ii) dynamic (full hydrograph) 1D models (both pipe and surface/open channel flow);

(iii) 2D dynamic (surface flow);

(iv) 1D/2D dynamic (combined surface and pipe flow).

There are many specialist 1D peak flow, steady state models available that take account of pressure flow, pipe, pit and inlet losses, pit bypass and inlet and outlet losses. In general, these models are designed for road and trunk drainage systems of localised catchments, where design flows are less than 15 m^3/s.

For large open drain and creek systems, where flow paths are well defined and contained, dynamic 1D modelling is recommended. Steady-state analysis may only be applicable where storage/attenuation and flood peak timing is not critical.

For floodplains or urban flooding situations with complex flow patterns, dynamic 2D modelling is recommended. 1D/2D modelling is also preferred for complex urban flow situations with significant sub-surface flow networks, particularly where there is the potential for significant overland flow that may not follow the road and pipe systems.

(c) Water quality models

Available water quality models are generally either catchment pollutant yield models—which use continuous hydrologic simulation—or in-channel/water body process models. Examples of the former are MUSIC and XP-

AQUALM, and of the latter are MIKE-11 WQ, MIKE-21 WQ, SOBEK and Delft 3D. More details are provided in ARQ (Engineers Australia, 2005).

6.03 Reporting of numerical model outcomes

Designers who use numerical models to design and/or support their design, have a duty of care to provide regulating authorities with sufficient information about the model and its outcomes to allow the regulating authority to adequately review the model's suitability and output.

In effect, the designer has two tasks; one, to operate the model appropriately and therefore obtain an appropriate model output; and two, to demonstrate that the model set-up and output are appropriate for the site conditions. It is noted that the latter task cannot be achieved if the regulating authority, their representative, or a third-party reviewer are either not familiar with the model, or are not supplied with sufficient information to review the model and its output.

It is noted that most "problems/errors" occur with the *application* of a numerical model rather than the initial development of the software program. If an *in-house* software model is used in the design of a drainage system, then it is not sufficient to simply indicate to the regulating authority that the software has been calibrated, or that the software is similar to another commercially available program.

As a minimum, when a numerical model is used in the design of a stormwater system, then the following information should be supplied to a regulating authority:

(i) Name and version of software package.

(ii) Full details of the modelling assumptions.

(iii) Review of model calibration.

(iv) Copy of the model's "error listing" output file.

(v) Copies of input data should be made available to the local government (i.e. supplied on request).

6.04 References

Engineers Australia, 2005. *Australian Runoff Quality – A Guide to Water Sensitive Urban Design*, Engineers Australia, Canberra.

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