20 COMMUNITY AND REGIONAL DETENTION

20.1 INTRODUCTION

This chapter provides guidelines for the design of the community/regional-based stormwater detention facilities outlined in Chapter 18. The designer is advised to read the general principles for detention storage provided in Chapter 18 before proceeding with detailed analysis and design.

A typical configuration of a detention basin is illustrated in Figure 20.1.

20.2 SITE SELECTION

Potential sites should be screened before undertaking a detailed analysis for any given location. Factors to consider in the screening process are presented as follows in the approximate order in which they should be considered.

(a) *Establish Land Ownership*

In anticipation of screening of land for potential detention sites, land ownership should be determined for large or otherwise significant parcels of land. Large tracts of undeveloped publicly owned land are most desirable, followed by undeveloped privately held land. Fully or partially developed public or private parcels of land in need of redevelopment may also offer opportunities for siting a detention facility.

The ownership of large parcels of land in the catchment, particularly potential detention sites, should be determined as early as possible in the planning process. Careful identification of current ownership and intended use, in combination with an assessment of recreational and other needs of a community can lay the groundwork for successful negotiation for purchase and development or redevelopment of property for flood control possibly in combination with other uses.

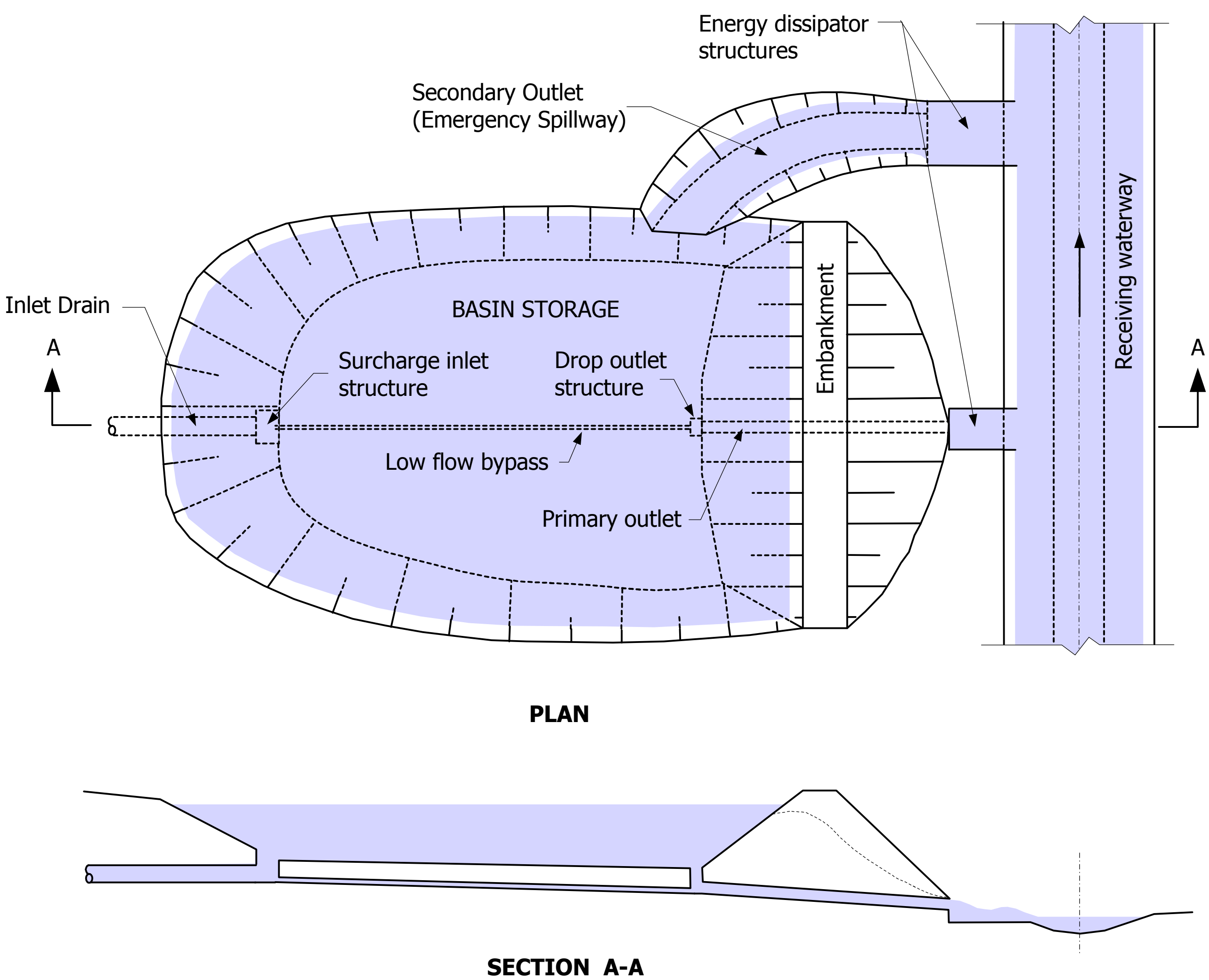


Figure 20.1 Typical Dry Detention Basin Components

(b) Assess Proximity to Flood-prone Areas

A primary consideration for a detention facility is that it be located upstream of and as close as possible to an area requiring flood protection. The nearer the storage site is to the flood-prone area, the greater the portion of the tributary area that will be controlled by the site.

(c) Determine if Site Size is Adequate

A potential site should have, in an approximate sense, adequate size as determined by the areal extent of the site and the volume of water that could be stored temporarily on the site. Given the size and characteristics of the catchment for the site, a rough estimate can be made of the volume of surface water runoff that will be delivered to the site.

As discussed under *(a)*, the collection and analysis of the catchment data should determine ownership of and plans for large or otherwise significant parcels of land. Furthermore, and also treated in *(a)*, potential related community problems and opportunities should be identified. Given the preceding information, a preliminary decision can be made as to whether or not a particular parcel of land is available or could be made available for a detention facility.

(d) Evaluate Topography and Likelihood of Gravity Flow

Gravity-driven inflow and outflow are most desirable. Potential detention sites should be examined to determine if gravity-driven inflow and outflow, or at least gravity inflow, can be accomplished through appropriate site layout and vertical positioning of inflow and outflow works.

Evaluating the likelihood of gravity inflow and outflow at a given potential detention site includes considering the means by which flow would be conveyed into and out of the detention facility.

In a related matter, it is preferable to have a site that is already topographically low, thus minimising the excavation and earthwork needed to achieve the desired storage volume. Sites with favourable topographic features are most likely to be found in undeveloped or newly developing areas. Such sites are less likely to exist in developed areas because existing development pre-empts the topographically low sites or results in their being filled in. Therefore, extensive excavation is common when retrofitting detention facilities into urbanised areas.

(e) Other considerations

Other relevant factors should be addressed in the site screening process, at least in a preliminary fashion, depending on the circumstances. For example, some potential detention sites may already serve at least a partial storage function either by design or by the nature of the site. A hypothetical example is a topographically low area on the upstream side of a roadway embankment drained by a small culvert. If the site is selected for further investigation, subsequent hydrologic-hydraulic analysis should determine the downstream benefit of the incremental storage, not the total storage, which could be developed at the site.

Some potential detention sites may be isolated from public roadways or public right-of-ways, thus presenting potential access problems for inspection, maintenance, and repair. Regardless of the ultimate ownership of a detention facility, access must be provided for inspection and maintenance, either from adjacent publicly owned land or by means of an access easement through privately owned land.

20.3 GENERAL DESIGN CONCEPTS

20.3.1 Outlet Flows

(a) Primary Outlets

Primary outlets for detention basins shall be designed to reduce post-development peak flows to match pre-development peak flows for both the minor and major system design storm ARI in accordance with Section 4.5. Design storm ARIs for the minor and major drainage systems shall be selected in accordance with Table 4.1.

This will require a two-staged outlet configuration (not including the emergency spillway), one outlet configuration to control the minor system design flow and an additional outlet configuration to control the major system design flow in conjunction with the minor system outlet.

This requirement can readily be achieved for new development areas as sufficient land can be set aside in the planning stages of the development to accommodate the necessary storage requirements. However, in existing development areas, reducing the major system design ARI flow to the pre-development rate may not be practical in some cases due to limited availability of suitable detention sites. In such cases, detention facilities should be sized to attenuate the minor system design ARI as well as the largest ARI flow that is possible given the maximum storage available. In addition more on-site detention facilities are encouraged.

(b) Secondary Outlets (Emergency Spillways)

A hazard rating for the basin should be determined and a secondary outlet design ARI selected in accordance with the Federal Government or relevant State Government dam safety guidelines and ANCOLD (1986).

Notwithstanding the above requirements, secondary outlets for all detention basins shall be designed to safely pass a minimum design storm of 100 year ARI through the basin.

20.3.2 Bypass Flows

Provision should be made in a dry detention basin to bypass low flows through or around the basin. This is necessary to ensure that the basin floor, particularly if it is grassed, is not inundated by small storms or continually wetted by dry weather baseflow. The minimum amount of bypass should be one half the 1 month ARI flow.

Flows may be bypassed by a variety of methods depending on the inflow system into the basin. The most commonly used methods for open waterway systems are low flow pipes passing around or under the basin, or low flow inverts connected to the primary outlet structure. Low flows in pipe inlet systems are normally bypassed by providing a smaller pipe connected between a surcharge structure at the basin inlet and the primary outlet. An example of a pipe bypass arrangement is shown in Figure 20.1.

In existing areas, it may be desirable to bypass a larger amount of flow than just the dry weather baseflow if a chosen site has insufficient capacity to attenuate both the minor and major design storms. However, the level of flow bypassed should not exceed the downstream minor system design ARI. It should also be noted that the larger the amount of flow bypassed, the more difficult it will be to reduce the post-development minor system design flow to the pre-development level.

In detention ponds, low flows and dry weather baseflow should be allowed to pass through the pond for treatment and to provide water circulation between storms.

20.4 DETENTION DESIGN CONCEPTS

The sizing of a detention facility requires an inflow hydrograph, a stage-storage curve, and a stage-discharge curve (sometimes called a rating curve). Inflow hydrographs for a range of design storm durations must be routed through the basin to determine the maximum storage volume and water level in the basin corresponding to the maximum allowable outflow rate.

The design storm duration that will produce the maximum storage volume in a basin will vary depending on catchment, rainfall, and basin outflow characteristics, and is typically somewhere between one and three times the peak flow time of concentration for the basin catchment. The design storm duration that produces the maximum storage volume is called the critical duration.

20.4.1 Inflow Hydrographs

Procedures for the estimation and routing of design hydrographs are presented in Chapter 14. However, the complexity of the calculations and the number of hydrographs that need to be estimated and routed through the basin make manual calculation methods very tedious and time consuming. These calculations are best performed using a computer model. A number of suitable computer models are described in Chapter 17. The Rational Method is not suitable for estimating inflow hydrographs for sizing community or regional detention facilities and must not be used under any circumstances.

The sequences of storm events with which a basin will have to cope during its life are unknown. Recorded storm events or a continuous rainfall record over an extended period may be used to analyse the behaviour of the basin and to determine the type and size of the basin and the optimum outlet configuration.

20.4.2 Stage-Storage Relationship

A stage-storage relationship defines the relationship between the depth of water and storage volume in the storage facility. The volume of storage can be calculated by using simple geometric formulas expressed as a function of storage depth.

The storage volume for natural basins in irregular terrain may be developed using a topographic map and the double-end area formula (see Figure 20.2):

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2} \right] \Delta d \tag{20.1}$$

where,

$V_{1,2}$ = storage volume between elevations 1 and 2 (m^3)

A_1 = surface area at elevation 1 (m^2)

A_2 = surface area at elevation 2 (m^2)

Δd = change in elevations between points 1 and 2 (m)

This relationship between storage volume and depth defines the stage-storage curve. An example of a simple stage-storage curve is illustrated in Figure 20.3.

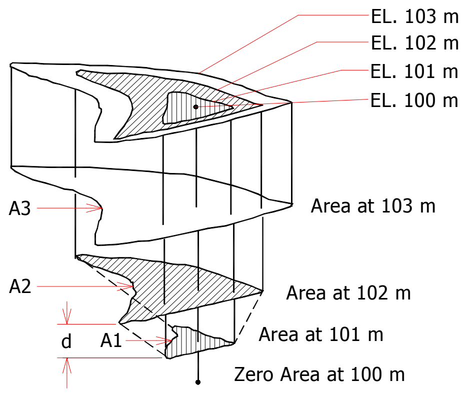


Figure 20.2 Double-end Area Method

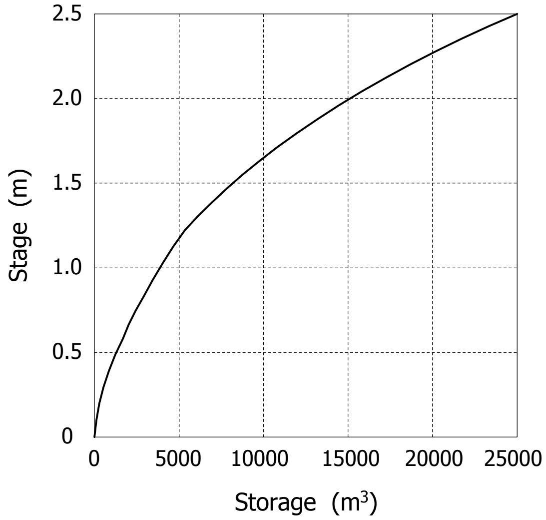


Figure 20.3 Typical Stage-Storage Curve

20.4.3 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the storage water depth and the discharge or outflow from a storage facility. A single composite stage-discharge curve should be developed for each design storm outlet arrangement, which requires consideration of the stage and discharge rating relationship for each outlet component.

Development of a stage-discharge curve for a particular outlet control structure will depend on the interaction of the individual ratings for each component of the control structure. Figure 20.4 illustrates the construction of a stage-discharge curve for an outlet control device (consisting of a low flow orifice and a riser pipe connected to an pipe outlet culvert) and an emergency spillway.

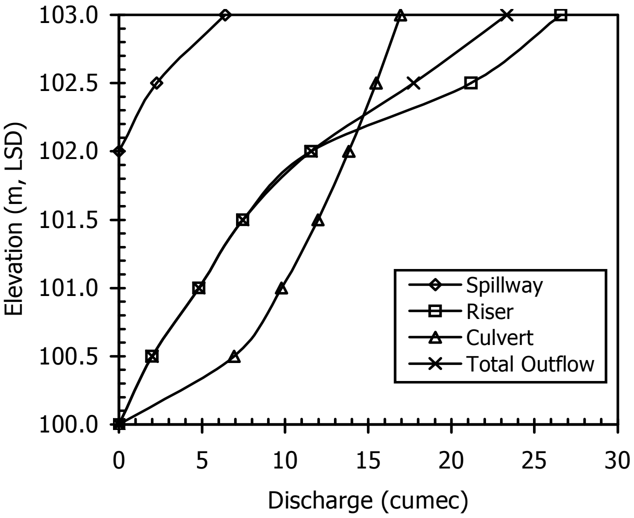


Figure 20.4 Composite Stage-Discharge Curve

20.5 EMBANKMENTS

20.5.1 Classification

Dry detention basins are intermittent water-retaining structures and their embankments do not need to be designed rigorously as dams unless they are high, or special soil problems exist.

An embankment that raises the water level a specified amount as defined by the appropriate dam safety group (generally 1.5 m to 3 m or more above the usual mean low water height, when measured along the downstream toe of the embankment to the emergency spillway crest), is classified as a dam. Such embankments must be designed, constructed, and maintained in accordance with the Federal Government or relevant State Government dam safety standards.

Water quality control ponds, regardless of whether or not they incorporate active storage for flood control, should be designed fully as dams (refer Chapter 35).

All other detention basins with embankments that are *not classified as dams should be designed in accordance with the following criteria*, which are not intended as a substitute for a thorough, site-specific engineering evaluation.

20.5.2 Maximum Pond Depth

The maximum pond depth within the basin should not exceed 3.0 m under normal operating conditions for the maximum design flow for which the primary outlets have been designed, i.e. the maximum design storm ARI flow that does not cause the emergency spillway to operate under normal design conditions.

20.5.3 Top Widths

Minimum recommended embankment top widths are provided in Table 20.1.

Table 20.1 Minimum Recommended Top Width for Earthen Embankments (USDA, 1982)

Height of Embankment (m)	Top Width (m)
Under 3	2.4
3 to 4.5	3.0
4.5 to 6	3.6
6 to 7.5	4.2

20.5.4 Side Slopes

For ease of maintenance, the side slopes of a grassed earthen embankment and basin storage area should not be steeper than 4(H):1(V). However, to increase public safety and facilitate ease of mowing, side slopes of 6(H):1(V) (or flatter) are recommended.

20.5.5 Bottom Grades

The floor of the basin shall be designed with a minimum grade of 1% to provide positive drainage and minimise the likelihood of ponding. Adequate drainage of the basin floor between storms is essential if the facility is to be used for recreation. Where high groundwater occurs, subsoil drains may be required to prevent soggy ground conditions (Chapter 44).

20.5.6 Freeboard

The elevation of the top of the settled embankment shall be a minimum of 0.3 m above the water surface in the detention basin when the emergency spillway is operating at maximum design flow.

20.5.7 Fill Material

All fill material in earthen embankments should be free from brush, roots and other organic material subject to decomposition. The fill material should be compacted to at least 95% of the Modified Proctor method (ASTM D698). Special attention is needed for spillways, where the required compaction should be specified, and to pipe outlets through embankments to prevent piping failures.

20.6 PRIMARY OUTLET DESIGN

Primary outlets are designed for the planned release of water from a detention basin. Basin outlets are ordinarily uncontrolled (i.e. without gates or valves), and may be a single stage outlet structure or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the facility is typically designed as a simple culvert (refer Chapter 27). For multi-stage control structures, the inlet control structure is designed considering a range of design flows. A stage-discharge curve is developed for the full range of flows that the structure would experience. The design flows are typically orifice flow through whatever shape chosen, while the higher flows are typically weir flow over the top of the control structure. The outlets are typically housed in a riser structure connected to a single outlet conduit that passes through the basin embankment and discharges to the downstream conveyance system. Orifices and weirs can be designed using the equations provided in the following sections. The outlet conduit must be designed to carry all flows considered in the design of the riser structure.

The outlet hydraulics for multi-outlet riser systems may be complicated and difficult to analyse. Care must be taken to ensure that the stage-discharge relationship adequately reflects the range of different flow regimes that the structure will operate under. In some cases, particularly if the consequences of failure of the structure are high, the stage-discharge characteristics may need to be verified by physical modelling.

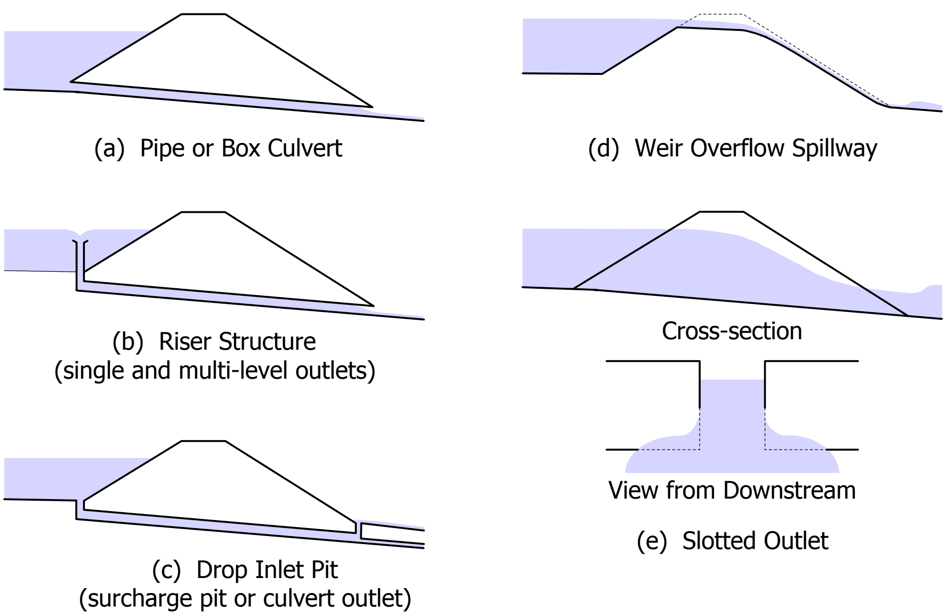


Figure 20.5 Typical Detention Basin Primary Outlets

Alternatively, several pipe or box culvert outlets may be provided at different levels in the basin that either discharged separately to the downstream conveyance system or are combined to discharge at a single location. The stage-discharge relationships for these systems are generally easy to estimate since the separate outlets are hydraulically independent of each other. The capacity of culvert outlets may be determined from Chapter 27.

Figure 20.5 shows some typical primary outlets. The most commonly used outlets are the pipe/box culvert, riser structure, and drop inlet. These outlets may be used singularly or in various combinations which allows a great deal of flexibility in providing flow control over a range of design storm ARIs.

20.6.1 Orifices

For a single circular orifice, illustrated in Figure 20.6(a), the orifice flow can be determined using Equation 20.2.

$$Q = C_d A_o \sqrt{2g\, H_o} \qquad (20.2)$$

where,

Q = the orifice flow rate (m^3/s)

C_d = orifice discharge coefficient (0.40 - 0.62)

A_o = area of orifice (m^2), $\pi\, D_o^2/4$

D_o = orifice diametre (m)

H_o = effective head on the orifice measured from the centre of the opening (m)

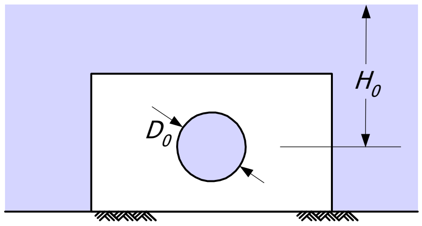
g = acceleration due to gravity (9.81 m/s^2)

If the orifice discharges as a free outfall, the effective head is measured from the centreline of the orifice to the upstream water surface elevation Figure 20.6(a). If the orifice discharge is submerged, the effective head is the difference between the upstream and downstream water surface levels. This latter condition is shown in Figure 20.6(b).

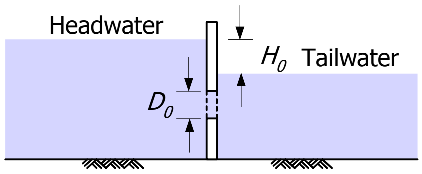
For square-edged uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used for D_d < 50 mm or 0.62 for $D_d \geq$ 50 mm. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

Pipe outlets smaller than 0.3 m diameter may be analysed as a submerged orifice as long as H_o/D_o is greater than 1.5. Pipes greater than 0.3 m diameter should be analysed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

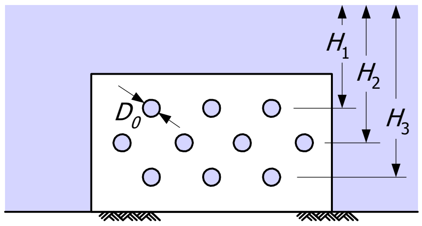
Flow through multiple orifices (see Figure 20.6(c)) can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.



(a) Free Fall



(b) Single (Submergd)



(c) Multple

Figure 20.6 Definition Sketch for Orifice Flow

20.6.2 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided in the following sections:

(a) Sharp-Crested Weirs

Typical sharp-crested weirs are illustrated in Figure 20.7. Equation 20.3 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure 20.7(a)).

$$Q = C_{SCW}\, B\, H^{1.5} \qquad (20.3)$$

where,

Q = weir discharge (m^3/s)

C_{SCW} = 1.81 + 0.22 (H/H_c), sharp-crested weir discharge coefficient

B = weir base width (m)

H = head above weir crest excluding velocity head (m)

As indicated in Equation 20.3, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 20.7(c) for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 may be used.

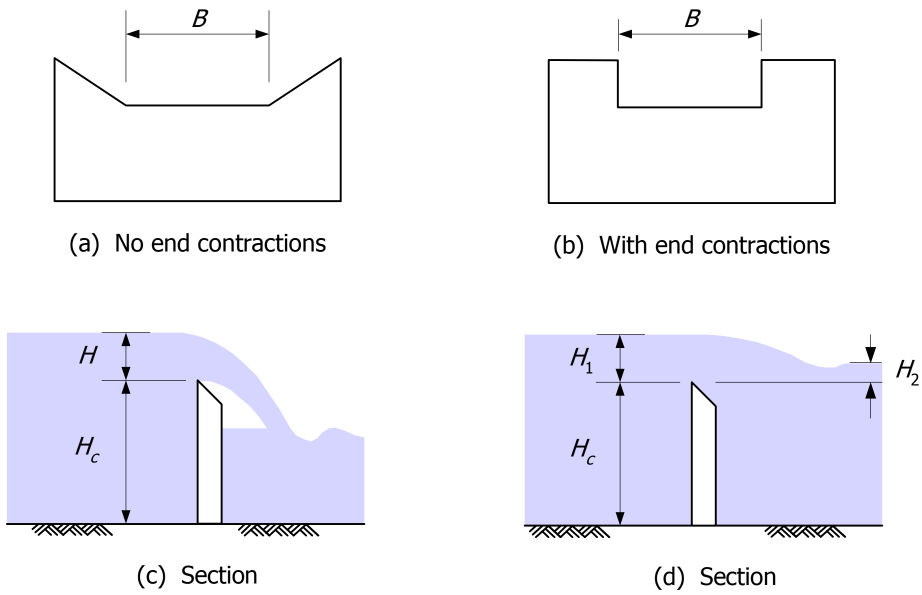


Figure 20.7 Sharp-Crested Weirs

Equation 20.4 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure 20.7(b)). As noted above, a constant C_{SCW} of 1.84 may be used for values of the ratio H/H_c less than 0.3.

$$Q = C_{SCW}\left(B - 0.1\,n\,H\right) H^{1.5} \tag{20.4}$$

where,

n = number of end contractions

Sharp-crested weirs will be affected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure 20.7(d), resulting in a reduced discharge. The discharge equation for a submerged sharp-crested weir is:

$$Q_s = Q_\tau \left(1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right)^{0.385} \tag{20.5}$$

where,

Q_s = submerged weir discharge (m^3/s)

Q_τ = unsubmerged weir discharge from Equation 20.3 or 20.4 (m^3/s)

H_1 = upstream head above weir crest (m)

H_2 = downstream head above weir crest (m)

Flow over the top edge of a riserpipe is typically treated as flow over a sharp-crested weir with no end contractions. Equation 20.3 should be used for this case.

(b) Broad-Crested Weir

The equation typically used for a broad-crested weir is:

$$Q = C_{BCW}\, B\, H^{1.5} \tag{20.6}$$

where,

Q = weir discharge (m^3/s)

C_{BCW} = broad-crested weir coefficient

B = weir base width (m)

H = effective head above weir crest (m)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and, if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth y_c at the weir crest; a value of 1.70 may be used for C_{BCW}. For sharp corners on the broad-crested weir, a value of 1.44 may be used. Additional information on C_{BCW} values as a function of weir base width and head is provided in Chart 20.1 in Appendix 20.A.

(c) V-Notch Weir

The discharge through a V-notch weir is shown in Figure 20.8 and can be calculated using:

$$Q = 1.38\,\tan\left(\frac{\theta}{2}\right) H^{2.5} \tag{20.7}$$

where,

Q = weir discharge (m^3/s)

θ = angle of V-notch (degrees)

H = head on apex of V-notch (m)

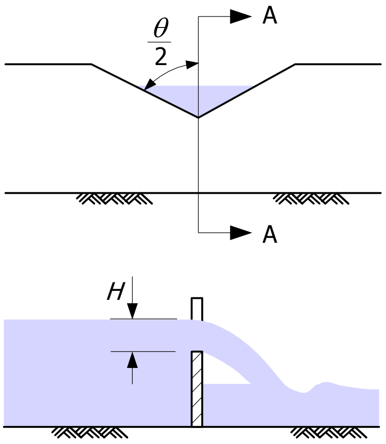


Figure 20.8 V-Notch Weir

(d) Proportional Weir

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship. This relationship is achieved by allowing the discharge area to vary non-linearly with head. A typical proportional weir is shown in Figure 20.9. Design equations for proportional weirs are as follows:

$$Q = 2.74\, a^{0.5}\, b \left(H - \frac{a}{3} \right) \tag{20.8a}$$

$$\frac{x}{b} = 1 - \left(0.315 \right) \left[\arctan \left(\frac{y}{a} \right)^{0.5} \right] \tag{20.8b}$$

where,

Q = weir discharge (m^3/s)

H = head above horizontal sill (m)

Dimensions a, b, x and y are as shown in Figure 20.9.

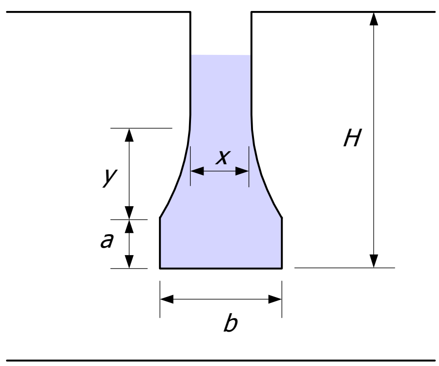


Figure 20.9 Proportional Weir Dimensions

20.6.3 Culverts

Pipe or box culverts are often used as outlet structures for detention facilities. The design of these outlets can be for either single or multi-stage discharges. A single stage discharge system typically consists of a single culvert entrance system, which is not designed to carry emergency flows. A multi-stage inlet typically involves the placement of a control structure at the inlet end of the culvert. The inlet structure is designed in such a way that the design discharge passes through a weir or orifice in the lower levels of the structure and the emergency flows pass over the top of the structure. The culvert needs to be designed to carry the full range of design storm flows from the basin catchment area.

Rubber ring jointed pipes without lifting holes are recommended for pipe culverts. All culverts should be provided with suitable bedding and cutoff walls or seepage collars to prevent possible failure due to piping.

20.6.4 Trash Racks

The susceptibility of inlets, such as small orifices, to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice. For very small outlets, an even larger ratio is usually necessary to control the onrush of debris at the onset of a storm, and a high degree of maintenance is required.

An example of a trash rack is shown in Figure 20.10. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure. Cage type racks or racks with horizontal members inhibit this type of debris removal. A maximum angle of 60^o to the horizontal is therefore recommended for all trash racks.

The surface area of all trash racks should be maximised and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

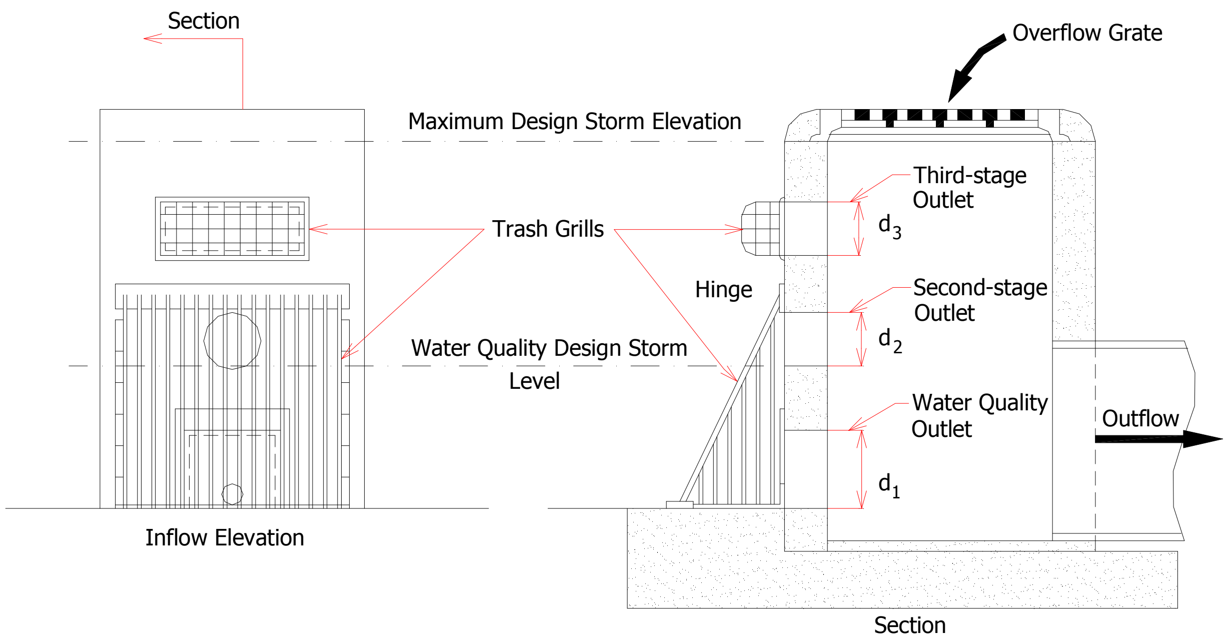


Figure 20.10 Typical Primary Outlet Trash Racks

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the invert of the outlet structure for a dry basin should be depressed below the ground level to minimise clogging due to sedimentation. Depressing the outlet invert to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

20.6.5 Mechanical Devices

On large regional detention basins, such as flood storage reservoirs and flood storage in urban lakes, electrically or mechanically controlled devices are often used to regulate the basin outflow. Ideally, these types of devices are designed to pass all discharges up to a limiting figure without hindrance. Thereafter, this limiting flow is maintained precisely and all excess flow is held in storage until it can be cleared. Some typical devices used are briefly described as follows.

(a) Vertical Gate

An electrically operated vertical sluice gate can be used as an effective control. Gates of this type are normally only used on large facilities, but their use could be extended to smaller installations where precise, automatic control of the storage is being considered.

Two types of vertical gate are normally used, namely: the sliding sluice gate, and the 'fixed-roller gate'. The sliding sluice gate is the simplest form. This is fabricated in steel and slides in a metal frame built into the concrete walls and base of the control structure. Sealing normally is achieved by direct metal-to-metal contact between the gate face and bottom and the sideframes and sill. The contact faces of the gate and frame normally consist of machined bronze strips. The pressure of water on the gate usually is relied upon to seal the gate when closed. Where improved control over leakage is required, a flexible rubber L-shaped or 'musical note' seal fastened to the sides of the gate can be used with a rubber or Neoprene strip on its bottom edge. Friction loads increase rapidly as the size of the gate increases. To minimise this and reduce operating loads, rollers are usually mounted on larger sized vertical gates. This form of gate is described as a fixed-roller sluice gate. Operating loads can be reduced still further by using counterbalance weights.

Vertical gates can be operated by chains or wire ropes, the deadweight of the gate being used to effect closure. A direct drive through screws or racks can also be employed, which has the advantage of providing a more positive closing action. Limit switches must be fitted to prevent accidental damage to the gate or drive mechanism if over-run occurs during the raising or lowering cycle. Further details of gates of various types are described by Leliavsky (1981) and Steele (1968), and the design literature of gate manufacturers such as Boving (Newton Chambers) Ltd.

Floating debris can be a problem with a vertical 'undershot' gate, and booms or trash screens might be necessary to prevent blockage in some circumstances. While electrically driven and electronically controlled gates are capable of giving precise control over flow, a power failure during a major storm would have serious consequences. Provision should always be made for emergency operation by hand or by the use of a portable mechanical drive unit.

Formulae for the discharge through vertical gates or similar ungated openings are based largely on observations and model tests. Performance information for vertical gates may be found in Henderson (1966) and Chow (1959).

(b) Radial or Tainter Gates

Radial or tainter gates provide further means of control. While their use is generally more applicable to large facilities, a gate of this type has been used on a comparatively small reservoir in the area of the River Thames in England.

The advantage of the radial gate, over the vertical gate is that the resultant of the hydraulic forces on the curved gate surface acts through the centre of rotation or hinge, placing no load on the lifting mechanism. A further advantage over the vertical gate is grooves, which are a possible source of undesirable turbulence, are not required in the sidewalls. Sealing at the bottom of the gate can consist of a rubber or Neoprene strip held in place by bolts and mild steel flats. Side sealing can be achieved by the use of a 'music note' seal, fastened to the skin plate by studs and mild steel flats, with the seal moving against steel 'rubbing plates' set flush in the control structure sidewalls. Operating loads can be kept to a minimum, as with the vertical gate, by the provision of counterbalance weights. Electric motors generally are employed to operate radial gates, hydraulic rams being a frequently used alternative.

Crable (1976) describes a self-closing radial gate system developed by Hydraulics Research Ltd, designed for the diversion of flood water, which could be modified for use as a control for an on-stream flood storage reservoir. The general principles of radial gate design are fully described by Thorn (1966).

(c) Tilting Gates

Tilting gates or weirs can also be used as flood storage controls. They have the advantage over undershot vertical and radial gates in that debris can pass over them in times of storm and are therefore less susceptible to blockage. Older gates of this type were often raised by electric winches mounted on overhead gantries, the self-weight of the gate being relied upon for return to its 'parking' position at bed level. More positive control can be obtained by employing rising screw equipment – twin lifting screws being operated by a headstock unit placed centrally on the gantry. In recent years, increasing use has been made of the 'fishbelly' type of tilting gate. This has a semi-elliptical section, giving it a high torsional rigidity and allowing operation from one end. Gates as long as 33.5 m, designed and installed by Boving (Newton Chambers) Ltd, have been operating successfully in the UK for some time. Tilting gates, actuated by hydraulic cylinders, do not need overhead gantries, making the structure less obtrusive.

20.6.6 Erosion Protection

(a) Primary Outlets

When the dimensions of the detention basin and outlet structures have been finalised, maximum exit velocities should be calculated and consideration given to the need to protect the downstream bed and banks from erosion.

The outlet velocity from a primary outlet on a small basin, operating at low head (i.e. the difference in upstream and downstream water levels), is comparatively small. The only measures required are generally the protection of the bed and banks for a few metres downstream by stone pitching or other means. Where the head exceeds 1 m, a structure for dissipating energy should be provided in order to prevent erosion which might otherwise lead to the failure of the basin embankment.

Below a pipe outlet, a suitable device is the 'impact energy dissipator' developed by the US Bureau of Reclamation (refer Chapter 29). This standard stilling basin can be constructed in various sizes to deal with flows ranging from 0.3 to 11 m^3/s with maximum outlet velocities of 9 m/s. The basin is effective over a wide range of tailwater levels (US Department of the Interior, Bureau of Reclamation, 1987; Bradley and Peterka, 1957). Another energy-dissipating basin for pipe diameters ranging from 450 mm to 1800 mm and heads of up to three times the pipe diameter has been developed in Australia by the Public Works Department, New South Wales (Argue, 1961). Refer to Chapter 29 for details of this basin.

An open stilling basin is generally more suitable downstream of weirs, gates and sluices, and other large control structures. The stilling basin must be of sufficient depth below the downstream tailwater level so as to drown the flow, thus enabling the hydraulic jump to be retained within the stilling basin at all flows. The detailed design of stilling basins is covered in many papers and hydraulic manuals (US Department of the Interior, Bureau of Reclamation, 1974; Chow, 1959; Bradley and Peterka, 1957).

While satisfactory stilling basins or control structures may be designed from theory, there is often a degree of uncertainty as to how closely prototype performance will match that predicted by the theoretical calculations. The errors are unlikely to be of great significance on small structures but can be so on large ones, particularly where approach and discharge conditions, or the structure shape, differ from those considered in previous hydraulic tests. Any failure of the structure to meet its theoretical performance on a major project could lead to serious financial and safety problems. It is prudent under these circumstances to take specialist advice or to verify calculations by physical model tests.

(b) Downstream Waterway

The channel bed and banks immediately downstream of stilling basins should be protected by stone pitching or riprap. Where the outfall from the basin is a culvert, this should be provided for a distance of at least four times the diameter or height of the culvert. Information for stone pitching and rip rap is provided in Chapter 29.

20.7 SECONDARY OUTLET DESIGN

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm ARI for the storage facility.

In many cases, stormwater detention structures do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. The catchment areas of many sites are so small that only very short, sharp thunderstorms are apt to threaten overtopping or dam failure, and such storms are very localised. Also, capacities of the structures are usually too small to create a flood wave.

By contrast, regional on-line facilities with homes immediately downstream may pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor. The potential for loss of life or property damage must be categorised early in the design effort and a spillway design ARI up to and including the Probable Maximum Flood (PMF) may be warranted.

In many instances, the small size of a detention basin virtually precludes a spillway designed for the PMF. In those cases, means to mitigate the adverse effects of overtopping of the embankment can include:

(a) flattening of the downstream embankment face

(b) armouring the embankment crest and downstream face

(c) using regulated floodplain delineation and occupancy restrictions downstream representative of conditions without the detention storage

(d) providing extra waterway capacity downstream

(e) using a wide embankment crest such as is common with urban roads and streets (where rapid failure seldom occurs due to modest overtopping depths)

(f) using non-eroding embankment material such as roller compacted concrete

(g) using small tributary basins, where the rate and volume of discharge involved are limited, resulting in overtopping flows of short duration and non-hazardous proportions

The emergency spillway is proportioned to pass flows in excess of the design flood without allowing overtopping of the embankment. Flow in the emergency spillway is open channel flow. Normally, it is assumed that critical depth occurs at the control section. For the larger structures, to avoid the possibility of an eroded channel developing in the spillway when the spillway is not lined, it is good practice to put a small concrete kerb and cutoff wall in the throat as a point of hydraulic control.

Soil Conservation Service manuals (USDA, 1982) provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year assume minimum retardance.

20.7.1 Overflow Weir

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated in Figure 20.11.

Equation 20.9 presents a relationship for computing the flow through a broad-crested emergency spillway. The dimensional terms used in the equation are illustrated in Figure 20.11.

$$Q = C_{SP}\, B\, H_p^{1.5} \qquad (20.9)$$

where,

Q = emergency spillway discharge (m^3/s)

C_{SP} = spillway discharge coefficient

B = emergency spillway base width (m)

H_p = effective head on the spillway crest (m)

The discharge coefficient C_{SP} in Equation 20.9 varies as a function of spillway base width and effective head. Design values for C_{SP} are provided in Design Chart 20.2 in Appendix 20.A.

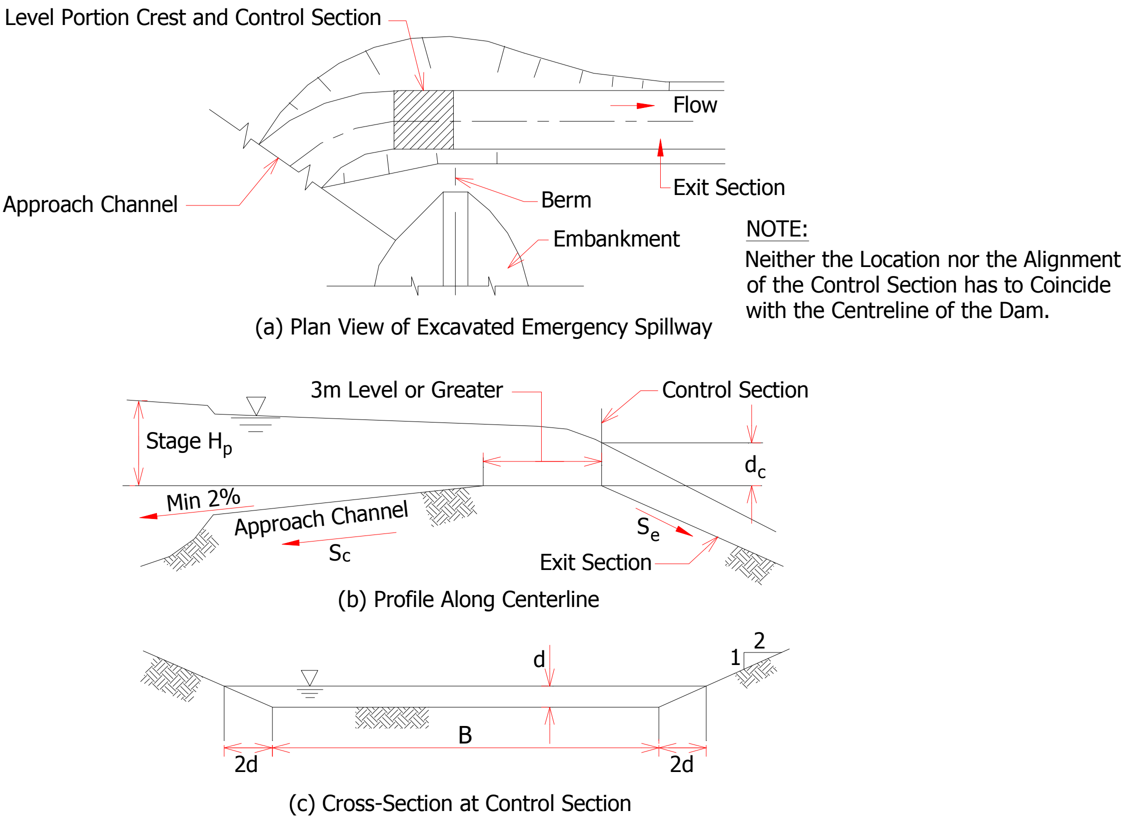


Figure 20.11 Emergency Spillway Design Schematic

Equations 20.10 and 20.11 can be used to compute the critical velocity V_c and critical slope S_c at the control section of an emergency spillway:

$$V_c = 2.14 \left(\frac{Q}{B}\right)^{0.33} \tag{20.10}$$

$$S_c = 9.84\ n^2 \left(\frac{V_c\ B}{Q}\right)^{0.33} \tag{20.11}$$

Where,

n = Manning's roughness coefficient

Note that for a given effective head H_p, flattening the exit slope S_e to less than S_c decreases spillway discharge, but steepening S_e greater than S_c does not increase discharge. Also, if a slope S_e steeper than S_c is used, the velocity V_e in the exit channel will increase according to the following relationship:

$$V_e = V_c \left(\frac{S_e}{S_c}\right)^{0.3} \tag{20.12}$$

20.7.2 Erosion Protection

(a) Embankment and Spillway Channel

The surfaces of embankments and spillway channels must be protected against damage by scour when subject to high velocities. The degree of protection required depends on the velocity of flow to which the bank will be subjected. Well-established turf will provide protection against velocities of up to 3 m/s for as long as 9 hours. Where water is unlikely to flow against a newly excavated surface for some months, the protecting turf covering can be grown from seed. The bank should be covered with a 0.15 m layer of topsoil, incorporating a suitable grass fertiliser and sown with an approved seed mixture. Turf forming native grasses which require little maintenance and provide a dense well-knit turf are most suitable whilst bunch grasses are least suitable. One or more of the following permanent grasses are recommended for permanent seed mixes:

- *Axonophus compressus* (Cow grass)
- *Vertiver grass*
- *Brachiaria sp.*
- *Cynodon dactylon* ((Bermuda grass)
- *Panicum virgatum* (Switch grass)

The quality of the grass seed used is important. Grass seed should be fresh, recleaned grass seed of the latest crop available. Grass seed may range from 20% to 100% purity. Compensation for purity and germination is made by furnishing sufficient additional seed to equal the specified pure live seed product.

In places where immediate cover and protection of the completed earthworks against erosion is required, turf can be laid and held down by coarse-meshed wire netting and wire pins until it is firmly rooted. With all turfing methods, adequate subsoil cultivation is essential to encourage good root penetration. Not only does this enable the grass to survive periods of drought, but provides long-term continuity between the turf and the subsoil.

A large number of geotextiles are available which can be used for reinforcing turf enabling it to withstand velocities of up to 7 m/s. This performance enables these materials and systems to be considered in some situations as viable alternatives to traditional concrete construction and can lead to significant savings in cost. Some products are available that simplify the wire mesh method of reinforcing turf by growing it on a mat of nylon filaments. The turf is then lifted in 1 m wide strips up to 10 m long and can be pegged down to give immediate protection.

The use of concrete reinforcement in conjunction with grass should be considered where turf alone will not provide sufficient protection to a bank or spillway subject to high velocities for long periods. A number of proprietary systems are available which cover the use of in-situ concrete placed in special fabric formers and various types of precast concrete blocks.

The four key points for reinforced grass design are as follows (Hewlet, Boorman and Bramley, 1987):

1. *Hydraulic design* : the main parameters are:
   (a) velocity
   (b) duration of flow
   (c) erosion resistance of the various armour layers
2. *Geotechnical considerations* : related to the effects that water entering the embankment (or cutting) will have on the subsoil and cover:
   (a) soil sampling (to identify the soil type)
   (b) testing (to determine the soil parameters to be used in design)
3. *Botanical considerations* : concern the choice of a suitable grass mixture and its establishment and subsequent management
4. *Detailing and specification* : involves the consideration of a number of factors which combine points 1 to 3 above and completes the design process

Included in point 4 are requirements for anchorages at the edge of all reinforcement and within concrete systems, as well as the possible need for additional shear connections between concrete armour layers and the subsoil. Attention must also be given to crest, channel, and downstream toe details, as well as the joints in geotextiles and concrete reinforcement, and the preparation of the formation.

While careful design, selection of seed, and soil preparation are necessary for the design of a satisfactory grass-lined spillway channel, proper management is essential if a dense sward is to be maintained on those sections where the grass is relied upon to give protection against high velocities. This entails regular cutting, weed control, and the application of fertiliser.

(b) Downstream Waterway

An open stilling basin, as discussed in Chapter 29, may be required at the bottom of the spillway prior to discharge into the downstream waterway. It may be possible, and more cost effective, to provide a single stilling basin for both the emergency spillway and the primary outlet to the basin.

20.8 PUBLIC SAFETY

It should be recognised that a detention basin will form an integral part of the total infrastructure for an urban community. It is inevitable that people will have access to a detention basin, especially if it is designed for multi-purpose usage incorporating active or passive recreation, or sporting facilities. Accordingly, a detention basin must be designed with public safety in mind when the facility is in operation and also during periods between storms when the facility is empty. Appropriate ways must be considered to prevent and to discourage the public from being exposed to high-hazard areas during these periods.

Retarding basins should be provided with signs that clearly indicate their purpose and their potential danger during storms. Signs should be located such that they are clearly visible at public access points and at entrances and exits to outlet structures.

The inlet of a primary outlet structure creates a potential hazard when in operation due to the possibility of a person being carried into the opening. Gratings or trash racks may be used to help prevent this happening. These should be inclined at an angle of 60^{o} to the horizontal and placed a sufficient distance upstream of the inlet where the velocity through the rack is low. This should ensure that a person would not become held under the water against the grating or trash rack.

The downstream end of a primary outlet structure can also be a potentially hazardous area as an energy dissipator device is often provided for scour protection. A pipe rail fence should be provided on steep or vertical drops such as headwalls and wingwalls at the inlet and outlet to a primary outlet structure to discourage public access. Pipe rail fencing can also prevent a person inadvertently walking into of falling off these structures during periods when the basin is not operating.

During the periods of no operation, there is little hazard at most outlet works, although they can be attractive to playing children or curious adults. The visibility of an outlet could be minimised by screening with bunds or shrubs to reduce its attraction potential.

20.9 LANDSCAPING

Detention facilities should be tastefully incorporated into the urban setting in which they reside. This is not a hydrologic consideration, but is a consideration, which will be used by the public to judge these facilities.

Aesthetics of the finished facility is therefore extremely important. Wherever possible, designs should incorporate naturally shaped basins with landscaped banks, footpaths, and selective planting of vegetation to help enrich the area and provide a focal point for surrounding development. The services of a landscape architect should be sought to assist with this aspect of the design.

Sympathetic landscaping and the resulting improvement in local visual amenity will also encourage the public to accept detention basins as an element of the urban natural environment and not as a target for vandalism.

Trees and shrubs should not be planted on basin embankments as they may increase the danger of bank failure by 'piping' along the line of the roots.

20.10 OPERATION AND MAINTENANCE

A detention basin, in common with other methods of flood alleviation, will prove effective only if it is designed correctly, constructed properly, and maintained regularly. This not only requires a firm grasp of hydrological, hydraulic, and structural design principles, but also a sound understanding of operational and maintenance requirements.

20.10.1 Consultation

It is important that lines of communication and contacts be established during the planning period and maintained thereafter, so that any problems regarding the operation, maintenance, or use of the detention basin can be brought to the notice of operational staff quickly and prompt action taken.

It is also very important that the person or team responsible for the design liaises closely with and seeks the advice of the staff who will be responsible for its future operation and maintenance. A schedule of points covering operational, inspection, and maintenance requirements should be drawn up and agreed. This should cover questions of safety, access for personnel and plant, and methods of dealing with blockages and the possible failure of equipment or power supplies. Inquiries should be made about any problems experienced with previous installations and the design amended where necessary to devise improvements. On completion, the works must be handed over formally after ensuring that operational staff are fully conversant with the installation, have been trained in the operation of special equipment, and are aware of all maintenance requirements.

20.10.2 Planned Maintenance and Inspection

The design storm for most detention basins is a relatively rare occurrence and the basin and its outlet structures must be kept in good working order in the intervening period so that it performs satisfactorily when required. It is essential, therefore, that all detention basins are subject to regular inspection and maintenance. In some circumstances, failure to carry out routine maintenance could result in blockage of the primary outlets and premature filling of the basin under normal flow conditions, leaving no storage available for flood control. It is essential that the responsibility for future maintenance should be clearly established and formal arrangements should be drawn up for inspection and maintenance. In addition to basic engineering requirements, the arrangements must cover the amenity of the basin wherever recreational usage of the basin is provided.

Inspection and maintenance must be carried out on a regular basis in order to minimise risks. The frequency and requirements for routine inspection will depend on the type and size of the basin, the local circumstances, and the type and complexity of the primary outlets. The frequency of inspections and maintenance visits may vary widely and should be reviewed continually in the light of any problems experienced on site and any long-term changes in maintenance requirements. A formal inspection and maintenance programme should be drawn up, staff allocated, and the duties and responsibilities confirmed in writing.

20.10.3 Effect of Design on Maintenance Costs

Maintenance costs can be kept to a minimum by the careful design of the basin, outlet structures, and any adjoining amenity area. Adequate car parking space and strategically located footpaths can control public access and limit damage to grassed areas. The judicious planting of shrubs and trees can be used to guide the public along preferred routes. Grass-cutting costs can be kept to a minimum (in areas used for formal recreation or where grass is used for scour protection) by keeping the slopes of embankments and other areas flat enough for machine mowing. Equipment can operate on slopes of up to 4(H):1(V) but slopes of 6(H):1(V) or flatter are preferred.

20.10.4 Grassed Areas and Embankments

Where embankments and/or spillways are subject to scour caused by high velocities of flow, regular mowing (at least twice a year) is required to keep the grass sward in good condition and discourage woody growth. Similar treatment is necessary in areas used for formal recreation. Maintaining turf quality, where hydraulic protection is to be provided, requires a good supply of nutrients, which may require the use of fertilisers. The frequency of application depends on the quality of the soil. Normal soils may only require fertilising in the first year of growth while poor ones may demand annual treatment for a number of years. Some weeds can break up the turf cover and have serious effects on its ability to withstand erosive forces and must be controlled; such weeds can be eradicated by using selective weed-killers.

20.10.5 Waterways

Engineered waterways upstream and downstream of a detention basin will require regular attention, particularly in urban districts. Banks below flood level should be mown where necessary to promote good grass growth, thereby providing protection against scour. Any rubbish, debris, and silt should be removed in order to prevent the blockage of screens and primary outlet structures.

20.10.6 Primary Outlets

All screens on primary outlets should be inspected and cleaned on a regular basis, particularly following a storm event. Particular problems can occur in urban areas where rubbish is often deposited in watercourses by residents.

20.10.7 Sediment Removal

Regular removal of any accumulated silt and sediment from a detention basin is essential, particularly where the basin floor is used for recreational purposes. Removal of accumulated debris, trash, paper, etc. should take place every 6 months or so and vegetation growing within the basin should not grow taller than 0.5 m. No standing water should be allowed in the basin beyond a period of 72 hrs after a storm event. If such conditions occur, corrective maintenance should be undertaken.

20.10.8 Structural Repairs and Replacement

Inlet and outlet devices and riser structures have been known to deteriorate with time, and may have to be replaced. The actual life of a structural component will depend on site specific criteria, such as soil conditions, type of construction, and frequency of operation.

20.11 SIZING PROCEDURE

A general procedure for sizing a detention basin, shown diagrammatically in Figures 20.13 and 20.14, is described as follows:

Step 1 *Determine design storm criteria for the basin*

Select the minor and major design storm ARI for the basin appropriate for the type of development in the basin catchment in accordance with Table 4.1.

Select the secondary outlet design storm ARI in accordance with Section 20.3.1 (b).

Select the amount of bypass flow that will not be routed through the basin.

Any physical constraints at the basin site should be identified including maximum permissible depths of ponding, acceptable depths of flooding in downstream conveyance systems.

Step 2 *Determine the basin outflow limits*

For each design storm ARI, the basin outflow limits are set as the maximum pre-development flow less any non-routed post-development bypass flow. Peak flows for the pre-development design storms and non-routed post-development bypass may be determined by a hydrograph estimation technique or by the Rational Method.

Step 3 *Compute the basin inflow hydrographs*

For each design storm ARI, inflow hydrographs for a range of storm durations will need to be routed through the basin to determine the critical duration that produces the greatest storage and water level within the basin for a particular basin grading and outlet configuration. The basin inflow hydrographs are obtained by subtracting the non-routed bypass flow from the total inflow hydrographs.

Step 4 *Make a preliminary estimate of the required basin volume*

When initially sizing a detention facility, the required storage volume to accomplish the necessary peak reduction is unknown and a preliminary storage volume must be estimated. Estimating the required storage volume is an important task since an accurate first estimate will reduce the number of trials involved in the sizing procedure.

A preliminary estimate may be obtained based on the post-development basin inflow hydrographs for the major system design ARI and the required outflow rate as shown in Figure 20.12. The outflow hydrograph can be approximated by drawing a straight line from the beginning of substantial runoff on the inflow hydrograph to the point on the receding limb corresponding to the maximum allowable peak outflow rate. The amount of storage required is equal to the representative volume (shaded area) between the inflow and outflow hydrographs. To determine the necessary storage, the shaded area can be planimetered or computed mathematically.

Inflow hydrographs for the major system design ARI overa range of durations should be examined and the largest estimated volume selected.

Alternatively, a preliminary estimate of the storage volume required may be obtained using the following regression equation (US Department of Transportation, 1996) for each estimated inflow hydrograph and selecting the largest value:

$$V_s = 1.291 V_i \left(1 - \frac{Q_o}{Q_i}\right)^{0.753} \left(\frac{t_i}{t_p}\right)^{-0.411} \quad (20.13)$$

Where,

V_s = estimated storage volume (m^3/s)

V_i = inflow hydrograph runoff volume (m^3/s)

Q_i = inflow hydrograph peak flow rate (m^3/s)

Q_o = allowable peak outflow rate (m^3/s)

t_i = time base of the inflow hydrograph (minutes)

t_p = time to peak of the inflow hydrograph (minutes)

In some locations, the available storage volume may be limited due to site or other constraints. If a storage volume is known, a preliminary estimate of the maximum possible peak outflow may be obtained by using the following regression equation (US Department of Transportation, 1996) for each estimated inflow hydrograph and selecting the largest value:

$$Q_o = Q_i \left[1 - 0.712 \left(\frac{V_s}{V_i}\right)^{1.328} \left(\frac{t_i}{t_p}\right)^{0.546}\right] \quad (20.14)$$

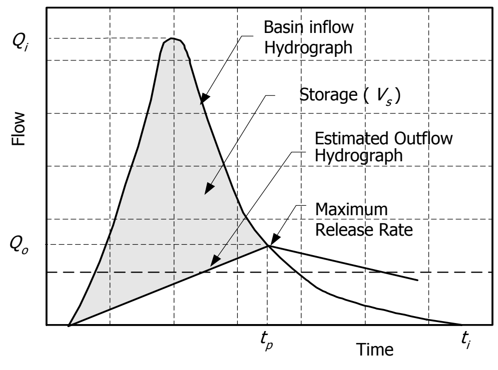


Figure 20.12 Preliminary Estimate of Required Storage

Step 5 *Develop a basin grading plan*

A grading plan to accommodate the storage volume estimated in Step 4 should be prepared keeping in mind any site constraints that may have been identified in Step 1 and the slope criteria for embankments and basin floors.

Step 6 *Compute the stage-storage relationship*

The stage-storage relationship can be defined from the basin geometry using the double-end area method outlined in Section 20.4.2. The maximum stage selected should extend above the top of the basin embankment to ensure that it is not exceeded in the routing calculations.

Step 7 *Size the minor design storm primary outlet*

Since the flow performance criteria requires control over both the minor and major system ARI, multiple outlet control consisting of an arrangement of devices placed at appropriate stages (levels) within the basin will need to be provided. Matching this flow performance criterion will require careful selection of the type and arrangement of outlets to be used. Arriving at the best multiple outlet arrangement to achieve the level of control required will normally involve a trial and error process and gradual refinement until a satisfactory design is found.

(i) Select trial outlet arrangement

Select a trial outlet arrangement with an invert at or below the lowest level in the floor of a dry basin, or at water level in a detention pond, to ensure the storage completely empties after each storm event.

(ii) Compute the stage-discharge relationship

Compute the stage-discharge relationship by summing the individual discharge ratings for each outlet adopted.

The maximum stage selected must be greater than the expected maximum water level in the basin so that it will not be exceeded in the routing calculations in the following step.

(iii) Route the inflow hydrographs through the basin

Route the inflow hydrographs through the basin, using a suitable computer model or the procedures presented in Chapter 14, to determine the maximum basin outflow and water level.

The routing time step adopted should be a uniform integer value and should be small enough so that the change in inflow and outflow between time steps is relatively linear. A value of $2t_i / 300$ may be used as a rough guide. However, for manual calculations, a minimum value of one minute is recommended.

(iv) Check if results are acceptable

If the maximum basin outflow is greater than or excessively smaller than the limit determined in Step 2, or the basin water depth exceeds that permissible, return to Step 5 or 7 and modify the geometry of the basin and/or the outlet arrangement or configuration as necessary.

Step 8 *Size the major design storm primary outlet*

(i) Select trial outlet arrangement

Select a trial outlet arrangement and set the lowest level for the major system outlet(s) at or slightly above the maximum basin water level estimated for the minor design storm.

(ii) Compute the stage-discharge relationship

Compute the stage-discharge relationship by summing the individual discharge ratings for each outlet adopted including the minor design storm outlets.

(iii) Route the inflow hydrographs through the basin

Route the inflow hydrographs through the basin, using a suitable computer model or the procedures presented in Chapter 14, to determine the maximum basin outflow and water level.

(iv) Check if results are acceptable

If the maximum basin outflow is greater than or excessively smaller than the limit determined in Step 2, or the basin water depth exceeds that permissible, return to step 5 or 8 and modify the geometry of the basin and/or the outlet arrangement or configuration as necessary.

Note: if the basin geometry is altered, the minor design storm routing in Step 7 will need to be redone to check if the minor system outlet performance is still satisfactory and to establish the revised maximum basin water level for setting the major outlet invert level.

Step 9 *Size the secondary outlet arrangement*

Once a basin configuration meets the selected flow control performance criterion, the emergency outlet will need to be sized to contain the selected secondary outlet design ARI.

(i) Select trial outlet arrangement

Select a trial secondary outlet arrangement. Set the minimum outlet level at the maximum basin water level estimated for the major design storm plus a freeboard of at least 200 mm.

(ii) Compute the stage-discharge relationship

Compute the stage-discharge relationship by summing the individual discharge ratings for all the basin outlets (i.e. the secondary outlet plus the minor and major system outlets)

(iii) Route the inflow hydrographs through the basin

Route the inflow hydrographs through the basin, using a suitable computer model or the procedures presented in Chapter 14, to determine the maximum basin outflow and water level.

(iv) Check if results are acceptable

A flow control criterion for the 100 year ARI design storm has not been specified. However, the water depth in the basin will determine the maximum height of the embankment. The outlet arrangement may need to be refined until a satisfactory balance in terms of cost or public safety is found between the height of the embankment and the size of the secondary outlet.

Step 10 *Check behaviour under extreme conditions*

The basin's behaviour under extreme conditions may also need to be checked. These conditions may be larger floods than the design flood, possibly up to the PMF, and/or conditions under which partial or total blockage of the basin primary outlet(s) occurs.

Step 11 *Size downstream erosion protection measures*

Calculate the exit flow velocities from the basin outlets under the range of design storms. If these velocities will cause erosion in the downstream waterway, select and size energy dissipation and waterway erosion protection measures.

Step 12 *Prepare design drawings and specifications*

When the basin performance is deemed acceptable for all operating conditions, including its behaviour under extreme flood events, detailed design drawings and specifications should be prepared. These should include grading plans, embankment design details, landscape plans, structural details of all primary and secondary outlets, and written details of maintenance procedures and schedules.

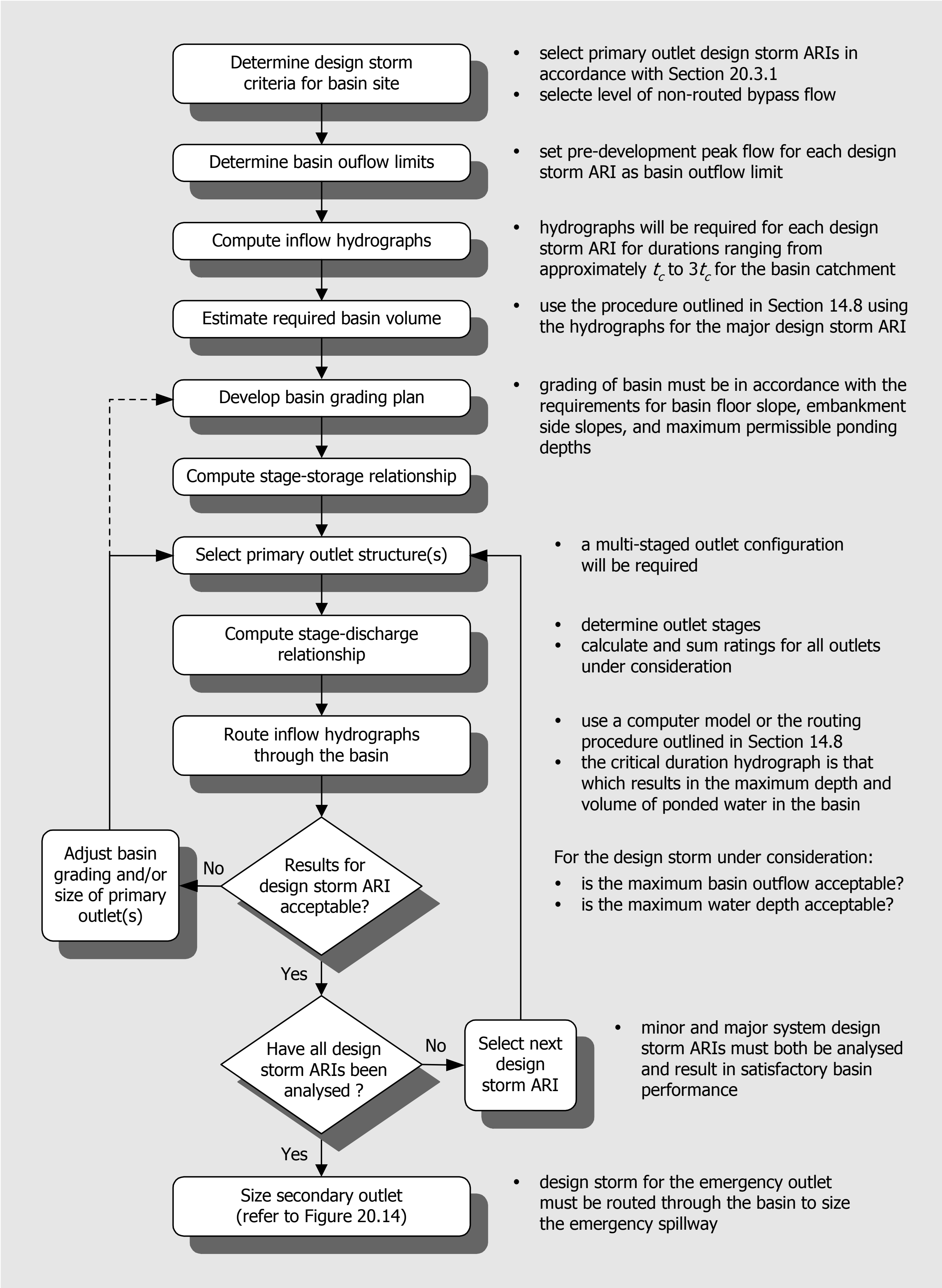


Figure 20.13 Detention Basin Sizing Procedure for Basin and Primary Outlets

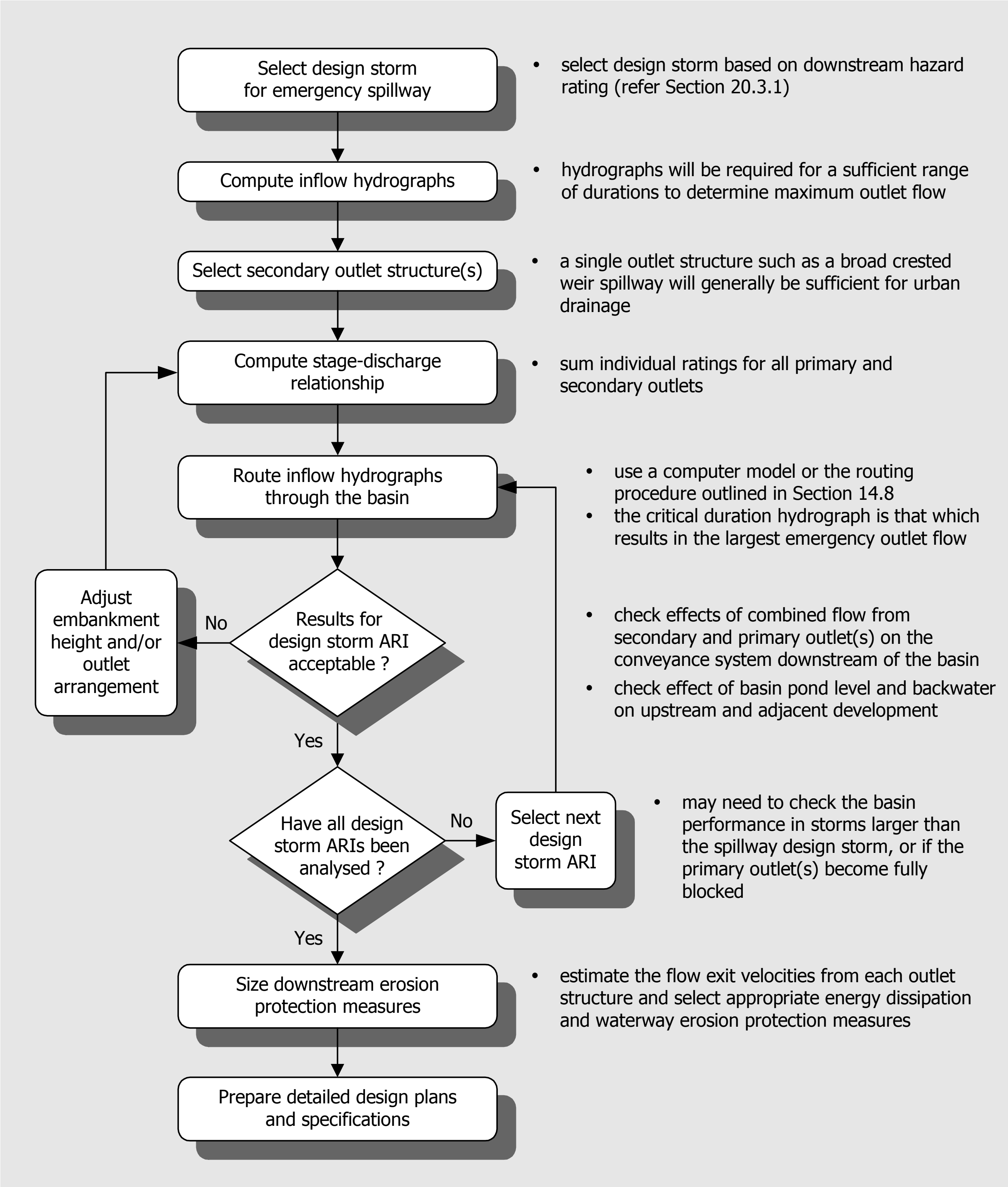


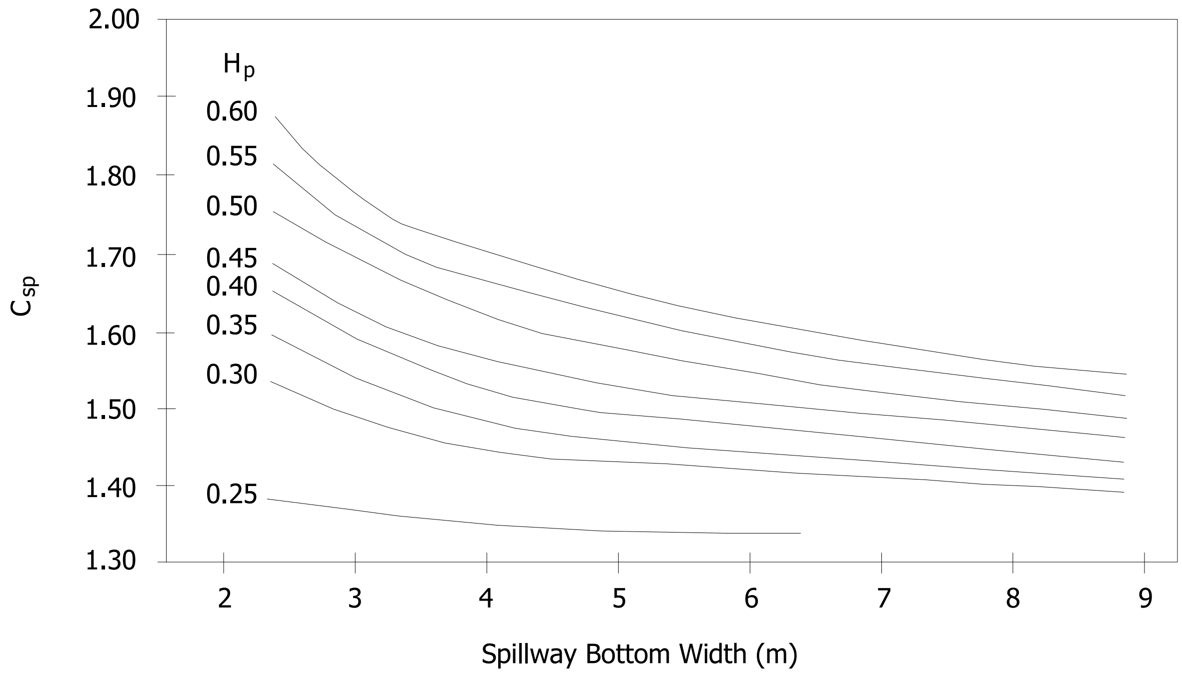
Figure 20.14 Detention Basin Sizing Procedure for Secondary Outlet and Downstream Erosion Protection

APPENDIX 20.A DESIGN CHARTS

Design Chart 20.1 Broad-Crested Weir Coefficient CBCW Values as a Function of Weir Base Width and Head (coefficient has units of $m^{0.5}/sec$) (US Department of Transportation, 1996)

Head *H* (m) [(1)]	Weir Base Width *B* (m)														
	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.25	1.50	2.00	3.00	4.00
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39	1.37	1.35	1.36	1.40	1.45
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.44	1.45	1.45	1.44	1.43	1.44	1.45	1.45
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.46	1.48	1.48	1.49	1.49	1.49	1.45
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47	1.47	1.48	1.48	1.48	1.45
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46	1.47	1.47	1.47	1.48	1.45
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48	1.47	1.46	1.46	1.46	1.45
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34	1.48	1.46	1.46	1.46	1.45
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45	1.49	1.47	1.47	1.46	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55	1.50	1.47	1.47	1.46	1.45
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58	1.50	1.47	1.47	1.46	1.45
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60	1.51	1.48	1.47	1.46	1.45
1.10	1.83	1.83	1.83	1.83	1.83	1.83	1.80	1.75	1.66	1.62	1.52	1.49	1.47	1.46	1.45
1.20	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.70	1.65	1.53	1.49	1.48	1.46	1.45
1.30	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.82	1.77	1.71	1.56	1.51	1.49	1.46	1.45
1.40	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.77	1.60	1.52	1.50	1.46	1.45
1.50	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.66	1.55	1.51	1.46	1.45
1.60	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.81	1.74	1.58	1.53	1.46	1.45

[(1)] Measured at least 2.5 H_c upstream of the weir



Design Chart 20.2 Discharge coefficients for Emergency Spillways

APPENDIX 20.B WORKED EXAMPLE

Problem:

Design a dry detention basin for a 140 hectare catchment, 60% of which is proposed to be developed for commercial and urban areas. Flows will be directed to the basin via a grassed floodway along the alignment of the existing stream. The floodway will have a low flow pipe system with a maximum capacity of 3.8 m^3/s (0.5 times the 1 month ARI flow) which will bypass the basin and combine with the basin outflow in the downstream floodway. As the catchment is undeveloped, ample area is available for the detention basin to be incorporated into the landuse planning for the development.

Solution:

Step (1) Determine design storm criteria for the basin

As specified in Section 4.5.1, the runoff quantity control criterion for new development is to reduce post-development peak flows for the minor and major system ARI to at least the pre-development values. The majority of the proposed development upstream of the basin will be medium density housing and commercial lots. The minor and major system design storms are 5 year ARI and 50 year ARI respectively in accordance with Table 4.1. The design storm for the secondary outlet spillway is 100 years ARI in accordance with Section 20.3.1 (b).

Step (2) Determine the basin outflow limits

The pre-development peak flow time of concentration (t_c) for the 5 year ARI and 50 year ARI has been estimated to be 40 minutes. The pre-development flows for this duration will become the post-development flow limits in the floodway immediately downstream of the basin, i.e. the outflow from the basin plus the non-routed low flow bypass. The basin flow limits are therefore the downstream floodway limits minus the non-routed bypass flow. The estimated pre-development total flow hydrographs are provided in Table 20.B1 (only the 10 minute time increment flow values are shown). The basin outflow limits for 5 and 50 year ARI are **7.6 m^3/s** and **12.0 m^3/s** respectively.

Table 20.B1 Pre and Post-Development Total Flow Hydrographs

Time (mins)	Pre-development		Post-development											
	ARI (years)													
	5	50	5				50				100			
	Storm duration (minutes)													
	40		30	45	60	75	30	45	60	75	30	45	60	75
	Total Flow (m^3/s)													
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	1.1	1.9	3.3	3.1	2.7	0.8	5.3	5.0	4.3	1.3	5.6	6.6	4.6	1.4
20	3.4	5.7	9.4	7.3	6.4	3.9	15.2	11.7	10.4	6.3	16.1	15.3	11.0	6.7
30	8.2	11.3	**20.9**	13.7	11.7	6.7	**33.8**	21.8	18.9	10.8	**35.8**	28.7	20.0	11.4
40	**11.4**	**15.8**	12.0	**19.2**	**15.3**	10.8	19.4	**29.1**	**24.6**	17.4	20.5	**37.1**	**26.1**	18.4
50	10.1	13.2	6.4	11.4	11.9	**12.8**	10.4	18.3	19.2	**20.7**	11.0	24.0	20.4	**21.9**
60	8.3	10.6	3.4	7.0	8.9	9.9	5.6	11.2	14.4	16.0	5.9	14.8	15.3	17.0
70	6.8	8.6	2.0	4.8	6.6	7.7	3.2	7.7	10.7	12.4	3.3	10.1	11.3	13.1
80	5.4	7.0	0.8	3.5	4.9	6.2	1.4	5.5	7.9	10.0	1.4	7.0	8.4	10.6
90	4.1	5.4	0.0	2.3	3.5	4.8	0.0	3.7	5.6	7.8	0.0	4.6	5.9	8.3
100	2.8	3.9		1.2	2.0	3.8		1.9	3.3	6.2		2.5	3.5	6.6
110	1.8	2.6		0.5	1.1	3.0		0.7	1.8	4.8		1.0	1.9	5.1
120	1.0	1.5		0.0	0.5	2.4		0.0	0.8	3.9		0.0	0.8	4.1
130	0.5	0.6			0.2	1.6			0.3	2.6			0.3	2.8
140	0.0	0.0			0.0	1.1			0.0	1.7			0.0	1.8
150						0.6				0.9				1.0
160						0.0				0.0				0.0

Step (3) <u>Compute the basin inflow hydrographs</u>

The critical storm duration for maximum basin storage has to be determined by routing post-development inflow hydrographs of different durations (longer than t_c) through the basin. The basin inflow hydrographs shown in Table 20.B2 are obtained by subtracting the non-routed low flow bypass of 3.8 m^3/s from the post-development total flow hydrographs provided in Table 20.B1 (only the 10 minute time increment flow values are shown).

Table 20.B2 Basin Inflow Hydrographs

Time (mins)	ARI (years)											
	5				50				100			
	Storm duration (minutes)											
	30	45	60	75	30	45	60	75	30	45	60	75
	Basin Inflow (total flow less low flow bypass) (m^3/s)											
0					0.0	0.0	0.0		0.0	0.0	0.0	
10	0.0	0.0	0.0	0.0	1.5	1.2	0.5	0.0	1.8	2.8	0.8	0.0
20	5.6	3.5	2.6	0.1	11.4	7.9	6.6	2.5	12.3	11.5	7.2	2.9
30	**17.1**	9.9	7.9	2.9	**30.0**	18.0	15.1	7.0	**32.0**	24.9	16.2	7.6
40	8.2	**15.4**	**11.5**	7.0	15.6	**25.3**	**20.8**	13.6	16.7	**33.3**	**22.3**	14.6
50	2.6	7.6	8.1	**9.0**	6.6	14.5	15.4	**16.9**	7.2	20.2	16.6	**18.1**
60	0.0	3.2	5.1	6.1	1.8	7.4	10.6	12.2	2.1	11.0	11.5	13.2
70		1.0	2.8	3.9	0.0	3.9	6.9	8.6	0.0	6.3	7.5	9.3
80		0.0	1.1	2.4		1.7	4.1	6.2	0.0	3.2	4.6	6.8
90			0.0	1.0		0.0	1.8	4.0		0.8	2.1	4.5
100				0.0			0.0	2.4		0.0	0.0	2.8
110								1.0				1.3
120								0.1				0.3
130								0.0				0.0

Step (4) <u>Make a preliminary estimate of the required basin volume</u>

A preliminary estimate of the required basin volume is obtained using Equation 20.13 for the largest design storm ARI (i.e. major system ARI). The preliminary basin volume is estimated for each basin inflow hydrograph and the largest value selected. The results are summarised in Figure 20.B1.

Parameter	Storm Duration (minutes)			
	30	45	60	75
V_i (m^3)	40030	48020	48690	45880
Q_i (m^3/s)	30.0	25.3	20.8	16.9
Q_o (m^3/s)	11.3	11.3	11.3	11.3
t_i (minutes)	70	90	100	130
t_p (minutes)	30	40	40	50
V_s / V_i	0.638	0.592	0.491	0.379
Preliminary V_s (m^3)	25556	**28450**	23908	17409

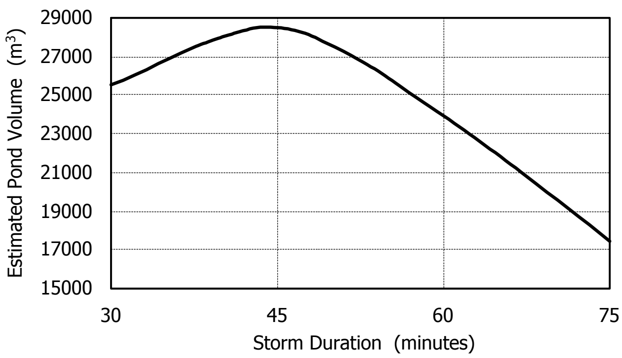


Figure 20.B1 Preliminary Determination of Critical Storm

Step (5) Develop a basin grading plan

The location and grading of the basin embankment and storage area is selected by trial and error so that at least the preliminary estimated volume of 28,450 m^3 is available in the basin to cater for the critical 50 year ARI design storm. The floor of the basin is graded at 1.4% toward the primary outlet and the basin and floodway side slopes are 6(H):1(V). A preliminary grading plan for the basin is shown in Figure 20.B2.

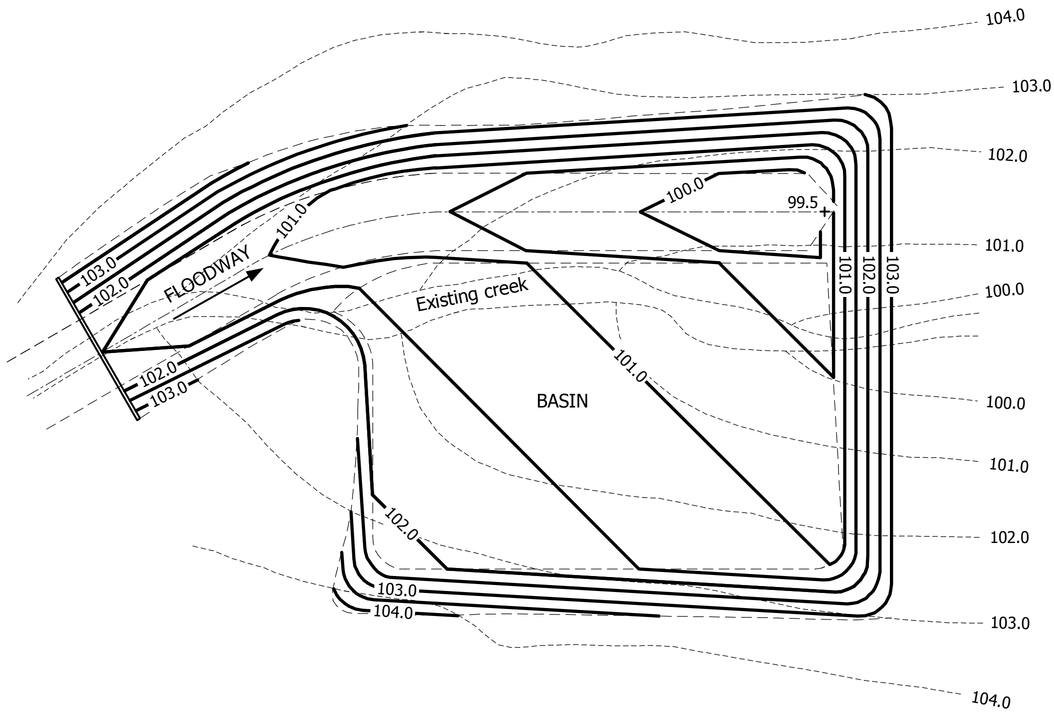


Figure 20.B2 Preliminary Grading Plan at the Detention Basin Site

Table 20.B3 Basin Stage-Storage Relationship

Elevation (m, LSD)	Stage (m)	Area (m^2)	Δ Storage (m^3)	Total Storage (m^3)
99.50	0.00	0	0	0
99.75	0.25	360	45	45
100.00	0.50	800	145	190
100.25	0.75	1230	254	444
100.50	1.00	1880	389	833
100.75	1.25	3950	729	1562
101.00	1.50	6650	1325	2887
101.25	1.75	9670	2040	4927
101.50	2.00	12690	2795	7722
101.75	2.25	15140	2176	11226
102.00	2.50	16370	3939	15165
102.25	2.75	17100	4184	19349
102.50	3.00	17610	4339	23688
102.75	3.25	18670	4535	28223
103.00	3.50	21450	5015	33238

Step (6) Compute the stage-storage relationship

For the grading plan developed in Step 5, the water surface area was calculated at 0.25 m intervals of basin depth (stage) from elevation 99.50 m, LSD. The segmental storage volume between successive stages was calculated using Equation 20.2 and summed to obtain the total stage-storage relationship as shown in Table 20.B3.

Step (7) Size the minor design storm primary outlet

The optimum size of the minor design storm primary outlet needs to be determined by trial and error to produce a maximum basin outflow that is as close as practicable to the required flow limit of 7.6 m^3/s. This involves selecting an initial outlet arrangement and size, determining the stage-discharge relationship, and routing the basin inflow hydrographs through the basin to determine the maximum outflow and water level produced. The outlet arrangement and/or size is then refined, if necessary, and the process repeated until an acceptable maximum outflow and water depth is reached.

(i) Select trial outlet arrangement

To provide flow reduction for the 5 year ARI post-development design storm, a single 2.4 m x 1.2 m reinforced concrete box culvert was initially selected. The invert level of the upstream end of the culvert was set at stage 0.00 in the basin, which is at elevation 99.50 m, LSD.

(ii) Compute the stage-discharge relationship

The stage-discharge relationship may be estimated using the procedure outlined in Chapter 27 or using a suitable computer program. The culvert was determined to be inlet controlled.

(iii) Route the inflow hydrographs through the basin

Using a routing time step of one minute, the 45 minute storm was found to be the critical duration storm. After successive trials, the optimum culvert size was determined to be 2.4 m x 0.9 m. This culvert produced a maximum discharge of **7.45 m^3/s** which is acceptable as it is close to but less than the 5 year ARI basin outflow limit of **7.6 m^3/s**. The maximum water elevation in the basin is **101.55 m**, LSD.

The stage-discharge relationship for the 2.4 m x 0.9 m culvert is shown in Figure 20.B3 and a summary of the routing results for the critical storm is provided in Table 20.B4.

Stage (m)	Discharge (m^3/s)
0.00	0
0.25	0.51
0.50	1.45
0.75	2.66
1.00	3.87
1.25	4.84
1.50	5.81
1.75	6.76
2.00	7.33
2.25	7.88
2.50	8.40
2.75	8.91
3.00	9.39
3.25	9.86
3.50	10.32

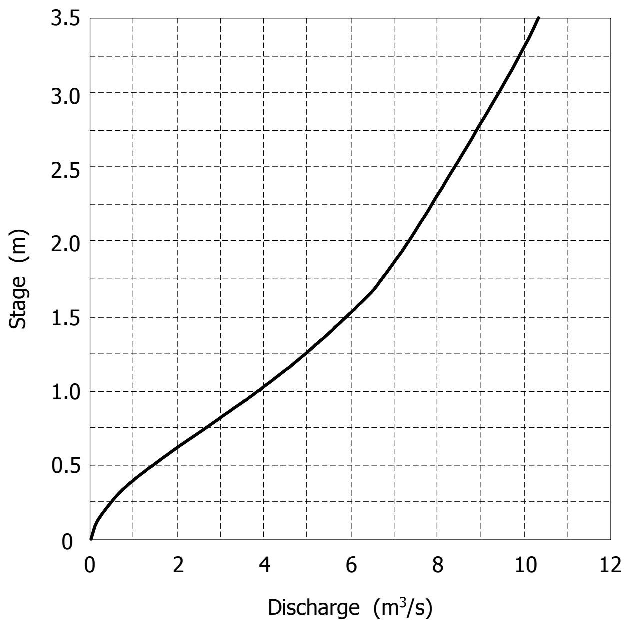


Figure 20.B3 5 Year ARI Stage-Discharge Relationship

Table 20.B4 Routing Results for 5 year ARI, 45 minute Basin Inflow Hydrograph

Time, t (mins)	Inflow, I (m^3/s)	I_j+I_{j+1} (m^3/s)	$(2S_j/\Delta t)-Q_j$ (m^3/s)	$(2S_{j+1}/\Delta t)-Q_{j+1}$ (m^3/s)	Outflow (m^3/s)	Water Level (m, LSD)	Stage (m, LSD)	Discharge, Q (m^3/s)	Storage, S (m^3)	$(2S/\Delta t)+Q$ (m^3/s)
12.0	0.00	0.00	0.00	0.00	0.00	99.50	0.00	0.00	0	0.00
13.0	0.21	0.21	0.10	0.21	0.05	99.53	0.25	0.51	45	2.01
(omitted)	:	:	:	:	:	:	0.50	1.45	190	7.78
:	:	:	:	:	:	:	0.75	2.66	444	17.46
:	:	:	:	:	:	:	1.00	3.87	833	31.64
48.0	8.88	18.45	271.09	285.96	7.43	101.55	1.25	4.84	1562	56.91
49.0	8.22	17.10	273.31	288.20	7.44	101.55	1.50	5.81	2887	102.04
50.0	7.60	15.82	274.25	289.14	**7.45**	**101.55**	1.75	6.76	4927	170.99
51.0	7.06	14.66	274.02	288.90	7.44	101.55	2.00	7.33	7722	264.73
52.0	6.54	13.60	272.74	287.61	7.44	101.55	2.25	7.88	11201	381.58
(omitted)	:	:	:	:	:	:	2.50	8.40	15140	514.49
:	:	:	:	:	:	:	2.75	8.91	19324	655.99
:	:	:	:	:	:	:	3.00	9.39	23663	802.95
93.0	0.00	0.00	0.03	0.06	0.02	99.51	3.25	9.86	28198	958.46
94.0	0.00	0.00	0.01	0.03	0.01	99.50	3.50	10.32	33213	1132.08
95.0	0.00	0.00	0.01	0.01	0.00	99.50				

The above calculations have been done using a spreadsheet. The steps in the calculation procedure are illustrated in Figure 20.B4.

Starting with $D_j = 0$, the values in columns D to G for time step $j+1$ are calculated as follows:

1. $E_{j+1} = D_j + C_{j+1}$
2. F_{j+1} and G_{j+1}, are interpolated from the stage-discharge relationship columns H and I on the left hand side of the routing table using the value of E_{j+1} in column K. The basin zero elevation of 99.50 is added to the interpolated stage value to obtain the water level.
3. $D_{j+1} = E_{j+1} - 2\ F_{j+1}$

This process is repeated for each consecutive time step until the outflow reduces to zero.

Time, t	Inflow, I	I_j+I_{j+1}	$(2S_j/\Delta t)-Q_j$	$(2S_{j+1}/\Delta t)-Q_{j+1}$	Outflow	Water Level	Stage	Discharge, Q	Storage, S	$(2S/\Delta t)+Q$
j	0.00	0.00	0.00		0.00	99.50	0.00	0.00	0	0.00
$j+1$	0.21	0.21	0.11	0.21	0.05	99.53	0.25	0.51	45	2.01
$j+2$	0.57	0.78	0.43	0.88	0.22	99.61				
A	B	C	D	E	F	G	H	I	J	K

Figure 20.B4 Tabular Routing Procedure

Step (8) <u>Size the major design storm primary outlet</u>

The procedure for sizing the major design storm primary outlet is the same as the minor design storm outlet.

(i) Select trial outlet arrangement

To provide flow reduction for the 50 year ARI post-development design storm, an additional single 2.4 m x 1.2 m reinforced concrete box culvert was initially selected with an invert level at the upstream end of 101.60 m, LSD that corresponds to a stage of 2.10 m. The assumed length of this culvert was also approximately 30 m.

(ii) Compute the stage-discharge relationship

The stage-discharge relationship is the summation of the 5 year ARI and the 50 year ARI culvert capacities. The culvert capacity may be estimated using the procedure outlined in Chapter 27 or using a suitable computer program. The culvert was also determined to be inlet controlled.

(iii) Route the inflow hydrographs through the basin

Using a routing time step of one minute, the 45 minute storm was also found to be the critical duration storm. After successive trials, the optimum culvert size was determined to be 3.6 m x 0.9 m. This culvert, in conjunction with the lower level minor storm culvert, produced a maximum discharge of **11.84 m^3/s**. This is acceptable as it is close to but less than the 50 year ARI basin outflow limit of **12.0 m^3/s**. The maximum water elevation in the basin is **102.25 m**, LSD that corresponds to a maximum water depth of 2.75 m. This is also acceptable, as it is less than the recommended maximum depth of 3.0 m.

The stage-discharge relationship for the optimum culvert arrangement is shown in Figure 20.B5 and a summary of the routing results for the critical storm is provided in Table 20.B5.

Stage (m)	2.4 m x 0.9 m Box Culvert (m^3/s)	3.6 m x 0.9 m Box Culvert (m^3/s)	Total Discharge (m^3/s)
0.00	0	-	0.00
0.25	0.51	-	0.51
0.50	1.45	-	1.45
0.75	2.66	-	2.66
1.00	3.87	-	3.87
1.25	4.84	-	4.84
1.50	5.81	-	5.81
1.75	6.76	-	6.76
2.00	7.33	-	7.33
2.10	7.56	0.00	7.56
2.25	7.88	0.33	8.21
2.50	8.40	1.42	9.82
2.75	8.91	2.95	11.86
3.00	9.39	4.79	14.18
3.25	9.86	6.12	15.98
3.50	10.32	7.45	17.77

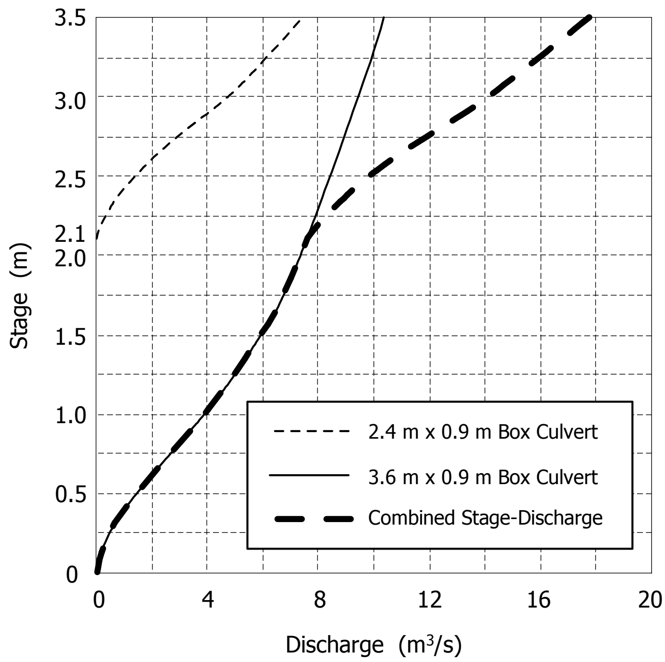


Figure 20.B5 50 Year ARI Stage-Discharge Relationship

Table 20.B5 Routing Results for 50 year ARI, 45 minute Basin Inflow Hydrograph

Time, t (mins)	Inflow, I (m^3/s)	I_j+I_{j+1} (m^3/s)	$(2S_j/\Delta t)-Q_j$ (m^3/s)	$(2S_{j+1}/\Delta t)-Q_{j+1}$ (m^3/s)	Outflow (m^3/s)	Water Level (m, LSD)	Stage (m, LSD)	Discharge, Q (m^3/s)	Storage, S (m^3)	$(2S/\Delta t)+Q$ (m^3/s)
9.0	0.00	0.00	0.00	0.00	0.00	99.50	0.00	0.00	0	0.00
10.0	1.20	1.20	0.59	1.20	0.30	99.65	0.25	0.51	45	2.01
(omitted)	:	:	:	:	:	:	0.50	1.45	190	7.78
:	:	:	:	:	:	:	0.75	2.66	444	17.46
:	:	:	:	:	:	:	1.00	3.87	833	31.64
51.0	13.62	28.12	626.87	650.43	11.78	102.24	1.25	4.84	1562	56.91
52.0	12.79	26.41	629.64	653.28	11.82	102.25	1.50	5.81	2887	102.04
53.0	11.99	24.78	630.74	654.42	**11.84**	**102.25**	1.75	6.76	4927	170.99
54.0	11.24	23.23	630.31	653.97	11.83	102.25	2.00	7.33	7722	264.73
55.0	10.52	21.75	628.46	652.07	11.80	102.24	2.10	7.56	9050	309.23
(omitted)	:	:	:	:	:	:	2.25	8.21	11201	381.58
:	:	:	:	:	:	:	2.50	9.82	15140	514.49
:	:	:	:	:	:	:	2.75	11.86	19324	655.99
120.0	0.00	0.00	0.02	0.05	0.01	99.51	3.00	14.18	23663	802.95
121.0	0.00	0.00	0.01	0.02	0.01	99.50	3.25	15.98	28198	958.46
122.0	0.00	0.00	0.01	0.01	0.00	99.50	3.50	17.77	33213	1132.08

Step (9) <u>Size the secondary outlet arrangement</u>

As there is no required limit for the 100 year ARI, the main criterion for selecting the secondary outlet size is to minimise the overall height of the embankment without having an excessively large secondary outlet.

(i) Select trial outlet arrangement

A 12 m wide broad-crested weir spillway with 3(H):1(V) side slopes was initially selected as the basin secondary outlet. The spillway was set at the side of the embankment at an elevation of 102.50 m, LSD (50 year ARI maximum water level of 102.25 m plus 250 mm freeboard) which corresponds to a stage of 3.00 m.

(ii) Compute the stage-discharge relationship

The stage-discharge relationship as shown in Figure 20.B6 is the combined discharge for all the basin outlets.

(iii) Route the inflow hydrographs through the basin

Using a routing time step of one minute, the 45 minute storm was again found to be the critical duration storm. The maximum water level in the basin is **102.70 m**, LSD that corresponds to a water depth of 3.2 m. Allowing a freeboard of 300 mm for wave action, the embankment crest elevation is set at **103.00 m**, LSD (height 3.5 m) which is considered acceptable.

The basin will provide a large reduction in the 100 year ARI flow, being reduced from 33.3 m^3/s to 17.69 m^3/s through the basin and from 37.1 m^3/s to 21.49 m^3/s in the downstream floodway.

A summary of the routing results for the critical storm is provided in Table 20.B6.

Stage (m)	50 year ARI Flow (m^3/s)	Weir Flow (m^3/s)	Stage-Storage (m^3/s)
0.00	0.00	-	0.00
0.25	0.51	-	0.51
0.50	1.45	-	1.45
0.75	2.66	-	2.66
1.00	3.87	-	3.87
1.25	4.84	-	4.84
1.50	5.81	-	5.81
1.75	6.76	-	6.76
2.00	7.33	-	7.33
2.10	7.56	-	7.56
2.25	8.21	-	8.21
2.50	9.82	-	9.82
2.75	11.86	-	11.86
3.00	14.18	0.00	14.18
3.25	15.98	2.55	18.53
3.50	17.77	7.21	24.98

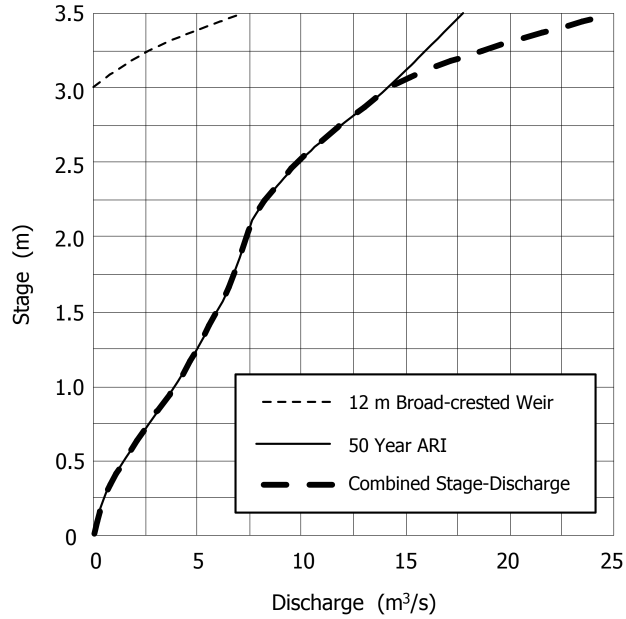


Figure 20.B6 100 Year ARI Stage-Discharge Relationship (Secondary Outlet)

Table 20.B6 Routing Results for 100 year ARI, 45 minute Inflow Hydrograph

Time, t (mins)	Inflow, I (m^3/s)	I_j+I_{j+1} (m^3/s)	$(2S_j/\Delta t)-Q_j$ (m^3/s)	$(2S_{j+1}/\Delta t)-Q_{j+1}$ (m^3/s)	Outflow (m^3/s)	Water Level (m, LSD)	Stage (m, LSD)	Discharge, Q (m^3/s)	Storage, S (m^3)	$(2S/\Delta t)+Q$ (m^3/s)
6.0	0.00	0.00	0.00	0.00	0.00	99.50	0.00	0.00	0	0.00
7.0	0.60	0.60	0.30	0.60	0.15	99.58	0.25	0.51	45	2.01
(omitted)	:	:	:	:	:	:	0.50	1.45	190	7.78
:	:	:	:	:	:	:	0.75	2.66	444	17.46
:	:	:	:	:	:	:	1.00	3.87	833	31.64
50.0	20.20	41.48	887.28	922.32	17.52	102.69	1.25	4.84	1562	56.91
51.0	19.07	39.27	891.27	926.55	17.64	102.70	1.50	5.81	2887	102.04
52.0	17.99	37.06	892.95	928.33	**17.69**	**102.70**	1.75	6.76	4927	170.99
53.0	16.96	34.95	892.55	927.90	17.68	102.70	2.00	7.33	7722	264.73
54.0	15.98	32.94	890.28	925.49	17.61	102.70	2.10	7.56	9050	309.23
(omitted)	:	:	:	:	:	:	2.25	8.21	11201	381.58
:	:	:	:	:	:	:	2.50	9.82	15140	514.49
:	:	:	:	:	:	:	2.75	11.86	19324	655.99
132.0	0.00	0.00	0.02	0.05	0.01	99.51	3.00	14.18	23663	802.95
133.0	0.00	0.00	0.01	0.02	0.01	99.50	3.25	15.98	28198	958.46
134.0	0.00	0.00	0.01	0.01	0.00	99.50	3.50	17.77	33213	1132.08

The basin inflow and outflow hydrographs for all critical 45 minute storms are shown in Figure 20.B7.

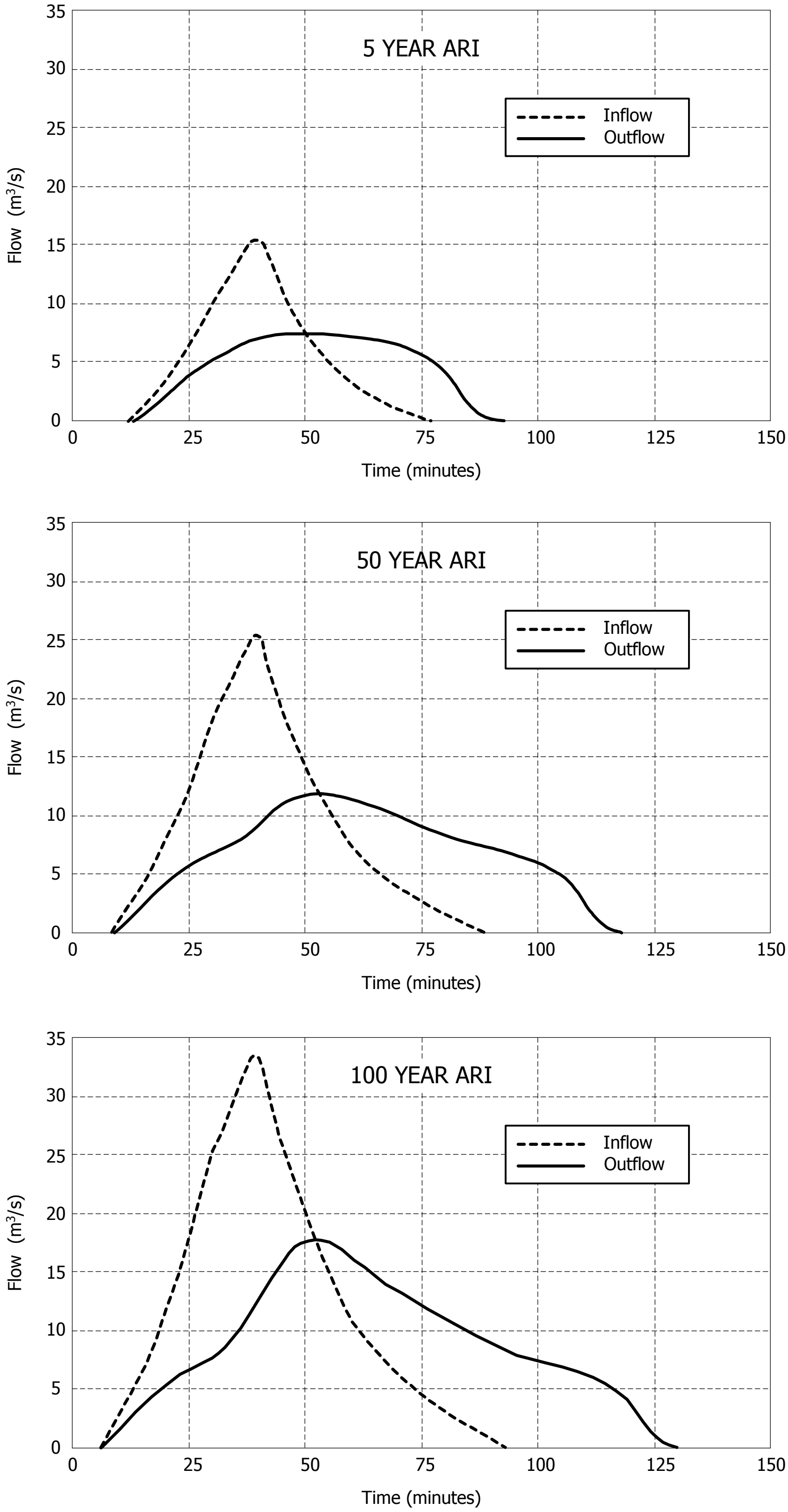


Figure 20.B7 Basin 45 minute Inflow and Outflow Hydrographs

Step (10) Check behaviour under extreme conditions

The behaviour of the basin under extreme storms or blocked outlets may need to be checked. This analysis has not been included in the example but would involve determining an inflow hydrograph or alteration to the stage-discharge relationship and routing this through the basin to see what effect it would have on the basin embankment.

Step (11) Size downstream erosion protection measures

The sizing of erosion protection measures has also not been included in the example. The reader should refer to Chapter 29 for information on designing erosion protection measures and energy dissipator structures.

Step (12) Prepare design drawings and specifications

The main components of the detention basin are shown in Figure 20.B8. For construction purposes, more detailed drawings and specifications need to be produced.

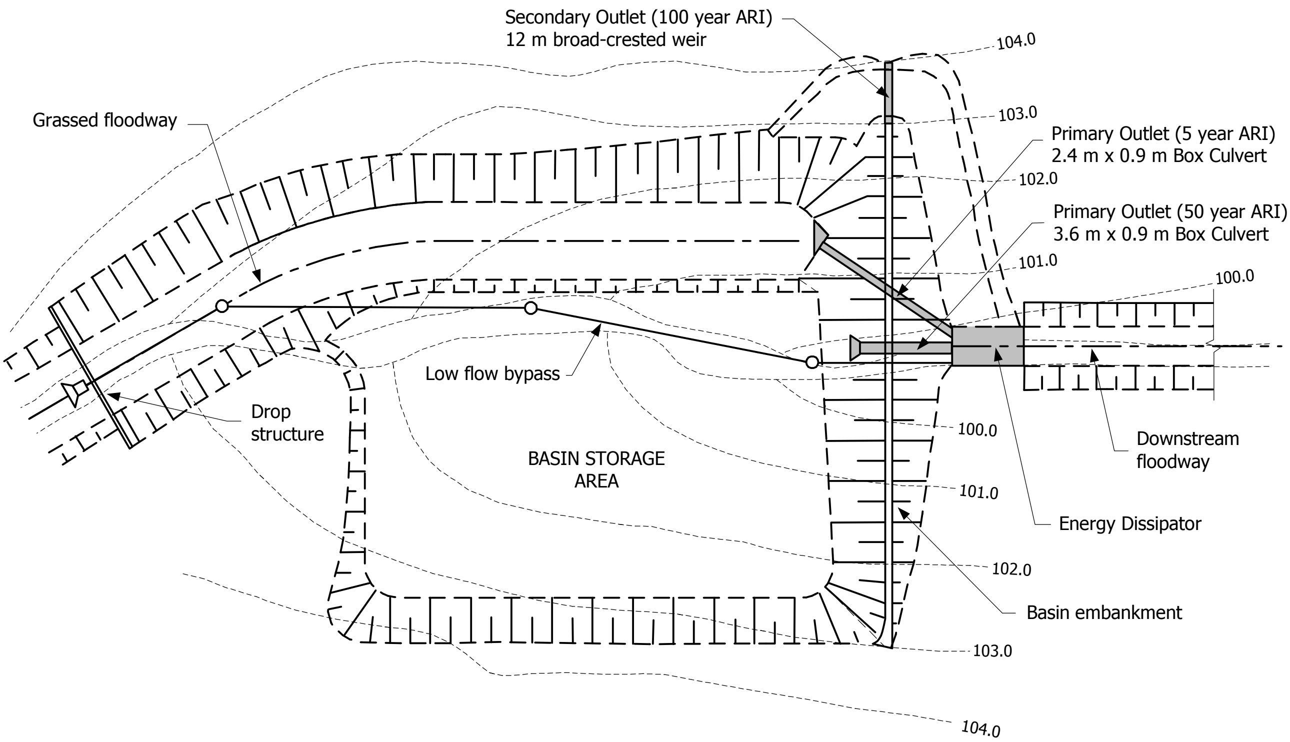


Figure 20.B8 Detention Basin Schematic

An alternative primary outlet arrangement could be a single box culvert with a rectangular baffled inlet for the 5 year ARI flow and a circular riser to provide additional outlet capacity for the 50 year ARI flow as shown in Figure 20.B9.

The stage-discharge relationship for this type of arrangement can be difficult to estimate, as there are a number of different flow regimes that the outlet will be subjected to in operation. For example, the flow through the baffled inlet, as well as the riser, will change from weir flow to orifice flow as the inlet becomes submerged. When the riser begins to operate, the relative head on the baffled inlet will be reduced and the thus the outflow will be reduced. The outflow characteristics of both outlets will also be dependent on the discharge capacity of the culvert. Depending on the size, location, and downstream hazard rating for a particular basin, the stage-discharge relationship for such an arrangement may need to be verified by physical model testing.

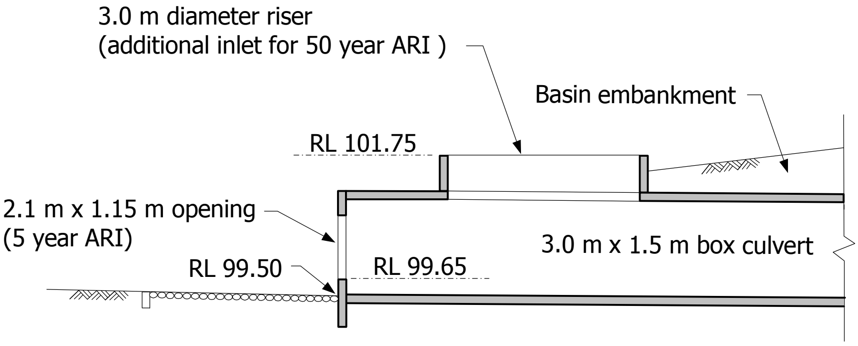


Figure 20.B9 Alternative Primary Outlet Arrangement