

CHAPTER 7 DETENTION POND

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7.1 INTRODUCTION

This Chapter provides guidelines and procedures for designing detention pond, a facility presently regarded as the most important measure in the stormwater management practice. Detention ponds are used for controlling stormwater quantity impacts resulting from larger urbanising catchment. The facilities are commonly located in public areas by the construction of embankment across stream, channel or by the excavation of a potential storage area. Ponds can be developed as “Dry” or “Wet” type as shown in Figure 7.1. For wet pond, the catchment area served is greater than 10ha. It is to ensure that the area generates enough baseflow to replenish and maintain the permanent pool level.

The pond reduces flood peak discharge downstream by temporary storage and gradual release using control outlet, usually ungated structure, riser or culvert, located at the base of the embankment. An overflow spillway, set near the top of the embankment is required to safely pass storms that exceed the pond capacity. The spillway protects the embankment from possible failure and subsequently downstream life and resources.

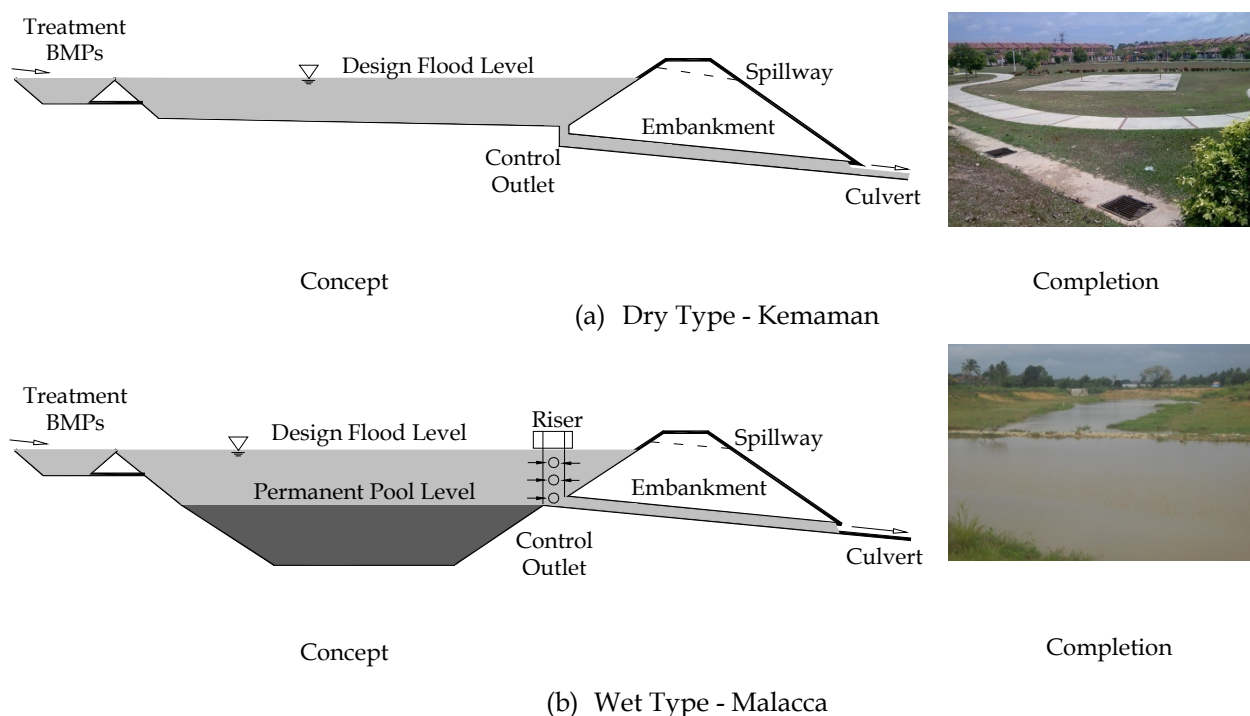


Figure 7.1: Typical Detention Ponds

7.1.1 System Components and Configurations

A detention pond, primarily wet and multi-purposes, ideally requires the following components (Figure 7.2) to be included in the system:

- Inlet zone – inlet structure, GPT, sediment forebay, water quality pond or wetlands, maintenance ramp and rock weir;
- Storage zone – low flow channel/drain, maintenance ramp, pond body and recreational facilities; and
- Outlet zone – primary outlet (usually a multi-level riser with culvert), secondary outlet (usually a spillway), embankment, outfall/energy dissipator.

The outlet structures are usually fixed. For larger reservoirs the structures are gated, movable and generally operated based on automation. The inlet and outlet zones must be protected against erosion. The system components should be configured, with shapes harmonised with the natural surroundings. Geometrically square or rectangular shapes are not recommended. The length to width ratio of the storage zone should be at least 3:2.

7.1.2 Water Quality Control Requirement

All detention ponds require pretreatment facilities (BMPs) which are to be located at the inlet zone or upstream of the ponds to ensure that the ponds are not polluted and their water quality levels always meet regulatory standards. The facilities shall include the followings:-

- GPTs/trash rack and sediment forebay
- Water quality pond or wetlands; and
- Other types of BMPs as needed to reduce pond's contamination.

Water quality control structures (BMPs) shall be designed and constructed based on 40mm runoff depth. Depending on the level of pollutant the treatment at each inlet zone could be in single BMPs and/or in series/treatment train BMPs. Design of the facilities shall be carried out based on procedures presented in Chapter 10 and 11.

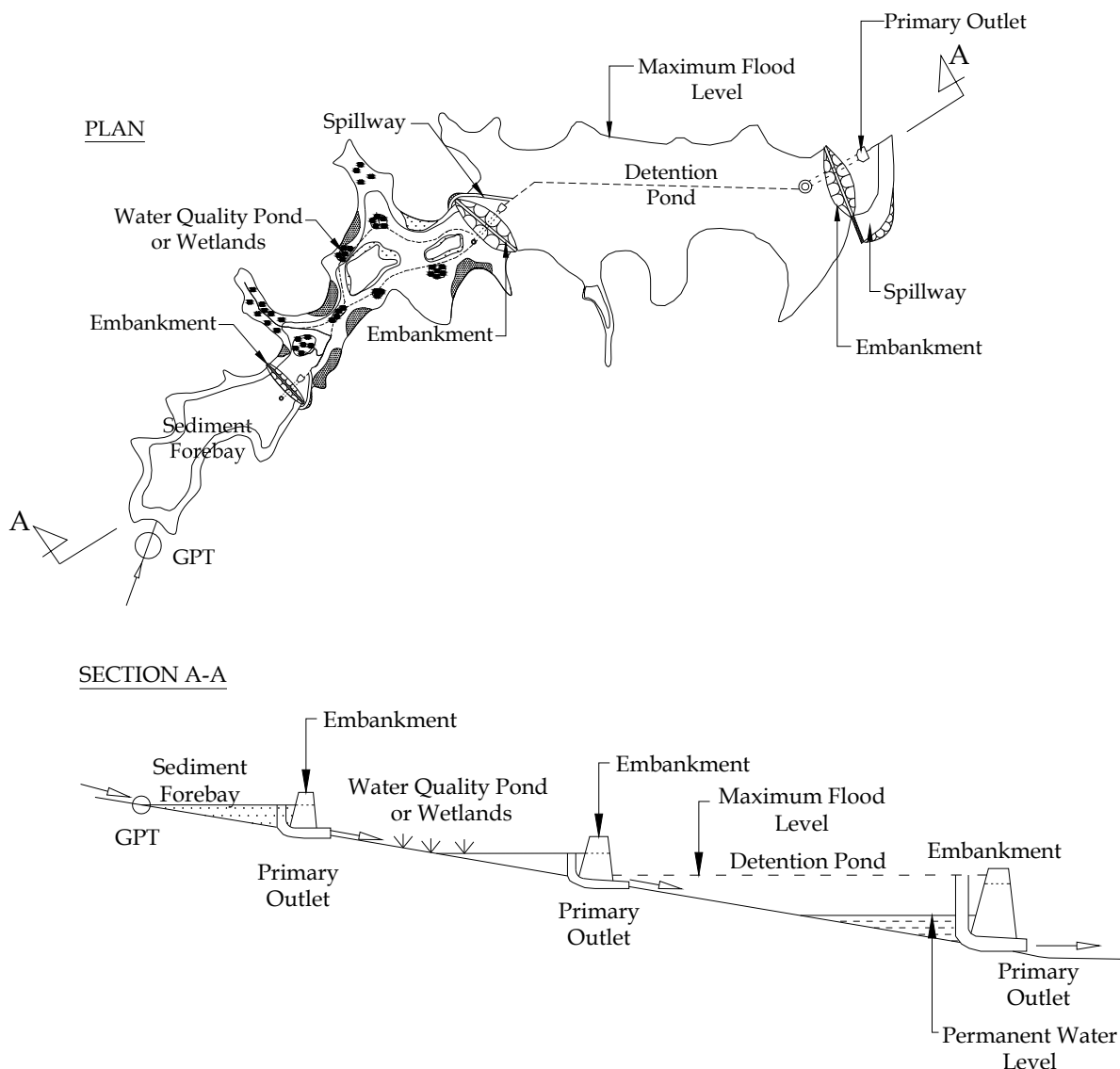


Figure 7.2: Typical Detention Pond Components

7.2 DESIGN CONSIDERATION

7.2.1 General

(a) Design and Analysis

In designing a flood detention facility to meet the flow control objectives stated in Chapter 1, it is necessary to consider the behaviour of the pond storage by examining:

- the degree of reduction of flows from the catchment;
- the depths and duration of ponding; and
- the frequency at which the overflow spillway comes into operation

When considering the required levels of operation, it is necessary to design underdrains beneath storages to bypass low flows. In addition, it may be necessary to consider extreme events such as probable maximum precipitation (PMP) and to examine the effects of an embankment failure.

(b) Release Timing

A stormwater detention structure increases both the peak discharge release time and total outflow hydrograph duration. Though there is reduction in the peak flow, the time shifting may result in adverse flow effects further downstream if the reduced peak from the controlled subcatchment coincides with the main stream peak. This is true in locations where multiple detention facilities have been installed within developing subcatchments, downstream stormwater flooding continue to occur. Therefore, it is critical that an optimum release timing be considered in the analysis of multiple stormwater control facilities to ensure that the desired result is obtained for design use. The outflow hydrograph release time or empty time of a detention pond is recommended to be within 24 hours.

(c) Multiple Design Storms

A particular rainfall pattern which provides the worst case for a detention pond shall be found by trial and error of pond modelling behaviour applying storms of different durations and magnitudes ARIs.

(d) Extreme Floods

There is a risk of failure of the embankment or wall of a detention storage during a very large flood, resulting in water wave, mud and debris. For earth embankments the most common failure mechanism is erosion of the downstream face due to overflows. If a rill is formed and rapidly growing, water pressure may cause stored water to burst through the weakened embankment. Designers should perform dam break analysis using PMP data and a probable maximum flood (PMF) calculated at the pond site. Analysis should be carried out to find minimum dam height, pool surface area, storage volume, overtopping depth, likely flow velocities, etc.

(e) Public Safety

The risk of persons drowning in detention ponds has usually been the most important safety consideration. Recommended depths of dry pond are limited to 1.2 m or 1.5 m or otherwise fence the ponds from public excess.

Other requirements are set as follows:

- The side slope of ponds should not be steeper than 1(V):4(H). Ponds with steeper slope may require a fence or rail. Special attention should be paid to the outlets, to ensure that persons are not allowed into the area. Rails, fences, crib-walls, anti-vortex devices and grates should be provided where necessary. Many safe designs are desirable such as introduction of trees and mounds within ponds as refuges.

- Special attention must be given to pond overflow spillways in design and operation to avoid catastrophic failures, which may cause loss of property and life at downstream areas. PMF studies should be conducted for ponds with vulnerable downstream areas or for large, multiple-pond systems to ensure adequate sizing of secondary outlet (emergency spillway) to serve for extreme events.

7.2.2 Inlet

7.2.2.1 Inlet Structure

The size of the inlet should be such that the capacity is equal to or higher than the design capacity of the approach channel. The invert level of the pond should be same or less than the invert of the incoming channel. Wings of the inlet structures should be protected (by concrete wall, stone pitching, gabion, etc.) against erosion.

7.2.2.2 GPTs and Sediment Forebay

GPTs are used mainly for removal of litter, debris and coarse sediment from stormwater. Some designs also provide for oil separation. For large catchment and pond system the facilities shall be designed according to site requirement while for small drainage system proprietary devices can be adopted. Refer to Chapter 10 for GPT design procedure. Further a sediment forebay is recommended to remove finer soil particulates. Its size is about one-third ($1/3$) of water quality pond or wetlands volume and with a minimum length to width ratio of 2:1.

7.2.2.3 Water Quality Pond or Wetlands

A sediment forebay together with water quality pond or wetlands are common practice to trap particulate and dissolved pollutants before entering the detention pond. Occasionally the system requires a volume of about 30% of the detention pond volume. The design of the water quality pond or wetland facilities should follow the procedure presented in Chapter 11.

7.2.3 Primary Outlet

Primary outlets are designed for the planned control and release of water from a detention pond. They may be a single stage outlet structure or several outlet structures combined to provide multi-stage outlet control. Figure 7.3 shows some typical primary outlets. The outlet conduit must be designed to carry all flows considered in the design of the riser structure.

For a single stage, the facility is typically designed as a simple culvert. For multi-stage system, the outlet control structure is designed by 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 pond embankment and discharges to the downstream conveyance system. Orifices and weirs can be designed using the equations provided in Chapter 2.

Primary outlets for detention ponds shall be designed to reduce post-development peak flows to pre-development peak flows for the minor and major system design storm ARI (Table 1.1). For this will require a two-staged outlet configuration, one to control the minor system design flow and the other to control the major system design flow, in combination. However, to prevent possible failure of the embankment due to piping, the pipe downstream of the outlet works has to be sufficiently large to pass the major design flow without developing hydraulic surcharge pressure within it.

This requirement can readily be achieved for new development areas as sufficient land can be set aside to accommodate the necessary storage requirements. However, in existing developed 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 only while addition of more OSD facilities are encouraged to lower peak flow for the major system instead.

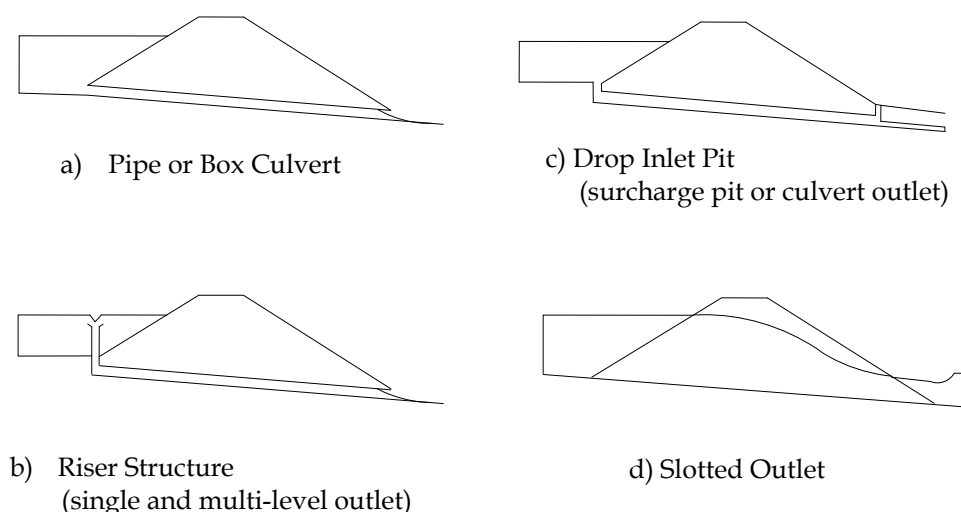


Figure 7.3: Typical Pond Primary Outlets

7.2.3.1 Culverts

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 pond catchment area, without developing any internal pressure. Such surface flow condition within the culvert is necessary to avoid culvert failure that may lead to embankment failure. 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. Detail procedure for culvert design is given in Chapter 18.

7.2.3.2 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 case trash racks is required to control the clogging. 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 pond outlets. For very small outlets, an even larger opening 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 7.4.

The following points should be followed for the trash racks:

- The trash rack should have an area at least ten (10) times larger than the control outlet opening.
- The inclined bar racks are recommended.
- A maximum angle of 60° to the horizontal can be allowed for the trash racks.
- The trash racks should be located at 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.
- Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

- The racks should have hinged connections.
- The invert of the outlet structure for a dry pond 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.

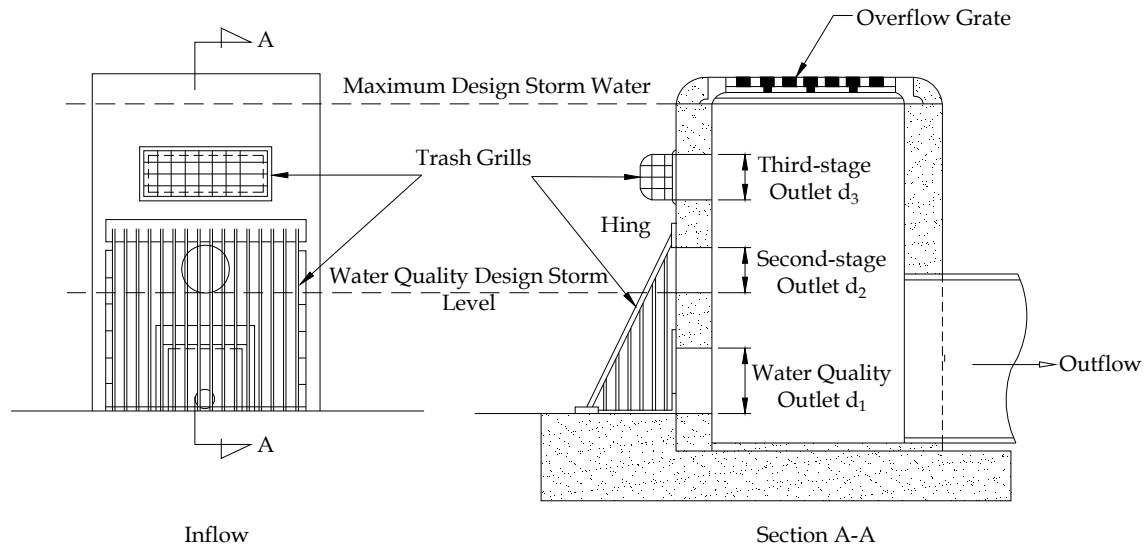


Figure 7.4: Typical Multistage Primary Outlet Trash Racks (DID, 2000)

7.2.4 Secondary Outlet (Emergency Spillway)

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 small that only very short, sharp thunderstorms are apt to threaten overtopping or embankment failure, and such storms are localised. Also, capacities of the structures are usually too small to create a flood wave.

By contrast, sizable on-line facilities with settlement/valuable assets downstream may pose a significant hazard if failure was 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. A hazard rating for the pond should be determined and a secondary outlet (emergency spillway) design ARI selected in accordance with the Federal Government or relevant State Government Dam Safety Guidelines. The secondary outlets for all non-hazard small detention ponds shall be designed to safely pass a minimum design storm of 100 year ARI through the pond. For hazard ponds they must be designed as per the approved guidelines (ANCOLD, 1986; USBR, 1987; USBR, 1992; FEMA, 1987 and Golzé, 1977).

7.2.5 Storage Zone

7.2.5.1 Dry Time

Drawdown time of the pond will depend on the type and purpose of the pond. The drawdown time is controlled by appropriate sizing of the primary and secondary outlets. Dry pond is proposed to be emptied within 12 hours after rain stops while water level of a wet pond should be back to normal 24 hours after the rain ceases.

7.2.5.2 Low Flow Channel

Provision should be made in a dry detention pond to bypass low flows through or around the pond using low flow channel or pipe. This is necessary to ensure that the pond floor, particularly if it is grassed, is not inundated by small storms or continually wetted by dry weather baseflow. The minimum rate of bypass flow should be computed based on 10mm runoff depth from the contributing catchment.

7.2.6 Embankment

7.2.6.1 Classification

Dry detention ponds 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.

7.2.6.2 Design Criteria

All other detention ponds 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.

a) Pond Water Depth

The maximum pond water depth 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.

b) Embankment Top Widths

Minimum recommended embankment top widths are provided in Table 7.

Table 7.1: 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

c) Side Slopes

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

d) Bottom Grades

The floor of the pond shall be designed with a minimum grade of 1% to provide positive drainage and minimise the likelihood of ponding. Adequate drainage of the pond 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.

e) Freeboard

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

7.3 EROSION PROTECTION

7.3.1 Primary Outlet and Downstream

When the dimensions of the detention pond 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 of a small pond, 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 pond embankment.

Below a pipe outlet, a suitable device is the 'impact energy dissipator'. 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 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 riprap is provided in Chapter 20.

7.3.2 Secondary Outlet and Downstream

The surfaces of embankments and spillway channels must be protected against damage by scour when they are subjected 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 acceptable.

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. 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. 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. Details on erosion control practices can be referred to Chapter 12.

For reducing erosion an open stilling pond, as discussed in Chapter 20, 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.

7.4 SAFETY AND AESTHETICS

7.4.1 Public Safety

A detention pond must be designed with public safety in mind when the facility is in operation and also during periods between storms when the facility is emptied. Appropriate ways must be considered to prevent and to discourage the public from being exposed to high-hazard areas during these periods.

Ponds 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 from happening. These should be inclined at an angle of 60° 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 or falling off these structures during periods when the pond 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.

7.4.2 Landscaping

Aesthetics of the finished facility is extremely important. Wherever possible, designs should incorporate naturally shaped ponds with landscaped banks, footpaths, and selective planting of vegetation to help enrich the area and provide a focal point for surrounding development.

Sympathetic landscaping and the resulting improvement in local visual amenity will also encourage the public to accept detention ponds as an element of the urban natural environment and not as a target for vandalism. Trees and shrubs should not be planted on pond embankments as they may increase the danger of bank failure by 'piping effect' along the line of the roots.

7.5 SIZING FLOOD DETENTION SITE

For early planning purpose pond area can be approximated using Figure 7.5 for a catchment having different landuses, as represented by average runoff coefficient C (refer Chapter 2) and for various pond depths. The final design of a detention facility involves an inflow hydrograph, a stage vs. storage curve, a stage vs. discharge and storage indicator number curve. However, before a stage vs. storage and a stage vs. discharge curve can be developed, a preliminary estimate of the needed storage capacity and the shape of the storage facility are required. Trial computations will be made to determine if the estimated storage volume will provide the desired outflow hydrograph.

7.5.1 Pond Volume Estimate

The design storm for estimating the required storage volume shall be for major system, up to 100 yr ARI. The following sections present Rational Hydrograph Method (RHM) for determining an initial estimate of the storage required to provide a specific reduction in peak discharge. The method provides preliminary estimates only and a degree of judgement is needed to determine the initial storage estimate.

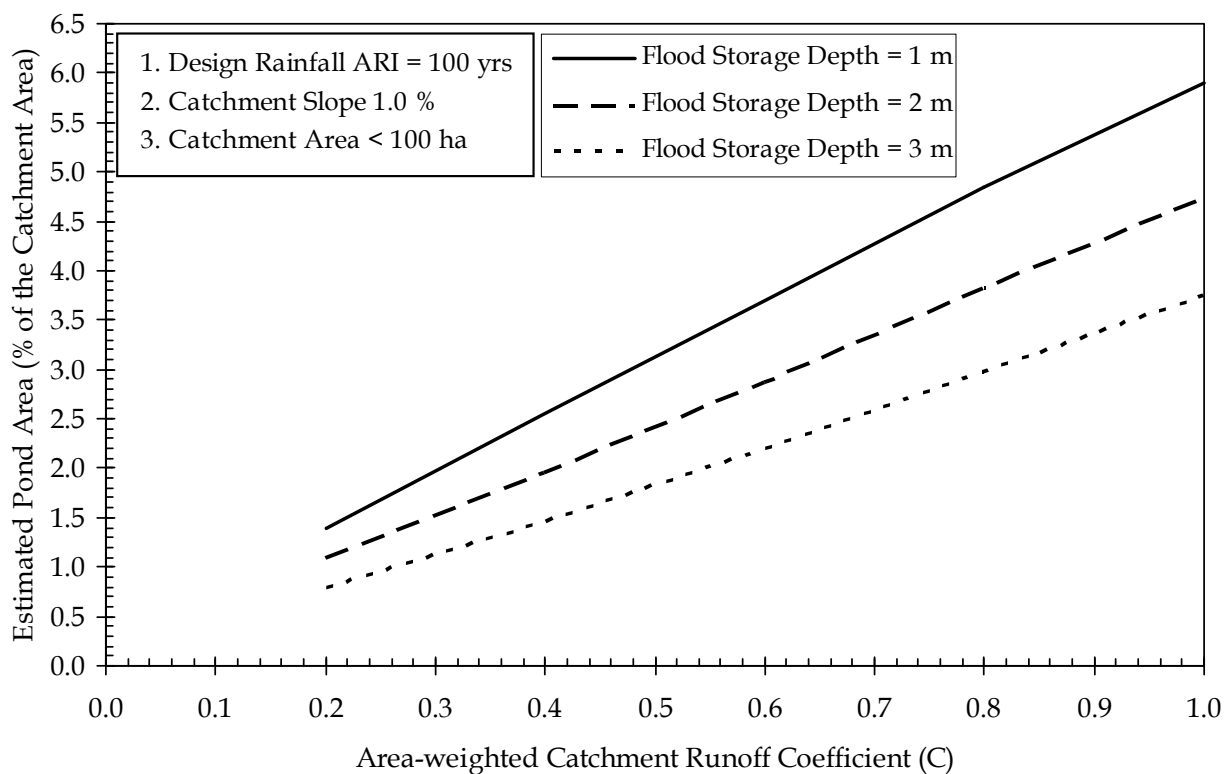


Figure 7.5: Estimate of Pond Area for Planning Purpose

To work with detention storage design the inflow hydrograph, derived using RHM, is to be provided along with the intended release rate. With these values, the facility discharge curves can be approximated using the procedures as shown in Figure 7.6 and 7.7.

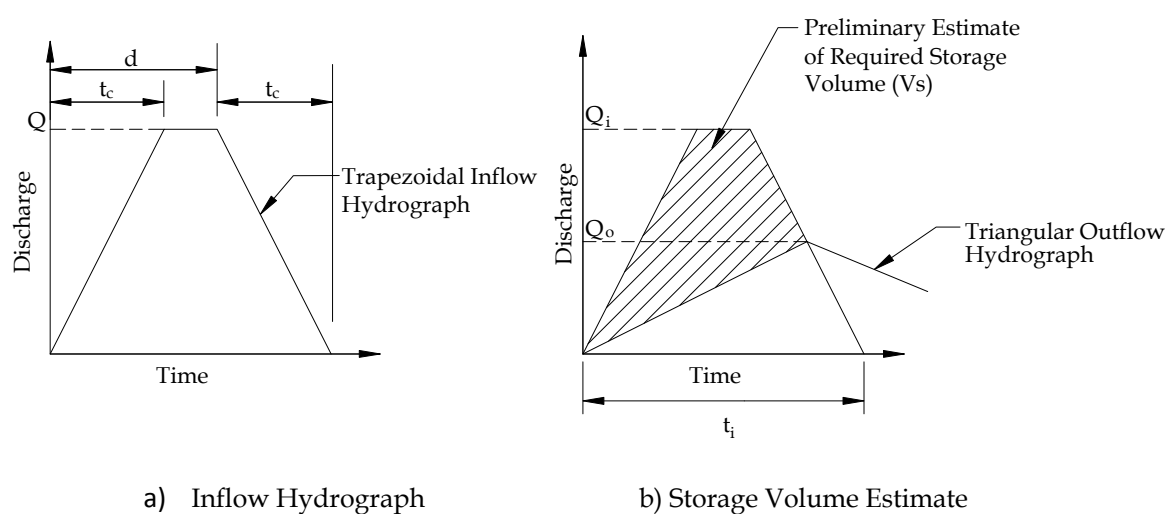
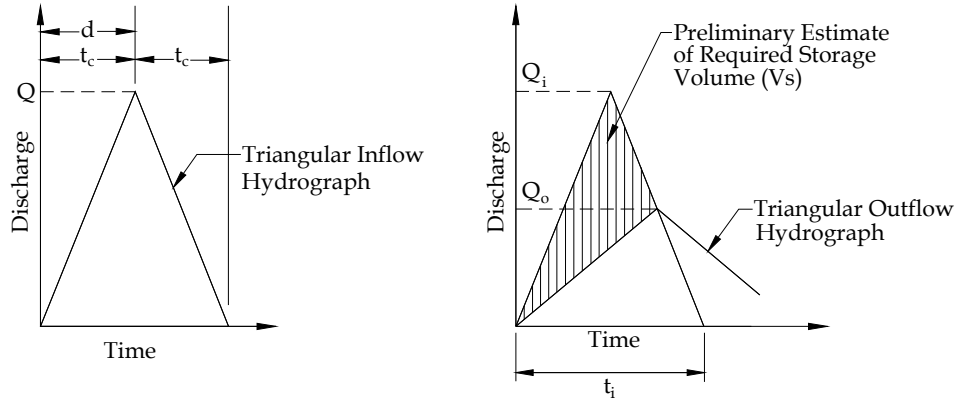


Figure 7.6: Estimating Detention Pond Storage by RHM for Type 1 Hydrograph



a) Inflow Hydrograph

b) Storage Volume Estimate

Figure 7.7: Estimating Detention Pond Storage by RHM for Type 2 Hydrograph

The storage volume (the shaded area), for each type of hydrograph (Type 1 and Type 2) can be readily computed using Equation 7.1.

$$V_s = dQ_i - 0.5t_iQ_o \quad \text{for Type 1} \quad (7.1a)$$

$$V_s = 0.5t_i(Q_i - Q_o) \quad \text{for Type 2} \quad (7.1b)$$

Where,

- V_s = Storage volume (m^3);
- Q_i = Peak inflow (m^3/s);
- Q_o = Peak outflow (m^3/s); and
- t_i = Total duration of inflow; $d + t_c$ (sec).

7.5.2 Stage - Storage Curves Development

The storage volume for natural ponds in irregular terrain is usually developed for detention pond using a topographic map and the double-end area. The double-end area formula is expressed as:

$$V_{1,2} = \left[\frac{A_1 + A_2}{2} \right] h \quad (7.2)$$

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); and
- h = Change in elevation between points 1 and 2 (m).

The storage volume for excavated ponds with regular geometric shape (usually pyramid), the frustum indicates storage between the layers, is shown in Figure 7.7 and is expressed as:

$$V_{1,2} = \frac{h[A_1 + (A_1A_2)^{0.5} + A_2]}{3} \quad (7.3)$$

where,

- V = Volume of frustum of a pyramid (m^3);
- A_1 = Surface area at elevation 1 (m^2);
- A_2 = Surface area at elevation 2 (m^2); and
- h = Change in elevation between points 1 and 2 (m).

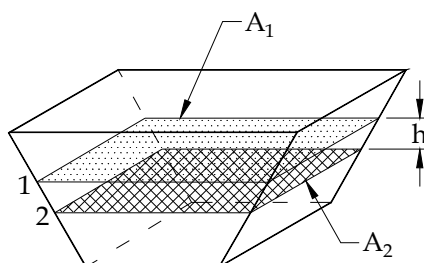


Figure 7.7: Frustum of a Pyramid

7.5.3 Stage – Discharge Curves

A stage-discharge (performance) curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility will have both a primary and an emergency outlet. The primary outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway.

The structure for the primary outlet will typically consist of a pipe culvert, weir, orifice, or other appropriate hydraulic control device. Multiple outlet control devices are often used to provide discharge controls for multiple frequency storms.

Development of a combined stage-discharge curve requires consideration of the discharge rating relationships for each component of the outlet structure. The design relationships for typical outlet controls are presented in Chapter 2.

7.5.4 Sizing Steps

A general procedure for sizing a detention pond, shown diagrammatically in Figures 7.8 and 7.9, is described as follows:

Step 1: Determine design storm criteria for the pond

Select the minor and major design storm ARI for the pond appropriate for the type of development in the catchment in accordance with Table 1.1, Chapter 1. Select the secondary outlet design storm ARI and the amount of bypass flow that will not be routed through the pond.

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

Step 2: Determine the pond outflow limits

For each design storm ARI, the pond 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 to be determined by the Time-Area hydrograph estimation technique.

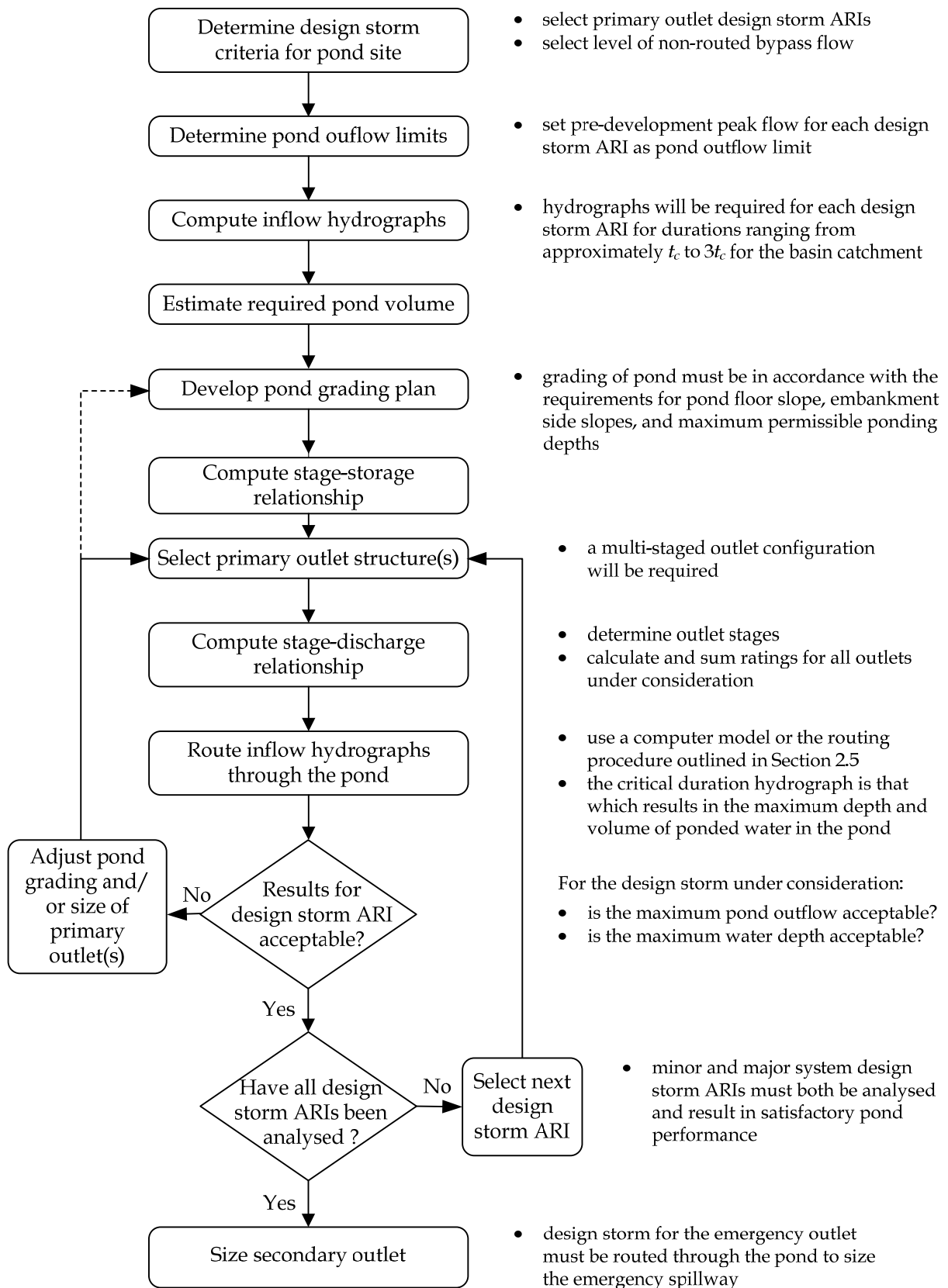


Figure 7.8: Detention Pond Sizing Procedure for Volume and Primary Outlets (DID, 2000)

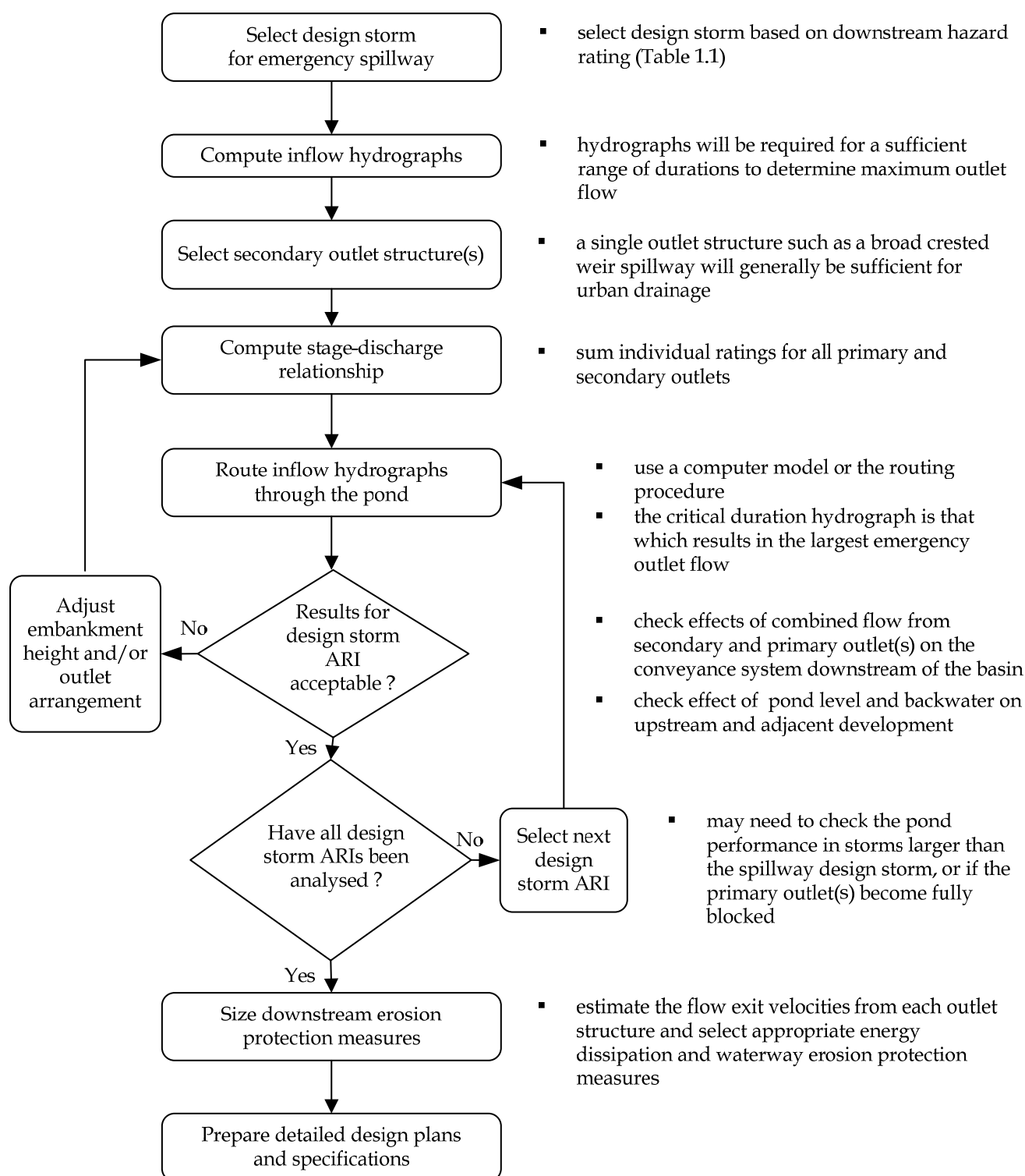


Figure 7.9: Detention Pond Sizing Procedure for Secondary Outlet (DID, 2000)

Step 3: Compute the pond inflow hydrographs

For each design storm ARI, inflow hydrographs for a range of storm durations will need to be routed through the pond to determine the critical duration that produces the greatest storage and water level within the pond for a particular pond grading and outlet configuration. The pond 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 pond 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 pond inflow hydrographs for the major system design ARI and the required outflow rate as shown in Figure 7.8. 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 over a range of durations should be examined and the largest estimated volume selected.

Step 5: Develop a pond 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 pond floors.

Step 6: Compute the stage-storage relationship

The stage-storage relationship can be defined from the pond geometry using the double-end area method. The maximum stage selected should extend above the top of the pond 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 pond 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 pond, 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 pond so that it will not be exceeded in the routing calculations in the following step. Step 3: Compute the pond inflow hydrographs

For each design storm ARI, inflow hydrographs for a range of storm durations will need to be routed through the pond to determine the critical duration that produces the greatest storage and water level within the pond

for a particular pond grading and outlet configuration. The pond 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 pond 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 pond inflow hydrographs for the major system design ARI and the required outflow rate as shown in Figure 7.8. 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 over a range of durations should be examined and the largest estimated volume selected.

Step 5: Develop a pond 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 pond floors.

Step 6: Compute the stage-storage relationship

The stage-storage relationship can be defined from the pond geometry using the double-end area method. The maximum stage selected should extend above the top of the pond 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 pond 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 pond, 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 pond so that it will not be exceeded in the routing calculations in the following step.

(iii) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond 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 five (5) minute is recommended.

(iv) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond 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 five (5) minute is recommended.

(iv) Check if results are acceptable

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

Step 8 Size the major design storm primary outlet

(vi) 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 pond water level estimated for the minor design storm.

(vii) 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.

(viii) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond outflow and water level.

(ix) Check if results are acceptable

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

Note: if the pond 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 pond water level for setting the major outlet invert level.

Step 9 Size the secondary outlet arrangement

Once a pond 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 pond 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 pond outlets (i.e. the secondary outlet plus the minor and major system outlets)

(iii) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond 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 pond 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 pond'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 pond primary outlet(s) occurs.

Step 11 Prepare design drawings and specifications

When the pond 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.

REFERENCES

1. Australian National Committee on Large Dams – ANCOLD (1986). *Guidelines for Operation Maintenance and Surveillance of Dams*. Australia.
2. Department of Irrigation and Drainage – DID (2000). *Urban Stormwater Management Manual for Malaysia*. PNMB, Government of Malaysia.
3. Federal Emergency Management Agency – FEMA (1987). *Dam Safety: An Owner's Guidance Manual*. FEMA, USA.
4. Golzé, A.R. (1977). *Handbook of Dam Engineering*. New York: Van Nostrand Reinhold.
5. Knox County (2008). *Knox County Tennessee Stormwater Management Manual*. Volume 2 (Technical Guidance), Tennessee, USA.
6. United States Bureau of Reclamation – USBR (1987). *Design of Small Dams*. 3rd Edition. Washington DC, USA.
7. United States Bureau of Reclamation – USBR (1992). *Safety Evaluation of Existing Dams (SEED Manual)*. Washington DC, USA.

APPENDIX 7.A EXAMPLE - DETENTION POND DESIGN

Problem:

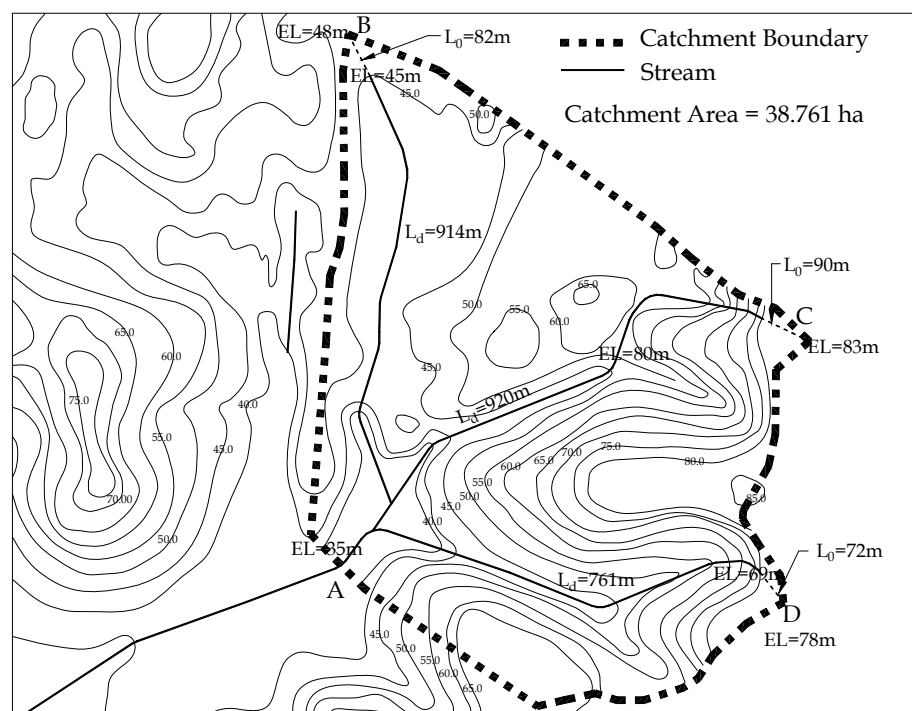
A catchment located in upstream of Sungai Buloh, Selangor, with area of 38.761 ha, is proposed to be developed for residential consisting Bungalow dwellings. A wet detention pond is to be incorporated to control the increased runoff resulting from the development. Flows will be directed to the pond via grassed channels built on existing streams. The existing and proposed triangular channel with slope = 1:1. From the site visit the depth of the channel is 0.3 m and width = 0.6 m, while the depth of the proposed channel is 0.42 m and width = 0.84 m. Design the pond and its outlet facilities with initial water level set at 31.00 m LSD and initial water depth of at least 1.0 m. The catchment is as shown in Figure 7.A1. The pond is required to be designed for 50 year ARI with primary outlets in the riser to control 5 and 50 year ARIs. The secondary outlet, a broad crested weir spillway, is to be provided to cater for the 100 year ARI event. The followings are data for the catchment and the streams/channels:

Pre-development catchment and stream properties

Reach	Overland			Natural Stream			
	L_o (m)	n^*	S_o (%)	L_d (m)	n	S_d (m/m)	R (m)
AB	82.0	0.045	3.66	914.0	0.05	0.011	0.11
AC	90.0	0.045	3.33	920.0	0.05	0.049	0.11
AD	72.0	0.045	4.17	761.0	0.05	0.053	0.11

Post-development catchment and channel properties

Reach	Overland			Grass Channel			
	L_o (m)	n^*	S_o (%)	L_d (m)	n	S_d (m/m)	R (m)
AB	82.0	0.035	3.66	914.0	0.035	0.009	0.15
AC	90.0	0.035	16.67	920.0	0.035	0.024	0.15
AD	72.0	0.035	13.89	761.0	0.035	0.026	0.15

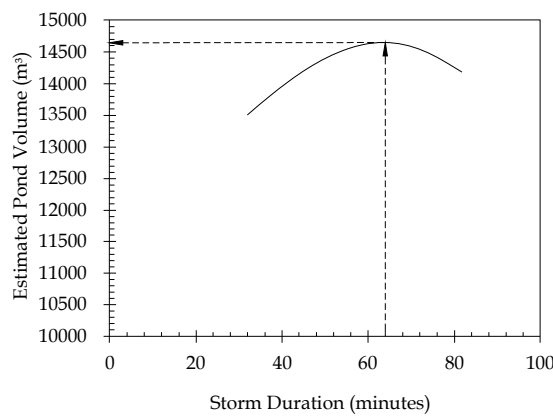
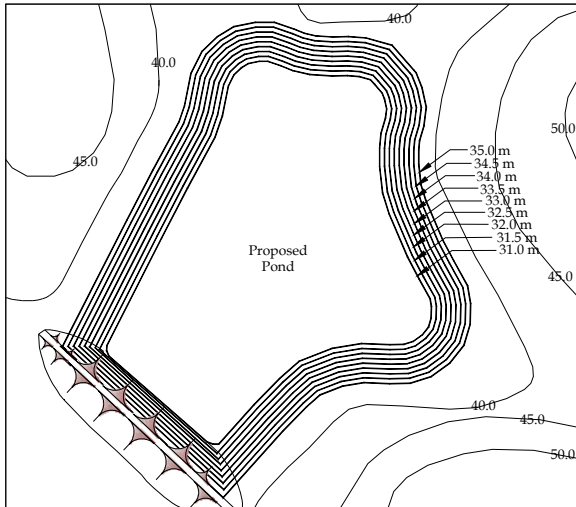


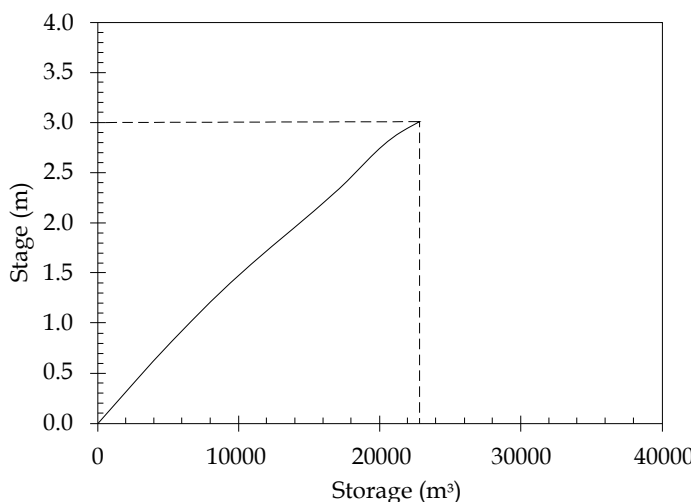
Solution:

Reference	Calculation	Output
	Step 1 : Setting the design storm criteria for the pond	
	As specified in Section 1.2.1, the runoff quantity control criterion for new development is to reduce post-development peak flows to less than or equal to the pre-development values. The majority of the proposed development upstream of the pond will be medium density housing lots. The minor and major design storms for primary outlet are 5 and 50 year ARI, while the secondary outlet is 100 year ARI (in accordance with Table 1.1, Section 7.2.3 and 7.2.4).	
	Step 2 : Compute the pond inflow hydrograph	= 16.1 min
	<u>Compute the Pre-development Time of Concentration, t_c</u>	
	<u>Reach AB:</u>	
	$t_o = \frac{107n^* L_o^{1/3}}{S_o^{1/5}}$	= 32.5 min
		= 48.6 min
Table 2.1	$L_o = 82.0 \text{ m}, \quad S_o = 3.66 \%, \quad n^* = 0.045, \quad t_o$	
Table 2.2	$t_d = \frac{n L_d}{60 R^{2/3} S_d^{1/2}}$	= 17 min
Table 2.1	$L_d = 914 \text{ m}, \quad S_d = 0.011 \%, \quad n = 0.050, \quad R = 0.11 \quad t_d$	= 15.5 min
Table 2.3	$t_c = t_o + t_d$	= 32.4 min
	<u>Similarly for Reach AC:</u>	
	$L_o = 90 \text{ m}, \quad S_o = 3.33 \%, \quad n^* = 0.045, \quad t_o$	= 15.1 min
	$L_d = 920 \text{ m}, \quad S_d = 0.049 \%, \quad n = 0.050, \quad R = 0.11 \quad t_d$	= 12.3 min
	$t_c = t_o + t_d$	= 27.4 min
	<u>Similarly for Reach AD:</u>	
	$L_o = 72.0 \text{ m}, \quad S_o = 4.17 \%, \quad n^* = 0.045, \quad t_o$	
	$L_d = 761 \text{ m}, \quad S_d = 0.053 \%, \quad n = 0.050, \quad R = 0.11 \quad t_d$	= 12.6 min
	$t_c = t_o + t_d$	= 20.2 min
	Thus, the pre-development t_c is 48.6 min (the longest).	= 32.7 min
	<u>Compute the Post-development Time of Concentration, t_c</u>	
	<u>Reach AB:</u>	
	$L_o = 82 \text{ m}, \quad S_o = 3.66 \%, \quad n^* = 0.035, \quad t_o$	= 13.2 min
		= 12.2 min
	$L_d = 914 \text{ m}, \quad S_d = 0.009 \%, \quad n = 0.035, \quad R = 0.15 \quad t_d$	

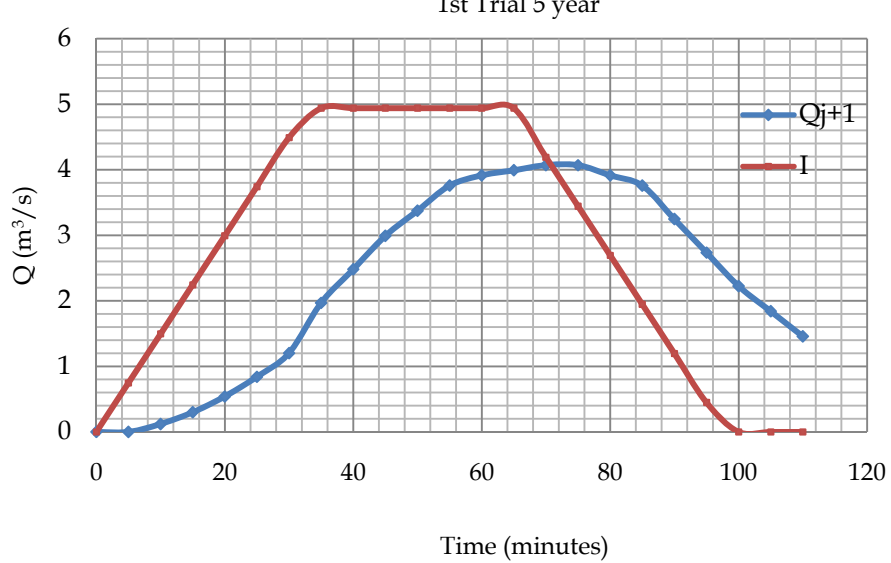
Reference	Calculation	Output																																
Table 2.B1.4	$t_c = t_o + t_d$	= 25.3 min																																
	<u>Similarly for Reach AC:</u>																																	
	$L_o = 90 \text{ m}, \quad S_o = 3.33 \%, \quad n^* = 0.035, \quad t_o$	= 11.7 min																																
	$L_d = 920 \text{ m}, \quad S_d = 0.024 \%, \quad n = 0.035, \quad R = 0.15 \quad t_d$	= 9.7 min																																
	$t_c = t_o + t_d$	= 21.4 min																																
	<u>Similarly for Reach AD:</u>																																	
	$L_o = 72 \text{ m}, \quad S_o = 4.17 \%, \quad n^* = 0.035, \quad t_o$																																	
	$L_d = 761 \text{ m}, \quad S_d = 0.026 \%, \quad n = 0.035 \quad R = 0.15 \quad t_d$																																	
	$t_c = t_o + t_d$																																	
	Thus, the post-development t_c is 32.7 min, which is 15.9 minute shorter than the the pre-development t_c .																																	
Equation 2.2	<u>Compute the rainfall intensity, i</u>																																	
	The nearest rainfall station is Station ID 3216004, SK Jenis Keb. Kepong. Referring to Table 2.B1, the fitting constants for the IDF Empirical Equation of this station are:																																	
	<table><tr><td>λ</td><td>κ</td><td>θ</td><td>η</td></tr><tr><td>73.602</td><td>0.164</td><td>0.330</td><td>0.874</td></tr></table>		λ	κ	θ	η	73.602	0.164	0.330	0.874																								
	λ	κ	θ	η																														
	73.602	0.164	0.330	0.874																														
	The rainfall intensity for various durations and ARIs are given in Table 7.A2 and Table 7.A3 using Equation 2.2.																																	
	Step 3: Determine the pond outflow limits																																	
	Table 7.A1: Runoff Coefficient																																	
	<table><tr><td>Condition</td><td>C_{post}</td><td>C_{pre}</td></tr><tr><td>ARI < 10 yr</td><td>0.65</td><td>0.30</td></tr><tr><td>ARI > 10 yr</td><td>0.70</td><td>0.40</td></tr></table>		Condition	C_{post}	C_{pre}	ARI < 10 yr	0.65	0.30	ARI > 10 yr	0.70	0.40																							
	Condition	C_{post}	C_{pre}																															
ARI < 10 yr	0.65	0.30																																
ARI > 10 yr	0.70	0.40																																
<u>Compute the pre and post-development discharge</u>																																		
Table 7.A2: Calculated Pre-development Discharge																																		
Table 2.5	<table><tr><td>ARI (year)</td><td>Storm duration, d</td><td>d (min)</td><td>i (mm/hr)</td><td>C_{pre}</td><td>A (ha)</td><td>Q_{pre} (m³/s)</td></tr><tr><td>5</td><td>t_c</td><td>48.6</td><td>85.5</td><td>0.30</td><td>38.761</td><td>2.8</td></tr><tr><td>50</td><td>t_c</td><td>48.6</td><td>124.7</td><td>0.40</td><td>38.761</td><td>5.4</td></tr><tr><td>100</td><td>t_c</td><td>48.6</td><td>139.7</td><td>0.40</td><td>38.761</td><td>6.0</td></tr></table>						ARI (year)	Storm duration, d	d (min)	i (mm/hr)	C_{pre}	A (ha)	Q_{pre} (m ³ /s)	5	t_c	48.6	85.5	0.30	38.761	2.8	50	t_c	48.6	124.7	0.40	38.761	5.4	100	t_c	48.6	139.7	0.40	38.761	6.0
	ARI (year)	Storm duration, d	d (min)	i (mm/hr)	C_{pre}	A (ha)	Q_{pre} (m ³ /s)																											
	5	t_c	48.6	85.5	0.30	38.761	2.8																											
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100	t_c	48.6	139.7	0.40	38.761	6.0																												
Table 7.A3: Calculated Post-development Discharge																																		

Reference	Calculation							Output
Equaion 2.2 Equaion 2.3 								

Reference	Calculation	Output																																
	 Figure 7.A2: Storm Duration vs Estimated Pond Volume Curve From Figure 7.A2, the estimated pond volume required is 14630 m³. Step 5: Develop a pond grading plan The location and grading of the pond embankment and storage area is selected by trial and error so that at least the preliminary estimated volume of 14630 m³ is available in the pond to cater for the critical 50 year ARI design storm event.  Step 6: Compute stage-storage relationship Table 7.A4: Stage-Storage Relationship <table><tr><th>Stage (m)</th><th>Elevation (m)</th><th>Area (m²)</th><th>Storage Volume (m³)</th></tr><tr><td>0.00</td><td>31.00</td><td>6250</td><td>0</td></tr><tr><td>0.50</td><td>31.50</td><td>6435</td><td>3171</td></tr><tr><td>1.00</td><td>32.00</td><td>7275</td><td>6599</td></tr><tr><td>1.50</td><td>32.50</td><td>7360</td><td>10258</td></tr><tr><td>2.00</td><td>33.00</td><td>8245</td><td>14159</td></tr><tr><td>2.50</td><td>33.50</td><td>8365</td><td>18311</td></tr><tr><td>3.00</td><td>34.00</td><td>9278</td><td>22722</td></tr></table> <td></td>	Stage (m)	Elevation (m)	Area (m ²)	Storage Volume (m ³)	0.00	31.00	6250	0	0.50	31.50	6435	3171	1.00	32.00	7275	6599	1.50	32.50	7360	10258	2.00	33.00	8245	14159	2.50	33.50	8365	18311	3.00	34.00	9278	22722	
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3.00	34.00	9278	22722																															

Reference	Calculation	Output																																
	 Figure 7.A4: Stage – Storage Curve 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 pond outflow that is as close as practicable to the required flow limit of 2.8m³/s. This involves selecting an initial outlet arrangement and size, determining the stage-discharge relationship, and routing the pond inflow hydrographs through the pond 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. (1) Select trial outlet arrangement To provide flow reduction for the 5 year ARI post-development design storm, a rectangular orifice measuring 2.0m (width) x 1.0m (depth) was initially selected. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond. (2) Compute the stage-discharge relationship and storage indicator number Calculate the stage-discharge relationship and storage indicator number from 5yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve. Table 7.A5: Stage-Discharge Relation and Storage Indicator Numbers (5yr ARI 1st Trial) <table border="1"> <thead> <tr> <th>H (m)</th> <th>O (m³/s)</th> <th>S (m³)</th> <th>(S/Δt)+O/2 (m³/s)</th> </tr> </thead> <tbody> <tr><td>0.00</td><td>0.00</td><td>0</td><td>0.00</td></tr> <tr><td>0.50</td><td>1.20</td><td>3171</td><td>53.44</td></tr> <tr><td>1.00</td><td>3.76</td><td>6599</td><td>111.85</td></tr> <tr><td>1.50</td><td>5.32</td><td>10258</td><td>173.62</td></tr> <tr><td>2.00</td><td>6.51</td><td>14159</td><td>239.23</td></tr> <tr><td>2.50</td><td>7.52</td><td>18311</td><td>308.93</td></tr> <tr><td>3.00</td><td>8.40</td><td>22722</td><td>349.57</td></tr> </tbody> </table> $C_o=1.7$ (Weir Flow) and 0.6 (Orifice Flow), $A= 2.0\text{m}^2$, (Dimension=2.0m x 1.0m)	H (m)	O (m³/s)	S (m³)	(S/Δt)+O/2 (m³/s)	0.00	0.00	0	0.00	0.50	1.20	3171	53.44	1.00	3.76	6599	111.85	1.50	5.32	10258	173.62	2.00	6.51	14159	239.23	2.50	7.52	18311	308.93	3.00	8.40	22722	349.57	
H (m)	O (m³/s)	S (m³)	(S/Δt)+O/2 (m³/s)																															
0.00	0.00	0	0.00																															
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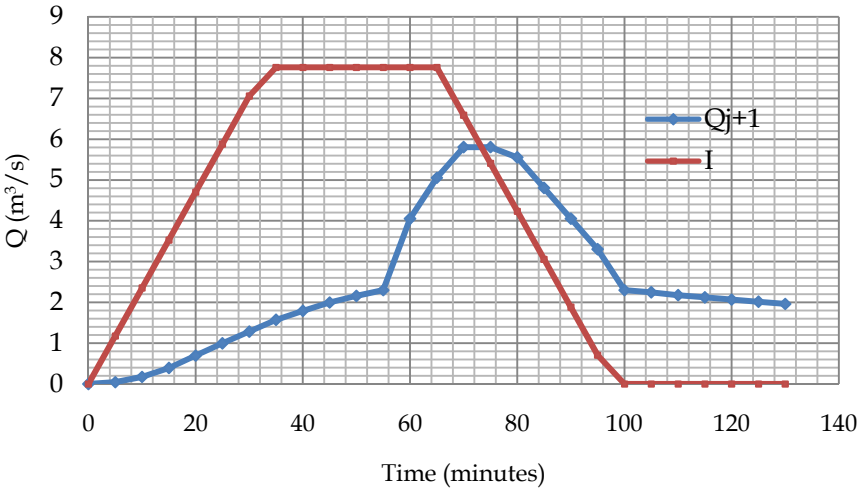
Reference	Calculation	Output																																																																																																																																																																																																																																																																			
	(3) Route the inflow hydrographs through the pond Using a routing time step of 1.0 minutes, the 5yr ARI orifice produced a maximum discharge of 4.07 m³/s which is NOT acceptable (Table 7.A6) as it is more than the 5 year ARI pond outflow limit of 2.8 m³/s. The maximum water elevation in the basin is 32.1 m, LSD. It is ok since it is less than the maximum allowable elevation (34.7m). However, the outlet needs to be resized since it exceeds the permissible discharge. Table 7.A6: Routing Table – 5 yr ARI (1st Trial) <table><tr><th>Time (min)</th><th>I (m³/s)</th><th>(I₁ + I₂)/2 (m³/s)</th><th>S₁/Δt + O₁/2 (m³/s)</th><th>O₁ (m³/s)</th><th>S₂/Δt + O₂/2 (m³/s)</th><th>O₂ (m³/s)</th></tr><tr><td>0</td><td>0.00</td><td>0.00</td><td>0</td><td></td><td>0</td><td>0</td></tr><tr><td>1</td><td>0.15</td><td>0.07</td><td>0.00</td><td>0.00</td><td>0.07</td><td>0.00</td></tr><tr><td>2</td><td>0.30</td><td>0.22</td><td>0.07</td><td>0.00</td><td>0.30</td><td>0.00</td></tr><tr><td>3</td><td>0.45</td><td>0.37</td><td>0.30</td><td>0.00</td><td>0.67</td><td>0.00</td></tr><tr><td>4</td><td>0.60</td><td>0.52</td><td>0.67</td><td>0.00</td><td>1.20</td><td>0.00</td></tr><tr><td>5</td><td>0.75</td><td>0.67</td><td>1.20</td><td>0.00</td><td>1.87</td><td>0.00</td></tr><tr><td>6</td><td>0.90</td><td>0.82</td><td>1.87</td><td>0.00</td><td>2.69</td><td>0.06</td></tr><tr><td>7</td><td>1.05</td><td>0.97</td><td>2.69</td><td>0.06</td><td>3.61</td><td>0.06</td></tr><tr><td>8</td><td>1.20</td><td>1.12</td><td>3.61</td><td>0.06</td><td>4.67</td><td>0.06</td></tr><tr><td>9</td><td>1.35</td><td>1.27</td><td>4.67</td><td>0.06</td><td>5.88</td><td>0.12</td></tr><tr><td>10</td><td>1.50</td><td>1.42</td><td>5.88</td><td>0.12</td><td>7.18</td><td>0.12</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr><tr><td>60</td><td>4.94</td><td>4.94</td><td>116.96</td><td>3.84</td><td>118.06</td><td>3.91</td></tr><tr><td>61</td><td>4.94</td><td>4.94</td><td>118.06</td><td>3.91</td><td>119.09</td><td>3.91</td></tr><tr><td>62</td><td>4.94</td><td>4.94</td><td>119.09</td><td>3.91</td><td>120.12</td><td>3.91</td></tr><tr><td>63</td><td>4.94</td><td>4.94</td><td>120.12</td><td>3.91</td><td>121.14</td><td>3.99</td></tr><tr><td>64</td><td>4.94</td><td>4.94</td><td>121.14</td><td>3.99</td><td>122.09</td><td>3.99</td></tr><tr><td>65</td><td>4.94</td><td>4.94</td><td>122.09</td><td>3.99</td><td>123.04</td><td>3.99</td></tr><tr><td>66</td><td>4.79</td><td>4.87</td><td>123.04</td><td>3.99</td><td>123.91</td><td>3.99</td></tr><tr><td>67</td><td>4.64</td><td>4.72</td><td>123.91</td><td>3.99</td><td>124.63</td><td>4.07</td></tr><tr><td>68</td><td>4.49</td><td>4.57</td><td>124.63</td><td>4.07</td><td>125.13</td><td>4.07</td></tr><tr><td>69</td><td>4.34</td><td>4.42</td><td>125.13</td><td>4.07</td><td>125.48</td><td>4.07</td></tr><tr><td>70</td><td>4.19</td><td>4.27</td><td>125.48</td><td>4.07</td><td>125.67</td><td>4.07</td></tr><tr><td>71</td><td>4.04</td><td>4.12</td><td>125.67</td><td>4.07</td><td>125.72</td><td>4.07</td></tr><tr><td>72</td><td>3.89</td><td>3.97</td><td>125.72</td><td>4.07</td><td>125.62</td><td>4.07</td></tr><tr><td>73</td><td>3.74</td><td>3.82</td><td>125.62</td><td>4.07</td><td>125.36</td><td>4.07</td></tr><tr><td>74</td><td>3.59</td><td>3.67</td><td>125.36</td><td>4.07</td><td>124.96</td><td>4.07</td></tr><tr><td>75</td><td>3.44</td><td>3.52</td><td>124.96</td><td>4.07</td><td>124.41</td><td>4.07</td></tr><tr><td>76</td><td>3.29</td><td>3.37</td><td>124.41</td><td>4.07</td><td>123.71</td><td>3.99</td></tr><tr><td>77</td><td>3.14</td><td>3.22</td><td>123.71</td><td>3.99</td><td>122.93</td><td>3.99</td></tr><tr><td>78</td><td>2.99</td><td>3.07</td><td>122.93</td><td>3.99</td><td>122.01</td><td>3.99</td></tr><tr><td>79</td><td>2.84</td><td>2.92</td><td>122.01</td><td>3.99</td><td>120.94</td><td>3.91</td></tr><tr><td>80</td><td>2.69</td><td>2.77</td><td>120.94</td><td>3.91</td><td>119.79</td><td>3.91</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr></table>	Time (min)	I (m³/s)	(I ₁ + I ₂)/2 (m³/s)	S ₁ /Δt + O ₁ /2 (m³/s)	O ₁ (m³/s)	S ₂ /Δt + O ₂ /2 (m³/s)	O ₂ (m³/s)	0	0.00	0.00	0		0	0	1	0.15	0.07	0.00	0.00	0.07	0.00	2	0.30	0.22	0.07	0.00	0.30	0.00	3	0.45	0.37	0.30	0.00	0.67	0.00	4	0.60	0.52	0.67	0.00	1.20	0.00	5	0.75	0.67	1.20	0.00	1.87	0.00	6	0.90	0.82	1.87	0.00	2.69	0.06	7	1.05	0.97	2.69	0.06	3.61	0.06	8	1.20	1.12	3.61	0.06	4.67	0.06	9	1.35	1.27	4.67	0.06	5.88	0.12	10	1.50	1.42	5.88	0.12	7.18	0.12	Continued							...	...	...	...	...	...	...	60	4.94	4.94	116.96	3.84	118.06	3.91	61	4.94	4.94	118.06	3.91	119.09	3.91	62	4.94	4.94	119.09	3.91	120.12	3.91	63	4.94	4.94	120.12	3.91	121.14	3.99	64	4.94	4.94	121.14	3.99	122.09	3.99	65	4.94	4.94	122.09	3.99	123.04	3.99	66	4.79	4.87	123.04	3.99	123.91	3.99	67	4.64	4.72	123.91	3.99	124.63	4.07	68	4.49	4.57	124.63	4.07	125.13	4.07	69	4.34	4.42	125.13	4.07	125.48	4.07	70	4.19	4.27	125.48	4.07	125.67	4.07	71	4.04	4.12	125.67	4.07	125.72	4.07	72	3.89	3.97	125.72	4.07	125.62	4.07	73	3.74	3.82	125.62	4.07	125.36	4.07	74	3.59	3.67	125.36	4.07	124.96	4.07	75	3.44	3.52	124.96	4.07	124.41	4.07	76	3.29	3.37	124.41	4.07	123.71	3.99	77	3.14	3.22	123.71	3.99	122.93	3.99	78	2.99	3.07	122.93	3.99	122.01	3.99	79	2.84	2.92	122.01	3.99	120.94	3.91	80	2.69	2.77	120.94	3.91	119.79	3.91	Continued							...	...	...	...	...	...	...	
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Reference	Calculation	Output																																
	1st Trial 5 year  Time (minutes) Figure 7.A5: Inflow and Outflow Hydrograph of 5yr ARI Storm Event (1st Trial) (4) Re-Select trial outlet arrangement To provide flow reduction for the 5 year ARI post-development design storm to a pre-development level, a smaller size outlet is selected. A 0.5 x 0.5 m rectangular orifice was selected. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond. (5) Re-compute the stage-discharge relationship and storage indicator number Calculate the stage-discharge relationship and storage indicator number from 5yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve. Table 7.A7: Stage-Discharge Relation and Storage Indicator Numbers (5yr ARI 2nd Trial) <table border="1"> <thead> <tr> <th>H (m)</th> <th>O (m³/s)</th> <th>S (m³)</th> <th>(S/Δt)+O/2 (m³/s)</th> </tr> </thead> <tbody> <tr><td>0.00</td><td>0.00</td><td>0</td><td>0.00</td></tr> <tr><td>0.50</td><td>0.87</td><td>3171</td><td>53.28</td></tr> <tr><td>1.00</td><td>1.51</td><td>6599</td><td>110.73</td></tr> <tr><td>1.50</td><td>1.94</td><td>10258</td><td>171.94</td></tr> <tr><td>2.00</td><td>2.30</td><td>14159</td><td>237.12</td></tr> <tr><td>2.50</td><td>2.61</td><td>18311</td><td>306.48</td></tr> <tr><td>3.00</td><td>2.88</td><td>22722</td><td>380.14</td></tr> </tbody> </table> $C_o=1.7$ (Weir Flow) and 0.6 (Orifice Flow), $A= 0.25\text{m}^2$, (Dimension=0.5m x 0.5m) (6) Re-route the inflow hydrographs through the pond Using a routing time step of 1.0 minutes, the 5yr ARI orifice produced a maximum	H (m)	O (m ³ /s)	S (m ³)	(S/Δt)+O/2 (m ³ /s)	0.00	0.00	0	0.00	0.50	0.87	3171	53.28	1.00	1.51	6599	110.73	1.50	1.94	10258	171.94	2.00	2.30	14159	237.12	2.50	2.61	18311	306.48	3.00	2.88	22722	380.14	
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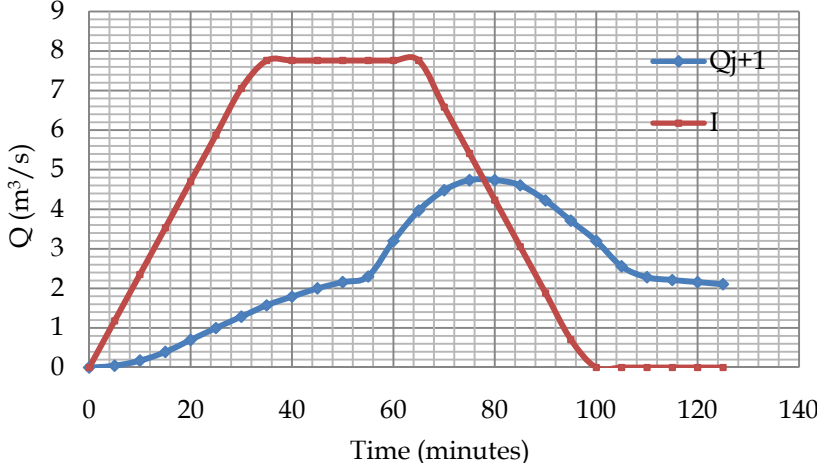
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Table 7.A8: Re-routing Table – 5 yr ARI (2nd Trial) <table><tr><th>Time (min)</th><th>I (m³/s)</th><th>(I₁ + I₂)/2 (m³/s)</th><th>S₁/Δt + O₁/2 (m³/s)</th><th>O₁ (m³/s)</th><th>S₂/Δt + O₂/2 (m³/s)</th><th>O₂ (m³/s)</th></tr><tr><td>0</td><td>0.00</td><td>0.00</td><td>0.00</td><td></td><td>0.00</td><td>0.00</td></tr><tr><td>1</td><td>0.15</td><td>0.07</td><td>0.00</td><td>0.00</td><td>0.07</td><td>0.00</td></tr><tr><td>2</td><td>0.30</td><td>0.22</td><td>0.07</td><td>0.00</td><td>0.30</td><td>0.00</td></tr><tr><td>3</td><td>0.45</td><td>0.37</td><td>0.30</td><td>0.00</td><td>0.67</td><td>0.00</td></tr><tr><td>4</td><td>0.60</td><td>0.52</td><td>0.67</td><td>0.00</td><td>1.20</td><td>0.00</td></tr><tr><td>5</td><td>0.75</td><td>0.67</td><td>1.20</td><td>0.00</td><td>1.87</td><td>0.00</td></tr><tr><td>6</td><td>0.90</td><td>0.82</td><td>1.87</td><td>0.00</td><td>2.69</td><td>0.04</td></tr><tr><td>7</td><td>1.05</td><td>0.97</td><td>2.69</td><td>0.04</td><td>3.62</td><td>0.04</td></tr><tr><td>8</td><td>1.20</td><td>1.12</td><td>3.62</td><td>0.04</td><td>4.70</td><td>0.04</td></tr><tr><td>9</td><td>1.35</td><td>1.27</td><td>4.70</td><td>0.04</td><td>5.93</td><td>0.09</td></tr><tr><td>10</td><td>1.50</td><td>1.42</td><td>5.93</td><td>0.09</td><td>7.27</td><td>0.09</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr><tr><td>80</td><td>2.69</td><td>5.54</td><td>322.51</td><td>2.70</td><td>325.35</td><td>2.73</td></tr><tr><td>81</td><td>2.54</td><td>5.24</td><td>325.35</td><td>2.73</td><td>327.85</td><td>2.75</td></tr><tr><td>82</td><td>2.40</td><td>4.94</td><td>327.85</td><td>2.75</td><td>330.05</td><td>2.76</td></tr><tr><td>83</td><td>2.25</td><td>4.64</td><td>330.05</td><td>2.76</td><td>331.93</td><td>2.77</td></tr><tr><td>84</td><td>2.10</td><td>4.34</td><td>331.93</td><td>2.77</td><td>333.50</td><td>2.79</td></tr><tr><td>85</td><td>1.95</td><td>4.04</td><td>333.50</td><td>2.79</td><td>334.75</td><td>2.80</td></tr><tr><td>86</td><td>1.80</td><td>3.74</td><td>334.75</td><td>2.80</td><td>335.69</td><td>2.80</td></tr><tr><td>87</td><td>1.65</td><td>3.44</td><td>335.69</td><td>2.80</td><td>336.34</td><td>2.80</td></tr><tr><td>88</td><td>1.50</td><td>3.14</td><td>336.34</td><td>2.80</td><td>336.68</td><td>2.80</td></tr><tr><td>89</td><td>1.35</td><td>2.84</td><td>336.68</td><td>2.80</td><td>336.72</td><td>2.80</td></tr><tr><td>90</td><td>1.20</td><td>2.54</td><td>336.72</td><td>2.80</td><td>336.47</td><td>2.80</td></tr><tr><td>91</td><td>1.05</td><td>2.25</td><td>336.47</td><td>2.80</td><td>335.91</td><td>2.80</td></tr><tr><td>92</td><td>0.90</td><td>1.95</td><td>335.91</td><td>2.80</td><td>335.06</td><td>2.80</td></tr><tr><td>93</td><td>0.75</td><td>1.65</td><td>335.06</td><td>2.80</td><td>333.90</td><td>2.79</td></tr><tr><td>94</td><td>0.60</td><td>1.35</td><td>333.90</td><td>2.79</td><td>332.47</td><td>2.77</td></tr><tr><td>95</td><td>0.45</td><td>1.05</td><td>332.47</td><td>2.77</td><td>330.74</td><td>2.77</td></tr><tr><td>96</td><td>0.30</td><td>0.75</td><td>330.74</td><td>2.77</td><td>328.72</td><td>2.76</td></tr><tr><td>97</td><td>0.15</td><td>0.45</td><td>328.72</td><td>2.76</td><td>326.41</td><td>2.73</td></tr><tr><td>98</td><td>0.00</td><td>0.15</td><td>326.41</td><td>2.73</td><td>323.82</td><td>2.72</td></tr><tr><td>99</td><td>0.00</td><td>0.00</td><td>323.82</td><td>2.72</td><td>321.11</td><td>2.70</td></tr><tr><td>100</td><td>0.00</td><td>0.00</td><td>321.11</td><td>2.70</td><td>318.40</td><td>2.68</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr></table>	Time (min)	I (m³/s)	(I ₁ + I ₂)/2 (m³/s)	S ₁ /Δt + O ₁ /2 (m³/s)	O ₁ (m³/s)	S ₂ /Δt + O ₂ /2 (m³/s)	O ₂ (m³/s)	0	0.00	0.00	0.00		0.00	0.00	1	0.15	0.07	0.00	0.00	0.07	0.00	2	0.30	0.22	0.07	0.00	0.30	0.00	3	0.45	0.37	0.30	0.00	0.67	0.00	4	0.60	0.52	0.67	0.00	1.20	0.00	5	0.75	0.67	1.20	0.00	1.87	0.00	6	0.90	0.82	1.87	0.00	2.69	0.04	7	1.05	0.97	2.69	0.04	3.62	0.04	8	1.20	1.12	3.62	0.04	4.70	0.04	9	1.35	1.27	4.70	0.04	5.93	0.09	10	1.50	1.42	5.93	0.09	7.27	0.09	Continued							...	...	...	...	...	...	...	80	2.69	5.54	322.51	2.70	325.35	2.73	81	2.54	5.24	325.35	2.73	327.85	2.75	82	2.40	4.94	327.85	2.75	330.05	2.76	83	2.25	4.64	330.05	2.76	331.93	2.77	84	2.10	4.34	331.93	2.77	333.50	2.79	85	1.95	4.04	333.50	2.79	334.75	2.80	86	1.80	3.74	334.75	2.80	335.69	2.80	87	1.65	3.44	335.69	2.80	336.34	2.80	88	1.50	3.14	336.34	2.80	336.68	2.80	89	1.35	2.84	336.68	2.80	336.72	2.80	90	1.20	2.54	336.72	2.80	336.47	2.80	91	1.05	2.25	336.47	2.80	335.91	2.80	92	0.90	1.95	335.91	2.80	335.06	2.80	93	0.75	1.65	335.06	2.80	333.90	2.79	94	0.60	1.35	333.90	2.79	332.47	2.77	95	0.45	1.05	332.47	2.77	330.74	2.77	96	0.30	0.75	330.74	2.77	328.72	2.76	97	0.15	0.45	328.72	2.76	326.41	2.73	98	0.00	0.15	326.41	2.73	323.82	2.72	99	0.00	0.00	323.82	2.72	321.11	2.70	100	0.00	0.00	321.11	2.70	318.40	2.68	Continued							...	...	...	...	...	...	...	
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Reference	Calculation	Output																																				
	2nd Trial 5 year Figure 7.A6: Inflow and Outflow Hydrograph of 5yr ARI Storm Event (2nd Trial) Step 8: Size the major design storm primary outlet The procedure for sizing the major design storm primary outlet is the same as the minor design storm outlet. (iii) Select trial outlet arrangement To provide flow reduction for the 50 year ARI post-development design storm, an additional 2.0 m x 2.0 m orifice was initially selected with an invert level of 33 m, LSD that corresponds to a stage of 2.0 m. (iv) Compute the stage-discharge relationship The stage-discharge relationship is the summation of the 5 year ARI and the 50 year ARI orifice capacities (Table 7.A9). Table 7.A9: Combined Stage-Discharge – 5 yr and 50 yr ARI Orifice <table> <tr> <th>H</th> <th>5 yr ARI Orifice</th> <th>50 yr ARI weir</th> <th>Total Discharge</th> </tr> <tr> <th>(m)</th> <th>(m³/s)</th> <th>(m³/s)</th> <th>(m³/s)</th> </tr> <tr> <td>0</td> <td>0.00</td> <td>0</td> <td>0</td> </tr> <tr> <td>0.5</td> <td>0.87</td> <td>0</td> <td>0.87</td> </tr> <tr> <td>1</td> <td>1.51</td> <td>0</td> <td>1.51</td> </tr> <tr> <td>1.5</td> <td>1.94</td> <td>0</td> <td>1.94</td> </tr> <tr> <td>2</td> <td>2.30</td> <td>0</td> <td>2.30</td> </tr> <tr> <td>2.5</td> <td>2.61</td> <td>4.70</td> <td>7.31</td> </tr> <tr> <td>3</td> <td>2.88</td> <td>6.64</td> <td>9.53</td> </tr> </table> $C_o=1.7$ (Weir Flow) and 0.6 (Orifice Flow), $A=4\text{ m}^2$ (Dimension=2.0m x 2.0m)	H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge	(m)	(m ³ /s)	(m ³ /s)	(m ³ /s)	0	0.00	0	0	0.5	0.87	0	0.87	1	1.51	0	1.51	1.5	1.94	0	1.94	2	2.30	0	2.30	2.5	2.61	4.70	7.31	3	2.88	6.64	9.53	
H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge																																			
(m)	(m ³ /s)	(m ³ /s)	(m ³ /s)																																			
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3	2.88	6.64	9.53																																			

Reference	Calculation	Output																																																																																																																																																																																																																						
	Table 7.A10: Combined Storage Indicator Numbers – 5 yr and 50yr ARI <table><tr><th>H (m)</th><th>Q (m3/s)</th><th>S (m3)</th><th>(S/dt)+Q/2</th></tr><tr><td>0</td><td>0</td><td>0</td><td>0.00</td></tr><tr><td>1</td><td>1</td><td>3171</td><td>53.28</td></tr><tr><td>1</td><td>2</td><td>6599</td><td>110.73</td></tr><tr><td>2</td><td>2</td><td>10258</td><td>171.94</td></tr><tr><td>2</td><td>2</td><td>14159</td><td>237.12</td></tr><tr><td>3</td><td>7</td><td>18311</td><td>308.83</td></tr><tr><td>3</td><td>10</td><td>22722</td><td>383.46</td></tr></table> (v) Route the inflow hydrographs through the pond Using a routing time step of 2.5 minutes, the 5 year and 50 year ARI orifices produced a maximum discharge of 5.80 m³/s. This is NOT acceptable (Table 7.A11) as it is more than the 50 year ARI pond outflow limit of 5.4 m³/s. The maximum water elevation in the pond is 33.35m, LSD that corresponds to a maximum water depth of 2.35m. Table 7.A11: Routing Table – 50 yr ARI (1st Trial) <table><tr><th>Time (min)</th><th>I (m³/s)</th><th>(I₁ + I₂)/2 (m³/s)</th><th>S₁/Δt + O₁/2 (m³/s)</th><th>O₁ (m³/s)</th><th>S₂/Δt + O₂/2 (m³/s)</th><th>O₂ (m³/s)</th></tr><tr><td>0</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td></tr><tr><td>1</td><td>0.24</td><td>0.12</td><td>0.00</td><td>0.00</td><td>0.12</td><td>0.00</td></tr><tr><td>2</td><td>0.47</td><td>0.35</td><td>0.12</td><td>0.00</td><td>0.47</td><td>0.00</td></tr><tr><td>3</td><td>0.71</td><td>0.59</td><td>0.47</td><td>0.00</td><td>1.06</td><td>0.00</td></tr><tr><td>4</td><td>0.94</td><td>0.82</td><td>1.06</td><td>0.00</td><td>1.88</td><td>0.00</td></tr><tr><td>5</td><td>1.18</td><td>1.06</td><td>1.88</td><td>0.00</td><td>2.94</td><td>0.04</td></tr><tr><td>6</td><td>1.41</td><td>1.29</td><td>2.94</td><td>0.04</td><td>4.19</td><td>0.04</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr><tr><td>65</td><td>7.76</td><td>7.76</td><td>276.09</td><td>4.86</td><td>278.99</td><td>5.1</td></tr><tr><td>66</td><td>7.52</td><td>7.64</td><td>278.99</td><td>5.11</td><td>281.52</td><td>5.4</td></tr><tr><td>67</td><td>7.29</td><td>7.41</td><td>281.52</td><td>5.37</td><td>283.55</td><td>5.4</td></tr><tr><td>68</td><td>7.05</td><td>7.17</td><td>283.55</td><td>5.37</td><td>285.36</td><td>5.6</td></tr><tr><td>69</td><td>6.82</td><td>6.94</td><td>285.36</td><td>5.63</td><td>286.67</td><td>5.6</td></tr><tr><td>70</td><td>6.58</td><td>6.70</td><td>286.67</td><td>5.63</td><td>287.74</td><td>5.8</td></tr><tr><td>71</td><td>6.35</td><td>6.47</td><td>287.74</td><td>5.88</td><td>288.33</td><td>5.8</td></tr><tr><td>72</td><td>6.11</td><td>6.23</td><td>288.33</td><td>5.88</td><td>288.68</td><td>5.8</td></tr><tr><td>73</td><td>5.88</td><td>6.00</td><td>288.68</td><td>5.88</td><td>288.80</td><td>5.8</td></tr><tr><td>74</td><td>5.64</td><td>5.76</td><td>288.80</td><td>5.88</td><td>288.68</td><td>5.8</td></tr><tr><td>75</td><td>5.41</td><td>5.53</td><td>288.68</td><td>5.88</td><td>288.32</td><td>5.8</td></tr><tr><td>76</td><td>5.17</td><td>5.29</td><td>288.32</td><td>5.88</td><td>287.73</td><td>5.8</td></tr><tr><td>77</td><td>4.94</td><td>5.06</td><td>287.73</td><td>5.88</td><td>286.90</td><td>5.6</td></tr><tr><td>78</td><td>4.70</td><td>4.82</td><td>286.90</td><td>5.63</td><td>286.10</td><td>5.6</td></tr><tr><td>79</td><td>4.47</td><td>4.59</td><td>286.10</td><td>5.63</td><td>285.06</td><td>5.6</td></tr><tr><td>80</td><td>4.23</td><td>4.35</td><td>285.06</td><td>5.63</td><td>283.78</td><td>5.6</td></tr></table>	H (m)	Q (m3/s)	S (m3)	(S/dt)+Q/2	0	0	0	0.00	1	1	3171	53.28	1	2	6599	110.73	2	2	10258	171.94	2	2	14159	237.12	3	7	18311	308.83	3	10	22722	383.46	Time (min)	I (m³/s)	(I ₁ + I ₂)/2 (m³/s)	S ₁ /Δt + O ₁ /2 (m³/s)	O ₁ (m³/s)	S ₂ /Δt + O ₂ /2 (m³/s)	O ₂ (m³/s)	0	0.00	0.00	0.00	0.00	0.00	0.00	1	0.24	0.12	0.00	0.00	0.12	0.00	2	0.47	0.35	0.12	0.00	0.47	0.00	3	0.71	0.59	0.47	0.00	1.06	0.00	4	0.94	0.82	1.06	0.00	1.88	0.00	5	1.18	1.06	1.88	0.00	2.94	0.04	6	1.41	1.29	2.94	0.04	4.19	0.04	Continued							...	...	...	...	...	...	...	65	7.76	7.76	276.09	4.86	278.99	5.1	66	7.52	7.64	278.99	5.11	281.52	5.4	67	7.29	7.41	281.52	5.37	283.55	5.4	68	7.05	7.17	283.55	5.37	285.36	5.6	69	6.82	6.94	285.36	5.63	286.67	5.6	70	6.58	6.70	286.67	5.63	287.74	5.8	71	6.35	6.47	287.74	5.88	288.33	5.8	72	6.11	6.23	288.33	5.88	288.68	5.8	73	5.88	6.00	288.68	5.88	288.80	5.8	74	5.64	5.76	288.80	5.88	288.68	5.8	75	5.41	5.53	288.68	5.88	288.32	5.8	76	5.17	5.29	288.32	5.88	287.73	5.8	77	4.94	5.06	287.73	5.88	286.90	5.6	78	4.70	4.82	286.90	5.63	286.10	5.6	79	4.47	4.59	286.10	5.63	285.06	5.6	80	4.23	4.35	285.06	5.63	283.78	5.6	
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	1st Trial 50 Year																																					
																																						
	Figure 7.A7: Resulting Outflow Hydrograph -50 yr ARI (1 st Trial)																																					
	(viv) Re-select trial outlet arrangement																																					
	To provide flow reduction for the 50 year ARI post-development design storm, a 1.2m x 1.2m square orifice was re-selected. The previous selection was a 2.0m x 2.0m square, which exceeded the 50yr ARI allowable discharge limit of 5.4 m³/s. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond.																																					
	(v) Re-compute the stage-discharge relationship																																					
	Re-calculate the stage-discharge relationship and storage indicator number from 50yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve.																																					
	Table 7.A12: Stage-Discharge – 5 yr and 50 yr ARI Orifice																																					
	<table><tr><th>H</th><th>5 yr ARI Orifice</th><th>50 yr ARI weir</th><th>Total Discharge</th></tr><tr><th>(m)</th><th>(m³/s)</th><th>(m³/s)</th><th>(m³/s)</th></tr><tr><td>0</td><td>0.00</td><td>0</td><td>0</td></tr><tr><td>0.5</td><td>0.87</td><td>0</td><td>0.87</td></tr><tr><td>1</td><td>1.51</td><td>0</td><td>1.51</td></tr><tr><td>1.5</td><td>1.94</td><td>0</td><td>1.94</td></tr><tr><td>2</td><td>2.30</td><td>0.00</td><td>2.30</td></tr><tr><td>2.5</td><td>2.61</td><td>2.26</td><td>4.86</td></tr><tr><td>3</td><td>2.88</td><td>3.19</td><td>6.07</td></tr></table>	H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge	(m)	(m³/s)	(m³/s)	(m³/s)	0	0.00	0	0	0.5	0.87	0	0.87	1	1.51	0	1.51	1.5	1.94	0	1.94	2	2.30	0.00	2.30	2.5	2.61	2.26	4.86	3	2.88	3.19	6.07	
H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge																																			
(m)	(m³/s)	(m³/s)	(m³/s)																																			
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0.5	0.87	0	0.87																																			
1	1.51	0	1.51																																			
1.5	1.94	0	1.94																																			
2	2.30	0.00	2.30																																			
2.5	2.61	2.26	4.86																																			
3	2.88	3.19	6.07																																			
	C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), A= 1.44 m² (Dimension =1.2m x 1.2m)																																					

Reference	Calculation	Output																																																																																																																																																																																																															
	Table 7.A13: Combined Storage Indicator Numbers – 5 yr and 50yr ARI <table><tr><th>H (m)</th><th>Q (m³/s)</th><th>S (m3)</th><th>(S/dt)+Q/2</th></tr><tr><td>0.00</td><td>0</td><td>0</td><td>0.00</td></tr><tr><td>0.50</td><td>0.87</td><td>3171</td><td>53.28</td></tr><tr><td>1.00</td><td>1.51</td><td>6599</td><td>110.73</td></tr><tr><td>1.50</td><td>1.94</td><td>10258</td><td>171.94</td></tr><tr><td>2.00</td><td>2.30</td><td>14159</td><td>237.12</td></tr><tr><td>2.50</td><td>4.86</td><td>18311</td><td>307.61</td></tr><tr><td>3.00</td><td>6.07</td><td>22722</td><td>381.74</td></tr></table> (vi) Re-route the inflow hydrographs through the pond Using a routing time step of 1.0 minutes, the 5yr and 50yr ARI orifices produced a maximum discharge of 4.9 m³/s, which is acceptable as it is less than the 5 year ARI pond outflow limit of 5.4 m³/s. The maximum water elevation in the pond is 33.48 m, LSD. It is ok since it is less than the maximum allowable elevation (33.7m). Table 7.A14: Routing Table – 50yr ARI (2nd Trial) <table><tr><th>Time (min)</th><th>I (m³/s)</th><th>(I₁ + I₂)/2 (m³/s)</th><th>S₁/Δt + O₁/2 (m³/s)</th><th>O₁ (m³/s)</th><th>S₂/Δt + O₂/2 (m³/s)</th><th>O₂ (m³/s)</th></tr><tr><td>0</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td></tr><tr><td>1</td><td>0.24</td><td>0.12</td><td>0.00</td><td>0.00</td><td>0.12</td><td>0.00</td></tr><tr><td>2</td><td>0.47</td><td>0.35</td><td>0.12</td><td>0.00</td><td>0.47</td><td>0.00</td></tr><tr><td>3</td><td>0.71</td><td>0.59</td><td>0.47</td><td>0.00</td><td>1.06</td><td>0.00</td></tr><tr><td>4</td><td>0.94</td><td>0.82</td><td>1.06</td><td>0.00</td><td>1.88</td><td>0.00</td></tr><tr><td>5</td><td>1.18</td><td>1.06</td><td>1.88</td><td>0.00</td><td>2.94</td><td>0.04</td></tr><tr><td>6</td><td>1.41</td><td>1.29</td><td>2.94</td><td>0.04</td><td>4.19</td><td>0.04</td></tr><tr><td colspan="7">Continued</td></tr><tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr><tr><td>65</td><td>7.76</td><td>7.76</td><td>280.13</td><td>4.22</td><td>283.67</td><td>4.38</td></tr><tr><td>66</td><td>7.52</td><td>7.64</td><td>283.67</td><td>4.38</td><td>286.94</td><td>4.53</td></tr><tr><td>67</td><td>7.29</td><td>7.41</td><td>286.94</td><td>4.53</td><td>289.81</td><td>4.53</td></tr><tr><td>68</td><td>7.05</td><td>7.17</td><td>289.81</td><td>4.53</td><td>292.45</td><td>4.69</td></tr><tr><td>69</td><td>6.82</td><td>6.94</td><td>292.45</td><td>4.69</td><td>294.69</td><td>4.61</td></tr><tr><td>70</td><td>6.58</td><td>6.70</td><td>294.69</td><td>4.85</td><td>296.54</td><td>4.61</td></tr><tr><td>71</td><td>6.35</td><td>6.47</td><td>296.54</td><td>4.85</td><td>298.15</td><td>4.73</td></tr><tr><td>72</td><td>6.11</td><td>6.23</td><td>298.15</td><td>5.01</td><td>299.37</td><td>4.73</td></tr><tr><td>73</td><td>5.88</td><td>6.00</td><td>299.37</td><td>5.01</td><td>300.35</td><td>4.73</td></tr><tr><td>74</td><td>5.64</td><td>5.76</td><td>300.35</td><td>5.01</td><td>301.10</td><td>4.86</td></tr><tr><td>75</td><td>5.41</td><td>5.53</td><td>301.10</td><td>5.17</td><td>301.45</td><td>4.86</td></tr><tr><td>76</td><td>5.17</td><td>5.29</td><td>301.45</td><td>5.17</td><td>301.57</td><td>4.86</td></tr><tr><td>77</td><td>4.94</td><td>5.06</td><td>301.57</td><td>5.17</td><td>301.45</td><td>4.86</td></tr><tr><td>78</td><td>4.70</td><td>4.82</td><td>301.45</td><td>5.17</td><td>301.10</td><td>4.86</td></tr><tr><td>79</td><td>4.47</td><td>4.59</td><td>301.10</td><td>5.17</td><td>300.51</td><td>4.73</td></tr></table>	H (m)	Q (m³/s)	S (m3)	(S/dt)+Q/2	0.00	0	0	0.00	0.50	0.87	3171	53.28	1.00	1.51	6599	110.73	1.50	1.94	10258	171.94	2.00	2.30	14159	237.12	2.50	4.86	18311	307.61	3.00	6.07	22722	381.74	Time (min)	I (m³/s)	(I ₁ + I ₂)/2 (m³/s)	S ₁ /Δt + O ₁ /2 (m³/s)	O ₁ (m³/s)	S ₂ /Δt + O ₂ /2 (m³/s)	O ₂ (m³/s)	0	0.00	0.00	0.00	0.00	0.00	0.00	1	0.24	0.12	0.00	0.00	0.12	0.00	2	0.47	0.35	0.12	0.00	0.47	0.00	3	0.71	0.59	0.47	0.00	1.06	0.00	4	0.94	0.82	1.06	0.00	1.88	0.00	5	1.18	1.06	1.88	0.00	2.94	0.04	6	1.41	1.29	2.94	0.04	4.19	0.04	Continued							...	...	...	...	...	...	...	65	7.76	7.76	280.13	4.22	283.67	4.38	66	7.52	7.64	283.67	4.38	286.94	4.53	67	7.29	7.41	286.94	4.53	289.81	4.53	68	7.05	7.17	289.81	4.53	292.45	4.69	69	6.82	6.94	292.45	4.69	294.69	4.61	70	6.58	6.70	294.69	4.85	296.54	4.61	71	6.35	6.47	296.54	4.85	298.15	4.73	72	6.11	6.23	298.15	5.01	299.37	4.73	73	5.88	6.00	299.37	5.01	300.35	4.73	74	5.64	5.76	300.35	5.01	301.10	4.86	75	5.41	5.53	301.10	5.17	301.45	4.86	76	5.17	5.29	301.45	5.17	301.57	4.86	77	4.94	5.06	301.57	5.17	301.45	4.86	78	4.70	4.82	301.45	5.17	301.10	4.86	79	4.47	4.59	301.10	5.17	300.51	4.73	
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2nd Trial 50 year 																																									
Figure 7.A8: Resulting Outflow Hydrograph -50 yr ARI (2nd Trial) Step 9: Size the secondary outlet arrangement 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. (vii) Select trial outlet arrangement A 10 m wide broad-crested weir spillway with 0(H):1(V) side slopes was initially selected as the pond secondary outlet. The spillway was set at the side of the embankment at an elevation of 33.5 m, LSD (50 year ARI maximum water level). (viii) Compute the stage-discharge relationship The stage-discharge relationship is the summation of the 5 yr and 50 yr ARI orifice and 100yr ARI spillway capacities. Table 7.A15: Combined Stage-Discharge - 5 and 50 yr Orifice and 100 yr ARI Spillway <table> <tr> <th>H</th> <th>5 yr ARI Orifice</th> <th>50 yr ARI Orifice</th> <th>spillway</th> <th>Total Discharge</th> </tr> <tr> <th>(m)</th> <th>(m³/s)</th> <th>(m³/s)</th> <th>(m³/s)</th> <th>(m³/s)</th> </tr> <tr> <td>0</td> <td>0.00</td> <td>0.00</td> <td>0</td> <td>0</td> </tr> <tr> <td>0.5</td> <td>0.87</td> <td>0.00</td> <td>0</td> <td>0.87</td> </tr> <tr> <td>1</td> <td>1.51</td> <td>0.00</td> <td>0</td> <td>1.51</td> </tr> <tr> <td>1.5</td> <td>1.94</td> <td>0.00</td> <td>0</td> <td>1.94</td> </tr> <tr> <td>2</td> <td>2.30</td> <td>0.00</td> <td>0</td> <td>2.30</td> </tr> </table>							H		5 yr ARI Orifice	50 yr ARI Orifice	spillway	Total Discharge	(m)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	0	0.00	0.00	0	0	0.5	0.87	0.00	0	0.87	1	1.51	0.00	0	1.51	1.5	1.94	0.00	0	1.94	2	2.30	0.00	0
H	5 yr ARI Orifice	50 yr ARI Orifice	spillway	Total Discharge																																					
(m)	(m³/s)	(m³/s)	(m³/s)	(m³/s)																																					
0	0.00	0.00	0	0																																					
0.5	0.87	0.00	0	0.87																																					
1	1.51	0.00	0	1.51																																					
1.5	1.94	0.00	0	1.94																																					
2	2.30	0.00	0	2.30																																					

Reference	Calculation						Output
		2.5	2.61	2.26	0	4.86	
		3	2.88	3.19	6	12.08	
	C _o =1.7 (Weir Flow) and 0.6 (Orifice Flow), 5yr Orifice=0.5m x 0.5m, 50yr Orifice=1.2m x 1.2m and 100yr Spillway=10m x 0.5m						
	Table 7.A16: Combined Storage Indicator Numbers – 5yr, 50yr and 100yr ARI						
		H (m)	Q (m³/s)	S (m³)	(S/dt)+Q/2		
		0.00	0	0	0.00		
		0.50	0.87	3170.5	53.28		
		1.00	1.51	6598.5	110.73		
		1.50	1.94	10258	171.94		
		2.00	2.30	14158.5	237.12		
	2.50	4.86	18310.5	307.61			
	3.00	12.08	20722	351.41			
(ix) Route the inflow hydrographs through the pond							
Using a routing time step of 1.0 minutes, the maximum water level in the pond is 33.65 m , LSD that corresponds to a water depth of 2.60 m. Allowing a freeboard of 300 mm for wave action, the embankment crest elevation is set at 34.0 m , LSD.							
(iv) Check if the results are acceptable							
The basin will provide a reduction in the 100 year ARI flow, being reduced from 8.69 m³/s to 7.03 m³/s (more than the 100yr ARI pre-development discharge of 6.0 m³/s). Should be ok since the pond is to cater for 50 year ARI only. Since the maximum water level (33.60 m LSD) is within the allowable level, the pond sizing and routing is ok.							
Table 7.A17: Routing Table – 5 yr, 50 yr and 100 yr ARI							
Time (min)	I (m³/s)	(I ₁ + I ₂)/2 (m³/s)	S ₁ /Δt + O ₁ /2 (m³/s)	O ₁ (m³/s)	S ₂ /Δt + O ₂ /2 (m³/s)	O ₂ (m³/s)	
0	0	0.00	0.00	0.00	0.00	0.00	
1	0.26	0.13	0.00	0.00	0.13	0.00	
2	0.53	0.40	0.13	0.00	0.53	0.00	
3	0.79	0.66	0.53	0.00	1.19	0.00	
4	1.05	0.92	1.19	0.00	2.11	0.00	
5	1.32	1.19	2.11	0.00	3.29	0.04	
Continued							
...	...	...	...	...	...	...	
67	8.16	8.30	312.43	6.07	314.65	6.65	
68	7.90	8.03	314.65	6.65	316.03	7.03	
69	7.64	7.77	316.03	7.23	316.57	7.03	
70	7.37	7.51	316.57	7.23	316.85	7.03	
71	7.11	7.24	316.85	7.23	316.87	7.03	
72	6.85	6.98	316.87	7.23	316.62	7.03	
73	6.58	6.72	316.62	7.23	316.11	7.03	
74	6.32	6.45	316.11	7.23	315.33	7.03	

Reference	Calculation							Output
	75	6.06	6.19	315.33	7.23	314.29	6.65	
	76	5.79	5.93	314.29	6.65	313.57	6.65	
	80	4.74	4.87	310.40	6.07	309.20	5.49	
	Continued							
	...	...	...	...	...	...	...	
1st Trial 100 year Figure 7.A9: Resulting Outflow Hydrograph - 100yr ARI Step 10: Check behaviour under extreme conditions As the pond is small in volume and bund height it is not a hazardous structure. Therefore, no checking was required for the bund failure due to PMF. Step 11: Prepare design drawings and specifications Refer Figure 7.A11 and 7.A12.								

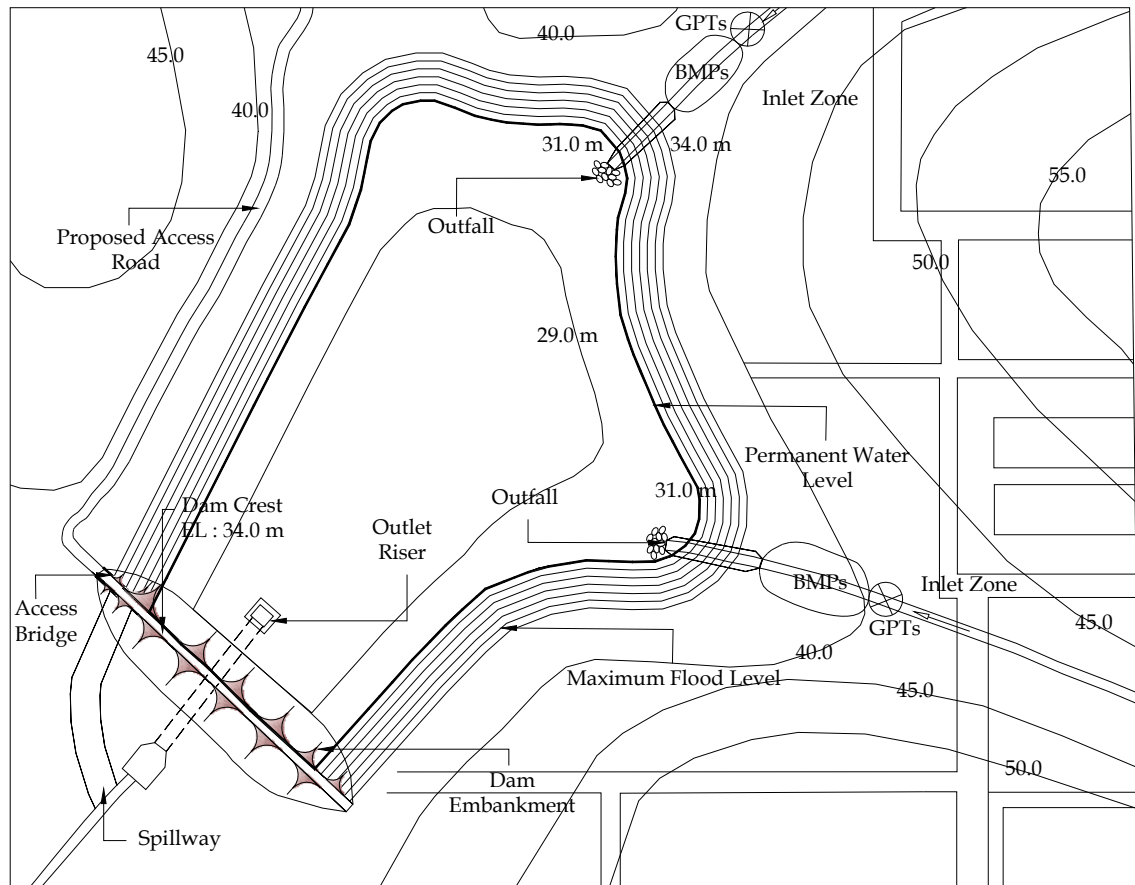


Figure 7.A10: Layout Plan of the Designed Detention Pond

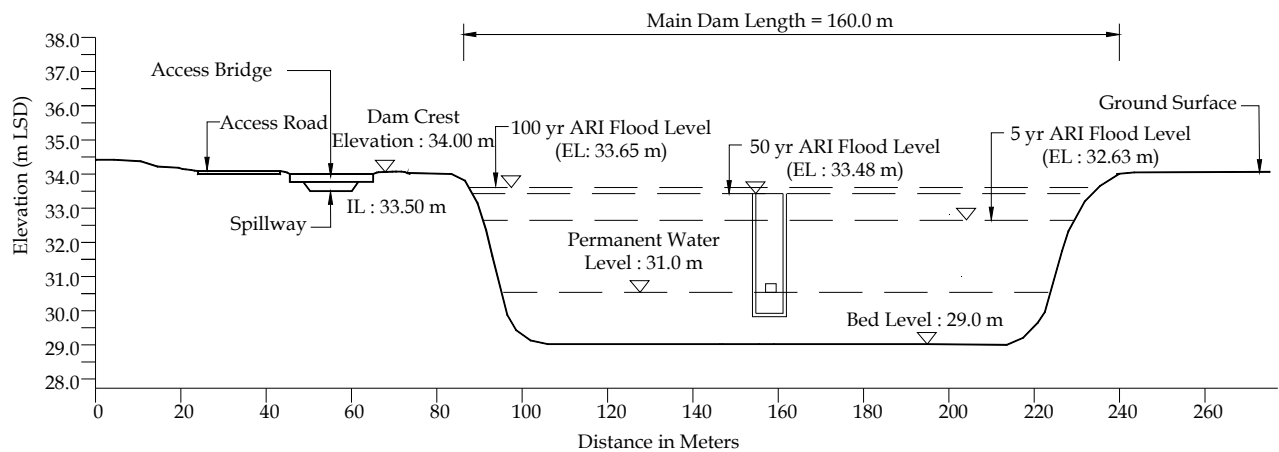


Figure 7.A11: Cross Section of the Designed Detention Pond

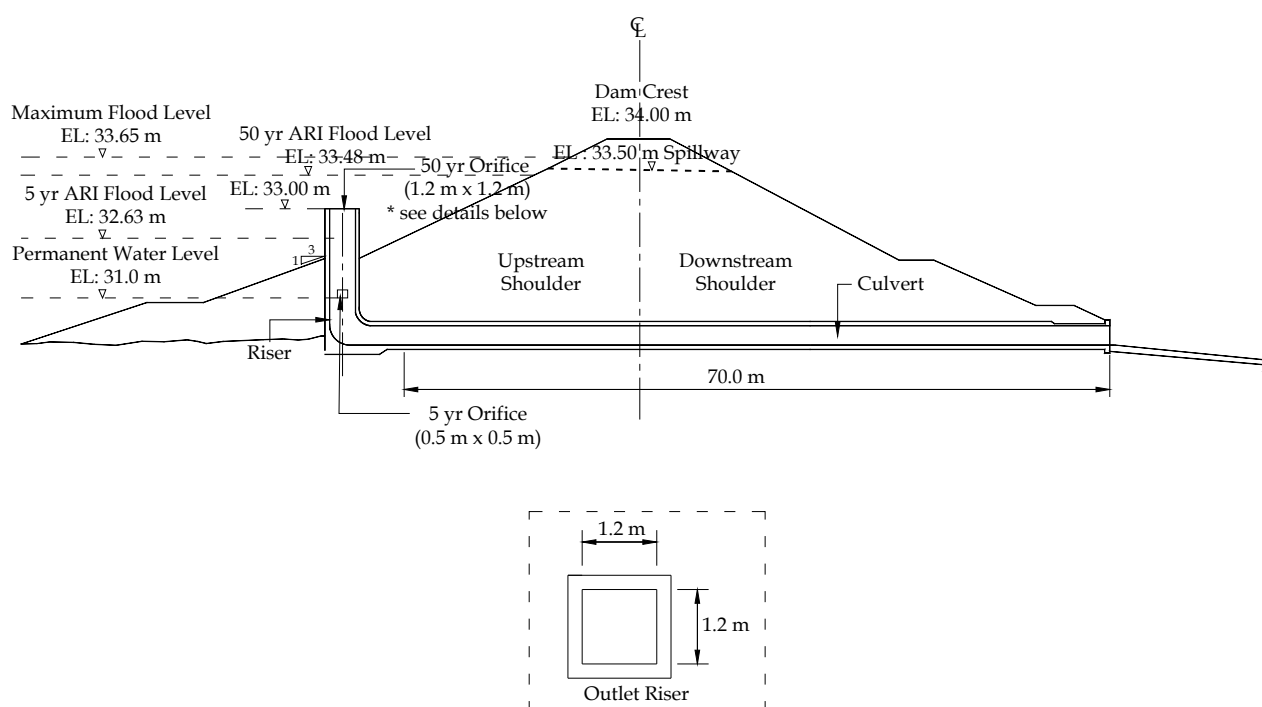


Figure 7.A12: Dam Cross-Section