CHAPTER 19 GATE AND PUMP

19.1 INTRODUCTION

Urban drainage and stormwater system design in low-lying and tidal areas involves a number of special considerations. Because of the difficulties of designing gravity systems in low-lying areas it may be necessary to use drainage gates/ tidal gates, and/or pumped systems. In some locations, there may be advantages in combining a tidal gate or drainage gate outlet with a pumped discharge. This would allow water to drain by gravity when the tailwater level is low, saving on pumping costs, and to be pumped when the tailwater level is high. A combined outlet system will be most practical where there is a large range in tailwater levels, typically 2.0 metres or more. A detailed analysis of the storage and pump requirements will require data on the stage hydrograph of the tailwater, whether it be a river flood or tide cycle, and the calculation should be performed by computer methods.

19.2 SYSTEM COMPONENTS

The general system components for gates and pump station (Figures 19.1 and 19.2) include the following:

- Gates and side-spill weir - Control gates are required at drainage outlet to avoid backflow during high tides or high flood levels at the receiving water bodies. Side-spill weir is to prevent stormwater from entering the pump sump during normal period where the flow can be discharged through gravity.

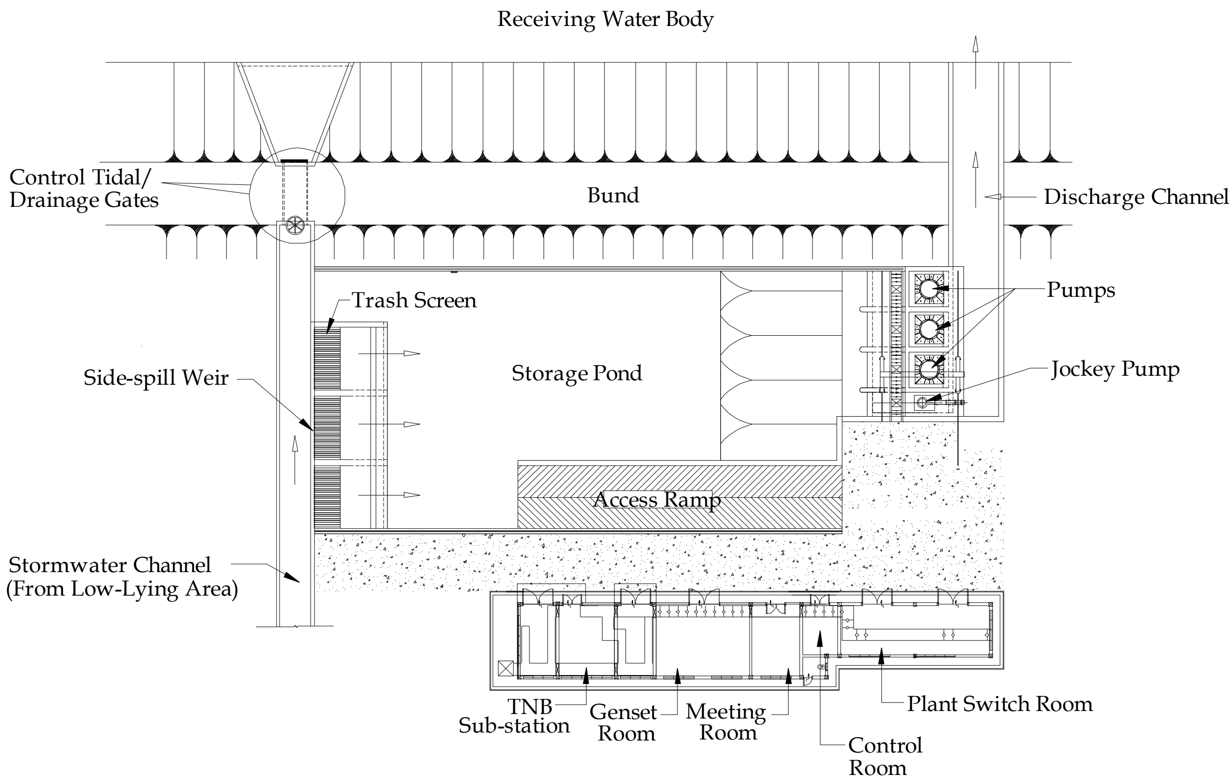


Figure 19.1: Typical Layout and System Components of Gates and Pump Station

Figure 19.2: Photograph of Pump Station

- Sumps and storages - The sump and storage receive the inflow of storm water prior to pumping. The storage can attenuate the storm hydrograph peak to reduce the required pumping rate. The station should design with provision for debris removal system and convenient access for the removal of accumulated debris and silt.
- Pumps and motors - Pump selection and numbers depend on station layout, required pump rate, wet well depth, and maintenance considerations. The size of each motor depends on the pump size, flow rate, pressure head, and duty cycle.
- Power sources - The power source is usually provided by the local utility that normally require electrical substation. Every pump station should have an on-site standby electrical generator because the type of storm that makes a pump station necessary is also the type of storm that interrupts utility power.
- Controls - Control circuitry includes the flood level at which the pump station will be activated, sequence of operation, activation of the standby generator when necessary, deactivation when the flood event has passed, and operation of any night security lighting. Controls may also include automatic communication with a central office on the station's status regarding water levels, pump readiness, utility electrical power, standby generator fuel level, security, or other central office concerns.
- Discharge channels or conduits - Direct the pump flows into the receiving water bodies and should design to avoid any backflow into the pump sump.

19.3 TIDAL AND DRAINAGE GATES

Tidal gates may be used in low-lying urban areas near the coast to prevent intrusion of tidal water into the drainage system at periods of high tide. These tidal waters decrease the storage capacities of the drains with the result that flooding may occur during storms.

Drainage gates may be used in low-lying urban areas to prevent intrusion of backflooding from rivers or other receiving waters (e.g. in the case of discharge through a leveed riverbank) into the drainage system at periods of high floodwater profiles at receiving water. In practice, there are many similarities in the design details of tidal gates and drainage gates.

19.3.1 Types of Gates

There are various types of tidal and drainage gate are used throughout Malaysia as presented below:

Culvert Type (Figure 19.3) - Has a flap gate that works automatically on the downstream side and a vertical penstock gate on the upstream side. The penstock gate is provided as a back-up that can be manually operated should the automatic gate jam, or when maintenance is being carried out. This type of gate is suitable generally for fairly low discharge and has the advantage of having minimal operation requirements but does, however, necessitate regular inspection and clearing since the flap gates are liable to obstruction by debris. The downstream outlet is normally maintained in drowned condition to prevent the high velocity jet from occurring and endangering the outlet channel stability. The value of C is usually assumed as 0.60. The peak flow will occur at the maximum differential head.

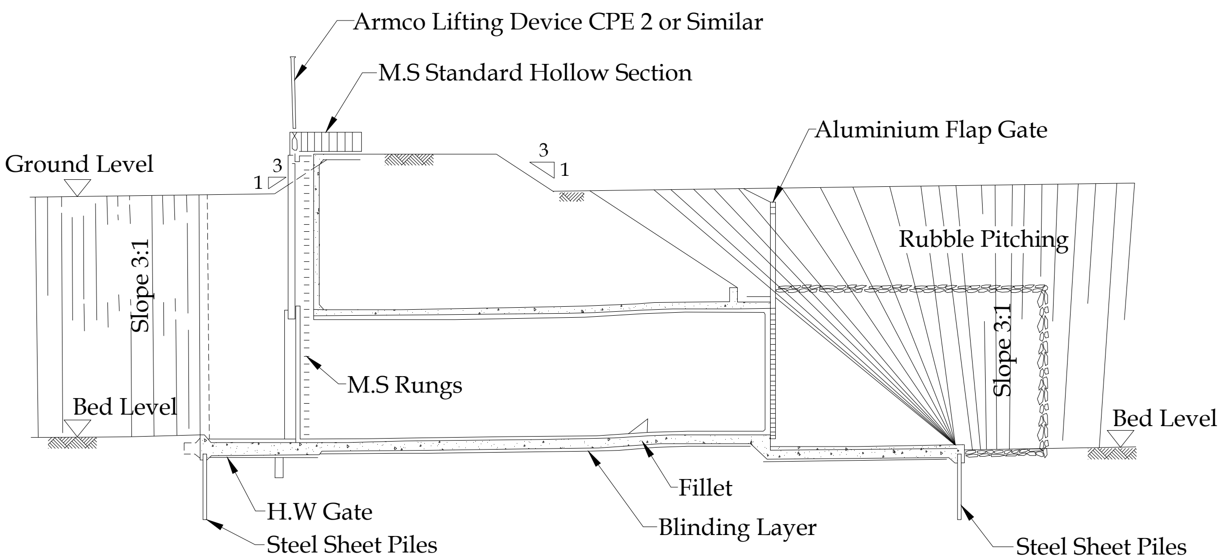


Figure 19.3: Typical Culvert Type Outlet Structure

Open Flume Type (Figure 19.4) - Consists of a vertical aluminium sliding gate that can either be mechanically operated or hand operated for the smaller sizes. Standard open flumes have much larger capacities than the culvert-type gates and are more reliable. However they put a greater burden on operating personnel. The discharge curve during controlled flow approximates a parabola and the gate discharge can be computed using weir formula.

Other Types - Flap gates, usually made of fibreglass, cast iron or cast steel (Figure 19.5), are used to permit flow in only one direction. A small differential pressure on the back of the gate will open it, allowing discharge in the desired direction. When water on the front side of the gate rises above that on the back side, the gate closes to prevent backflow. Flap gates are available for round, square, and rectangular openings and in various designs. Rubber flap gates and "duckbill" check valve types (Figure 19.5), which are less susceptible to this sort of clogging, are also being used. Periodic inspection and cleaning should be scheduled when the water flowing through the gate carries floating material. If the gate is to be kept clear of debris, it should be mounted 300-450 mm above the apron in front of the gate.

19.3.2 Design Consideration for Tidal Gates

Owing to the cyclic nature of the tides, the discharge of stormwater out of the compartment can only be effected for a certain duration of each tide cycle, unless pump drainage is incorporated. It is therefore necessary, to provide storage for the stormwater within the compartment during periods of high tides. Tidal gates should be provided in areas that are below Mean High Water Spring (MHWS) or Mean High High Water (MHHW). The invert of the outlet should preferably be above Lowest Astronomical Tide (LAT), between Mean Low Low Water (MLLW)/Mean High Water Spring (MLWS) and Mean Sea Level (MSL). High outfall will cause excessive scour unless protection measures are provided such as a dissipator chute. The obvert of the outlet should normally be below Highest Astronomical Tide (HAT).

A tidal outlet which will experience severe wave action may need to be extended through the beach zone to discharge beyond the breaker line and below the low-tide level. Structures subject to wave action must be designed to withstand wave loadings. The design of an outlet in the beach zone must also consider the possible undermining of the structure by wave action and longshore currents as well as the lateral loads that might be

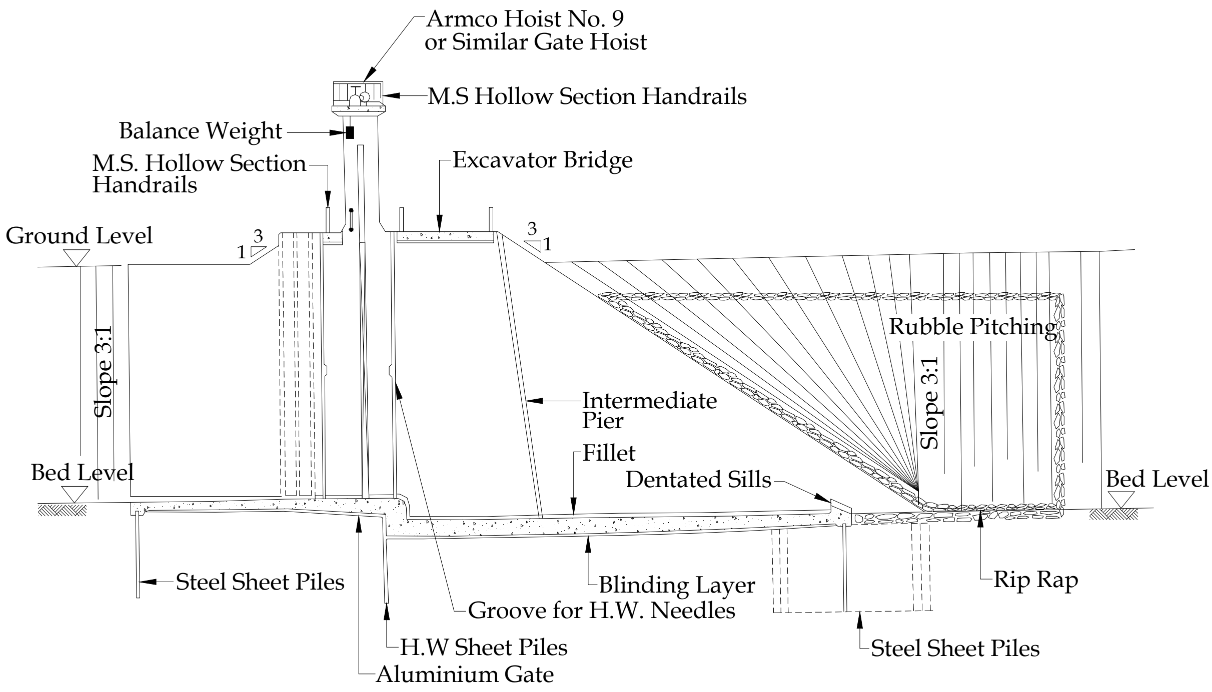


Figure 19.4: Typical Flume Type Outlet Structure

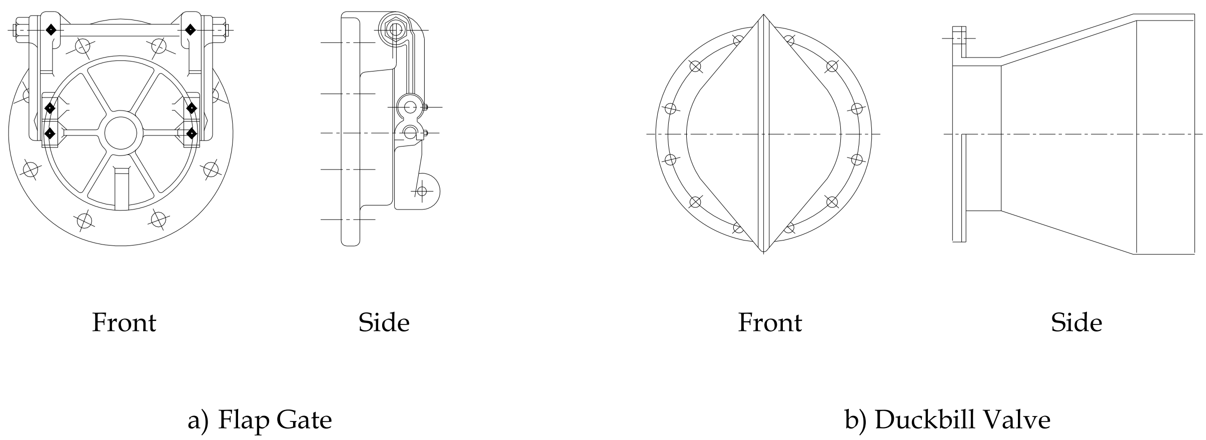


Figure 19.5: Typical Flap Gate and Duckbill Check Valve

allied by differential sand levels caused by longshore littoral drift. Outlets may be prone to siltation by beach sand accumulating against the outlet. Deposition may also occur in the channel/pipes leading to the outlet especially when high tailwater levels cause velocities in the channel/pipes to be reduced.

Tidal gates must be designed to operate automatically, as manual operation is usually not practical. For this reason flap-type gates are preferred. Generally, the flap gate should be fitted in a chamber just upstream of the outlet, to protect its operation from vandalism, wave attack, debris and sand blockage.

19.3.3 Design Consideration for Drainage Gates

Outlets should be located high enough to facilitate inspection and maintenance of the channel/pipes at low water level in the receiving waters. An energy-dissipating outfall should be provided where the velocity of the discharge from the conveyance system to the receiving water could cause scour of the receiving channel around the outlet. An outlet discharging to a river should also be located and designed with respect to possible changes in stream morphology, often best determined by reference to past maps and photographs (including aerial photographs) of the site. Rock mattresses or other flexible facing materials should be provided to counter local scour around the outlet.

Flap gates or drainage gates may be fitted to an outlet to prevent backflooding from the receiving waters (e.g. in the case of discharge through a leveed riverbank) and to control siltation within the channel/pipes from the receiving waters. Flap gates/Drainage gate may be fitted in a chamber just upstream of the outlet, if required for protection of their operation, but the chamber in such cases should be located within or on the riverside of the levee. Siltation by sediments which enter the channel/pipe system from the receiving waters can be controlled by locating the outlet as high as possible.

19.4 PUMPING STATION

Because of its high cost and the potential problems associated with pump stations, stormwater pumping is generally used only when gravity drainage is not feasible. When operation and maintenance cost are capitalised, a considerable expenditure can be justified for a gravity system. Keeping the drainage area as small as possible and providing storage in storm drains can reduce the pumping capacity required to handle peak runoff rates.

19.4.1 Planning Process

Planning of Site Location - In normal circumstances, the location of the pump station is at the drainage system outlet, just immediately upstream from the receiving waterbody to minimise the conveyance's head-loss. Sufficient space should be allocated all the facilities highlighted earlier and also for safe access and parking necessary for operation, maintenance, and emergency functions.

Station Types - Basically, there are two types of stations, wet-pit (Figure 19.6) and dry-pit (Figure 19.7). The main advantage over the wet-pit station is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. However dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are most often used. Dry-pit stations are more appropriate for handling sewage because of the potential health hazards to maintenance personnel.

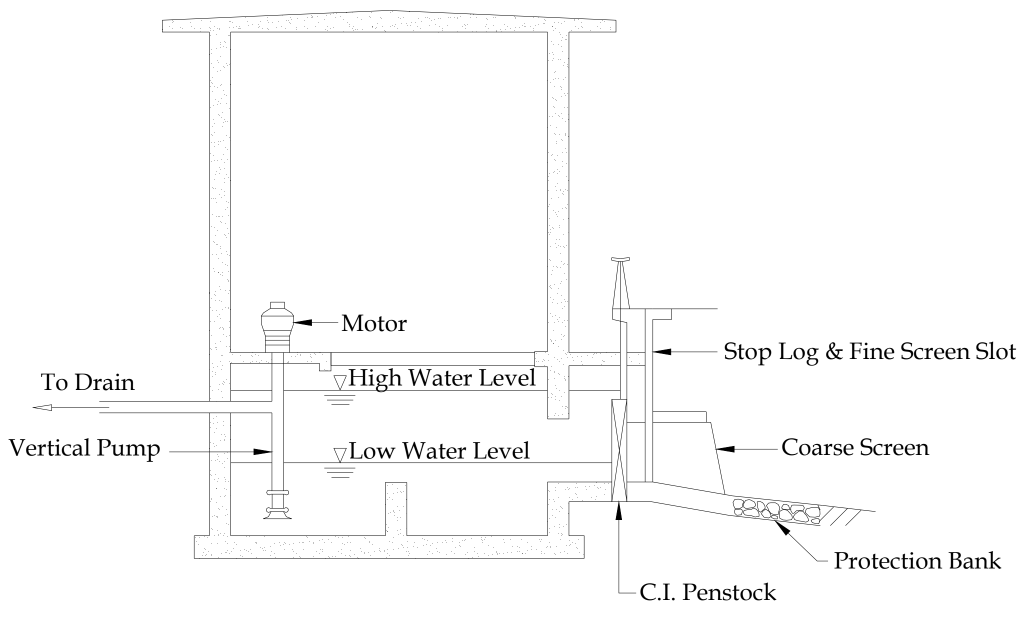


Figure 19.6: Typical Wet-pit Station

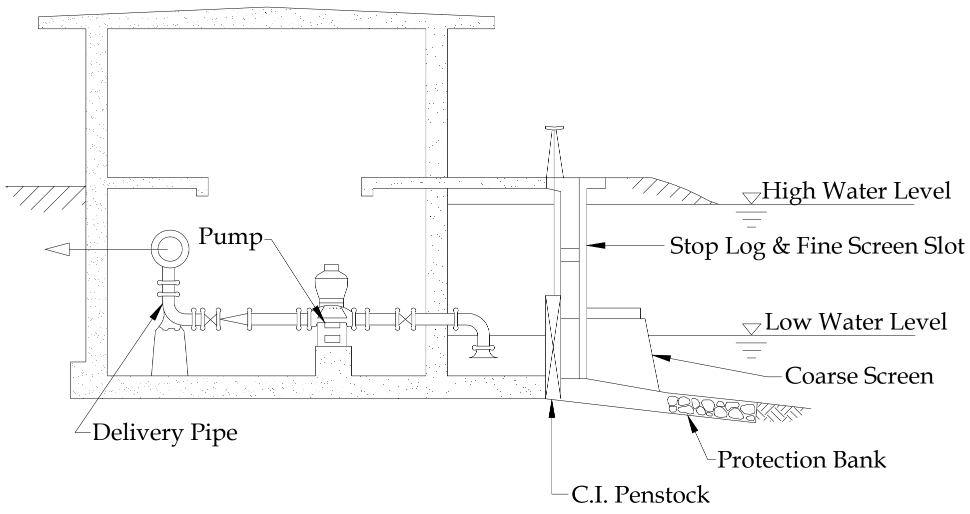


Figure 19.7: Typical Dry-pit Station

Pump Types - Various types of pumps are used and they are as listed out in Table 19.1 and shown in Figures 19.8 and 19.9. For centrifugal and axial pumps, submersible type frequently provides special advantages in simplifying the design, construction, maintenance and therefore the cost of the pumping station. Noise generated from the motors will also be less.

Pump Selection - The performance curve developed by the manufacturer should be obtained before selecting a particular pump. Capital costs are of more concern than operating cost in storm water pump stations since the operating periods during the year are relatively short. Ordinarily, providing as much storage as possible minimises capital costs. Either two to three pumps should be used, except in very large installations. Consideration may be given to over sizing the pumps to compensate, in part, for a pump failure. Large pumps should be avoided to prevent too frequent starting and stopping that can reduce the lifespan of them. All pumps in a station should be of the same size and type to enable all pumps to be freely alternated into service so that each pump is automatically redefine the lead pump after each pump cycle. This equalises wear and simplifies scheduling maintenance and allows pump parts to be interchangeable.

Pump Sump and Intake System - Pump positions in the sump should be as recommended by Figure 19.10 (The Hydraulic Institute, 1998) from manufacturer specification or even through physical modelling if recommended standards and specifications cannot be met. The primary function of the intake structure is to supply an even distribution of flow to the pumps to avoid reduced pump efficiency and undesirable operational characteristics.

Table 19.1: Pump Types for Stormwater Drainage

Pump Type	Descriptions
Centrifugal	Commonly used for high head applications and the impellers can be designed with large openings to avoid clogging
Axial	Commonly used for low head, high discharge applications. Do not handle debris particularly well because large and hard objects or fibrous material may damage and jam the propellers.
Mixed-flow	Combination of the above two types and they are used for intermediate head and discharge applications and handle debris slightly better than propellers.
Screw	Although low in efficiency and expensive, it is suitable for high pressures, and delivers fluid with little noise or pressure pulsation, thus fish that fit the water column can safely lifted up through it

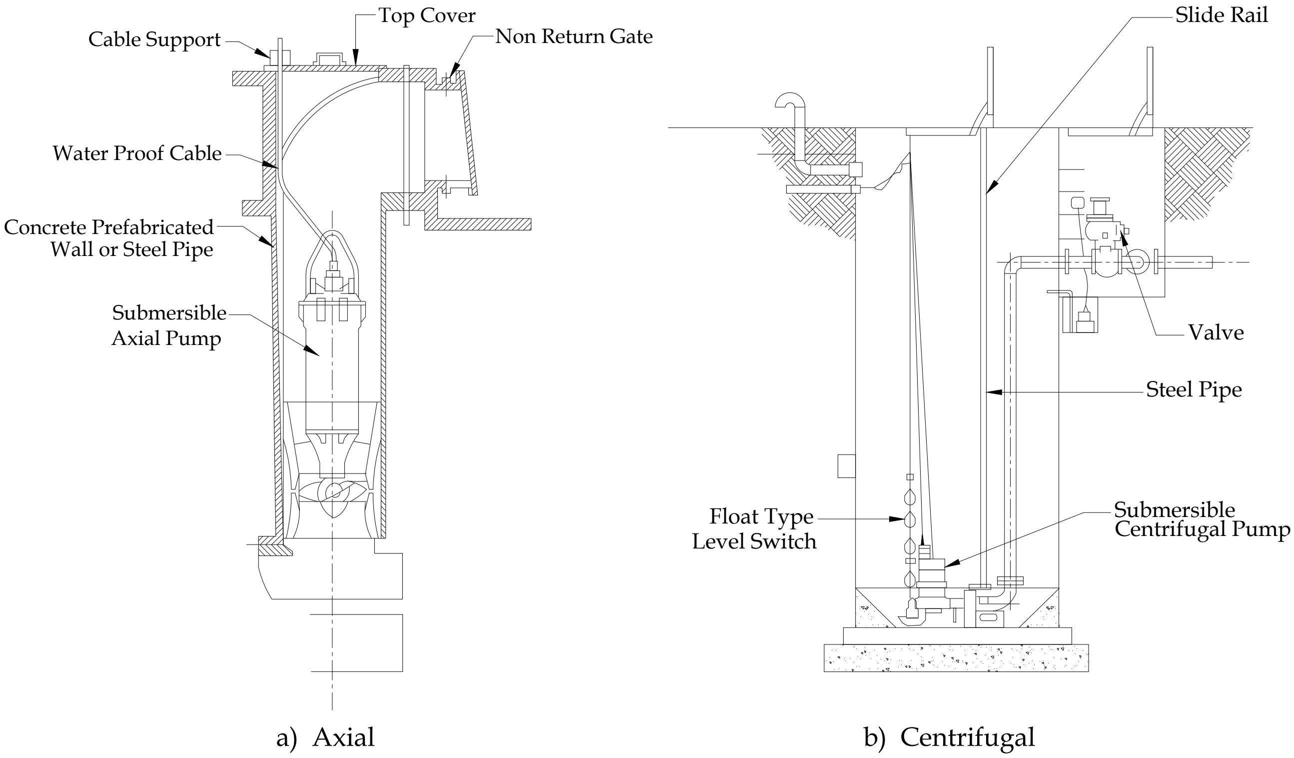


Figure 19.8: Typical Submersible Pumps

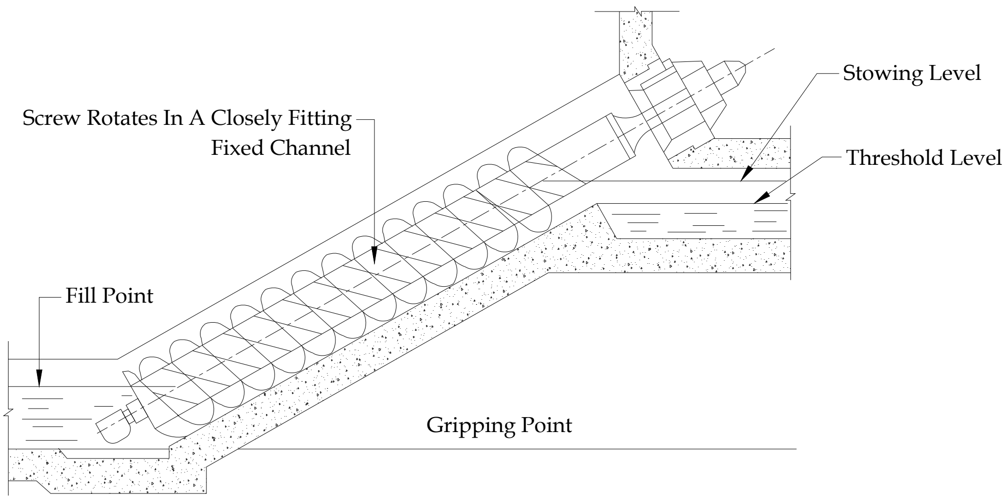
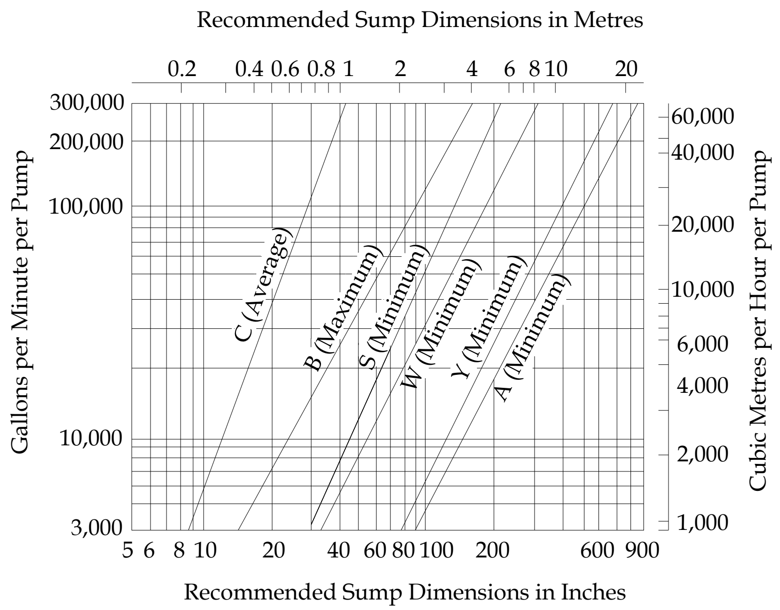
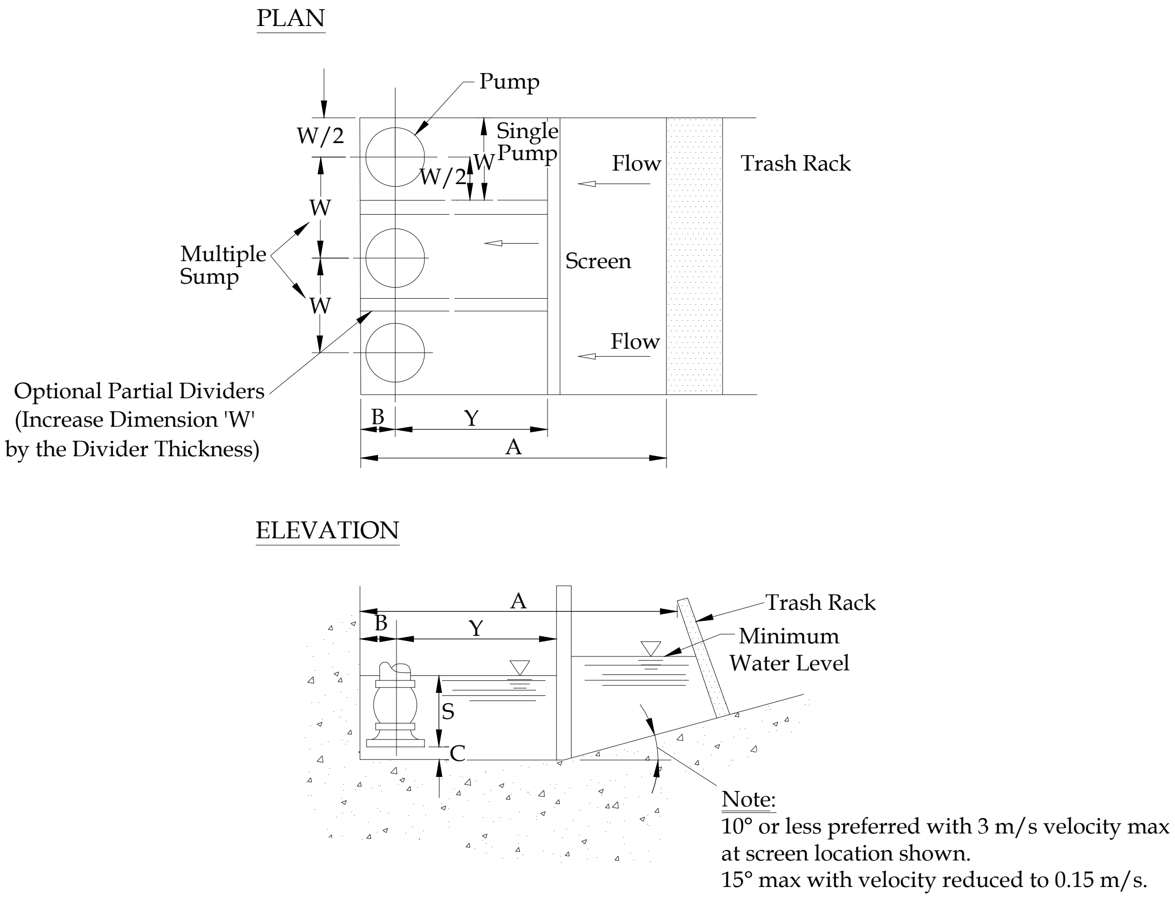


Figure 19.9: Typical Screw Pump



a) Sump Dimensions (see b for dimension location)



b) Plan and Elevation

Figure 19.10: Wet Pit Type Pump

Collection Systems - Stormwater drains leading to the pumping station are typically designed on mild grades to minimise depth, pumping head and associated construction cost while maintaining an average velocity of 1.0 m/s during flowing full to avoid siltation problems. To reduce the operation cost, by-pass with tidal/ drainage gates is constructed to enable the stormwater to be gravity drained when the waterlevel at receiving waterbody is low and only diverted to pump sump when water level at receiving waterbody is high.

Discharge System - The discharge piping should be kept as simple as possible. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Check valves must be provided on the individual lines to keep storm water from running back into the wet well. Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair, etc.

19.4.2 Design Considerations for Pump and Storage Sizing

Hydrology - The design standard and procedure to derive the design flood hydrographs for a stormwater pumping station should be the same as for the major drainage system. However, if storage is formed part of the system, not only the design peak discharges need to be considered but also the runoff volumes and hydrograph shapes for various rainstorm duration. Every attempt should be made to keep the drainage area tributary to the station as small as possible. By-pass or pass-through all possible drainage should be provided to reduce pumping requirements. Avoid future increase in pumping by isolating the pump drainage area with the gravity drainage area through high level drains or bunding to prevent off-site drainage from possibly being diverted to the pump station. Hydrologic design should be based on the ultimate development of the area, which must drain to the station.

Discharge Head and System Curve - The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head (TDH) for the pump discharge head. The TDH is computed as follows:

$$TDH = H_s + h_f + h_v + h_l \qquad (19.1)$$

where

TDH = Total dynamic head (m);

H_s = Static head (m);

h_f = Friction head (m);

h_v = Velocity head (m); and

h_l = Losses through fittings, valves, etc. (m)

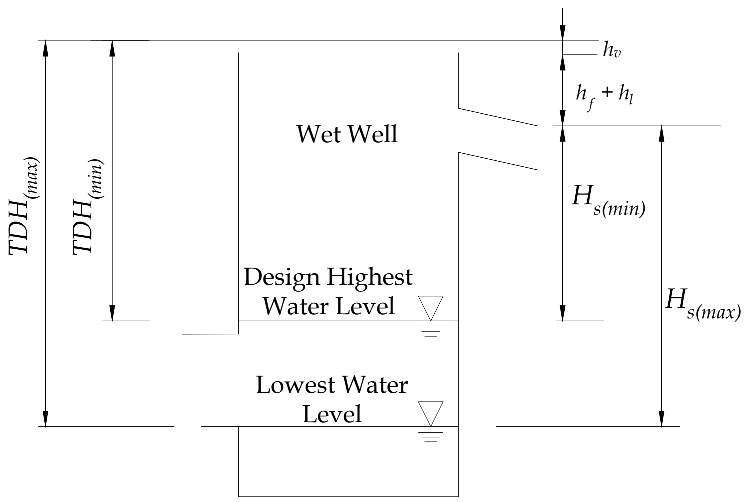
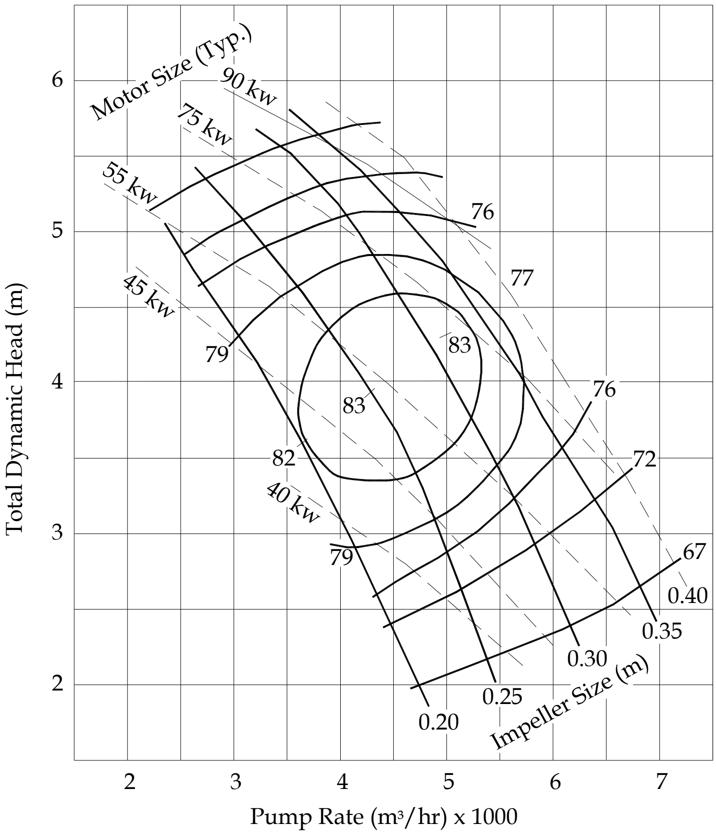
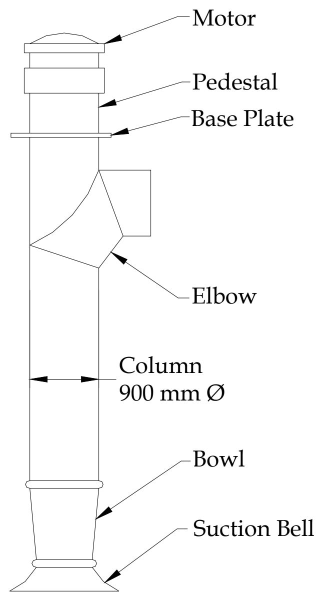


Figure 19.11: Components of Total Dynamic Head (TDH)

The total dynamic head (TDH) must be determined for a sufficient number of points to draw the system head curve where the system curve (Q vs. TDH) can be plotted. When overlaid with pump performance curves provided by manufacturer as given in (Figure 19.12), it will yield the pump operating points (Figure 19.13). Pumps should be selected to operate with the best efficiency at the design operating point.





Typical Outline of 900 mm Propeller Pump

Figure 19.12: Performance Curve for 900mm Pump Rotating

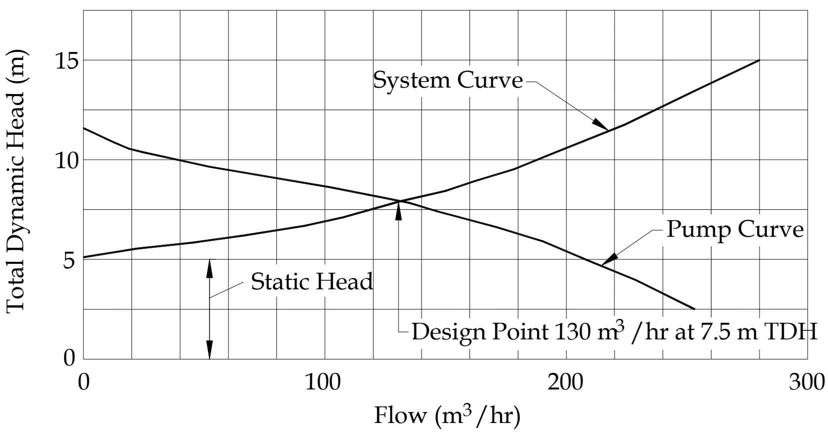


Figure 19.13: System Head Curve

Pump and Storage Sizing - There is a complex relationship between the variables of pumping rates, storage and pump on-off settings in pump station design. Storage capacity is usually required as a part of station design to permit the use of smaller, more economical pumps. The principle of minimum run time and pump cycling with increased storage volume should also be considered during the development of an optimum storage requirement. Comparing the inflow hydrograph to the controlling pump discharge rate as illustrated in Figure 19.14 gives an estimate of the required storage volume. If storage is used to reduce peak flow rates, a routing procedure must be used to design the system. A worked example of the pump rate and storage volume calculation is given in Appendix 19.A.

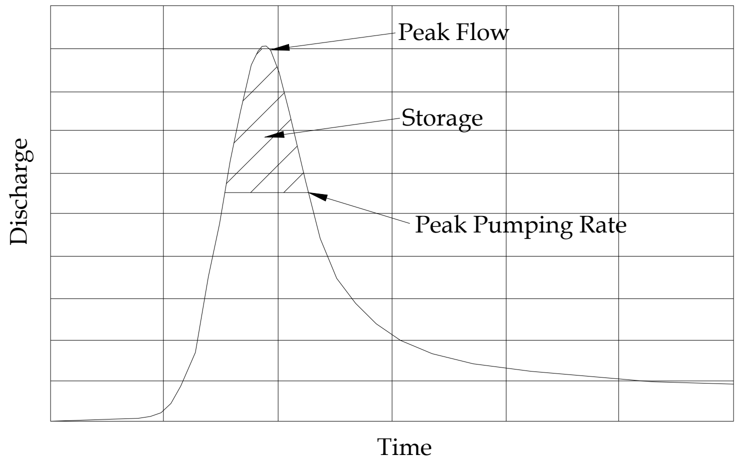


Figure 19.14: Estimation of Required Storage from Inflow Hydrograph.

Water-Level Sensors - Water-level sensors are used to activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch, and air pressure switch. The on-off setting for the pump is particularly important because it defines the most frequently used cycle. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement.

Allowable High Water Elevation - The allowable high water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 0.3 to 0.6m of freeboard below the roadway grate.

19.4.3 Other Facilities and Requirements

Power - Several types of power may be available for a pump station such as electric motors and petrol, diesel or natural gas engines. The designer should select the type of power that best meets the needs of the project. However, when readily available, electric power is usually the most economical and reliable choice. There generally is a need for backup power if the consequences of failure are severe. Motor voltages between 415 volts is recommended for pumping applications. Consequently, it is recommended that 225 kW be the maximum size motor used. This size is also a good upper limit for ease of maintenance.

Trash Racks and Grit Chambers - Trash racks should be provided at the entrance to the wet well if large debris is anticipated. Usually, the bar screens are inclined with bar spacing approximately 35mm. Automatic trash racks of various designs may also be used in area where trash problem is serious. If substantial amounts of sediment are anticipated, a easily maintained grit chamber may be provided to catch solids that are expected to settle out. This will minimise wear on the pumps and limit deposits in the wet well. The screen inlet should be adequately sized by taking into consideration of the partial clogging of the inlet by debris that prevents the full flood flows from entering the pump sump. If the among of sediment and debris are a lot, GPTs should be installed upstream of the stormwater system to minimise them from reaching the collection system.

Ventilation - Ventilation of dry and wet wells is necessary to ensure a safe working environment for maintenance personnel. If required, exhaust fan systems may be used when accessing the pit.

Roof Hatches and Monorails - It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment are a cost-effective way of providing this capability. Mobile cranes can simply lift the smaller equipment from the station onto maintenance trucks. Monorails are usually more cost-effective for larger stations.

Equipment Certification and Testing - Equipment certification and testing is required to ensure that the pumps supplied meet the design and specification. a crucial element of pump station design. It is good practice to include in the contract specifications the requirement for acceptance testing by the owner, when possible, to ensure proper operation of the completed pump station.

Monitoring and Maintenance - Pump stations are vulnerable to a wide range of operational problems from malfunction of equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimise such failures and their consequences. Telemetry and SCADA system are options that should be considered for monitoring critical pump station. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

Safety - Ladders, stairwells and other access points as well as adequate lightings should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering.

19.5 DESIGN PROCEDURE FOR TIDAL GATES

The method involves the initial estimation of the required waterway area (in the case of culverts) or the required width (in the case of flumes) to discharge the design stormwater runoff out of the compartment into the outlet channel whose tidal fluctuations are known. Based on this a suitable gate is selected and the design storm inflow is routed through the selected gate into the tidal channel. If the gate is adequate the whole of the design stormwater runoff should be discharged within the tidal period. The procedure comprises the stages as shown in Figure 19.15.

19.6 DESIGN PROCEDURE FOR DRAINAGE GATE

For the larger-size drainage gates, design is normally based on routing methods, which require a fairly detailed knowledge of the topography of the catchment area discharging through the gate in order to be able to assess accurately storage effects throughout the design flood hydrograph of the downstream receiving water. For urban stormwater drains, it is often sufficient to assume that the drainage gate is closed for the duration of the storm. For areas where temporary storage is not available flooding upstream of the gate would be unacceptable (in particular built-up urban areas) and where space for detention storage is not available, gates should provide as large an area as possible for discharge after periods of high river level. The area available for discharge should preferably be equivalent to the wetted area of the drainage channel. The procedure comprises the steps as shown in Figure 19.15.

19.7 DESIGN PROCEDURE FOR PUMPING STATION

It incorporates the design criteria discussed in previous sections and yields the required number and capacity of pumps as well as the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance. To initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then by routing the inflow hydrograph through the trial pump station its adequacy can be evaluated.

The hydraulic analysis of a pump station involves the interrelationship of three components:

- the inflow hydrograph
- the storage capacity of the wet well and the outside storage, and
- the discharge rate of the pumping system

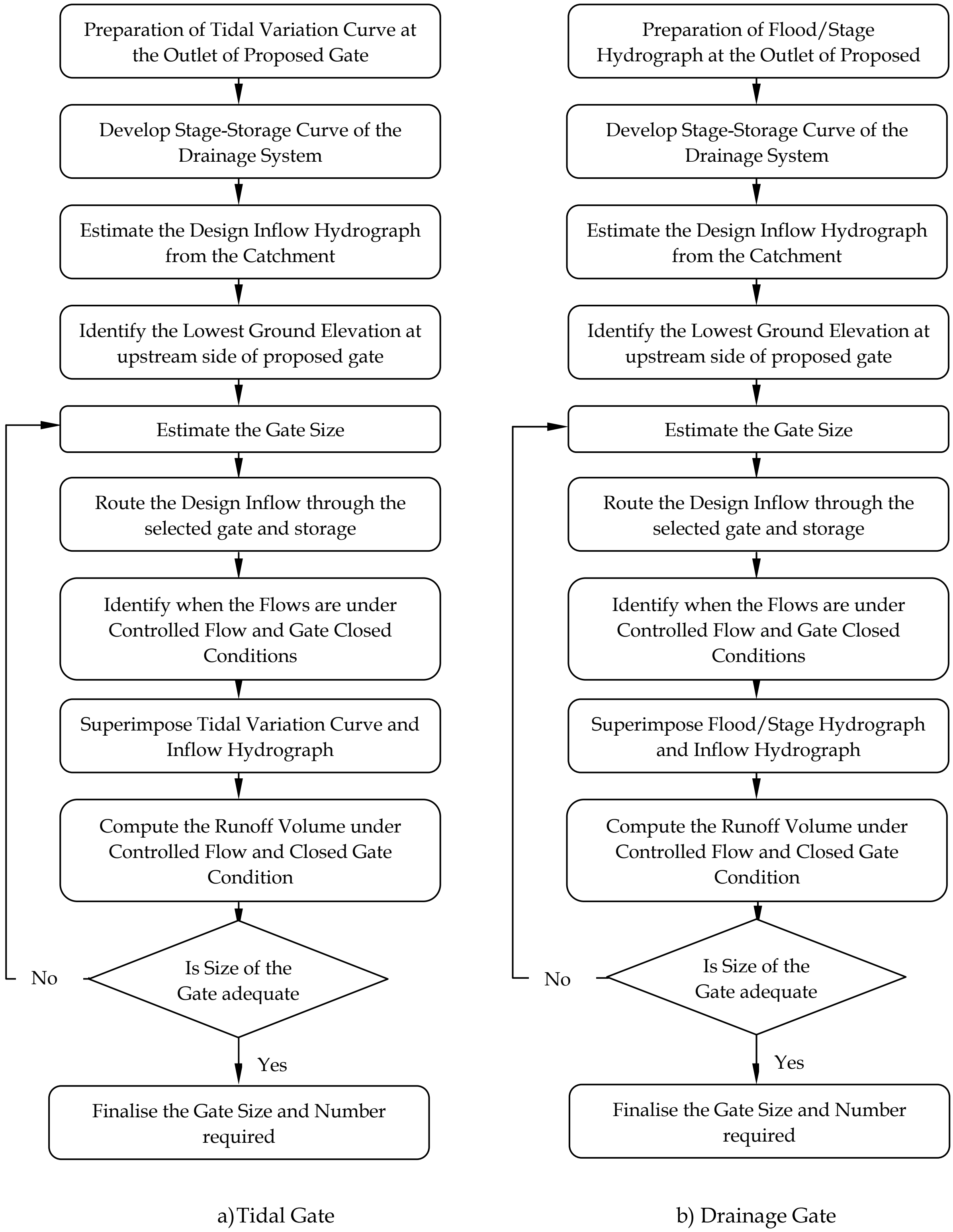


Figure 19.15: Flowcharts for Gate Design

The procedure for pump station design is illustrated in Figure 19.16. It comprises the following 11 steps.

Step 1: Inflow to Pump Station

Develop an inflow hydrograph representing the design storm as per Chapter 2.

Step 2: Estimate Pumping Rate, Volume of Storage, and Number of Pumps

A trial and success approach is usually necessary for estimating the pumping rates and storage required for a balanced design. The goal is to develop an economic balance between storage volume and pumping capacity. One approach to estimating storage volume was illustrated in Figure 19.14 and as given in Appendix 19.A. Once an estimated storage volume is determined, a storage facility can be estimated and a stage-storage relationship can be developed.

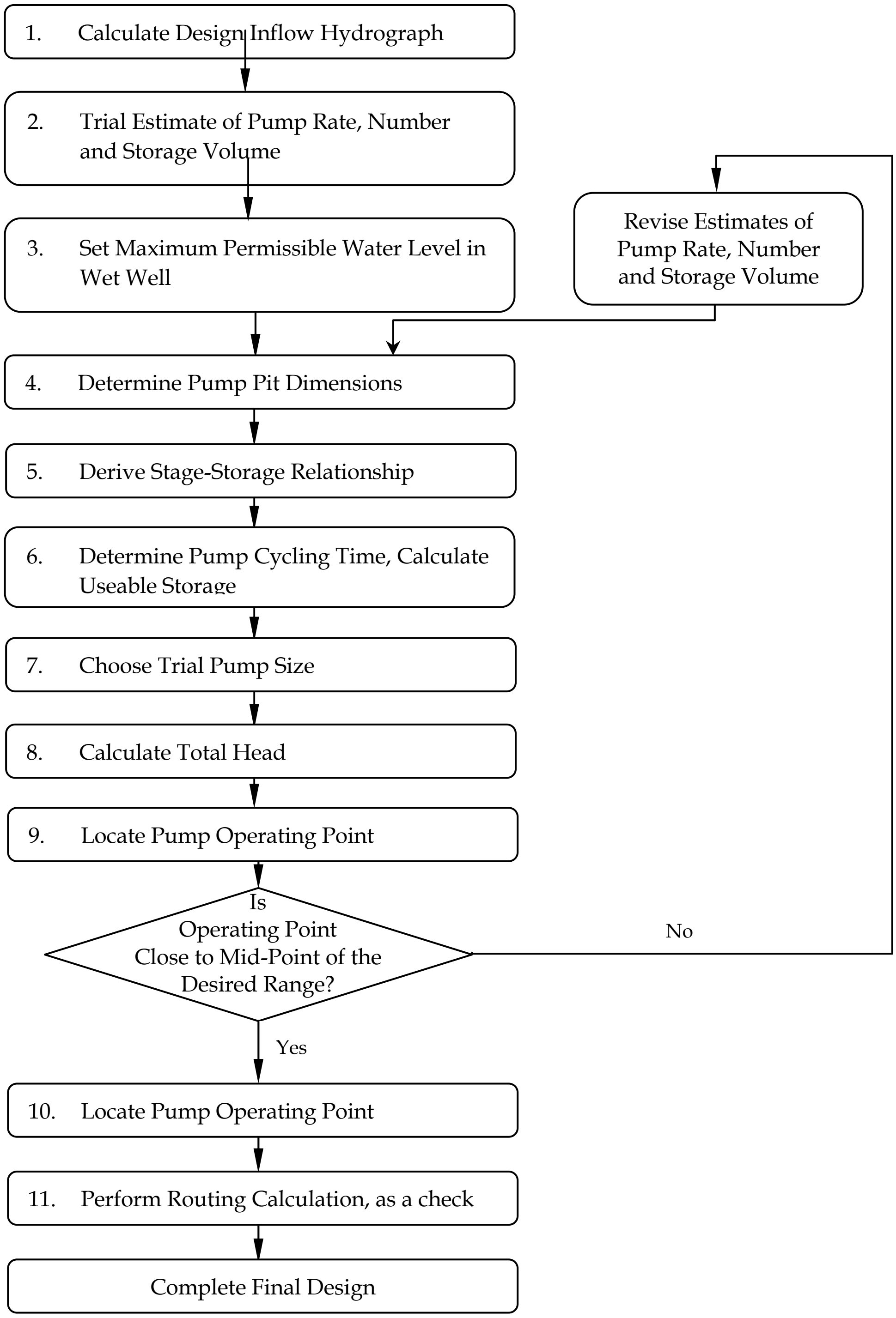


Figure 19.16: Flowchart for Pump Station Design

Step 3: Design High Water Level

The highest permissible water level should not be set higher than 0.3m to 0.6m below the finished pavement surface at the lowest pavement inlet. The lower the elevation the more conservatism is the design. Therefore a hydraulic gradient will be established and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient.

Step 4: Determine Pump Pit Dimensions

Actual dimensions of pump sump are normally recommended by the pump manufacturer to ensure conducive pumping condition and avoid damaging hydraulic problems such as vortices and swirl. The dimensions are usually determined by locating the selected number of pumps on a floor plan keeping in mind the need for clearances around pumps, valves, electrical panels and associated equipment that will be housed in the pump station building. As discussed, physical modelling maybe required in special conditions.

Step 5: Stage – Storage Relationship

Having roughly estimated the volume of storage required and a trial pumping rate, the configuration and elevations of the storage chamber can be initially set where the stage-storage relationship is established.

Step 6: Pump Cycling and Usable Storage

With the number of pumps set, the correct elevations must be chosen to turn each pump on and off to avoid rapid cycling. The pump cycling time can be related to usable volume as follows:

t = Time between starts
= Time to empty + time to fill usable storage volume V_t

when the inflow (I) is set to equal one half of the pump capacity (Q_P), then:

$$t = \frac{V_t}{(Q_p - I)} + \frac{V_t}{I} = \frac{V_t}{(Q_p - 0.5Q_p)} + \frac{V_t}{(0.5Q_p)} = \frac{4V_t}{Q_P} \tag{19.2}$$

$$t_{min} = \frac{4V_t}{Q_P}\left[\frac{1\,min}{60\,sec}\right] = \frac{V_t}{15Q_P} \tag{19.3}$$

where;

t = Time between starts (min);
V_t = Usable storage volume (m^3);
Q_p = Pump capacity (m^3/s); and
I = Inflow, I = ½ Q_P (m^3/s).

Generally, the minimum allowable cycling time, *t*, is designated by the pump manufacturer based on electric motor size. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development. However, Table 19.2 displays limits that may be used for estimating allowable cycle time during preliminary design.

Knowing the pumping rate and minimum cycling time (in minutes), the minimum necessary allowable storage, *V*, to achieve this time can be calculated from Equation 19.4.

$$V = 15Q_P t \tag{19.4}$$

Usually, the first pump stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer. The first pump start elevation will be a distance, Δh, above H. The minimum allowable storage would be calculated by Equation 19.4. The elevation associated with this volume in the stage-storage curve would be the lowest turn-on elevation that should be allowed for the starting point of the first pump. The second and subsequent pump start elevations will be determined by plotting the pump performance on the mass inflow curve. This distance between pump starts may be in the range of 0.3 to 1.0 metres for stations with a small amount of storage and 75mm to 150mm for larger storage configurations.

Table 19.2: Pump Cycle Time Limits

Motor Power (kW)	Cycling Time (Minutes)
0 – 11	5
15 – 22	6.5
26 – 45	8
49 – 75	10
112 - 149	13

Step 7: Trial Pumps and Pump Station Piping

The designer should study various manufacturers' literature in order to establish a reasonable balance between total dynamic head, discharge, efficiency, and energy requirements. This study will also give the designer a good indication of discharge piping needed since pumps will have a specific discharge pipe size. Each pump considered will have a unique performance curve that has been developed by the manufacturer. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. Figure 19.12 demonstrates a typical pump performance curve. Any point on an individual performance curve identifies the performance of a pump for specific Total Dynamic Head (TDH) that exists in the system. It also identifies the horsepower required and the efficiency of operation of pump (Figure 19.12). The designer must make certain that the design point be as close the eye as possible for optimum efficiency.

Step 8: Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances such as elbows and valves designated, total dynamic head for the discharge system can be calculated using Equation 19.1.

Step 9: Pump Design Operating Point

Using methods described in the previous step, the Total Dynamic Head of the outlet system can be calculated for a specific static head and various discharges. These TDH'S are then plotted vs. discharge. The plot is called a system head curve. A system head curve (Figure 19.13) is a graphical representation of total dynamic head plotted against discharge Q for the entire pumping and discharge system. The required operating point of the pump is given by the intersection of the system curve and the pump curve. The design operating point determined above should correlate with an elevation at about the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will thus operate in the best efficiency range.

Step 10: Power Requirements

To select the proper size pump motor, pump efficiency should be analyzed. Pump efficiency is defined as the ratio of pump power output to the power input applied to the pump. The efficiency of the pump is then expressed as:

$$\text{Efficiency, } \varepsilon = \frac{pump\ output\ power}{pump\ motor\ rating} \quad (19.5)$$

The pump power output can be determined as:

$$P = \frac{\gamma QH}{1000} \quad (19.6)$$

where;

P = Power output from the pump (kW);

γ = Specific weight of water (9800 N/m^3);

Q = Pump flow rate (m^3/s); and

H = Pump head (m).

Combining Equation 19.6 with the definition of efficiency and changing some of the units, the power put into the pump shaft can be expressed as:

$$P_m = \frac{Q.H}{6122\varepsilon} \quad (19.7)$$

where;

P_m = Pump motor rating (kW);

ε = Efficiency (%)

Each pump motor has a service factor with typical range between 1.15 and 1.25. This indicates that a motor can produce 1.15 or 1.25 times the rated kW for short periods of time and if operating above these limits will burn out the electric motor almost immediately.

Step 11: Storage Routing

The procedures described thus far will provide all the necessary dimensions, cycle times, appurtenances, etc. to complete a preliminary design for the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station by methods described in Chapter 2. In this way, the performance of the pump station can be observed at each hydrograph time increment and pump station design evaluated. Then, if necessary, the design can be “fine-tuned”.

REFERENCES

1. Baumgardner R.H. (1981). *Estimating Required Storage to Reduce Peak Pumping Rates at Stormwater Pumping Stations* , Federal Highway Administration Workshop Paper, Office of Engineering, Washington DC.

2. Baumgardner R.H. (1982). *Hydraulic Design of Stormwater Pumping Stations Considering Storage*, Federal Highway Administration Workshop Paper, Office of Engineering, Washington DC.

3. Beecham S.C. and Sablatnig S.J. (1994). *Hydraulic Modelling of Trash Racks*, Proceedings, 1994 International Conference on Hydraulics in Civil Engineering, IEAust. Nat. Conf. Pub. No. 94/1, 15th-17th February, Brisbane.

4. Jack.L (2001). *Hydraulic of Gates and Valves: in Free Surface Flow and Submerged Outlets*, published by Thomas Telford

5. Hydraulic Institute (1998), *Centrifugal/Vertical Pump Intake Design,* published by (ANSI/HI 9.8)

6. Lever W.F. (1982). *Manual for Highway Storm Water Pumping Stations*, Volume 1 & 2, FHWA-IP-82-17, U.S. Department of Transportation, FHWA, Washington DC.

7. Paulo C. F. E. (2004). *Design of Hydraulic Gates*, published by Taylor & Francis

8. Peter N. S. (2001). *Highway Stormwater Pump Station Design - Hydraulic Engineering Circular 24*, FHWA-NHI-01-007, U.S. Department of Transportation, Federal Highway Administration

9. Thorn R.B. (1966b). *Automatic Radial Sluice Gates and other Types of Sluice Gates*, In River Engineering and Water Conservation Works, R.B. Thorn (ed.), Butterworths, London.

10. U.S. Army Corps of Engineers (1994). *Mechanical And Electrical Design Of Pumping Stations*, Department of Army, EM 1110-2-3105

11. Wong W.S. (1999). *Urban Pumping Drainage Design and Operation – A Case Study*, Proc. 8th International Conference on Urban Storm Drainage (ICUSD), Sydney, Australia

APPENDIX 19.A STORAGE AND PUMP CAPACITY ESTIMATION

Stormwater storage can reduce the require peak capacity of a pump station. Selection of a peak pumping capacity is a trial and error process that considers the inflow hydrograph, available storage, and possible pump discharge rates.

The approximate method includes adjusting the inflow hydrograph to an equivalent triangular hydrograph, as shown in Figure 19.A1. An estimate of the storage required to reduce the peak pumping rate to a desired value can be found by assigning a peak pumping rate and plotting it as horizontal, as shown in Figure 19.A2.

The area of the triangular hydrograph above the peak pumping rate represents an estimate of the storage volume required. This assumes that storage below the last pump-on elevation will not affect the design. The effect of this storage on final design can be considered using the computer program, or a mass curve routing procedure as presented in next section. The required storage can be estimated by the equation:

$$\frac{V_s}{V_t} = \left(\frac{\Delta Q}{Q_p}\right)^2 \tag{19.A1}$$

where,

V_s = Required storage volume, (m^3);
V_t = Volume of triangular inflow hydrograph,(m^3);
ΔQ = Peak flow reduction, (cumec); and
Q_P = Peak flow of triangular inflow hydrograph, (cumec).

A graphical presentation of the relationship in Equation 19.8 is shown in Figure 19.A3. By selecting a peak reduction ratio ($\Delta Q/Q_p$), the storage ratio (V_s/V_t) can be obtained directly. When the inflow hydrograph volume (V_t) is known, the storage required is estimated as the product of the storage ratio and the inflow hydrograph volume.

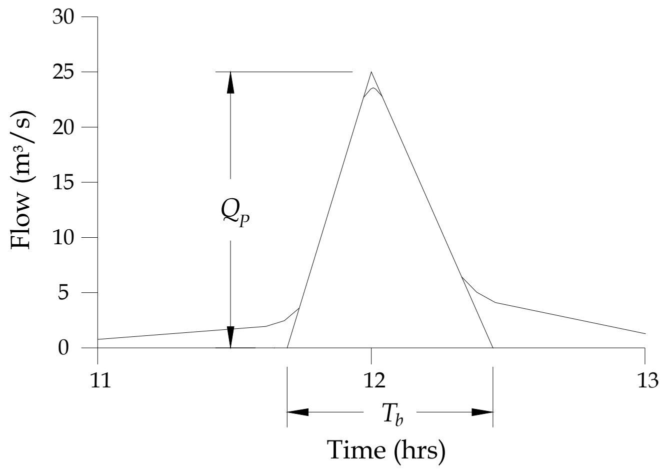


Figure 19.A1: Triangular Approximation of Inflow Hydrograph (Baumgardner, 1982)

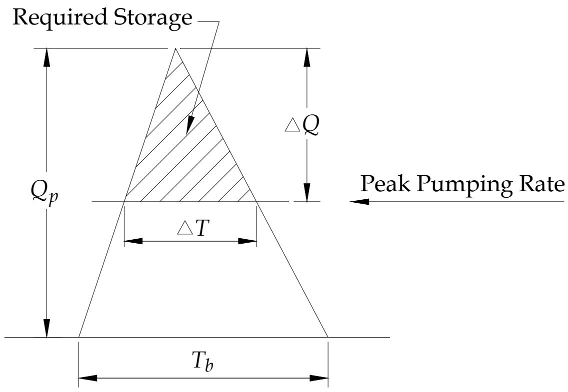


Figure 19.A2: Estimation of Required Storage Based on a Selected Peak Pumping Rate (Baumgardner, 1982)

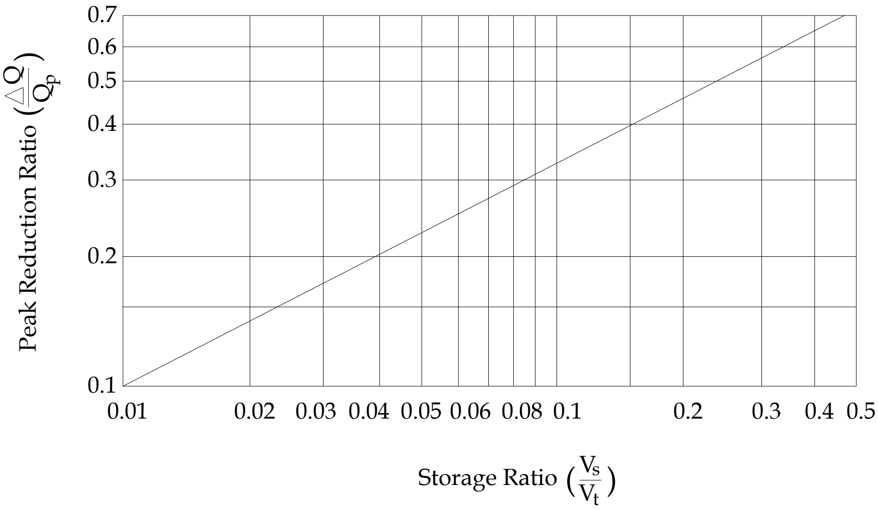


Figure 19.A3: Relationship between Peak Reduction Ratio and Storage Ratio : $Vs/V_T = (\Delta Q/Q_p)^2$ (Baumgardner, 1981)

APPENDIX 19.B EXAMPLE - PUMPING STATION DESIGN

Problem:

The catchment runoff of Kampung Pasir Baru is drained gravitationally through a gated outlet into Sungai Klang, 500 m upstream of the Jalan Klang Lama, Kuala Lumpur. Although the drainage system of the catchment has been improved, frequent flooding still occurs whenever heavy downpour happens that coincide with high flood level at Sungai Klang that impede the local runoff from discharging into the river. The catchment area at the outlet to Sungai Klang is approximately 57.8ha as shown in Figure 19.B1.

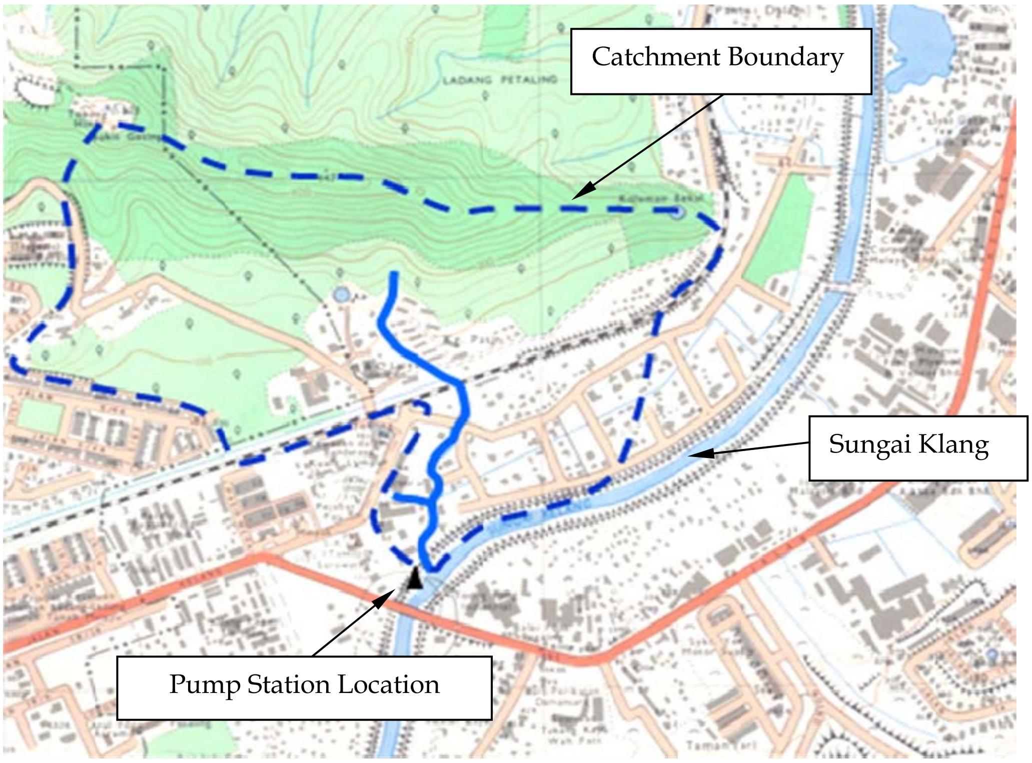


Figure 19.B1: Kampung Pasir Baru Pump Station

Solution:

The effective measure is to install pumps with storage to discharge the runoff from the Kampung Pasir Baru catchment into Sungai Klang when the water level at Sungai Klang is high.

Reference	Calculation		Output
Relevant Layout Plan	Catchment Area		
	Catchment area of Kampung Pasir Baru at outlet to Sg. Klang	=	57.8ha
	Design Protection Level		
	Design protection level	=	5-yr ARI
	Checking	=	100-yr ARI

Reference	Calculation		Output
Chapter 2	<u>Inflow Hydrographs</u>		
	The design discharges estimation adopted using Rational Hydrograph Method		
	The computed average runoff coefficient	=	0.625
	The time of concentration for the catchment at the outlet point	=	48min
	The hydrographs for the longer duration than the time of concentration are also considered because for the sizing of the pumps and storage the total volume of runoff is also an important design parameter to be considered.		
Chapter 2	The rainfall intensity adopted is from the Revised Hydrological Procedure No.1 (2010) for Station 3015001		
	The computed peak discharges and inflow hydrograph using Rational Hydrograph Method	=	Table 19.B1
	<u>Estimate Storage Volume and Stage – Storage Relationship</u>		
Equation 19.A1	Total available storage from elevation 16.5 mRL to 19.5 mRL (provide 0.5 m freeboard with platform level at 20.0 mRL) is about.	= =	Figure 19.B2 2,800m^3
	The Stage-Storage for the pump sump storage and channel storage	= =	Table 19.B2 Figure 19.B3
	<u>Estimate Pumping Rate and Number of Pumps</u>		
	Runoff volume for 48 min rainstorm Runoff peak for 48 min rainstorm	= =	25,468m^3 8.84m^3/s
	The estimated required pumping rate	= =	5.9m^3/s Figure 19.B4
	From this analysis, provides	=	3 x 2m^3/s submersible axial pumps

Reference	Calculation	Output
	<u>Estimate the Total Dynamic Head</u>	
Equation 19.1	Assuming that the lowest and highest levels the pumps need to pump are 16.5 mRL and 19.5 mRL respectively and the weir level need to pump over is 21.5 mRL as given in Figure 19.B3, hence the maximum and minimum static heads are as follows :	
	$H_s(max)$ = (21.5 -16.5)m =	5m
	$H_s(min)$ = (21.5 -19.5)m =	2m
	It is assumed that the diameter of the column pipe, D is 1 m and for 2 m/s discharge the velocity head is :	
	Velocity (v) = Q/A = $2/(D^2 \pi /4)$ =	2.55m/s
	h_v = $v^2/2g$ = $2.55^2/2/9.81$ =	0.33m
	The friction head is approximated using the Hazen-William equation. It is assumed that the column pipe is 4 m length and for steel pipe, the friction factor C is 100 :	
	h_f = $6.83\ v^{1.85}\ L\ /(C^{1.85}\ D^{1.165})$ =	0.03m
	There is no fittings for the arrangement and hence, h_L is zero. With all the above, the maximum and minimum Total Dynamic Head can be computed as follows :	
	TDH (max) = 5m + 0.33m + 0.03m =	5.36m
	TDH (min) = 2m + 0.33m + 0.03m =	2.36m
	<u>Selection of Pump Performance Curve</u>	
	With the pumping rate and TDH requirements, the pump performance curve can be selected and the pump chosen that suits the requirements. =	Figure 19.B5
	<u>Pump Start – Stop Levels and Stage – Pump Discharge Relationship</u>	
	The design start – stop levels for the pump =	Table 19.B3
	=	Figure 19.B2
	From the start – stop levels of the pumps, pump performance curve and TDH, the stage – pump discharge relationship	
	1. Water level rising in the pump sump storage =	Table 19.B4
	2. Water Level dropping in the pump sump storage =	Table 19.B5
	The stage – pump discharge relationships plot =	Figure 19.B6

Reference	Calculation		Output
	Routing of Pump Sump Storage		
Chapter 2	With the Stage – Storage and Stage – Discharge Relationships, routing procedure similar to pond routing as elaborated in Chapter 2 is engaged to determine the water level in the pump sump storage. The only difference is that as mentioned earlier, there are two different Stage – Pump Discharge Relationships for the water level rising and dropping conditions due to the different start – stop levels of the pumps.		
	The routing procedure to obtain the pump discharge and water level in the pump sump storage for the 5-year ARI 48 min inflow hydrograph	=	Table 19.B6
	and the maximum water level in the pump sump storage (lower than the design 19.50 mRL)	=	18.80 mRL
	The routing is also plotted:		
	1. 5-year ARI	=	Figure 19.B7
	2. 100-year ARI	=	Figure 19.B8
	Routings are also carried out for other rainstorm duration for 5-year ARI and also for the 100-year ARI events	=	Table 19.B7
	Layout Plan and Profile		
	Based on the available land, the layout plan and the profile of the pump	=	Figure 19.B9
	station are conceptualised	=	Figure 19.B10
	Discussions		
	From the pump routing analysis, the pumps provided are sufficient to meet the 5-year ARI flood protection criterion for Kampaung Pasir Baru. The checks done on 100-year ARI review that the flood level is 0.7 m above the platform level of 20 mRL and the maximum inundation period is about 50 min. It can be deduced from this analysis that it is sufficient to design for the pump station to cater for 5 to 10-year ARI protection where even during 100-year ARI rainstorm, the flooding will not be severe. With this, the capital cost can be optimised.		

Table 19.B1: Inflow Hydrographs using Rational Hydrograph Method

Rainfall Duration (min)	5-year ARI		100-year ARI	
	Rainfall Intensity (mm/hr)	Peak Discharge (m^3/s)	Rainfall Intensity (mm/hr)	Peak Discharge (m^3/s)
48	88	8.84	137	13.73
60	74	7.47	117	11.74
75	63	6.34	99	9.96
90	55	5.52	86	8.68
120	44	4.41	69	6.94

Table 19.B2: Stage – Storage Tabulation

Elevation (m)	Channel Storage (m^3)	Pump Sump Storage (m^3)	Total Storage (m^3)
16.00	0	0	0
16.50	0	0	0
16.75	0	111	111
17.00	0	222	222
17.25	0	333	333
17.50	0	444	444
17.75	23	555	578
18.00	94	666	760
18.25	211	777	988
18.50	375	888	1263
18.75	586	999	1585
19.00	844	1110	1954
19.25	1148	1221	2369
19.50	1500	1332	2832
19.75	1898	1443	3341
20.00	2344	1554	3898

Table 19.B3: Pump Controls and Operational Parameters

Pump No.	Pump Flow (m^3/s)	Pump-Start Elevation (mRL)	Pump-Stop Elevation (mRL)
1	2	17.0	16.5
2	2	17.5	17.0
3	2	18.0	17.5

Table 19.B4: Stage – Pump Discharge Curve Tabulation for Rising Water Level

Elevation (mRL)	TDH (m)	1st Pump (m^3/min)	2nd Pump (m^3/min)	3rd Pump (m^3/min)	Total (m^3/min)	Total (m^3/s)
16.00	5.86	0	0	0	0	0.00
16.50	5.36	0	0	0	0	0.00
16.75	5.11	0	0	0	0	0.00
16.95	4.91	0	0	0	0	0.00
17.00	4.86	122	0	0	122	2.03
17.25	4.61	124	0	0	124	2.07
17.45	4.41	126	0	0	126	2.11
17.50	4.36	127	127	0	254	4.23
17.75	4.11	129	129	0	258	4.30
17.95	3.91	131	131	0	261	4.36
18.00	3.86	131	131	131	393	6.56
18.25	3.61	133	133	133	399	6.66
18.50	3.36	135	135	135	405	6.76
18.75	3.11	137	137	137	411	6.85
19.00	2.86	139	139	139	417	6.96
19.25	2.61	141	141	141	424	7.07
19.50	2.36	144	144	144	431	7.18
19.75	2.11	146	146	146	439	7.31
20.00	1.86	149	149	149	447	7.46

Table 19.B5: Stage – Pump Discharge Curve Tabulation for Dropping Water Level

Elevation (mRL)	TDH (m)	1st Pump (m^3/min)	2nd Pump (m^3/min)	3rd Pump (m^3/min)	Total (m^3/min)	Total (m^3/s)
16.00	5.86	0	0	0	0	0.00
16.45	5.41	0	0	0	0	0.00
16.50	5.36	115	0	0	115	1.92
16.75	5.11	119	0	0	119	1.98
16.95	4.91	121	0	0	121	2.02
17.00	4.86	122	122	0	243	4.05
17.25	4.61	124	124	0	249	4.14
17.45	4.41	126	126	0	253	4.21
17.50	4.36	127	127	127	380	6.34
17.75	4.11	129	129	129	387	6.45
18.00	3.86	131	131	131	393	6.56
18.25	3.61	133	133	133	399	6.66
18.50	3.36	135	135	135	405	6.76
18.75	3.11	137	137	137	411	6.85
19.00	2.86	139	139	139	417	6.96
19.25	2.61	141	141	141	424	7.07
19.50	2.36	144	144	144	431	7.18
19.75	2.11	146	146	146	439	7.31
20.00	1.86	149	149	149	447	7.46

Table 19.B6: Routing Procedure for 5-year ARI, 48 minute Inflow Hydrograph

Time, t (mins)	I (m^3/s)	$(I_1+I_2)/2$ (m^3/s)	$S_1/\Delta t+Q_1/2$ (m^3/s)	Q_1 (m^3/s)	$S_2/\Delta t+Q_2/2$ (m^3/s)	Q_2 (m^3/s)	WL (mRL)	WL (mRL)	Discharge, Q (m^3/s)	Storage, S (m^3)	$S/\Delta t+Q/2$ (m^3/s)
0	0.00				0.00	0.00	16.50	16.500	0.000	0.00	0.00
2	0.37	0.18	0.00	0.00	0.18	0.00	16.55	16.750	0.000	111.00	0.93
4	0.74	0.56	0.18	0.00	0.74	0.00	16.70	16.950	0.000	199.80	1.67
6	1.11	0.92	0.74	0.00	1.66	0.00	16.95	17.000	2.027	222.00	2.86
8	1.47	1.29	1.66	0.00	2.95	2.03	17.02	17.250	2.072	333.00	3.81
10	1.84	1.66	2.95	2.03	2.57	1.54	16.99	17.450	2.105	421.80	4.57
12	2.21	2.03	2.57	1.54	3.06	2.04	17.05	17.500	4.227	444.00	5.81
14	2.58	2.40	3.06	2.04	3.42	2.05	17.15	17.750	4.302	578.44	6.97
16	2.95	2.77	3.42	2.05	4.13	2.09	17.33	17.950	4.358	723.49	8.21
18	3.32	3.13	4.13	2.09	5.18	3.14	17.47	18.000	6.557	759.75	9.61
20	3.68	3.50	5.18	3.14	5.53	3.75	17.49	18.250	6.658	987.94	11.56
22	4.05	3.87	5.53	3.75	5.65	3.95	17.49	18.500	6.756	1263.00	13.90
24	4.42	4.24	5.65	3.95	5.94	4.23	17.53	18.750	6.855	1584.94	16.64
26	4.79	4.61	5.94	4.23	6.31	4.26	17.61	19.000	6.957	1953.75	19.76
28	5.16	4.98	6.31	4.26	7.02	4.30	17.76	19.250	7.066	2369.44	23.28
30	5.53	5.35	7.02	4.30	8.06	4.35	17.93	19.500	7.184	2832.00	27.19
32	5.90	5.71	8.06	4.35	9.42	6.26	17.99	19.750	7.313	3341.44	31.50
34	6.26	6.08	9.42	6.26	9.24	5.97	17.99	20.000	7.456	3897.75	36.21
36	6.63	6.45	9.24	5.97	9.71	6.56	18.01	For Rising Water Level			
38	7.00	6.82	9.71	6.56	9.97	6.58	18.05				
40	7.37	7.19	9.97	6.58	10.58	6.61	18.12				
42	7.74	7.56	10.58	6.61	11.52	6.66	18.24				
44	8.11	7.92	11.52	6.66	12.79	6.71	18.38				
46	8.47	8.29	12.79	6.71	14.37	6.77	18.54				
48	8.84	8.66	14.37	6.77	16.26	6.84	18.72				
50	8.47	8.66	16.26	6.84	18.07	6.90	18.87	16.450	0.000	0.00	0.00
52	8.11	8.29	18.07	6.90	19.46	6.95	18.98	16.500	1.920	0.00	0.96
54	7.74	7.92	19.46	6.95	20.44	6.98	19.05	16.750	1.977	111.00	1.91
56	7.37	7.55	20.44	6.98	21.01	7.00	19.09	16.950	2.018	199.80	2.67
58	7.00	7.18	21.01	7.00	21.20	7.00	19.10	17.000	4.054	222.00	3.88
60	6.63	6.82	21.20	7.00	21.01	7.00	19.09	17.250	4.145	333.00	4.85
62	6.26	6.45	21.01	7.00	20.47	6.98	19.05	17.450	4.211	421.80	5.62
64	5.90	6.08	20.47	6.98	19.57	6.95	18.98	17.500	6.340	444.00	6.87
66	5.53	5.71	19.57	6.95	18.33	6.91	18.89	17.750	6.452	578.44	8.05
68	5.16	5.34	18.33	6.91	16.76	6.86	18.76	18.000	6.557	759.75	9.61
70	4.79	4.97	16.76	6.86	14.88	6.79	18.59	18.250	6.658	987.94	11.56
72	4.42	4.61	14.88	6.79	12.69	6.70	18.37	18.500	6.756	1263.00	13.90
74	4.05	4.24	12.69	6.70	10.22	6.59	18.08	18.750	6.855	1584.94	16.64
76	3.68	3.87	10.22	6.59	7.50	6.40	17.63	19.000	6.957	1953.75	19.76
78	3.32	3.50	7.50	6.40	4.60	4.12	17.19	19.250	7.066	2369.44	23.28
80	2.95	3.13	4.60	4.12	3.61	3.61	16.99	19.500	7.184	2832.00	27.19
82	2.58	2.76	3.61	3.61	2.77	2.18	16.95	19.750	7.313	3341.44	31.50
84	2.21	2.39	2.77	2.18	2.98	2.54	16.96	20.000	7.456	3897.75	36.21
86	1.84	2.03	2.98	2.54	2.47	2.01	16.90	For Falling Water Level			
88	1.47	1.66	2.47	2.01	2.12	1.99	16.80				
90	1.11	1.29	2.12	1.99	1.42	1.95	16.62				
92	0.74	0.92	1.42	1.95	0.39	0.79	16.47				
94	0.37	0.55	0.39	0.79	0.16	0.32	16.46				
96	0.00	0.19	0.16	0.32	0.03	0.03	16.45				
98	0.00	0.00	0.03	0.03	0.00	0.00	16.45				
100	0.00	0.00	0.00	0.00	0.00	0.00	16.45				

Table 19.B7: Summary of the Routing Results

Rainfall Duration (min)	5-year ARI		100-year ARI	
	Max. Pump Sump Storage WL (mRL)	Period Exceeded 20 mRL (min)	Max. Pump Sump Storage WL (mRL)	Period Exceeded 20 mRL (min)
48	19.10	0	20.71	50
60	18.80	0	20.43	46
75	18.00	0	20.24	38
90	17.98	0	20.01	4

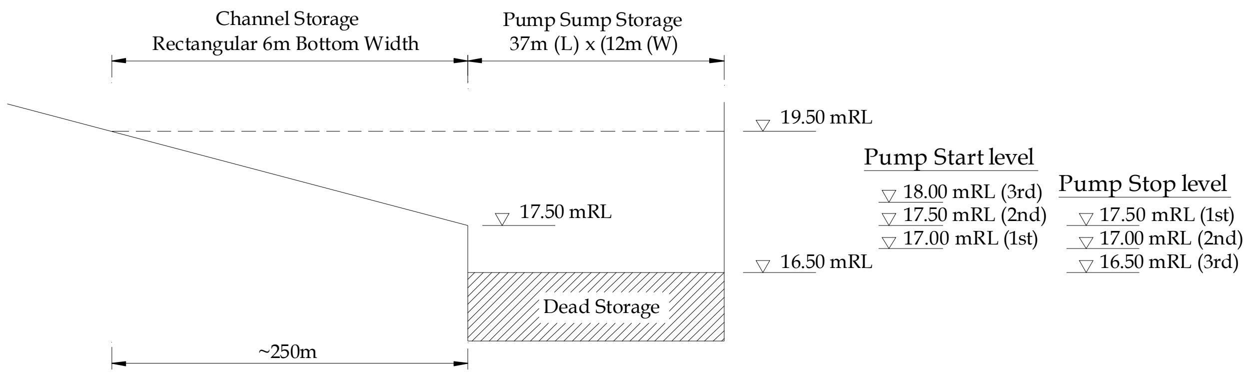


Figure 19.B2: Storage Sketch and Pump Start-Stop Levels

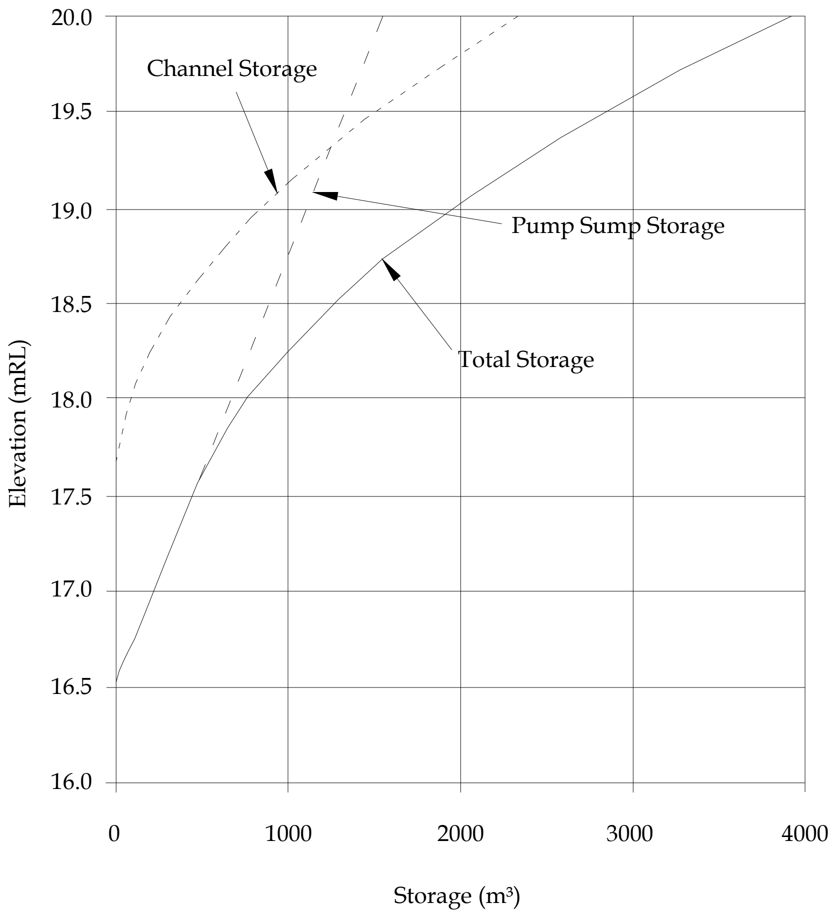


Figure 19.B3: Stage – Storage Curves

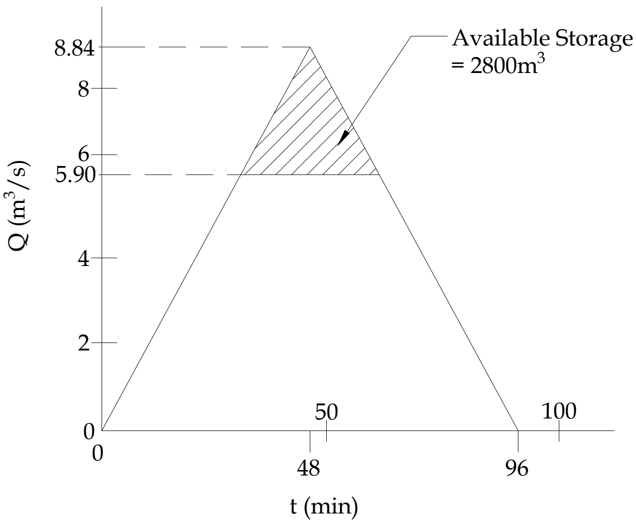
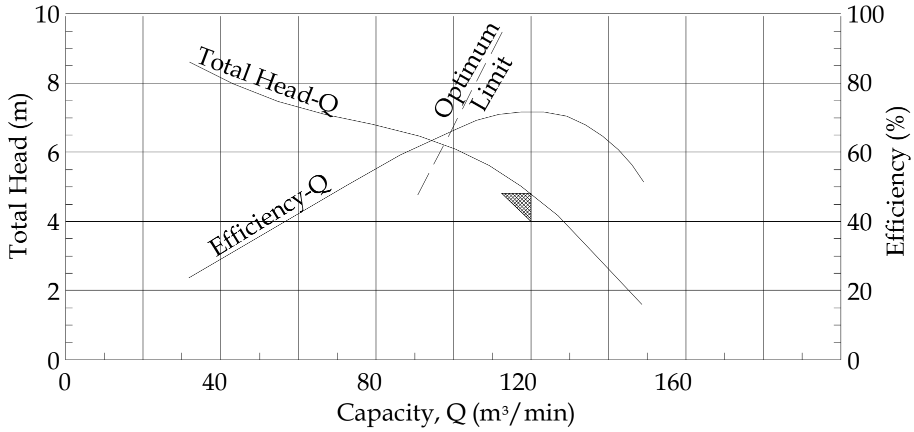


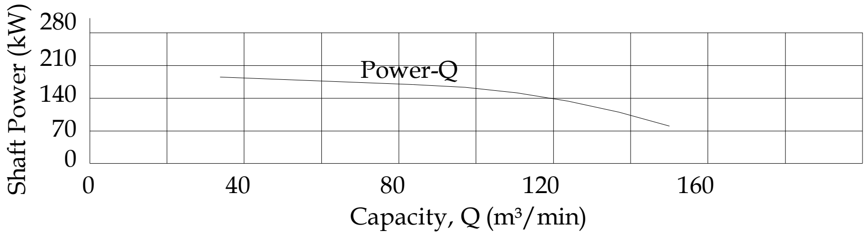
Figure 19.B4: Estimated Pumping Rate from Inflow Hydrograph and Available Storage

1. General			
Customer		Item No. (ID No.)	
Location		Model No.	1000KPL150 10T3
Service		Quantity	set
2. Design Data			
Motor		**Pump**	
Power Supply	3 ϕ 380 V, 50 Hz	Discharge Diameter	1000 mm
Rated Power	200 HP 150 kW 10 P	Total Head	5 m
Rated Current	324 A	Capacity	120 m^3/min
Type	Squirrel Cage		- USGL/min
Insulation Class	F	Efficiency	74%
Starting Method	Reactor	Revolution	580 R.P.M
Curve no.		Solid Size	

a) Motor and Pump Design Data



b) Head and Efficiency – Capacity Curves



c) Power – Capacity Curve

Figure 19.B5: Selected Pump Performance Curve Sheet from Manufacturer

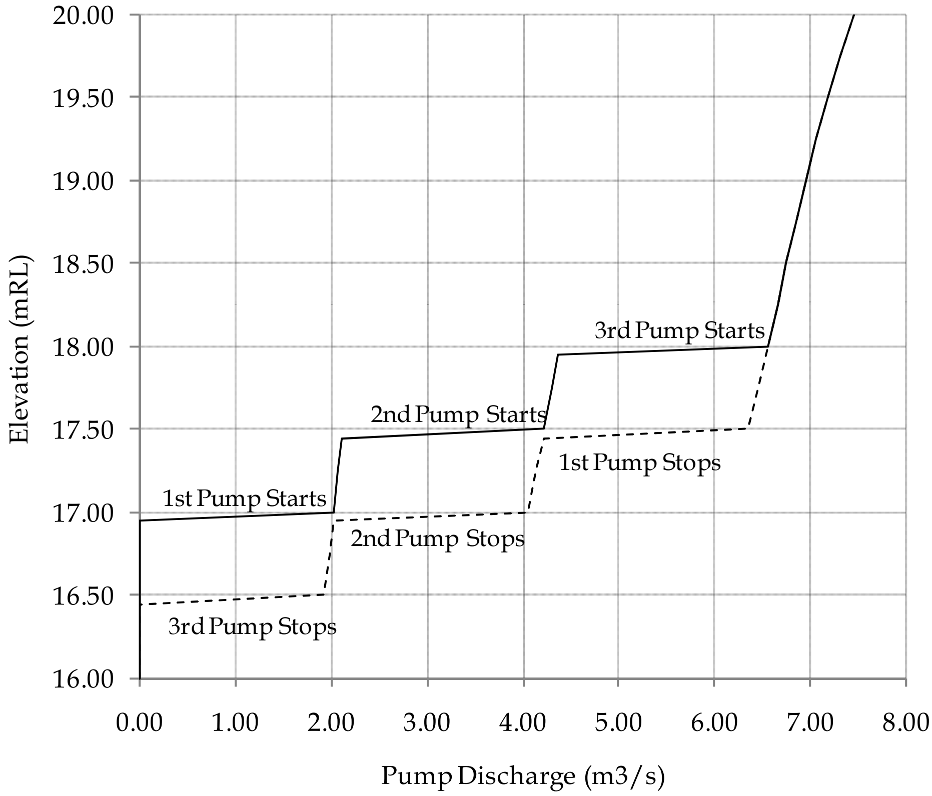


Figure 19.B6 Stage – Pump Discharge Curves

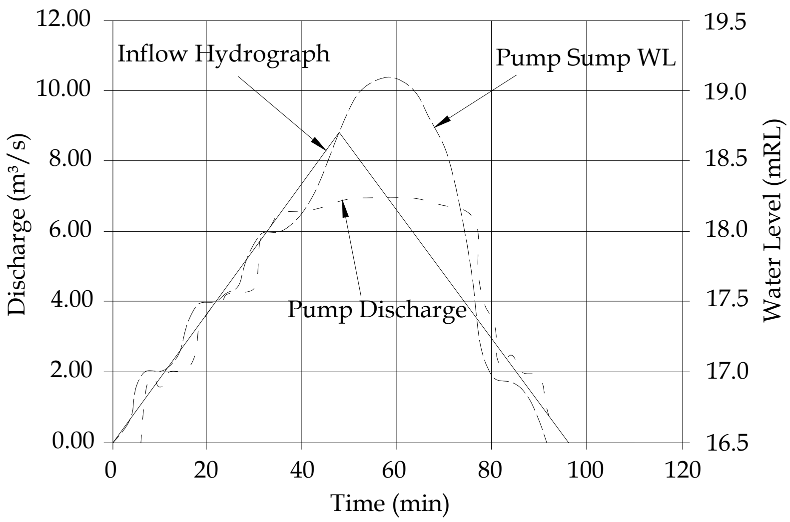


Figure 19.B7: Routing Results for the 5-year ARI 48 min Inflow Hydrograph

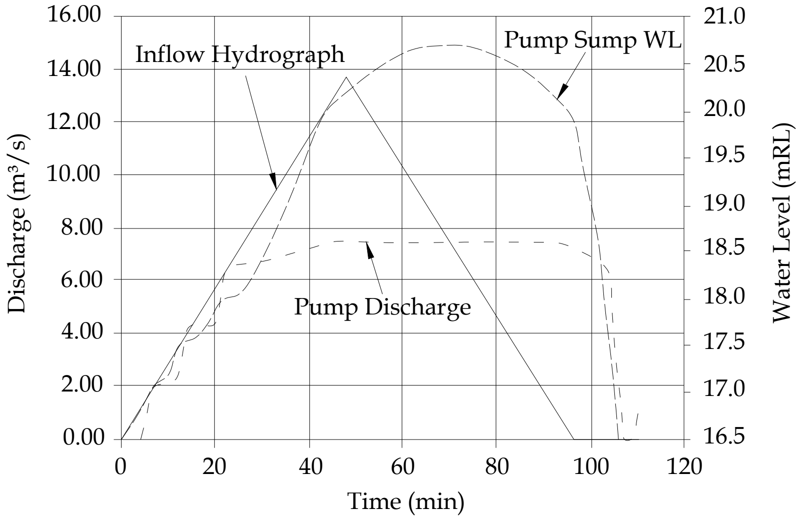
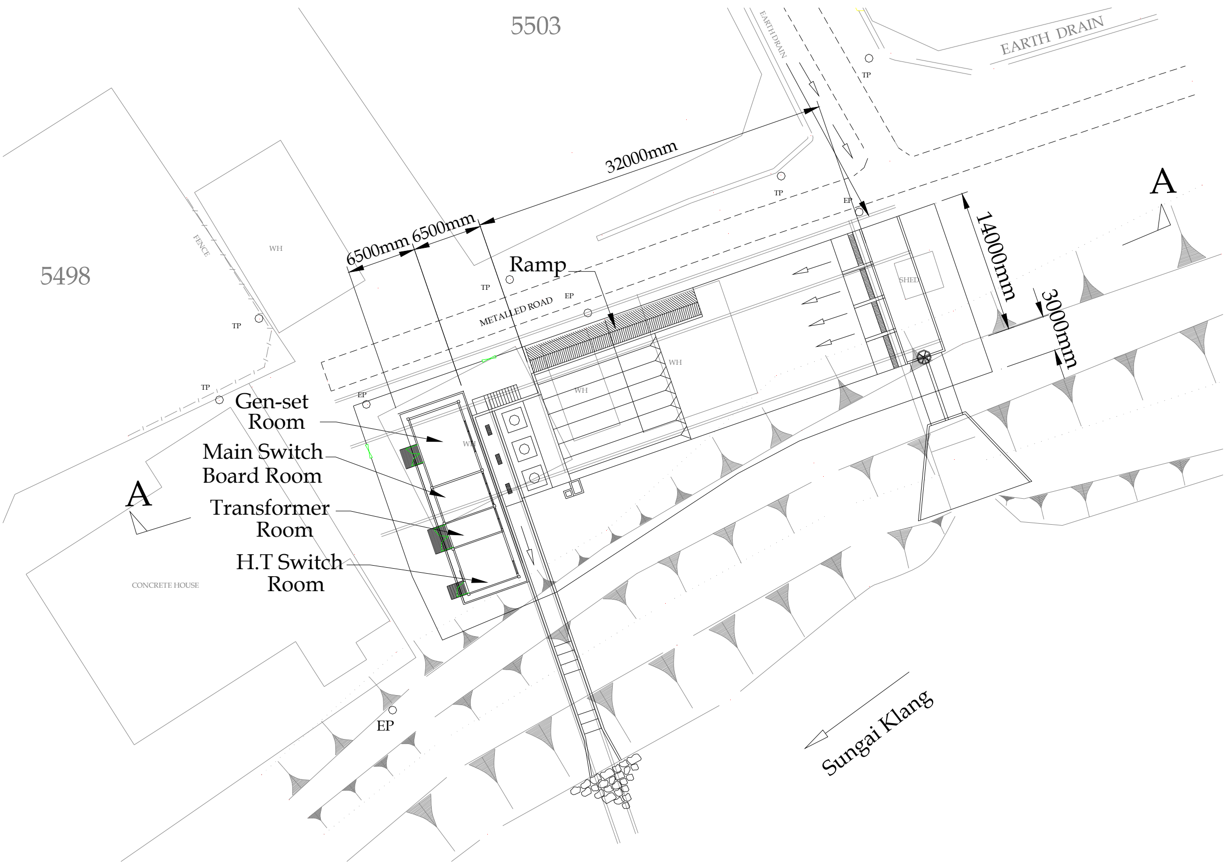
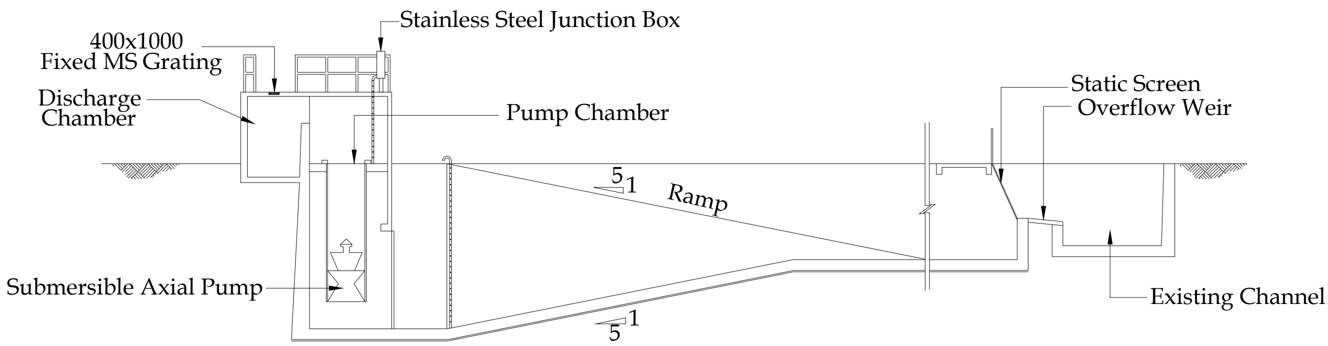


Figure 19.B8: Routing Results for the 100-year ARI 48 min Inflow Hydrograph



a) Plan



b) Longitudinal Profile (A-A)

Figure 19.B9: Layout of Pump Station

APPENDIX 19.C EXAMPLE - TIDAL GATE DESIGN

Problem:

The problem of flooding in the low-lying areas upstream of the Jalan Parit Bakri drainage outlet at Muar town (see Figure 19.C1) during exceptional high tide occurs frequently every year. A tidal gate is proposed to be constructed to solve the flooding problems. The upstream area is fully urbanised and considered to be the most important area of the town and drains to an outlet, which is situated in the town centre close to the main market. The peak discharge from the 5 year ARI critical duration design storm is 17.0m^3/s. Time of concentration at the proposed gate site is about 60 minute. The summarised design data for the area is given below.

Figure 19.C1: Location Plan of the Tidal Gate

Available maximum tide cycle data at spring tide from the observed tidal cycle, as shown in Figure 19.C2, are as follows:

(a) High Tide Level = 1.5m LSD

(b) Low Tide Level = -1.0m LSD

(c) Invert Level at Downstream of the Gate = -1.07m LSD

(d) Average Ground Level = 1.2m LSD

(e) Upstream Bed Level = -0.9m LSD

(f) Downstream Bed Level = -1.2m LSD

(g) Average Slope of the Drain = 1 in 1500

(h) Width of the Rectangular Drains = 5m

(i) Average Depth of the Drains = 2m

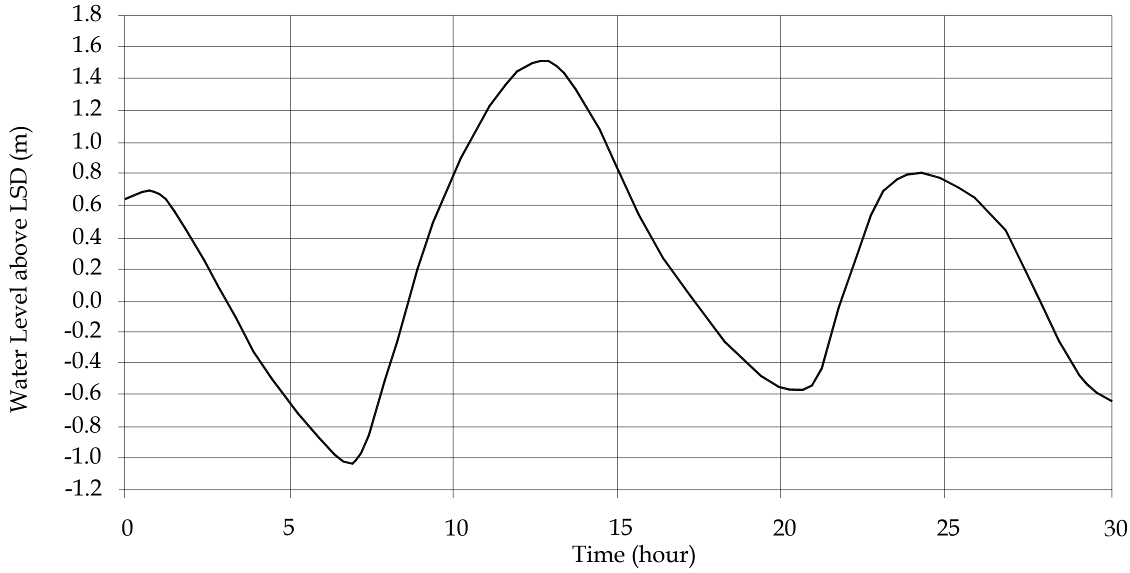


Figure 19.C2: Variation of Tide Levels at Proposed Outlet Gate

Check adequacy of the available storage capacity of the drainage system in the case when a high tide coincides with a heavy rainfall. If the storage capacity is inadequate design the flap tidal gate including pumping facilities if required.

<u>Solution:</u>

Reference	Calculation		Output
	Step - 1: Prepare the Tidal Variation Curve at the Outlet of Proposed Gate		
	Data for hourly tidal levels are obtained for Muar and the variation.	=	Figure 19.C2
	Step - 2: Develop Stage-Storage Curve of the Drainage System		
Chapter 7	The stage-storage relationship is developed based on the double-end area method.	=	Figure 19.C3
Chapter 2	Step - 3: Estimate the Design Inflow Hydrograph from the Catchment		
	The runoff inflow hydrograph	=	Figure 19.C4
	The critical storm in the catchment of 60 minute duration.	=	'abc'
	However, this will not be the critical storm when storage is considered during the tide period when the tidal gate is closed. Hydrograph assumed for the trial hydrograph to give the critical storage situation.	=	'abde'
	(Note: In practice, a number of different inflow hydrographs should be trialled.)		
	Step - 4: Identify the Lowest Ground Elevation at Upstream of Proposed Gate		
	Lowest ground level at the tidal gate site	=	1.2m LSD

Reference	Calculation	Output
	Step – 5: Estimate the Gate Size	
	Assume gate size. =	3.5m wide x 2.0m high
	Step - 6: Route the Design Inflow through the Selected Gate	
Chapter 20	Assuming culvert properties during controlled flow condition discharge through per unit width of the gate is,	
	$Q_c = C_d A_o \sqrt{2g H_o}$	
	$= 0.85x2x(2x9.81)^{0.5}xH_o^{0.5}$ cumecs / m width of gate =	$7.53xH_o^{0.5}$ m³/s/m
	Maximum flow through the gate will occur during weir flow condition and when the differential head between the upstream and downstream water level will be maximum. As such, maximum discharge per metre width is,	
	$Q_m = C_{SP} B H_p^{1.5}$	
	$= 1.25xH_p^{1.5}$ (here H = average GL – U/S Bed Level) =	$3.80\ m^3/s/m$
	$= 1.25x2.1^{1.5}$	
	So the differential head at which maximum discharge will occur at controlled flow condition is =	$7.53xH_o^{0.5} = 3.80$
	Thus, H_o =	0.25m
	As such with respect to the lowest safe ground level of 1.2m LSD maximum discharge may occur until the tide level rises at elevation of 1.2m LSD – 0.25 =	0.95m LSD
	In other words the controlled flow situation will start from =	0.95m LSD
	Step - 7: Identify when the Flows are under Controlled Flow and Gate Closed Conditions	
	The gate is partially closed when tide level rises at elevation =	0.95m LSD
	The gate is fully closed when the tide level rises at elevation =	1.2m LSD
	From the tide cycle (Figure 19.C2) period of controlled and no release are	
	1. Gate partially closed (i.e. 4.1 – 3.2) =	0.9hr
	2. Gate fully closed =	3.2hr
	Step - 8: Superimpose Tidal Variation Curve and Inflow Hydrograph	
	The critical inflow hydrograph is superimposed on the tide cycle to determine the period of fully open gate condition. =	Figure 19.C5
	Assuming that the stored runoff will be released within one tide cycle (12 hours), duration of fully open gate is (12 - 3.2) =	8.8hr

Reference	Calculation		Output
	Step - 9: Compute the Runoff Volume under Controlled Flow and Closed Gate Condition		
	From Figure 19.C4, runoff volume during controlled flow is the summation of the areas 'a' and 'd'.		
	Area 'a' = 0.5x27x7.7x60	=	6,237m^3
	Area 'd' = 0.5x(7.7+17)x27x60	=	20,007m^3
	So, total controlled runoff volume is (6,237+20,007)	=	26,244m^3
	Similarly, runoff volume during zero flow condition is the summation of the areas 'b' and 'c' (Figure 19.C5).		
	Area 'b' = 0.5x(7.7+17)x33x60	=	24,453m^3
	Area 'c' = 17x159x60	=	162,180m^3
	So total stored runoff during closed gate condition is (24,453+162,180)	=	186,633m^3
	Assuming 1/3 of the inflow runoff (8,748m^3) will be released due to the hydrostatic balance of the gate during partially controlled flow condition. So, the runoff volume trapped upstream of the gate is 2/3 of the runoff during the controlled flow. That is,		
	Volume during controlled flow = (2/3)x26,244	=	17,496m^3
	So, total runoff volume to be released during uncontrolled condition is 17,496 + 186,633	=	204,129m^3
	For actual release through the tidal gate, the stage-discharge curve should be prepared from the hydrostatic balance of the gate during the tide and that curve should be followed to estimate the runoff volume released during the controlled flow.		
	Step - 10: Is Size of the Gate Adequate?		
	Size of the gate should be such that it can safely release the assumed 8,748 m^3 of water within 0.9 hrs.		
	So, the average release rate is 8,748/(0.9x60x60x3.5) per m width	=	0.77 m^3/s/m
	Storage at controlled gate condition, i.e. at water level of 0.95m LSD is (from Figure 19.C3).	=	40,000m^3
	If no pump is used then water volume stored in the drains from the beginning of the controlled flow condition is (40,000 + 204,129)	=	244,129m^3
	Corresponding water level at this storage will spill over the banks where the ground levels are 1.2m LSD (from Figure 19.C3)		
	Storage at safe water level in the drain (1.2m LSD) is (From Figure 19.C3).	=	56,000m^3

Reference	Calculation		Output
	Step - 11: Finalise the Gate Size, Pump Capacity and Number Required		
	So, volume of stored runoff to be pumped is [204,129 – (56,000 - 40,000)]	=	188,129m^3
	Assuming that rise of another 300 mm of water at upstream of the gate will not cause any severe problems/losses at the commercial areas. Thus, volume of water can be stored safely within storage level from 1.2m to 1.5m LSD (from Figure 19.C3) is (87,000 – 54,000)	=	33,000m^3
	The excess volume of water to be pumped is (188129 – 33,000)	=	155,129m^3
	As such, the average required pumping rate during the controlled flow condition is :-		
	Controlled flow duration, 4.1 – (0.9/2)	=	3.65 hr
	Controlled flow rate, 155,129/(3.65x60x60)	=	11.81m^3/s
	So, proposed submersible pump capacity to discharge the flood volume safely.	=	3 nos x 4m^3/s
Appendix 19.B	The pumps can be operated at different stages (water level at the upstream side of the gate). The pump size selection, operation levels and storage routing are performed following the procedure as detailed out in the Appendix 19.B and is not shown here.		
	Any critical situation such as power failure, pump breakdown, etc. may occur during the storms and when the drainage gate is closed. Thus, gate size and number should be determined to safeguard against such critical conditions so that the runoff can be released without potential flooding at upstream of the gate. So, the required discharge rate per metre width of the gate :		
	Duration during, (12 – 4.1)	=	7.9 hr
	Discharge rate per m width, {204,129 m^3/(7.9 hr x60x60x3.5m)}.	=	2.05m^3/s/m
	Number of gate required to release peak inflow is {17/(2.05x3.5)}	=	2.37 nos
	So, the gate opening should be (2.37 No.x3.5m)	=	8.3m wide
	In this case the recommended size	=	2 gates of 4.5m wide
	The typical section and the downstream elevation of the tidal gates	=	Figure 19.C6 & Figure 19.C7
	The procedure followed here is an approximate method. For actual simulation of tide and flap gate it is recommended that the designer use available hydraulic computer software with tidal gate outlet and pumping options, which allows the rapid testing of a number of design storms and tidal gate and pump configurations.		

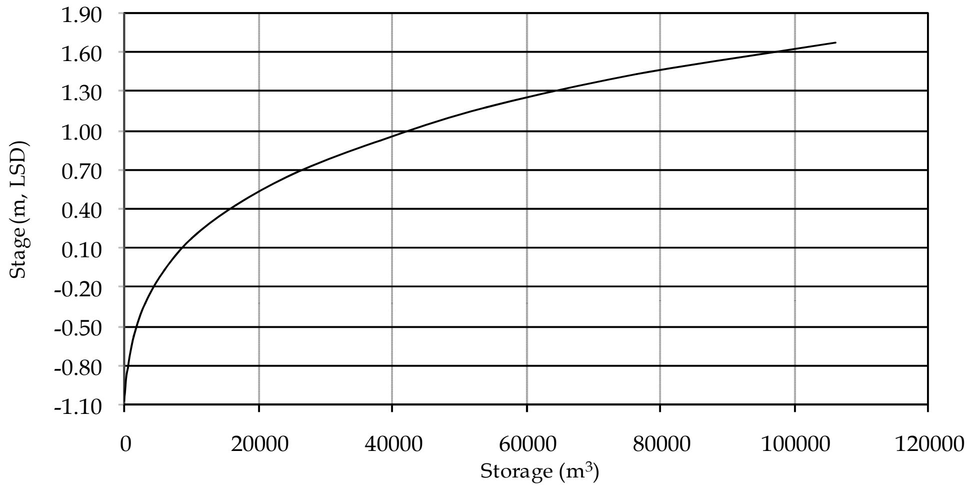


Figure19.C3: Stage-storage Relationship of the Existing Drainage System

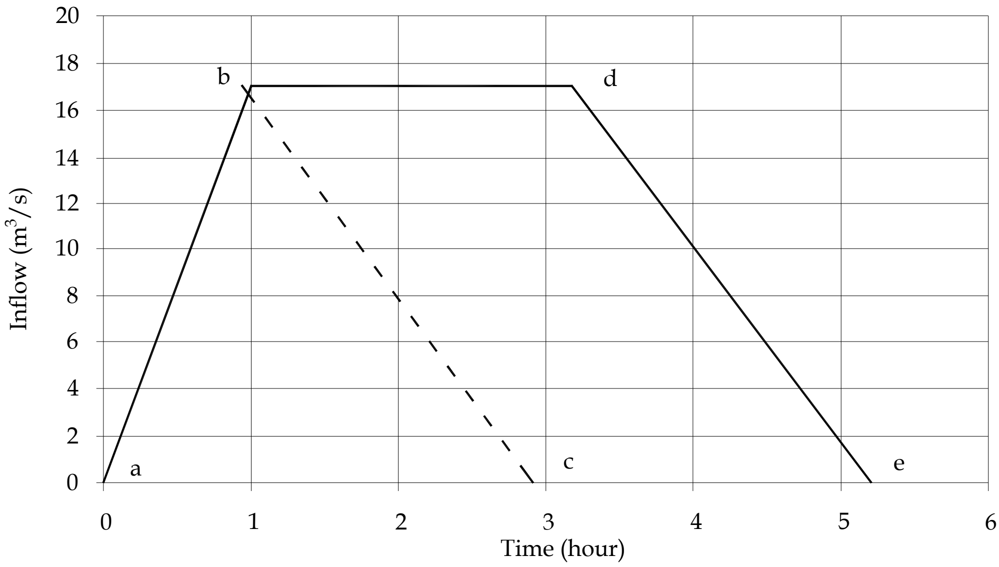


Figure 19.C4: Design Inflow Hydrograph at the Proposed Tidal gate

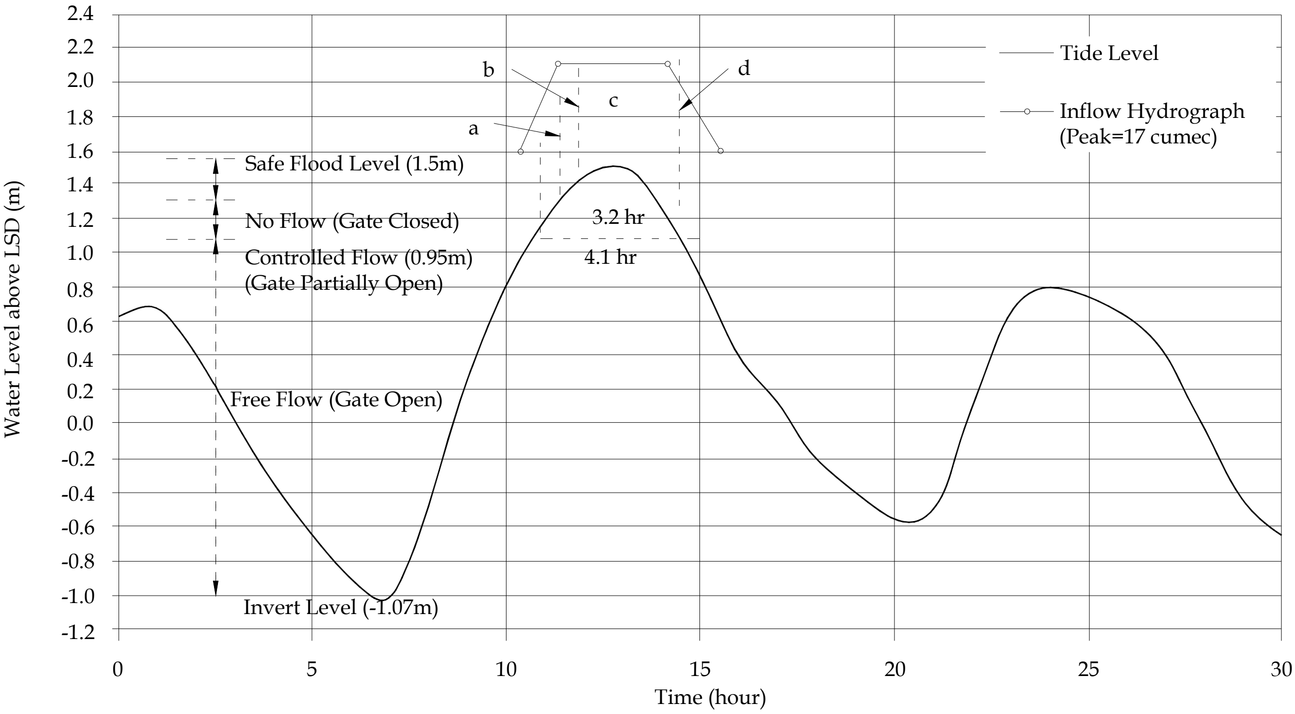


Figure 19.C5: Superimposed Inflow Hydrograph on the Tide Level Graph

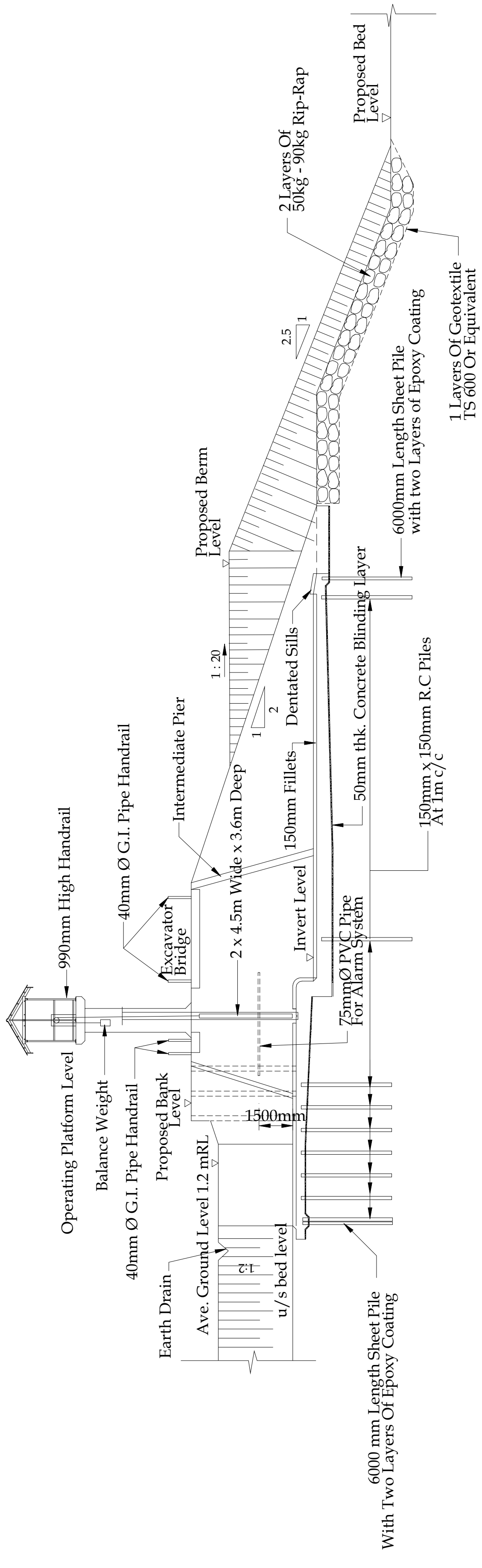


Figure 19.C6: Sectional Plan of the Tidal Gates

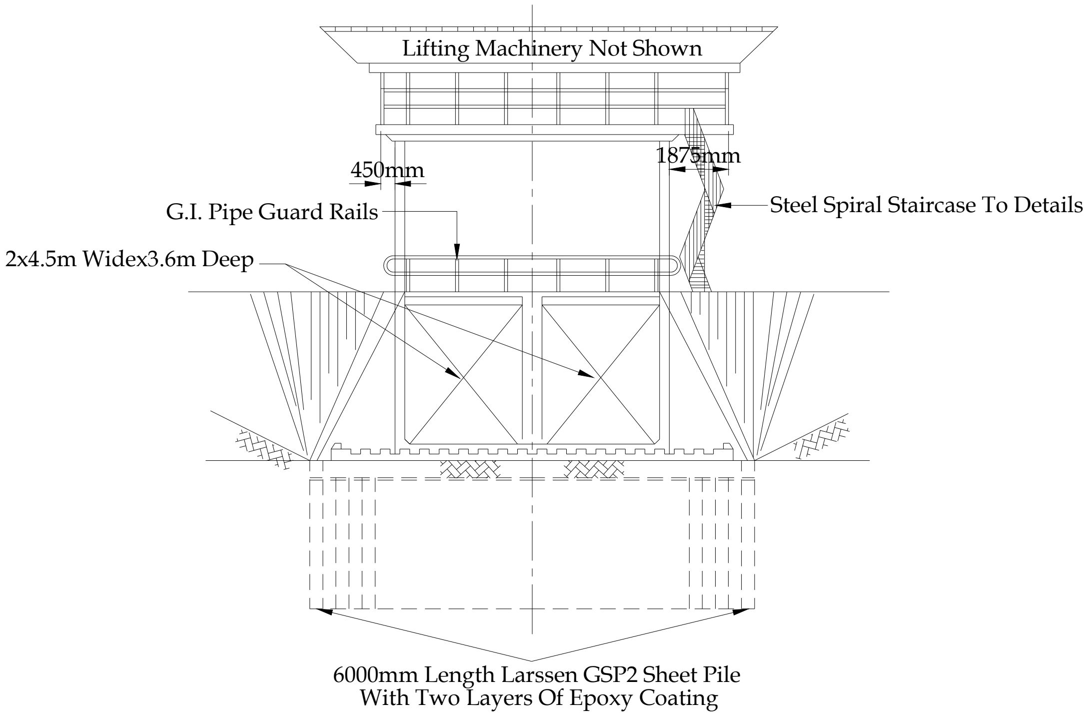


Figure 19.C7: Downstream Elevation View of the Tidal Gates