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## 46 LOWLAND, TIDAL & SMALL ISLAND DRAINAGE

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### 46.1 INTRODUCTION

Urban drainage and stormwater system design in low-lying and tidal areas involves a number of special considerations. Among the issues to which designers must pay attention are the following (see Figures 46.1 and 46.2):

- the hydraulic effects of flat grades and high tailwater levels;
- tidal effects;
- storm surge;
- greenhouse effect ;
- flood effects in river/estuaries;
- the combined probabilities of different modes of flooding.

Stormwater quality can also be influenced by the presence of salt water and saline groundwater in coastal areas. This Chapter describes these effects. Small islands have special problems that are also discussed.

Because of the difficulties of designing gravity systems in low-lying areas it may be necessary to use floodgates, tidegates, and/or pumped systems.

#### 46.1.1 Problems of Design in Low-lying/Flat Areas

Many low-lying tidal areas are characterised by flat gradients. This creates particular difficulties in hydraulic design.

All such systems must be checked by hydraulic grade line calculations (for pipe systems) or backwater analysis (for open channels), paying particular attention to the selection of appropriate tailwater levels. In flat areas, larger conveyance systems than would otherwise be the case are needed to carry stormwater flows. High velocities and scouring will not normally be a problem.

#### 46.1.2 Scope of this Chapter

The scope of this Chapter is to cover the issues directly associated with urban drainage system design in low-lying and/or tidal areas.

This Chapter includes sections on tidegates and floodgates. Pumped drainage systems are discussed in Chapter 45.

### 46.2 DEVELOPMENT IN THE TIDAL ZONE

Development of land in the tidal zone may result in a number of drainage and stormwater management problems, as well as potential for groundwater depletion. These problems need to be fully considered when preparing any proposals for such development.

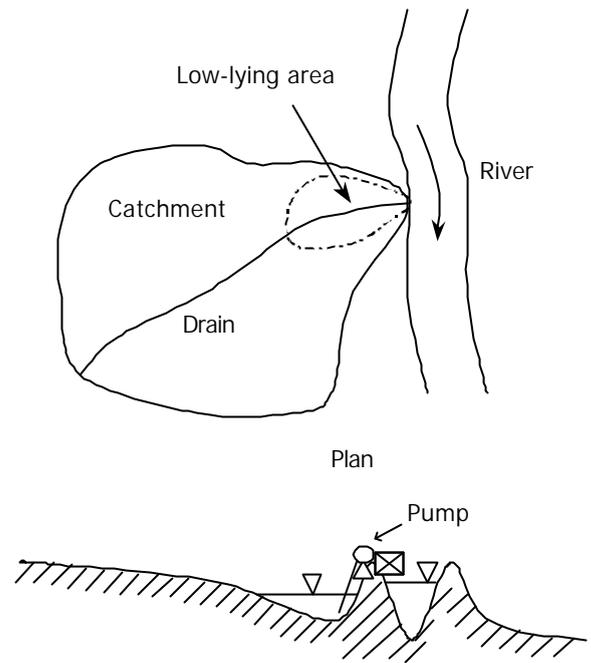


Figure 46.1 Urban Drainage Design Considerations in Low-lying Areas

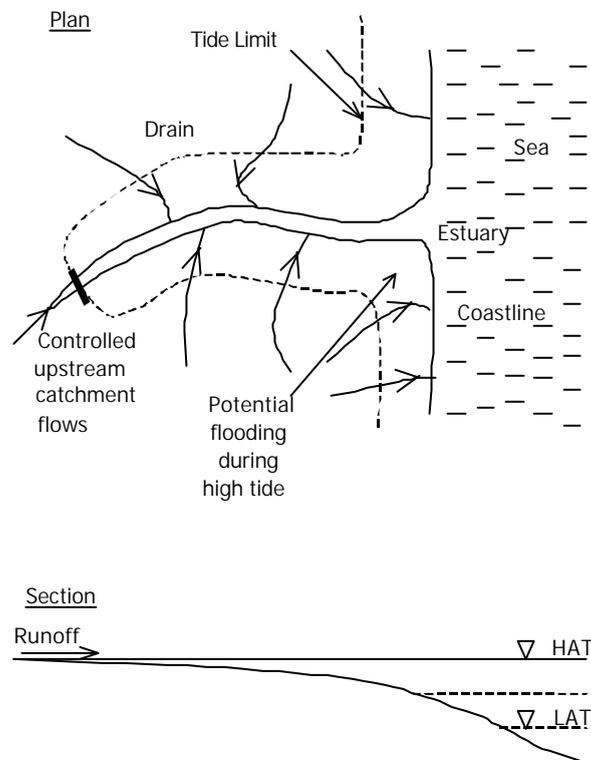


Figure 46.2 Urban Drainage Design Considerations in Tidal Areas

Lowland and coastal areas may be environmentally sensitive. Proposals for development of sensitive areas will need to comply with all applicable legislation and regulations (see Chapter 5).

**46.2.1 Land Reclamation, Filling and Grading**

The surface levels of reclaimed land should be raised, graded and designed with a minimum gradient, preferably 0.5%, to ensure good drainage and to limit the extent of tidal encroachment.

Associated with reclamation is the need to extend drainage systems, which may already have very low longitudinal gradients, further towards the sea. This produces even flatter gradients and the need to adopt large cross-sectional areas to handle the design flow.

Therefore, if at all possible reclamation should avoid the positions of existing drainage outfalls. The preferred arrangement is to reclaim a section of coastline between drainage outfalls, leaving the outfall areas untouched.

These areas must be protected against storm and wave action. This shoreline protection is outside the scope of this Manual, however appropriate guidance can be found in a number of specialised technical publications.

**46.2.2 Minimum Floor Levels for New Development**

For new developments, the designers should confer with the relevant Local Authority to establish an appropriate minimum floor and/or site filling levels taking into account the predicted Greenhouse Effect and thus compensate for the predicted rise in tailwater level. Table 46.1 shows the recommended minimum floor levels design for new development in coastal areas.

Table 46.1 Minimum Platform Levels for New Development

Design Condition	Criteria
Minimum platform levels for new development (including Greenhouse effect)	Highest Astronomical Tide + 0.5 m

**46.2.3 Integrated Measures for Small Islands**

Improved design of integrated drainage and stormwater management systems can greatly help to address water problems on small islands.

Offshore islands, in particular, usually suffer from water supply problems. Surface supplies are limited because of the lack of suitable catchment areas, while groundwater

extraction causes problems of saline groundwater intrusion. Pulau Pangkor, Pulau Tioman and Pulau Perhentian are examples of islands with existing or potential water problems.

Development on these islands needs to maximise freshwater retention and minimise saltwater intrusion, as shown in Figure 46.3.

The following SWM practices should be adopted:

- enhance freshwater runoff capture and retention. In sandy areas, concrete/lined drains can be omitted entirely and all fresh water should be directed to suitable infiltration basins to recharge groundwater. Use retention measures wherever possible;
- in clayey zones, provide more detention storage facilities for non-potable water reuse;
- protect and treat water quality. Every effort needs to be made to protect the quality of the water resource on islands. Structural quality controls (see Part G) should be used for non-point sources. Point sources should be controlled by a sewerage system that is designed to minimise water use and maximise re-use. Polluting industries, or those with high water consumption, should not be allowed on these islands;
- non-structural measures including community education to emphasise the importance of conserving and protecting fresh water. These measures should be specifically targetted to the local community who, it is hoped, would be well aware of the local issues.

In extreme cases, stormwater reuse for supplementary water supply should be considered. Treated and reclaimed stormwater can be used for non-potable or even potable purposes to reduce demand on the water supply. The potable water supply can also be supplemented by using tanks to collect roof runoff for drinking water, or by aquifer storage recovery.

**46.2.4 Polluted Stormwater**

Erosion and Sediment Control during construction is at least as vital in tidal areas as elsewhere; possibly more so, because construction is carried out in close proximity to (or even in) the receiving waters. Additional measures to those listed in Part H, such as silt curtains, may be required.

Polluted stormwater runoff can have a significant adverse impact on coastal areas. Typically these areas are environmentally sensitive, with a range of wildlife habitats. They form important fish breeding grounds. Coastal areas, particularly beaches, are highly valued by the public for recreation and aesthetic benefits.

Design of structural water quality control measures can be difficult in coastal areas. Control works such as booms and GPTs must be protected from backflow by means of

tidegates. This is to prevent material trapped in the device from being washed out by tides.

The high tailwater levels due to tides may make it difficult to include trash racks or GPTs because the additional head loss in the drainage system may increase flooding upstream. If the available head is limited it may be necessary to adopt pumped or combined pump/gravity systems, with storage.

The high water table and possible water salinity problems limit the opportunities for providing ponds or wetlands. A possible option is the inclusion of saline wetlands, as discussed later in this section. However very little is known about the pollutant removal efficiency of these wetlands

### 46.2.5 Levees or Bunds

Levees or bunds may be considered as a means of allowing development in low-lying or tidal land. Levee systems can cause the following problems:

- the enclosed low-lying land will have flat gradients and will be boggy and difficult to drain. It may be suitable for agriculture but will generally be unsuitable for urban development;
- the difficulty of removing stormwater from the area enclosed by the levee. Usually this will require pumping, which is prone to mechanical or power supply failure;

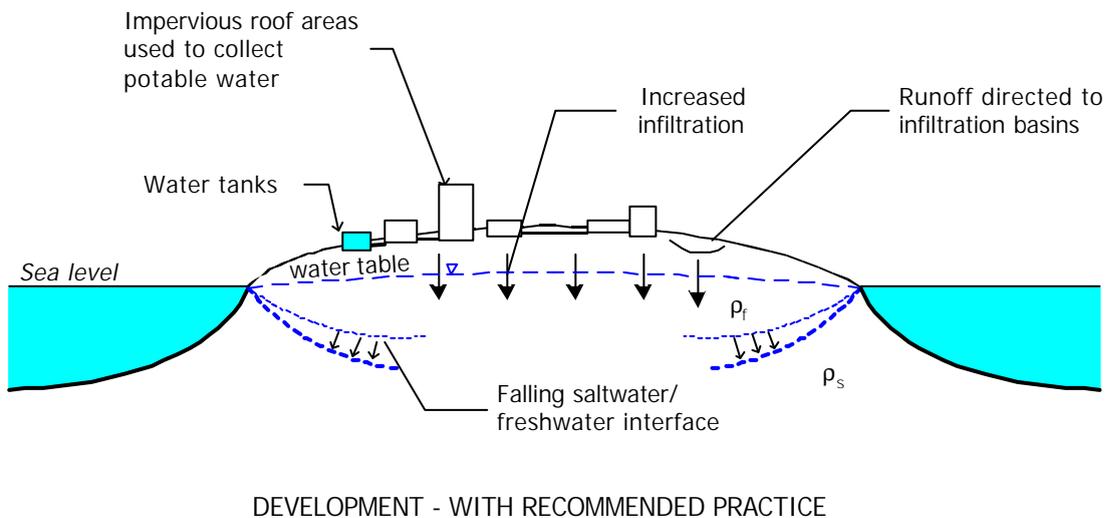
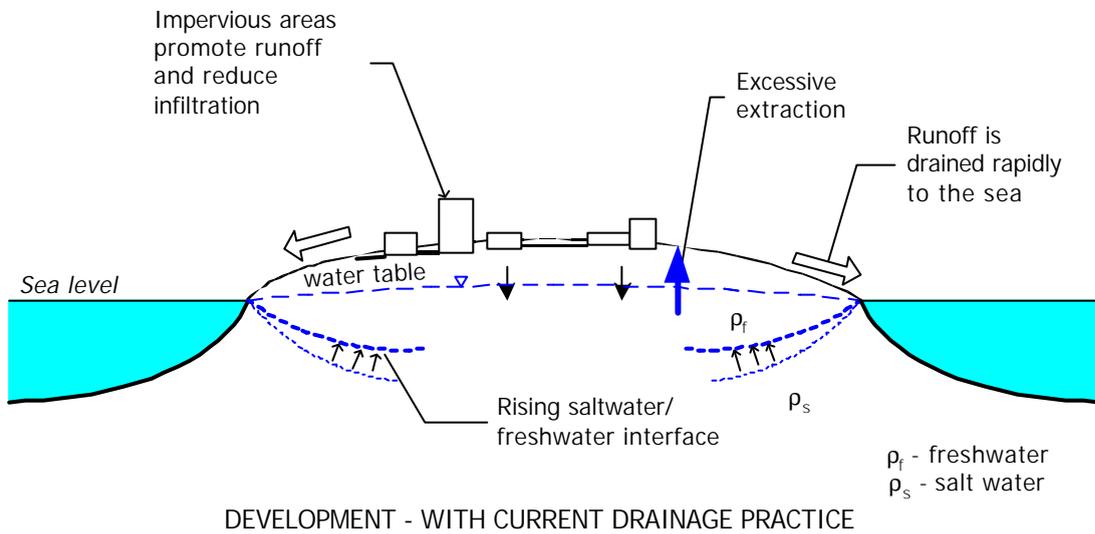


Figure 46.3 Stormwater Measures on Small Islands

- the potentially catastrophic effects of overtopping, which could lead to rapid flooding of large areas with a risk to life and property. In tidal areas overtopping could be caused by wave and storm action, subsidence, or by sea level rises due to the greenhouse effect (see Section 46.3). Flooding is the main cause of overtopping for levees protecting low-lying and alongside rivers.

For these reasons, levees are not generally recommended for use in low-lying and coastal areas. Filling is the preferred method of land reclamation for new development. If levees have to be used because of existing development, which cannot easily be moved, additional planning and design precautions are necessary.

#### 46.2.6 Detention/Retention

Detention storage is of little or no value for runoff control in a tidal area. In general for sites less than 500m from the coast or estuary shoreline, no detention storage is required except for water supply. Runoff infiltration facilities are, however, encouraged (see Section 46.2.1).

#### 46.2.7 Peat Soils

Development in coastal areas with peat soils can cause a number of drainage and other related problems. These include environmental impacts, low bearing capacity, difficulty of consolidation, and high water table making it difficult to provide drainage. Certain types of peat soils can produce acid runoff when exposed to air. This runoff can impact on the environment causing fish kill.

Any proposal for development in coastal areas containing peat soils will require specialised study including soil testing to identify whether Potential Acid Sulphate Soils (PASS) are present. In such cases either the development should be modified to avoid exposing the soils to the atmosphere, or preventative measures such as neutralisation will be required.

#### 46.2.8 Saline Wetlands

Chapter 35 on Constructed Wetlands, discussed the saline wetlands that occur naturally in some coastal areas. These differ from conventional wetlands in their water salinity and in the range of plant species that are present. Water quality in saline wetlands varies within a range from brackish to nearly fresh, and the plants are adapted to suit these particular environmental conditions.

Some examples exist overseas of constructed saline wetlands for water quality control. These wetlands often have multiple purposes, including landscaping and wildlife habitat. The hydraulic design of constructed saline wetlands requires considerable care, to ensure that the correct water levels are achieved. The lists of plant

species in Chapter 35 are not applicable to saline wetlands. A designer who needs to consider constructed saline wetlands should seek advice from biologists with specialist expertise in this area.

#### 46.2.9 River Aggradation

Sediment can be deposited from coastal rivers when they enter the tidal zone. This is due not only to the reduction in flow velocity, but also to chemical action (precipitation) caused by contact with salt water. The designer should examine the possibility of river aggradation (a rise in bed levels, due to sedimentation). River aggradation leads to high tailwater levels, possibly requiring the use of floodgates or pumped outfalls.

### 46.3 TIDE LEVELS

The design of stormwater channels in or near the tidal zone will normally involve the determination of an appropriate starting tailwater level as an initial step in the design procedure. Where the topography is flat, or where obstructions to the flow are present, the backwater effects of an elevated tailwater level could be extensive.

The determination of an appropriate starting tailwater level is facilitated at channel cross-sections where the flow velocity is effectively zero, and where the channel area can be regarded as infinite, such as a lake, dam or the sea. At locations where the proposed channel outlet discharges into a drainage system, the effects of coincident storm events in each system must be considered. In such cases, the joint probability of simultaneous runoff events occurring in both catchments needs to be assessed.

#### 46.3.1 Tidal Effects

One of the most obvious factors in low-lying, coastal and estuary areas is the rise and fall of tides. Backwater effects caused by tides and other factors can have a dominant influence on nearby drainage systems.

The actual water level at a given location in tidal zones at a given time will depend upon:

- astronomical effects caused by the gravitational action of the sun and the moon;
- meteorological effects, including atmospheric pressure variations and wind and wave set-up; and
- the effect of surface gradients associated with the flow pattern in the estuary.

Figure 46.4 shows the definition of terms for tidal levels in tidal affected areas. The mean sea level (the average level over a specified time period) at a given location tends to vary cyclically: the mean sea level determined over a period of one month at a given location may differ by

0.1 m or more from the mean level determined over one year at that location.

(a) Sources of Tide Data

Tide data is available from various government agencies in Malaysia, that include Royal Malaysian Navy, Survey Department, and Ports Authorities responsible for port management. The Royal Malaysian Navy and port authorities in the various States publish tidal predictions for particular locations. In the Malaysian National Tide Tables, the times and heights of predicted high were tabulated for each day of the year, together with data for the calculation of this information.

In hydraulic investigations, it is necessary to estimate the probable extreme tidal levels likely to occur at a given location. In the Malaysian National Tide Tables, the Highest Astronomical Tide (HAT) and the Lowest Astronomical Tide (LAT) are tabulated for the standard ports on the Malaysian coast (see Figure 46.5). These are defined respectively as the highest and lowest levels, which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions. Meteorological effects such as storm surge may cause actual extreme tidal levels to be higher than the HAT or lower than the LAT.

Predicted tidal levels take into account the astronomical effects, but are based upon average meteorological conditions. Consequently, actual tidal levels may differ significantly from the predicted levels, as a result of variations in meteorological conditions. The combined effect of surge and greenhouse effect is shown in Figure 46.6.

(b) Precautions when using Tide Data

The reliable estimation of probable extreme tidal levels at a particular location will generally require specialist advice; it

will generally require the examination and analysis of local records, and in many cases the collection of further data. In dealing with historical records, it should be borne in mind that the relations between local datum planes are often poorly defined, and that the history of many gauges is poorly documented: the possibility of undocumented changes in the datum or in the location of a gauge should be considered. Care must be taken when using tidal data, to ensure that it is adjusted to the same datum as the datum used for terrestrial levels.

46.3.2 Flood Level in Estuaries

Flood levels in estuaries may be affected by river flows and these must be taken into account in setting tailwater levels. Each individual situation is different and flood levels will need to be derived from local studies. In some estuaries, extreme ocean levels will dominate over any local floods. The user should also have regard to the discussion on combined flooding in Section 46.5.

46.3.3 Pressure Effects

If changes in atmospheric pressure persist for some time, they will cause corresponding changes in sea level. In particular, low-pressure systems such as cyclones and typhoons cause a rise in sea level. The likely rise is in the order of 0.3 m (AR&R, 1987). Fortunately however, because of global air circulation patterns Peninsular Malaysia does not normally experience these storms.

46.3.4 Storm Surge Setup Effects

Additional to the effects of air pressure, are the effects of wind. Strong persistent onshore winds will push up sea levels in an effect known as 'set-up'. The magnitude of the setup is likely to vary with the wind strength and duration, and the length of open sea in the direction of the prevailing wind (the 'fetch').

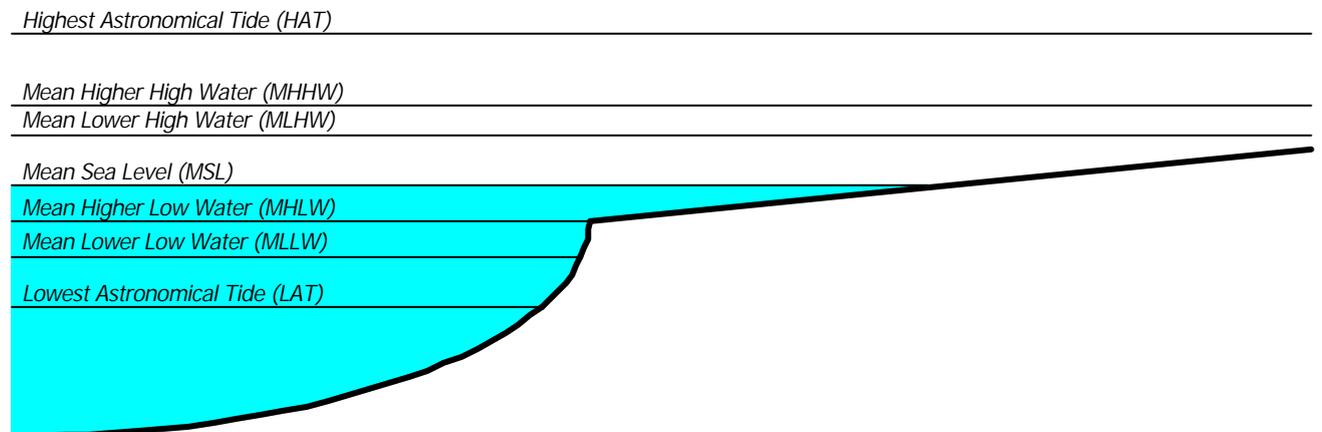


Figure 46.4 Tidal Variations

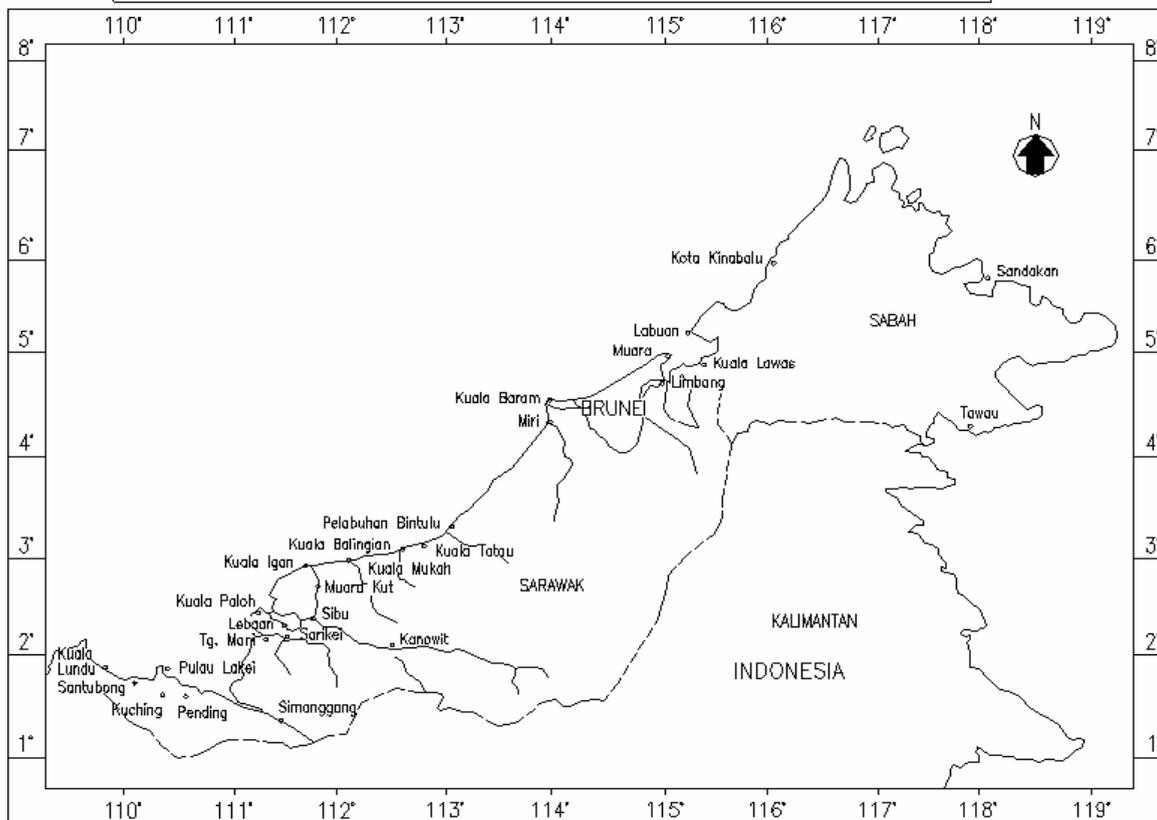
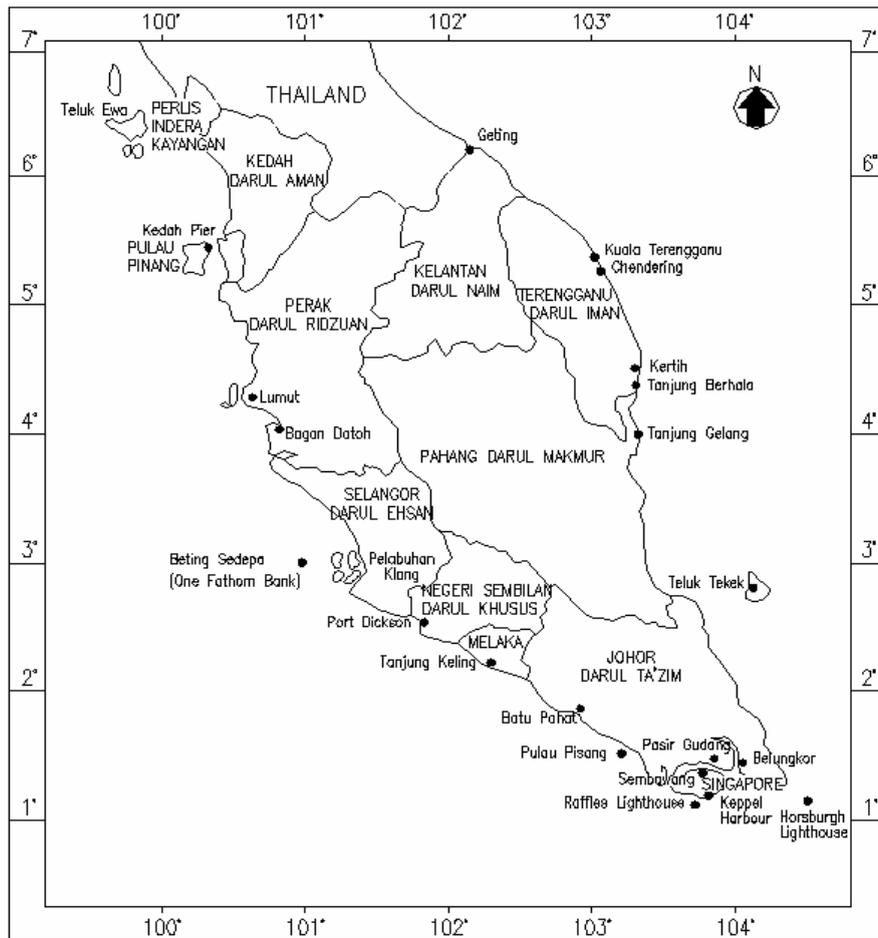


Figure 46.5 Locations of Standard Ports on the Malaysian Coast

The forecasting of storm surge and set-up requires specialist advice. Users requiring detailed information on these matters should consult with the Meteorological Service, or with maritime authorities that may also be able to provide information.

For minor studies where the effort of specialised investigation is not warranted, an allowance of 0.3m should be made for combined pressure and storm setup in coastal areas of Peninsular Malaysia (see Figure 46.6).

**46.3.5 Climate Change due to Greenhouse Effect**

The term Greenhouse Effect refers to measured increases in atmospheric carbon dioxide levels, and in mean global temperatures. It has been suggested that these effects will cause a worldwide rise in sea levels, climate change and an increase in storm severity.

*(a) Effect on Sea Level*

Based on the findings of the IPCC Scientific Assessment Working Group (1992), it is anticipated that "should greenhouse gas emissions continue to increase at the currently projected rate, an average rate of global mean sea level rise of between 3 and 10 centimetres per decade is predicted over the next century resulting from thermal expansion of the oceans. There would be significant regional variations."

For new development, which will have an expected life of 50 years or more, it is considered appropriate to allow for the greenhouse effect in setting minimum floor levels. The recommended allowance for sea level rise is 0.30 metres (see Figure 46.6). Note that this rise is *additional* to that for storm surge, given in the previous section.

*(b) Other Possible Effects*

The likely impact of climate changes caused by the Greenhouse Effect on storm rainfall magnitudes and intensities is difficult to predict.

At the time of writing, few long-term trend analyses have been published. This may be because there are few statistically valid rainfall intensity datasets, which cover a sufficiently long period to allow meaningful trend analysis.

One recently reported study of data for Yokohama, Japan, is that by Kawaguchi, Asada and Zhang (1999). The results appear to provide evidence for a significant increase in rainfall intensities in recent years, at least in large urban centres. The reported increase in rainfall intensity for a 20-minute duration storm, for example, is 20 mm/hr between the first and latest 20-year periods. This may be due as much to the 'heat island effect' in large cities as to the global Greenhouse effect.

**46.3.6 Subsidence**

Tide levels are expressed relative to the adjacent land level. Therefore, if the land subsides the effect will be equivalent to a rise in sea levels. Some coastal areas in some countries are known to be subject to long-term subsidence. Subsidence can be caused by water extraction, or in some coastal areas, oil extraction.

A detailed discussion of land subsidence is outside the scope of this Manual. However, the urban stormwater designer should be aware of the possibility that it will occur over the life of a drainage system. If there is reason to suspect that subsidence may be a problem, specialist advice should be sought from appropriate authorities.

**46.3.7 Saline Wedge Intrusion**

In an estuary, river water meets and interacts with seawater. The saline seawater will have a density about 3% greater than the river water. Interaction between the two water bodies may cause flocculation and settlement of suspended material, and may cause chemical precipitation of materials dissolved in the river water. These effects, together with the reduction in flow velocities, contribute to the soft muddy bed deposits often found in estuaries (AR&R, 1987).

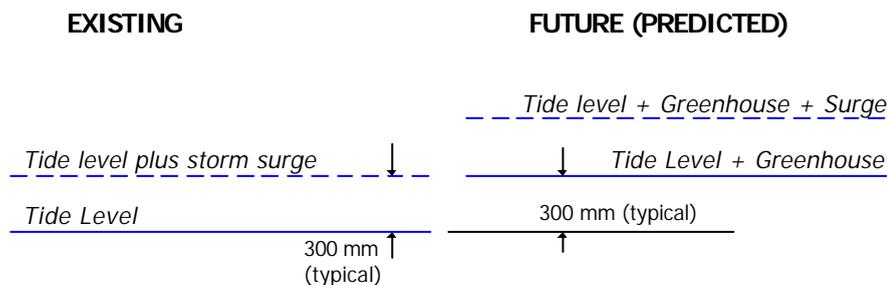


Figure 46.6 Tides, Surge and Greenhouse Effect

The difference in density of the two water bodies causes them to be relatively resistant to mixing. The density difference tends to cause stratification and consequently seawater tends to flow upstream on the riverbed forming a "saline wedge".

Under favourable conditions, the saline wedge may extend several kilometres upstream. However when a flood occurs in the river, river flow tends to flush the saline wedge out of the estuary and fully mixed conditions will occur.

For further information on the hydraulics of estuaries, the reader should refer to specialised publications.

The consequences of saline wedge intrusion include the exposure of drainage structures to salt water corrosion. The use of corrosion-resistant concrete and steel is always required in coastal areas.

#### 46.4 SETTING DESIGN TAILWATER LEVELS AT DRAINAGE OUTFALLS

The discharge point, or outfall, can be either a natural river or stream, or an existing or proposed stormwater drain or channel. The procedure for calculating the hydraulic grade line or backwater analysis through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

The determination of an appropriate starting tailwater level is facilitated at channel cross-sections where the flow velocity is effectively zero, and where the channel area can be regarded as infinite, such as a lake, dam or the sea. At locations where the proposed channel outlet discharges into a drainage system, the effects of coincident storm events in each system must be considered. In such cases, the joint probability of simultaneous runoff events occurring in both catchments needs to be assessed.

The design of stormwater channels in or near the tidal zone will normally involve the determination of an appropriate starting tailwater level as an initial step in the design procedure. Where the topography is flat, or where significant obstructions to the flow are present, the backwater effects of an elevated tailwater level could be extensive.

##### 46.4.1 Design Tailwater Levels for Sea and Estuary

Designers should confer with the relevant Local Authority to establish an appropriate level for outfalls to tidal waters. Consideration should be given to the joint probability of occurrence of the design storm, tide level and storm surge together with allowance for the Greenhouse Effect in determining the tailwater level for design.

Whilst it is not possible here to provide specific recommendations, some suggested levels are provided in Table 46.2. These suggestions should in no way replace the need to confer with the Local Authority and for the application of engineering judgement.

This approach recognises that local runoff can occur at any time, including at the time of maximum tidal level.

Table 46.2 Design Tailwater Levels in Coastal Areas

Design Condition	Design Tailwater Level (see Figure 46.4)
Minor storm	Range from Mean Lower High Water (MLHW) to Higher High Water (MHHW)
Major storm	* Range from Higher High Water (MHHW) to Highest Astronomical Tide (HAT)

NOTE: \* the user should assess the probability of this level occurring in conjunction with the major storm. For catchments less than 10 km<sup>2</sup>, the probability of joint occurrence is likely to be low.

##### 46.4.2 Design Tailwater Levels for Tidal River and Stream

In cases where the drainage outfall is located in the tidal reach of a river or stream, the design tailwater level should include an appropriate flood surcharge in addition to a high tide level. Adoption of any surcharge above tidal levels for the design tailwater involves consideration of the design ARI for the drainage system (see Chapter 4) and its relationship to the catchment size and flood behaviour of the receiving waters.

The procedures outlined in Section 46.4.3 permit corresponding critical discharge in the side drain (from the outfall) and main stream to be assessed. These procedures should be applied to the determination of the appropriate design tailwater in the main stream except that the tailwater adopted should be that obtained by adding the surcharge corresponding to the combined discharges, to the relevant tide level (including Greenhouse Effect) determined in accordance with Section 46.3.5.

Alternatively the Local Authority may determine appropriate tailwater levels or discharge conditions for particular reaches of tidal streams their area, based upon local experience and knowledge. These levels should not be any less stringent than the levels set out in this Chapter.

### 46.4.3 Design Tailwater Levels for Non-Tidal Outfalls

The design of a drainage system which discharges to a non-tidal outfall e.g. a lake, open channel, stream or river, should take into account the expected tailwater level in the receiving waters.

In cases where the tailwater level is affected by stormwater runoff from an external/upstream catchment, the critical design situation for surcharging of the drainage system may occur when the flow rate in the drainage system is less than the design flow rate. In such cases, the critical tailwater level and the drainage discharge should be determined by an investigation of the joint probabilities of flooding in both the subject drainage system and its receiving waters. In situations where the external catchment is relatively large in comparison with the catchment area of the drainage system, it will be inappropriate to assume that the peaks of the flow rates for each catchment will occur simultaneously .

Various procedures, which permit the assessment of the most critical combination of flow and tailwater, are described below. The appropriate maximum tailwater derived after consideration of each procedure should be adopted. Clearly the procedure providing the most appropriate result will depend upon the sizes of the catchment involved and their relative sizes.

The tailwater level or backwater envelope to be adopted should be the maximum of:

- The tailwater level corresponding to the combined peak discharge;
- The critical combination of tailwater level and discharge.

If the outfall is a river, it may be necessary to consider the joint probability of two hydrologic events occurring at the same time. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by comparing the catchment area of the river and the catchment area of the stormwater drain.

Design tailwater levels for urban drainage design in lowland areas should follow the recommendations in Table 46.3.

For example, if the receiving river has a drainage area of 2,000 hectares and the storm drainage system has a catchment area of 20 hectares, the ratio of catchment areas is 100 to 1. From Table 46.3, the flow rate in the main stream will be equal to that of a 5 year ARI storm when the drainage system flow rate reaches its 10 year ARI peak flow at the outfall. Conversely, when the flow rate in the river reaches its 10 year ARI peak flow rate, the flow rate in the drainage system will have fallen to that of the 5 year ARI peak flow.

The designer should prepare water surface profiles (or hydraulic grade lines) for both cases. The maximum water level to be used in setting minimum floor levels, is given by the *envelope* of the two profiles.

For other minor or major design flood ARIs, use similar ratios.

Table 46.3 Frequencies for Coincidental Occurrence

Catchment Area Ratio	10 year ARI Design		100 year ARI Design	
	Main River	Tributary	Main River	Tributary
10,000 to 1	1	10	2	100
	10	1	100	2
1,000 to 1	2	10	10	100
	10	2	100	10
100 to 1	5	10	25	100
	10	5	100	25
10 to 1	10	10	50	100
			100	50
1 to 1	10	10	100	100

NOTES: \* where two numbers are shown, calculate for both cases and develop the envelope curve of flood levels.

## 46.5 WATER SURFACE PROFILES IN AREAS OF COMBINED FLOODING

### 46.5.1 Background

A problem likely to be encountered in drainage design in tidal or lowland areas, is where a location is affected by two different modes of flooding. This may occur at sites subject to flooding from different catchments (urban and rural), or affected by flooding from combined tidal/ocean effects and stormwater runoff. These two flooding modes will, in general, have very different response times. Flood peaks from the two sources may occur separately or together. The problem arises of how to interpret and assign the correct *joint probability* to a combined flooding event.

If the two flooding modes were statistically independent, the joint probability  $P_C$  is the product of the respective probabilities  $P_1, P_2$  as given by Equation 46.1:

$$P_C = P_1 \times P_2 \tag{46.1}$$

However this is rarely the case. Usually, the meteorological events that lead to flooding in an urban

catchment will also contribute to high ocean levels or flooding on adjacent catchments, and therefore the assumption that the events are statistically independent is not valid.

In cases where the tailwater level is affected by stormwater runoff from an external catchment, the critical design situation for surcharging of the drainage system may occur when the flow rate in the drainage system is less than the design flow rate. In such cases, the critical tailwater level and the drainage discharge should be determined by an investigation of the joint probabilities of flooding in both the subject drainage system and its receiving waters. In situations where the catchment area of the receiving waters is relatively large in comparison with the catchment area of the drainage system, it is incorrect to assume that the peaks of the flow rates for each catchment will occur simultaneously.

The need to consider envelope curves is shown in Figure 46.7. Dahl, Harramoes and Jacobsen (1996) discussed the problem in the context of urban drainage systems. In the case study quoted, failure to consider the joint probability of flooding led to damaging consequences. The authors recommended that the solution is to carry out an analysis on the basis of a simulation of runoff from both catchments, using continuous historical time series as input. With the recent increases in computer power and

improvements in modelling techniques, this is now the preferred approach.

**46.5.2 Recommendation**

The design flood level to be adopted at any point is the higher of the HAT level, or the level in the stormwater drain due to a local storm. These calculations are most easily done by computer analysis.

In either case consideration should be given to the rules for determining start of tailwater. The critical combination of tailwater level and discharge is derived as set out below.

This procedure assumes that the tide level and the design storm for the stormwater drain, can be regarded as independent. If this is not the case the designer must consider the joint probability of the two events.

- (i) Project the level of the selected design tide level along the profile of the stormwater drain.
- (ii) Perform hydraulic calculations, such as backwater or HGL analysis, for the design storm in the stormwater drain using a starting water level selected in accordance with Table 46.2.
- (iii) the required design flood level is the envelope of the two profiles derived in (i) and (ii).

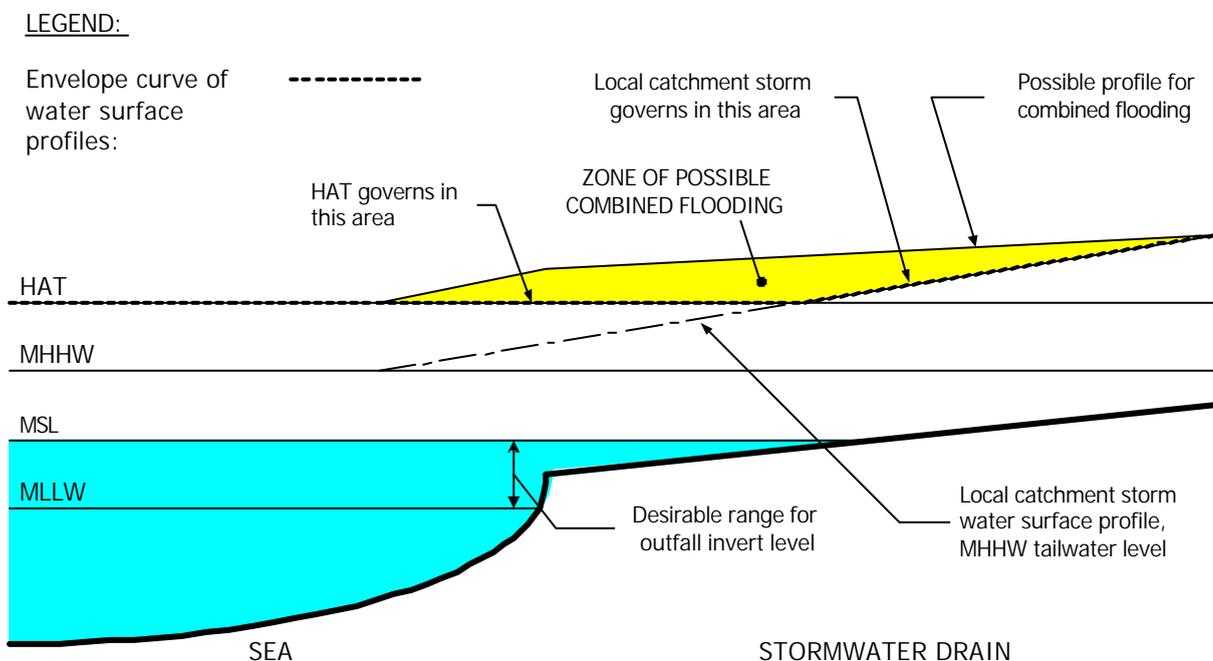


Figure 46.7 Envelope Method for Combined Flooding Analysis

## 46.6 DESIGN AND PROTECTION OF TIDAL OUTLETS

### 46.6.1 General Principles

Works constructed in tidal areas may need to comply with the requirements of a number of statutory authorities. There are a number of areas of particular concern and these include:

- All areas below MHHW ;
- Fish Habitat Reserves;
- Tidal Wetland Reserves;
- National Parks;
- Marine Parks;
- Area controlled by a Port Authority;
- Area controlled by a Waterways Authority.

Tidal outlets should be sufficiently evaluated to minimise the risks of sand blockage and to facilitate inspection and maintenance of the channel/pipes.

The invert of the outlet should preferably be above LAT, between MLLW and MSL. However, depending upon location, aesthetic considerations may dictate that outlets be located below low tide level. The outlet should not be so high that the outfall will cause excessive scour unless measures such as a dissipator chute are provided to convey the outflow down to the general level of the receiving water. The obvert of the outlet should normally be below 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. This may require the construction of breakwaters. 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 allied by differential sand levels caused by longshore littoral drift. Advice should be sought from the Local Authority in regard to the local beach behaviour and littoral processes in each instance.

An outlet discharging to a tidal estuary, should be located and designed with respect to possible morphological scour, often best determined by reference to past maps and photographs (including aerial photographs) of the site. 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.

Tidegates 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. The design of flap gates is discussed below.

### 46.6.2 Tidegate Design

Tidegates are outlet structures for the discharge of stormwater from reclaimed tidal compartments. These gates are closed during periods of high tides when the tide level outside the compartment is higher than the water level in the drainage system within the compartment. 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. The hydraulic design of such tidegates which are either culvert or flume type, entails careful consideration to ensure effective performance of the gate. A rational approach to the hydraulic design of such tidegates is provided in this section.

Detention storage is usually not required upstream of tidegates. Land upstream of the gates may be subject to temporary storage of runoff and flooding when the gates are closed, and it is therefore unsuitable for urban development unless it is filled. This low-lying land can be used for open space.

Filling of land upstream of a tidegate is acceptable provided that an adequate channel (or pipe) is installed to convey flow to the tidegate. The tidegate should have sufficient capacity to discharge the design major flood from the local catchment, during high tide.

A comprehensive description of the design method is provided in the DID Publication "Design of Tidegates" (Unpublished draft report).

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 46.8.

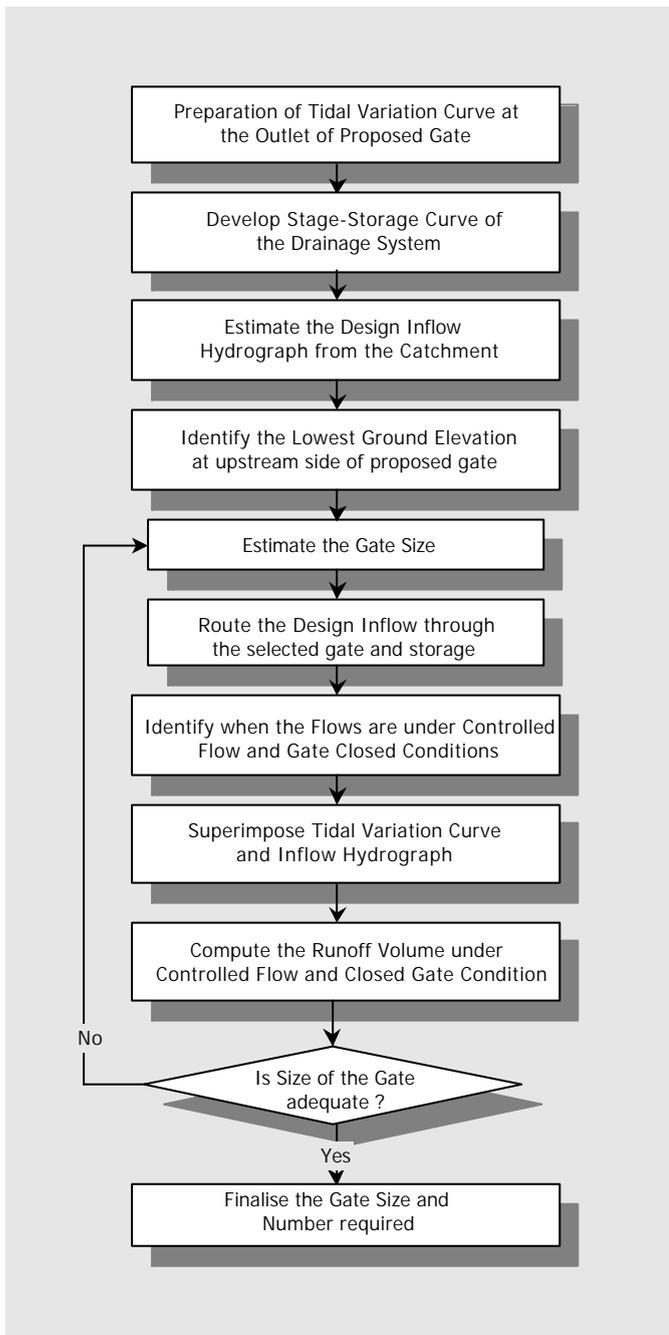


Figure 46.8 Tidegate Design Flowchart

## 46.7 DESIGN AND PROTECTION OF NON-TIDAL OUTLETS

### 46.7.1 General Principles

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 floodgates 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/Floodgate 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.

### 46.7.2 Floodgate Design

For the larger-size floodgates, 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 floodgate 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 46.9.

## 46.8 TIDEGATES AND FLOODGATES

Tidegates 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.

Floodgates 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 tidegates and floodgates.

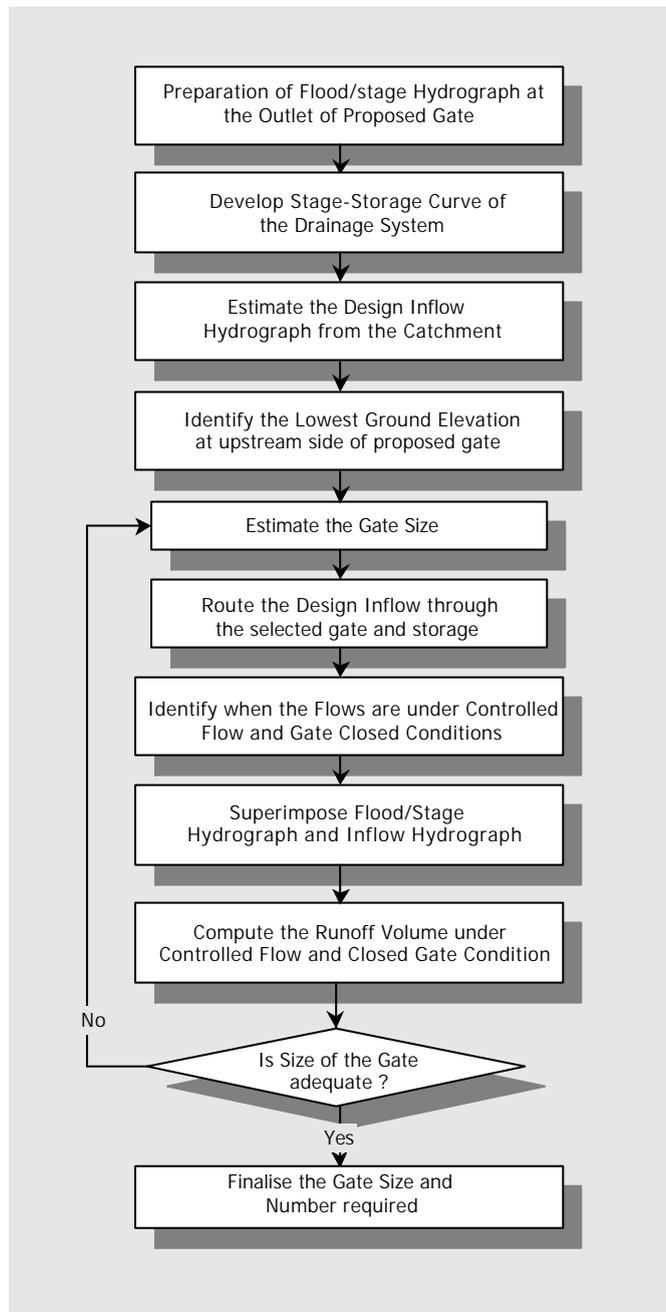


Figure 46.9 Floodgate Design Flowchart

#### 46.8.1 Types of Tidegate and Floodgate

Two types of tidegate are at present in widespread use throughout Malaysia (see Figure 46.10 and 46.11). Similar principles are used for floodgates for agricultural drains and also in the urban areas

##### (a) Culvert Type

Which has an aluminium flap gate with counterweight on the downstream side and a vertical penstock gate on the upstream side. The gate normally works automatically

with outlet discharge taking place when the water level is higher on the landward 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. Equation 20.1 (Chapter 20), which is applicable under all stages of tide/flood levels, can be used to calculate the discharge capacity through the gate. The value of  $C$  is usually assumed as 0.60. The peak flow will occur at the maximum differential head.

##### (b) Open Flume Type

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.

In the case of flume type weir behaviour occurs whenever the differential head between the water level in compartment and that of the outlet channel is equal to or greater than one third of the upstream head. The discharge curve during controlled flow approximates a parabola and the gate discharge can be computed using weir formula given in Equation 20.2 (Chapter 20).

Multiple gates, of either type, can be used to cope with discharge flow, which exceed the capacity of a single bay. Both these types of gates, in a range of sizes, are manufactured at the JPS workshops in Ipoh.

##### (c) Other Types

Flap gates, usually made of fibreglass, cast iron or cast steel, 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. The seat or ring of the flap gate is attached to a headwall or directly to the pipe that forms the opening through which flow passes. Larger flap gates are often mounted to a cast iron thimble, cast in the headwall or other structure through which flow is to pass.

Flap gates are available for round, square, and rectangular openings and in various designs. Flap gates can act as a skimmer and cause brush and trash to collect between the flap and the seat at low flow.

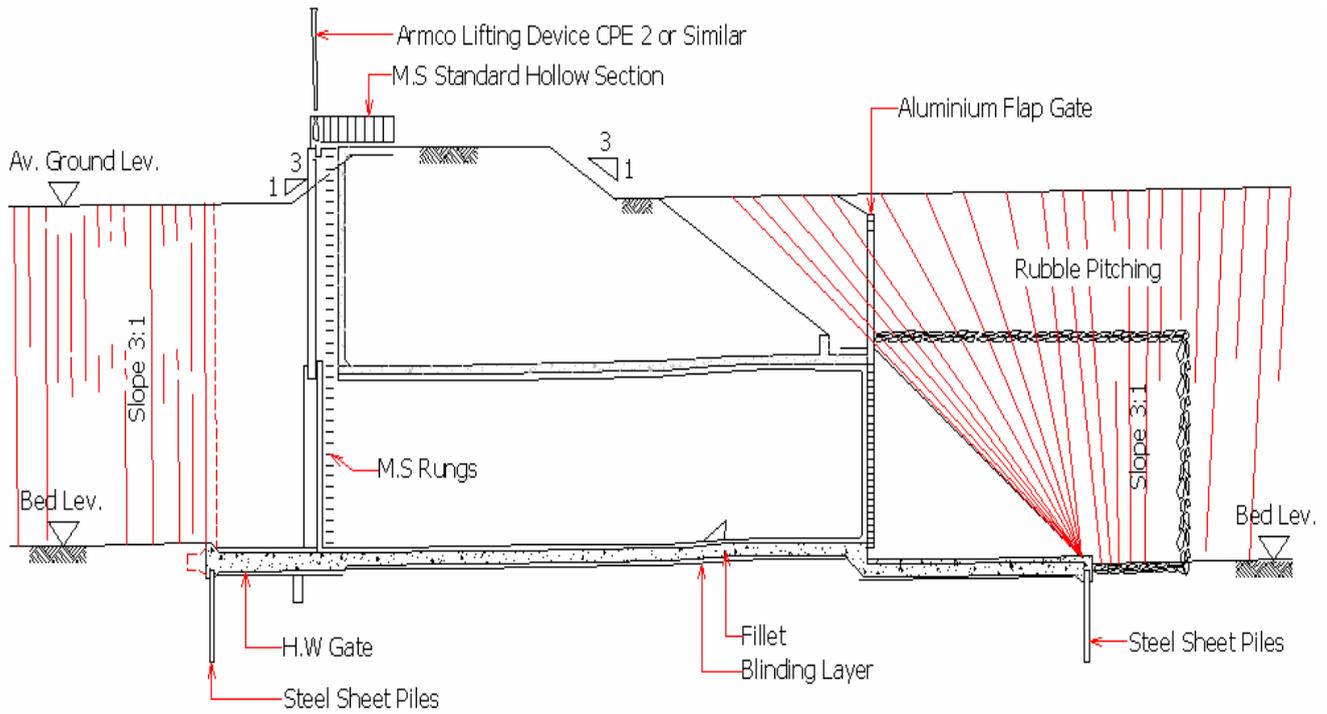


Figure 46.10 Typical Culvert Type Outlet Structure

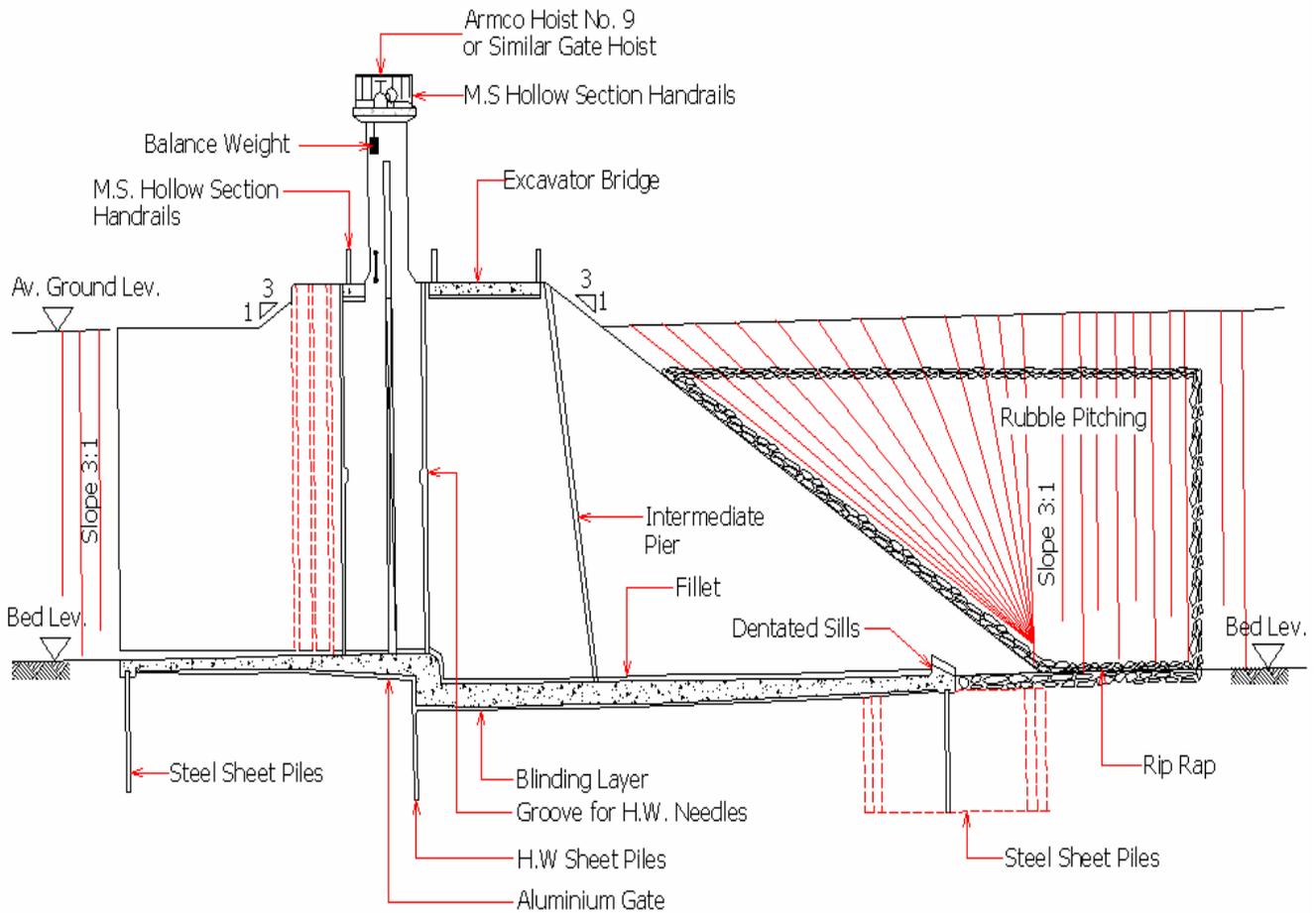


Figure 46.11 Typical Flume Type Outlet Structure

Rubber flap gates and "duck bill" types, 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. This allows at the bottom for floating material to work its way through the gate. When this clearance is not available, more frequent inspection and cleaning must be provided.

For those drainage structures that have a flap gate mounted on a pipe projecting into a stream, the gate must be protected from damage by floating logs during high flows.

There are several other types of tidegate presently in use throughout the world and which might be developed for use in Malaysia. However, for the purpose of the manual, the conventional types described above are preferred rather than on the newer types which are as yet untried in the country and which suffer from certain disadvantages at present, in particular cost (notably foreign exchange component), when compared with the home-manufactured tidegates.

#### **46.8.2 Combined Gates and Pumping Systems**

In some locations, there may be advantages in combining a tidegate or floodgate 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. Design principles for a combined outlet are a combination of the principles for tidegate outlets (see previous sub-section) and for pumps (see Chapter 45). 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.



**APPENDIX 46.A WORKED EXAMPLE FOR TIDEGATE DESIGN**

Problem:

The problem of flooding in the low-laying areas upstream of the Parit Jalan Bakti outlet during exceptional high tide occurs frequently every year. A tidegate is proposed to be constructed to solve the tidal 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 17.0 m<sup>3</sup>/s. Time of concentration at the proposed gate site is about 60 minute. The summarised design data for the area is given below.

Maximum tide cycle at spring tide from the observed tidal cycle as show in Figure 46.A1;

- (a) High Tide Level = 1.5 m LSD
- (b) Low Tide Level = -1.0 m LSD
- (c) Invert Level at Downstream of the Gate = -1.07 m LSD
- (d) Average Ground Level = 1.2 m LSD
- (e) Upstream Bed Level = -0.9 m LSD
- (f) Downstream Bed Level = -1.2 m LSD
- (g) Average Slope of the Drain = 1 in 1500
- (h) Width of the Rectangular Drains = 5 m
- (i) Average Depth of the Drains = 2 m

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 tidegate including pumping facilities if required.

Solution:

*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 can be seen in Figure 46.A1.

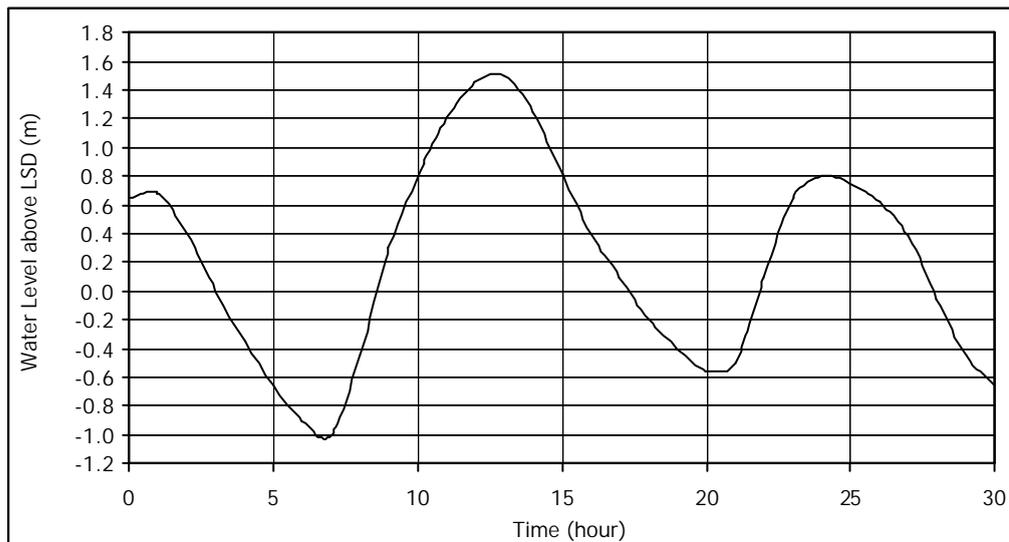


Figure 46.A1 Variation of Tide Levels at Proposed Outlet Gate

Step - 2: Develop Stage-Storage Curve of the Drainage System

The stage-storage relationship is developed based on the double-end area method described in Chapter 20 and shown in Figure 46.A2.

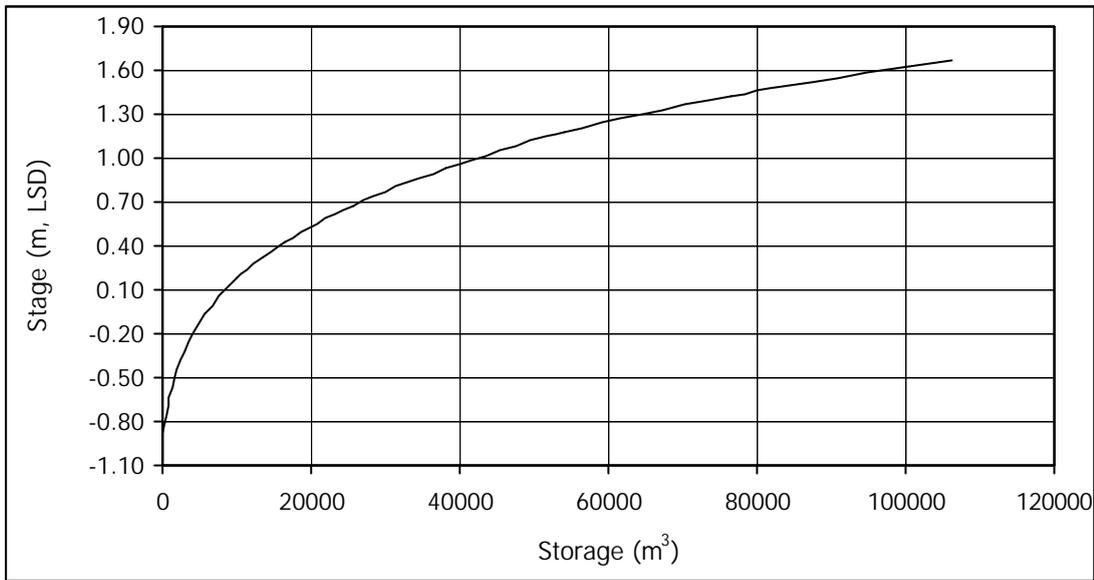


Figure 46.A2 Stage-storage Relationship of the Existing Drainage System

Step - 3: Estimate the Design Inflow Hydrograph from the Catchment

The runoff inflow hydrograph for the critical storm in the catchment of 60 minute duration is shown in Figure 46.A3 as hydrograph 'abc'. However, this will not be the critical storm when storage is considered during the tide period when the tidegate is closed. Hydrograph 'abde' is an assumed trial hydrograph to give the critical storage situation.

Note: In practice, a number of different inflow hydrographs should be trialled.

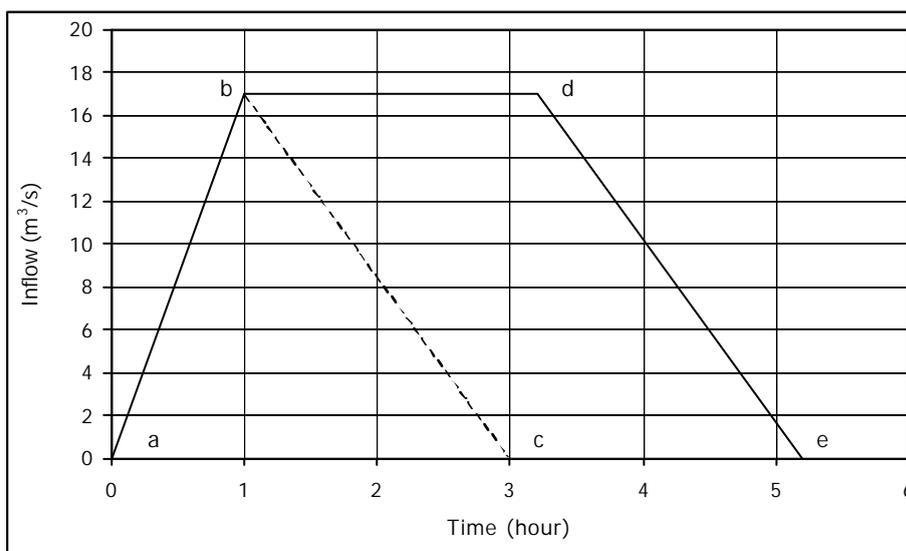


Figure 46.A3 Design Inflow Hydrograph at the Proposed Tidegate

*Step - 4: Identify the Lowest Ground Elevation at Upstream of Proposed Gate*

Lowest ground level at the tidegate site is 1.2 m LSD.

*Step – 5: Estimate the Gate Size*

Assume a 3.5 m wide and 3.0 m high gate.

*Step - 6: Route the Design Inflow through the Selected Gate*

Assuming culvert properties during controlled flow condition discharge through per unit width of the gate is (Equation 20.1),

$$Q_c = 0.85 \times 3.0 \times (2 \times 9.81)^{0.5} \times H^{0.5}$$

$$= 11.3 \times H^{0.5} \text{ cumec/m width of gate}$$

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, from Equation 20.2 maximum discharge per metre width is,

$$Q_m = 1.85 \times H^{1.5} \text{ (here H = average GL – U/S Bed Level)}$$

$$= 1.85 \times 2.1^{1.5}$$

$$= 5.63 \text{ cumec/m width}$$

So the differential head at which maximum discharge will occur at controlled flow condition is,

$$11.3 \times H^{0.5} = 5.63$$

$$H = 0.25 \text{ m}$$

As such with respect to the lowest safe ground level of 1.2 m LSD maximum discharge may occur until the tide level rises at elevation of 0.95 m LSD (1.2 – 0.25 = 0.95 m). In other words the controlled flow situation will start from 0.95 m level.

*Step - 7: Identify when the Flows are under Controlled Flow and Gate Closed Conditions*

The gate is partially and fully closed when the tide level rises at elevation 0.95 m and 1.2 m LSD, respectively. From the tide cycle (Figure 46.A1) period of controlled and no release are 0.9 (i.e. 4.1 – 3.2) and 3.2 hours, respectively.

*Step - 8: Superimpose Tidal Variation Curve and Inflow Hydrograph*

The critical inflow hydrograph is superimposed on the tide cycle (Figure 46.A4) to determine the period of fully open gate condition. Assuming that the stored runoff will be released within one tide cycle (12 hours), duration of fully open gate is 8.8 hour (12 - 3.2).

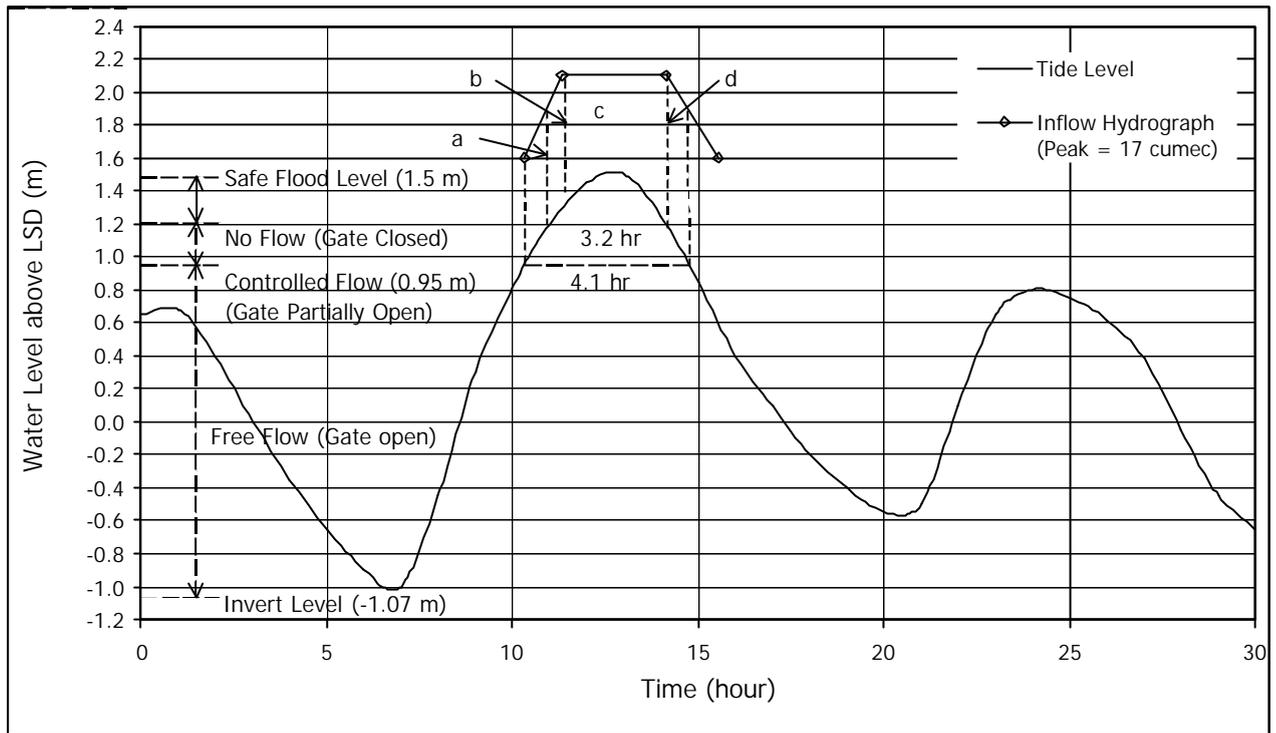


Figure 46.A4 Superimposed Inflow Hydrograph on the Tide Level Graph

*Step - 9: Compute the Runoff Volume under Controlled Flow and Closed Gate Condition*

From Figure 46.A4, runoff volume during controlled flow is the summation of the areas 'a' and 'd'.

Area 'a' =  $0.5 \times 27 \times 7.7 \times 60 = 6,237 \text{ m}^3$  and

Area 'd' =  $0.5 \times (7.7 + 17) \times 27 \times 60 = 20,007 \text{ m}^3$

So, total controlled runoff volume is  $26,244 \text{ m}^3$  ( $6,237 + 20,007$ ).

Similarly, runoff volume during zero flow condition is the summation of the areas 'b' and 'c' (Figure 46.A4).

Area 'b' =  $0.5 \times (7.7 + 17) \times 33 \times 60 = 24,453 \text{ m}^3$

Area 'c' =  $17 \times 159 \times 60 = 162,180 \text{ m}^3$

So total stored runoff during closed gate condition is  $186,633 \text{ m}^3$  ( $24,453 + 162,180$ ).

Assuming  $1/3$  of the inflow runoff ( $8,748 \text{ m}^3$ ) 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 =  $(2/3) \times 26,244 = 17,496 \text{ m}^3$

So, total runoff volume to be released during uncontrolled condition =  $17,496 + 186,633 = 204,129 \text{ m}^3$ .

For actual release through the tidegate, 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  $8,748 \text{ m}^3$  of water within 0.9 hrs. So, the average release rate is  $0.77 \text{ m}^3/\text{m width}$   $\{8,748 / (0.9 \times 60 \times 60 \times 3.5)\}$ . Storage at controlled gate condition, i.e. at water level of 0.95 m LSD is  $40,000 \text{ m}^3$  (from Figure 46.A2).

If no pump is used then water volume stored in the drains from the beginning of the controlled flow condition is 155,389 m<sup>3</sup> (40,000 + 204,129). Corresponding water level at this storage will spill over the banks where the ground levels are 1.2 m LSD (from Figure 46.A2)

Storage at safe water level in the drain (1.2 m LSD) is 56,000 m<sup>3</sup> (From Figure 46.A2).

*Step - 11: Finalise the Gate Size, Pump Capacity and Number Required*

So, volume of stored runoff to be pumped is 188,129 m<sup>3</sup> {204,129 – (56,000 - 40,000)}. 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, 33,000 m<sup>3</sup> (87,000 – 54,000) of water can be stored safely within storage level from 1.2 m to 1.5 m LSD (from Figure 46.A2). The excess volume of water to be pumped is 155,129 m<sup>3</sup> (188129 – 33,000).

As such, the average required pumping rate during the controlled flow condition {4.1 – (0.9/2) = 3.65 hr} is 11.81 cumec 155,129/(3.65x60x60).

So, three (3) large submersible pumps with effective capacity of 4 cumec is proposed to discharge the flood volume safely. The pumps can be operated at different stages (water level at the upstream side of the gate). For detail please refer to Chapter 45.

Any critical situation such as power failure, pump breakdown, etc. may occur during the storms and when the floodgate 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 during 7.9 hr (12 – 4.1) is 2.05 cumec/m width {204,129 m<sup>3</sup>/(7.9 hr x60x60x3.5 m)}.

Number of gate required to release peak inflow is 2.37 Nos. {17/(2.05x3.5)}. So, the gate opening should be 8.3 m wide (2.37 No.x3.5 m). In this case 2 gates of 4.5 m wide are recommended.

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 tidegate outlet and pumping options, which allows the rapid testing of a number of design storms and tidegate and pump configurations.