CHAPTER 6

CHAPTER 6

GROSS POLLUTANT TRAPS

6.1 INTRODUCTION

Gross pollutant traps are used to remove gross solids (including coarse sediment) and litter from stormwater. A number of definitions of gross solids are currently used in the industry most of which centre around the definition of the size of these solids. Allison *et al.* (1998) defines gross solids as any solids that are larger than 5 mm. Essentially gross pollutant traps combine the mechanisms of gross solid interception and retention. With the main mechanism of treatment being the process of settling and interception, the fundamental features of most GPTs are the utilisation of a energy dissipation device (to facilitate settlement of coarse sediment and non-floatable objects), a sedimentation basin to retain settled material and trash racks to intercept gross solids. GPTs would typically be comprised of a concrete lined wet basin and a trash rack with provision for maintenance and cleaning of the basin.

There are now a number of devices for the trapping of gross pollutants that are based on initially diverting stormwater to a separation and retention chamber in which these pollutants are subjected to the mechanisms of interception and sedimentation. The diversion device allows stormwater to by-pass the separation chamber in the event of blockage of the chamber due to excessive accumulation of gross pollutants and during above-design events.

There are limited definitive guidelines for the design of gross pollutant traps with the most amount of research work being carried out in the area of sedimentation basin design. Issues related to the size of trash rack, design flows, operation under above design conditions and optimal dimensions for the trap are often dealt with in a cursory manner. There are many individually designed GPTs currently in use, most of which have been designed to promote the key mechanisms of gross solid interception and retention. In this chapter, a broad overview of some of the common GPTs in use is presented to provide the reader with an appreciation of the key features of these devices to enable the reader to select and design desirable features applicable to their individual requirements. The chapter also outlines three possible standard types of GPTs for the Putrajaya project to facilitate ease of design, construction and maintenance.

6.2 OVERVIEW OF COMMON GROSS POLLUTANT TRAP

6.2.1 Trash Racks and Sediment Basin

Canberra GPTs

Conventional GPTs such as those developed for use in Canberra, Australia consist of a trash rack placed immediately downstream of a sediment basin. Two types are proposed for use, ie. the minor GPT, for installation at the pipe/open waterway interface and the major

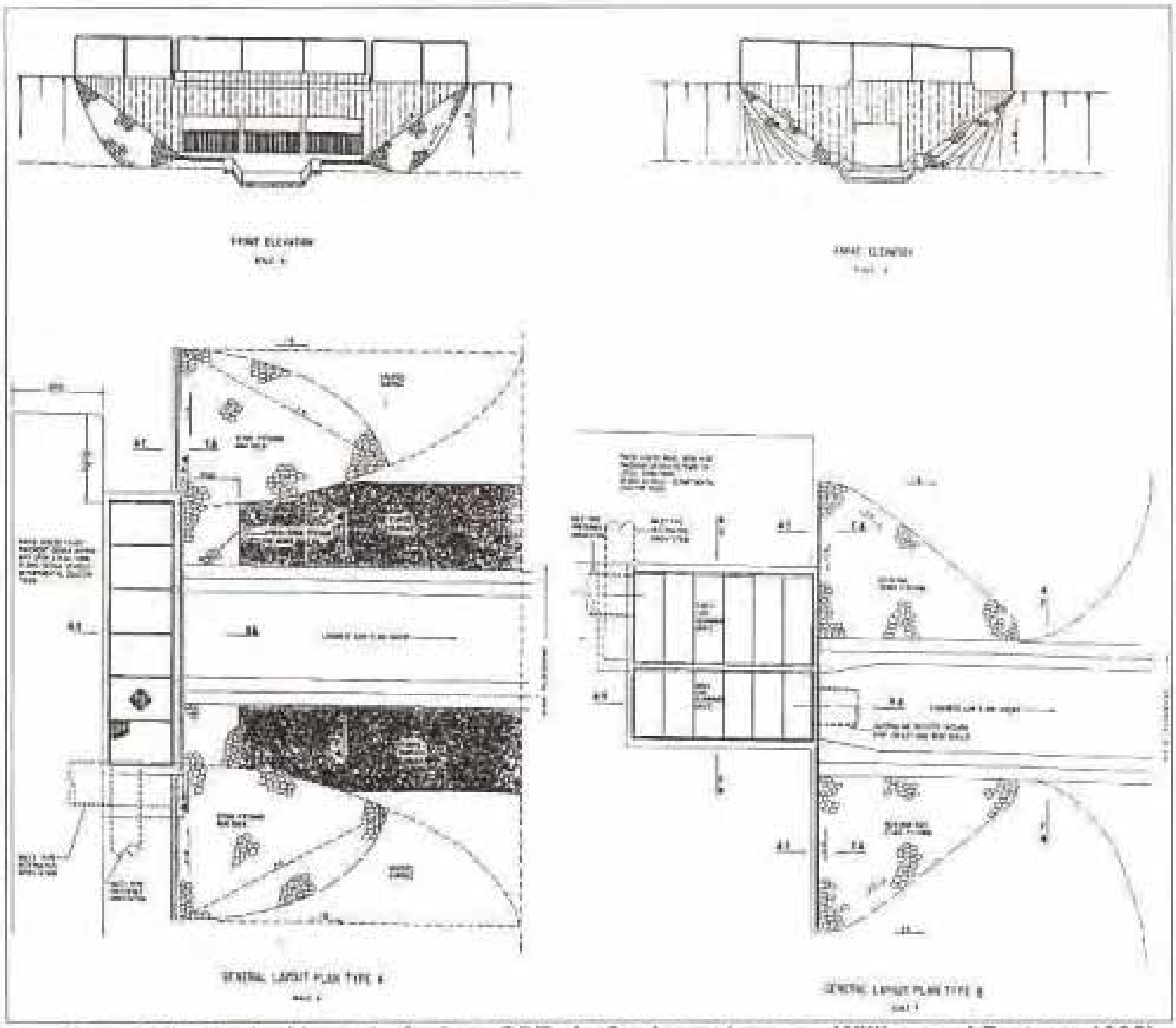


Figure 6.1 - Typical layout of minor GPTs in Canberra (source: Willing and Partner, 1988)

GPT which is intended for placement on open waterways. Figure 6.1 is a typical layout of a minor GPT recommended for Canberra stormwater systems as presented by Willing and Partners (1989). The prominent feature of the trap is the mechanism by which energy dissipation is promoted. Incoming flow is generally diverted at right angle such that the flow into the sediment basin is along the length of the trash rack. This allows for a high degree of energy to be dissipated by means of water impact on the wall of the sediment basin. If the incoming flow is aligned perpendicular to the trash rack, an energy dissipation chamber is used to divert the incoming flow to an alignment, which is along the length the trash rack. Figure 6.1 shows two possible orientations for the GPT.

A second type of GPT commonly used in Canberra is referred to as the major GPT as shown in Figures 6.2 and 6.3 This type of GPT is utilised on open channel systems and are generally much larger. The various components of the GPT remain much the same with the trash rack and the sedimentation basin being the two prominent feature of the GPT. No energy dissipation structure is used, instead, an expanded flow section reduces flow velocity in the sedimentation basin. The width of the sediment basin determines the flow velocity approaching the trash rack, which in turn defines the sediment capture efficiency of the sedimentation basin.

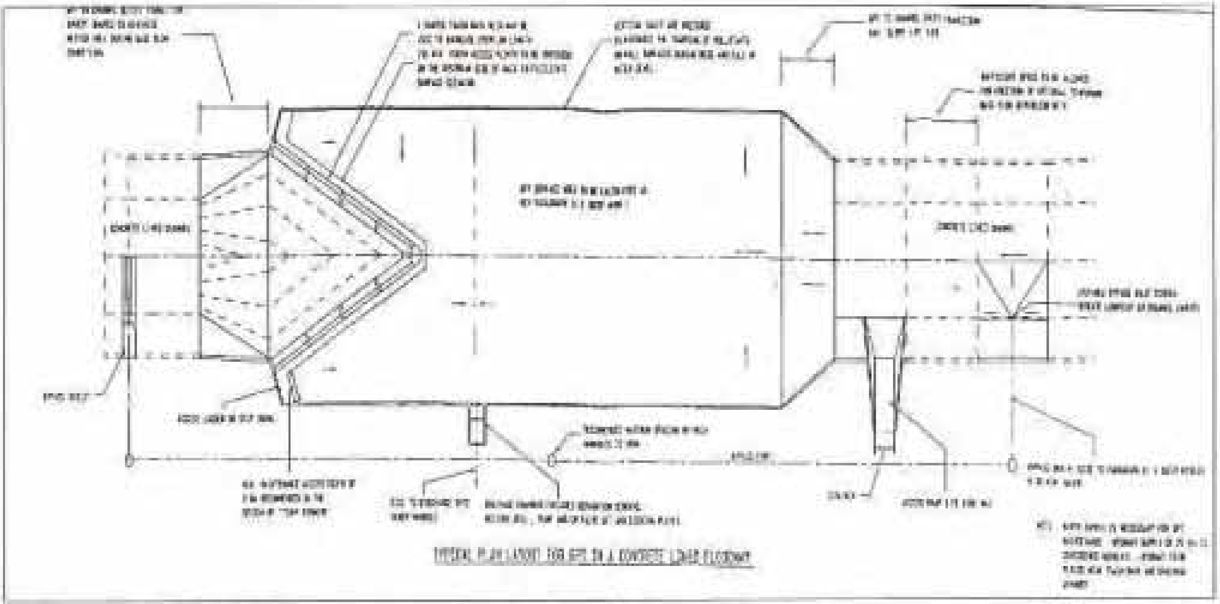


Figure 6.2 - Typical layout of a major GPTs in Canberra (source: Willing and Partners, 1989)

Vertical bars are commonly used the trash rack. Standard bar spacing is often between 50 mm and 150 mm depending on the minimum size of the gross pollutant to be captured. In all cases, the height of the trash rack needs to be carefully selected to meet a number of performance criteria including non-submergence during design flow conditions, minimum reduction of the discharge capacity of the upstream flow conveyance system in situations when the trash rack is fully blocked, public safety etc. Notional height of the trash rack is often between 0.3 m to 1.2 m.

Figure 6.3 Major gross pollutant trap in Canberra (ref. Allison *et al.*, 1997)

Recommended gross pollutant traps for the Putrajaya catchment are based on some features of the Canberra GPTs. This will be discussed in some detail in Section 6.3. Experience with the Canberra gross pollutant traps suggest that the traps have the tendency to deteriorate rapidly in their capture efficiency if trapped gross pollutants are not regularly removed. In some cases, removal of captured litter may be required as frequently as once a week.

Trash Racks

Trash rack can also be used as stand-alone units principally for the removal of flow debris in urban waterways. The design for these racks involve vertical bar screens placed across a channel perpendicular or inclined to the direction of flow. Beecham and Sablatnig (1994) investigated the efficiency of 24 litter racks, and identified six arrangements that had the potential to improve trapping rates and reduce maintenance requirements, given particular in Figure 6.6. These baskets can be made from metal or plastic.

Litter baskets are individually effective but require good coverage of the source area for this particular measure to have a significant regional impact. They are most appropriately used in Putrajaya as an additional measure in high source areas such as food courts and shopping centers and only when there is a clear program of maintenance, which must include emptying these baskets weekly. Emptying of the baskets is usually carried out using a vacuum truck.

Figure 6.6 Litter Control Devices located at the pipe outfall (ref.Allison et al, 1997)

6.2.3 Litter Booms

Litter booms are typically installed across a waterway (pond, channel or creek) to collect floating and partially submerged trash and debris. Trash and debris collected in booms is removed manually and success to date has been mixed.

Floating booms only collect floating material and are largely ineffective for capture of material that becomes waterlogged and partially sinks. Waterlogged material and naturally buoyant material such as plastic bags can be dragged under the boom by the flow of water. Pollutants which are soluble and heavier than water will not be trapped in the boom. Trapped material are generally unsightly. The use of litter booms in the Putrajaya project should be considered a "last line of defence" measure in isolating the flow of floating litter to a small section of the receiving waters.

Most floating litter boom installations have the boom attached at points on opposite sides of the channel with sufficient slack to allow the boom to form a semi-circle under normal flow conditions. This shape results in the collected litter accumulating at the centre of the boom, which is also the centre of the channel and the region of highest velocity. High flow velocities can drag collected litter under the boom and/or allow the boom to twist and become less effective at trapping and retaining trash and debris.

Boom performance can be improved by angling the boom across the channel to allow the collected trash to accumulate on one side of the channel, away from the high velocity area. In addition, the trash could be collected in a mesh container which will retain trash during high flows, and attached to the side of the channel within easy reach of the bank for cleaning. Freeman (1995) notes that an alternative boom configuration involves angling the boom across the channel and directing the litter to a meshed cage for collection.

6.2.4 Continuous Deflective Separation Devices

Continuous deflective separation devices are essentially a type of gross pollutant trap designed to capture coarse sediment, trash and debris. This device is installed below ground requiring a typical area of between 10 m^2 and 20 m^2, depending on the design operating flow. The device is highly efficient and does not have the problem of screen blockage by flow debris commonly associated with conventional forms of gross pollutant separation from stormwater. Maintenance requirements of the device is also lower than conventional devices as the materials captured within the trap do not inhibit the functionality of the continuous deflective separation mechanism. The device is a propriety device owned by CDS Technologies, an Australian public company based in Melbourne. The device can be expensive and is recommended for high profile areas within Putrajaya as well as at locations where low frequency of maintenance is desirable.

The CDS device consists of a solid deflection unit that deflects stormwater and associated gross pollutants into a circular separation and containment chamber as shown in Figure 6.7. The mechanism by which the CDS technology separates and retains gross pollutant is by first diverting flow and associated pollutants in a stormwater drainage system away from the main flow stream of the pipe or channel into a pollutant separation and containment chamber. The separation and containment chamber consist of a containment sump in the lower section and an upper separation section. Gross pollutants are separated within the chamber using a perforated plate allowing the filtered water to pass through to a volute return system and thence to the outlet pipe. The water and associated pollutant contained within the separation chamber are kept in continuous motion by the energy generated by the incoming flow. This has the effect of preventing the separation plate from being blocked by the gross solids separated from the inflow. The heavier solids ultimately settle into the containment sump.

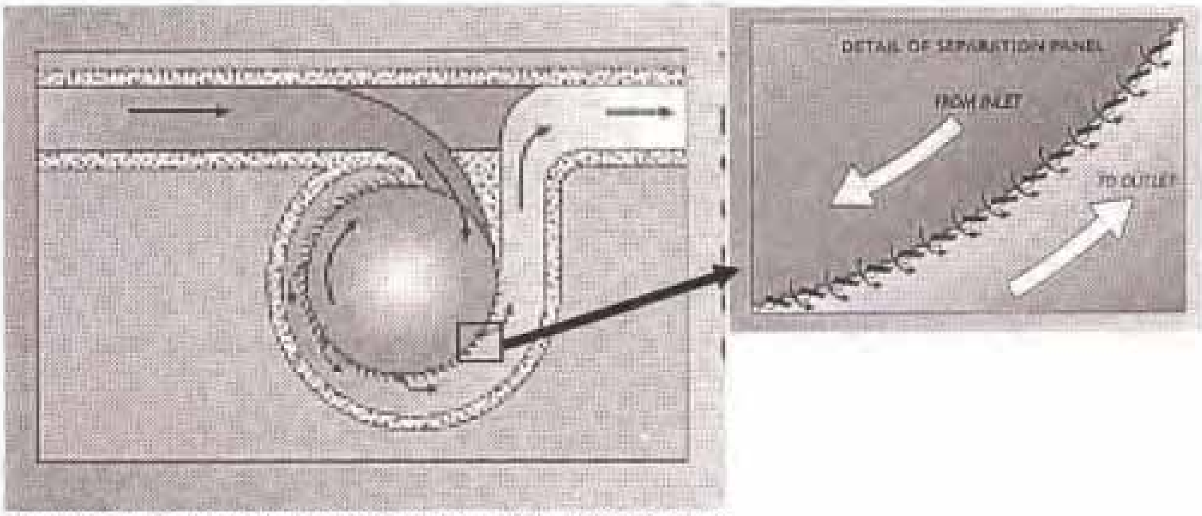


Figure 6.7 Schematic Representation of the CDS System

The diversion of the stormwater and associated pollutants into a separation chamber overcomes problems associated with the direct filtration systems of conventional gross pollutant traps. The present design of the CDS system utilises a simple solid diversion unit to divert flows into the separation chamber. The diversion unit is designed to divert all flows into the separation chamber as long as water levels are below the crest level of the diversion unit. As water levels exceed the crest of the diversion unit, some flows would by-pass the CDS system. The crest level of the diversion unit may be adjusted to suit individual installations.

The solid separation system consists of a large expanded stainless steel plate, which acts as a filter screen with an outer volute outlet passage. The perforations in the separation screen are typically elongated in shape and are aligned with the longer axis in the vertical direction. The size of the elliptical holes can be specified according to performance requirements and typical width of the short axis ranged from 0.6 mm to 4.7 mm. Typical size of the perforation for use in urban stormwater system is 4.7 mm by 10 mm. The separation screen is installed in the unit such that the leading edge of each perforation extends into the flow within the containment chamber. The bypass system can be designed to minimise the impact on upstream flood levels if the system becomes blocked. If the unit discharges directly to a watercourse, some type of outlet scour protection will be appropriate, such as gabions or riprap.

Laboratory tests by Wong *et al.* (1995) with graded sediment particles and a separation screen perforation size of 2.4 mm by 7 mm found the CDS device to be a very effective sediment trap. Close to 100% capture of particles of sizes greater than 1.5 mm or 63% of the minimum aperture size of the separation plate were found. Figure 6.8 shows the sediment performance curve of the CDS device derived from laboratory tests conducted by Wong *et al.* (1995).

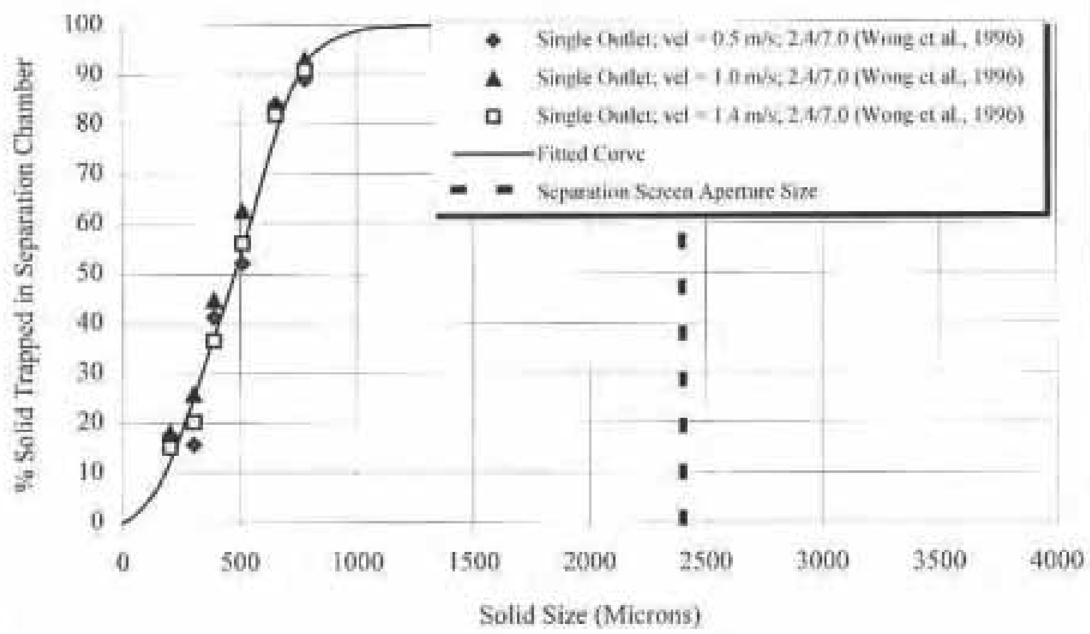


Figure 6.8 Sand Trapping Efficiency of CDS Unit with Aperture Size of 2.4 mm

The installation of the CDS device can be carried out while keeping the existing stormwater system in operation. Allison *et al.* (1996) describe the installation process as follows:-

1. Construction commenced with excavation of land adjacent to the stormwater pipe (Figure 6.9).
2. The steel reinforcements for the base of the CDS unit are then laid and concreted (Figure 6.10). In addition, the separation sump, which is made of a standard precast pipe, is also fitted to the base.
3. The shell of the separation chamber is then cast as shown in Figure 6.11.
4. The separation screens are installed then installed as shown in Figure 6.12. The separation screens are delivered in sheets and were mounted in the separation chamber with standard stainless steel bolts.
5. As the main structure neared completion, the existing stormwater pipe can then be cut open in preparation for its connection to the CDS unit. Figure 6.13 shows the stormwater pipe exposed and preparation made to extend the CDS unit diversion structure into the pipe. Figure 6.14 shows the completed unit fully contained underground.

Figure 6.9 Excavation down to the floor of the chamber (6x6x4m).

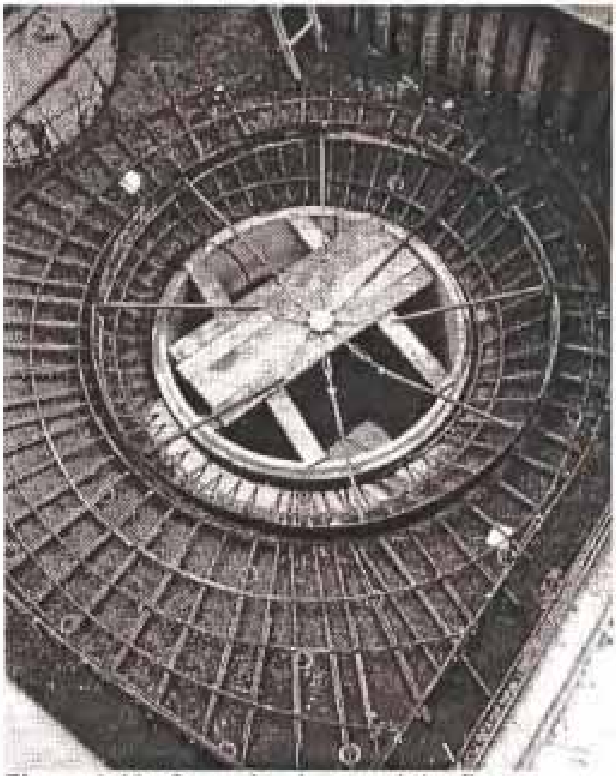


Figure 6.10 Sump in place and the floor reinforcement tied.

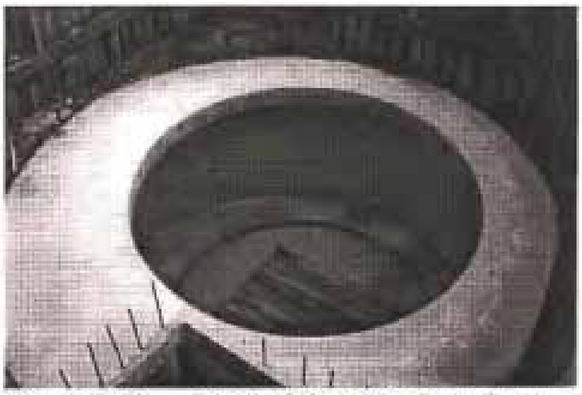


Figure 6.11 Shell of the chamber where the screens will be placed

Figure 6.12 Fitting the separation screens.

Figure 6.13 Breaking and connection to the adjacent to the stormwater pipe

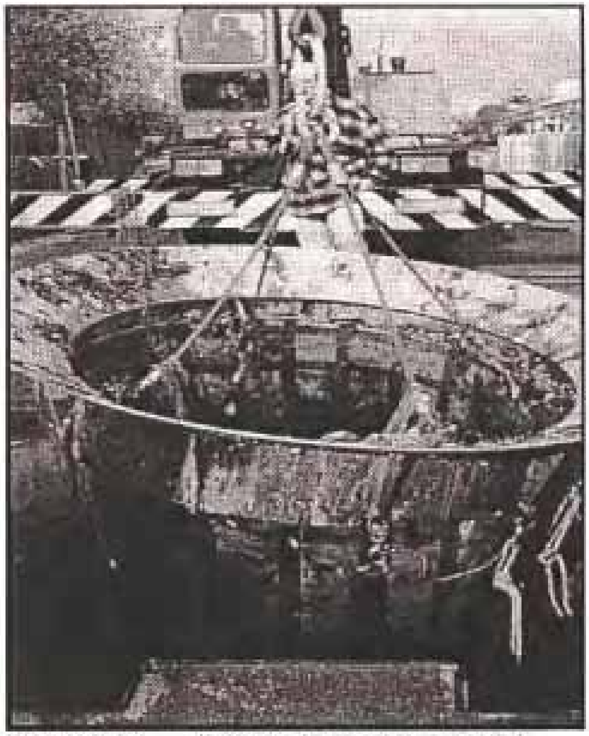


Figure 6.15 Removal trapped material from a CDS basket (ref. Allison et al, 1997)

Figure 6.14 View of the completed construction at road level.

Figure 6.16 Removal of trapped material in the CDS device using a vacuum truck (ref. Allison et al, 1997)

Removal of trapped materials from the containment sump of the CDS device can be carried out using any one of three possible methods, ie. containment basket, suction and excavator. For small CDS units, the use of a containment basket is often adopted. The basket is placed in the containment sump and clean out of the trapped material involves the use of a truck-mounted hydraulic crane to lift the basket out of the sump as shown in Figure 6.15 and the material emptied into the truck. Vacuuming of the trapped material is also often used for these devices in Australia as shown in Figure 6.16.

6.3 STANDARD GPTs FOR THE PUTRAJAYA PROJECT

Apart from the recommended use of the CDS device for high profile areas, three standard types of GPTs have been proposed for the Putrajaya project. This is to facilitate ease of design, construction and maintenance. The three standard types are as follows:-

GPT Type I	-	enclosed unit applicable for in-line minor stormwater conveyance systems, either open drains or underground pipes;
GPT Type II	-	enclosed systems applicable for underground pipe outfall to receiving waters
GPT Type III	-	open systems applicable for open channels

In designing GPTs, a number of design factors related to the performance of these devices need to be taken into consideration. These factors include:-

- Layout
- Design flow
- Above design flow performance
- Expected gross pollutant load
- Minimum dimensions
- Types and minimum size of litter to be intercepted
- Maintenance

The following are the general design specifications applicable to all three types of GPTs. Sections 6.4 to 6.6 outline the computation steps and selection considerations that are particularly directed at designing the respective gross pollutant traps. Worked examples in each case are also presented.

6.3.1 General Design Specifications

Design Flows (Q_{GPT})

The appropriate design flow for the GPT varies from one application to another. Quite often, the determination of the appropriate design flow is unclear. The design discharge is primarily used to define two features of the GPT, ie.

- the minimum height of the trash rack such that all flow at or below this design discharge will pass through the trash rack, ie the water surface levels for flow at or below this discharge are below the top of the trash rack; and
- the minimum dimension of the sediment basin.

The 1 year ARI peak discharge is often adopted as the design discharge.

Above Design Flow Conditions (Q_{minor})

All GPTs are required to operate adequately for larger events up to the discharge capacity of the conveyance system (Q_{minor}) in which the GPTs are placed without decreasing the discharge capacity of the conveyance system. Generally, $Q_{minor} \leq Q_{100}$.

Trash Rack

The trash rack is to be constructed from steel using rectangular bars welded at close spacing to form the grill. The grill spacing of the trash rack determines the size of gross pollutant to be trap for operating conditions up to the design flow. Current utilisation of trash racks employ a grill spacing which is typically between 25 mm and 100 mm. For the Putrajaya project, a grill spacing of 50 mm is recommended.

Under above-design flow conditions or when the trash rack is blocked, the trash rack or trash sill (in the cast of GPT Type 1) will have the characteristics of a weir with the crest of the weir being the top of the trash rack. Standard weir coefficient for a sharp-crested weir could be used to compute the discharge characteristics of the trash rack. This factor however will vary from between 1.2 to 1.7 depending on the alignment of the trash rack to the inflow streamline. These details are discussed in subsequent sections.

Adjustment to the weir coefficient will also need to be made under submerged weir flow conditions. This situation should ideally be avoided under design conditions but may occur during above-design conditions. Appropriate adjustment factors for different degree of weir submergence are shown in Figure 6.17 (Bradley, 1978). Submergence is measured as the

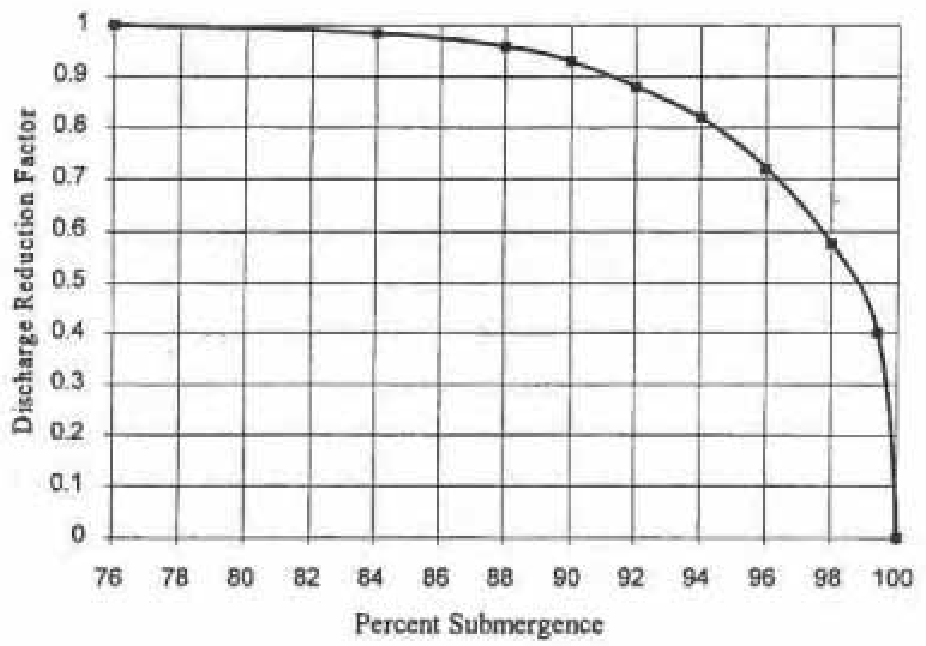


Figure 6.17 Factor for Reducing Weir Flow Coefficiency for Submergence

percentage ratio of the water depth above the crest of the weir on the downstream side of the weir to the corresponding depth on the upstream side, ie. (H_{ds}/H_{us} x 100%) where H is the depth of water above the crest of the weir.

Gross Pollutant Loading

The dimensions of the sediment basin defines the likely frequency for sediment basin cleaning. This is dependent on the expected rate of sediment and gross pollutant exported from the catchment and the capture efficiency of the GPT. Studies by Allison *et al.* (1998) have suggested a nominal annual gross pollutant load (defined as material greater than 5 mm in size) of approximately 90 kg/ha/yr (wet weight) with a typical pollutant density (wet) of 250 kg/m^3 and a wet to dry mass ratio of approximately 3.3 to 1. This gives the expected volume of total gross pollutant load of approximately 0.4 m^3/ha/yr. Allison *et al.* (1998) also found that the rate of gross pollutant loading is highly correlated with rainfall and suggests that the limiting mechanism is not the supply of gross pollutant but the mobilisation and transportation of these solids.

For Putrajaya which has some 3 times the annual rainfall of Melbourne, it is reasonable that the above pollutant loading rate should be adjusted to reflect the higher rainfall conditions of the Putrajaya catchment. In the absence of further information, a factor of 3 has been applied to the loading rate to give 1.2 m^3/ha/yr.

Allison *et al.* (1998) found a high proportion of the total gross pollutant load to consist of vegetation (ie. leaves) and that urban derived litter constitutes approximately 30% of the total gross pollutant load. This gives the expected litter load to be 81 kg/ha/yr (wet weight) or 0.3 m^3/ha/yr.

Data provided by Willing and Partners (1992) on the amount of expected sediment (particle size > 0.01 mm) export from urban catchments in the Canberra region suggests a rate between 1.2 tonnes/ha and 2.5 tonnes/ha depending on the degree of urbanisation in the catchment with a density of 2.65 tonnes/m^3 and a sediment porosity of 0.42. The expected volume of sediment exported from an urban catchment in the Canberra region is thus approximately 1.6 m^3/ha/yr. Again, this needs to be adjusted for higher rainfall conditions in Malaysia, giving a sediment loading rate of approximately 4.8 m^3/ha/yr. According to the Putrajaya Drainage Masterplan (AGHD, 1996), the adopted annual volumetric rate of gross pollutant and sediment exported is approximately 4.5 m^3/ha/yr.

It is recommended that a sediment & gross pollutant loading rate of 5 m^3/ha/yr be adopted. This rate is strictly applicable to catchments which are fully developed for residential and commercial landuse. Adjustments, by as much as 75%, will need to be made if applied to open grassed and forested areas.

6.4 GPT TYPE I

6.4.1 General

GPT Type I units are intended for in-line installation in the minor stormwater drainage system which may either be open drains of underground pipelines. Suitable site conditions for Type I GPT include relatively steep terrain where the slope along the alignment of the minor drainage system is sufficiently steep to allow a drop structure to be introduced.

Figure 6.18 shows a schematic diagram of the trap for an underground pipeline system. The mechanism of gross pollutant capture is based on using an inclined trash rack placed on the horizontal plane at or below the inlet pipe invert. The configuration of the Type I trap is different from conventional GPT in that flow passes through the trash rack vertically thereby allowing a smaller area requirement for the trap. The trash rack is slightly inclined to allow trapped material to move downstream towards the sill of the trash rack. The sill acts as a weir control such that above-design flow conditions can be efficiently discharged over the weir without reducing the discharge capacity of the drainage system. The sediment basin is located beneath the trash rack.

Specifications for the size of this type of GPT are based in part on the hydraulic performance of the unit under a range of operating conditions and in part on subjective judgement.

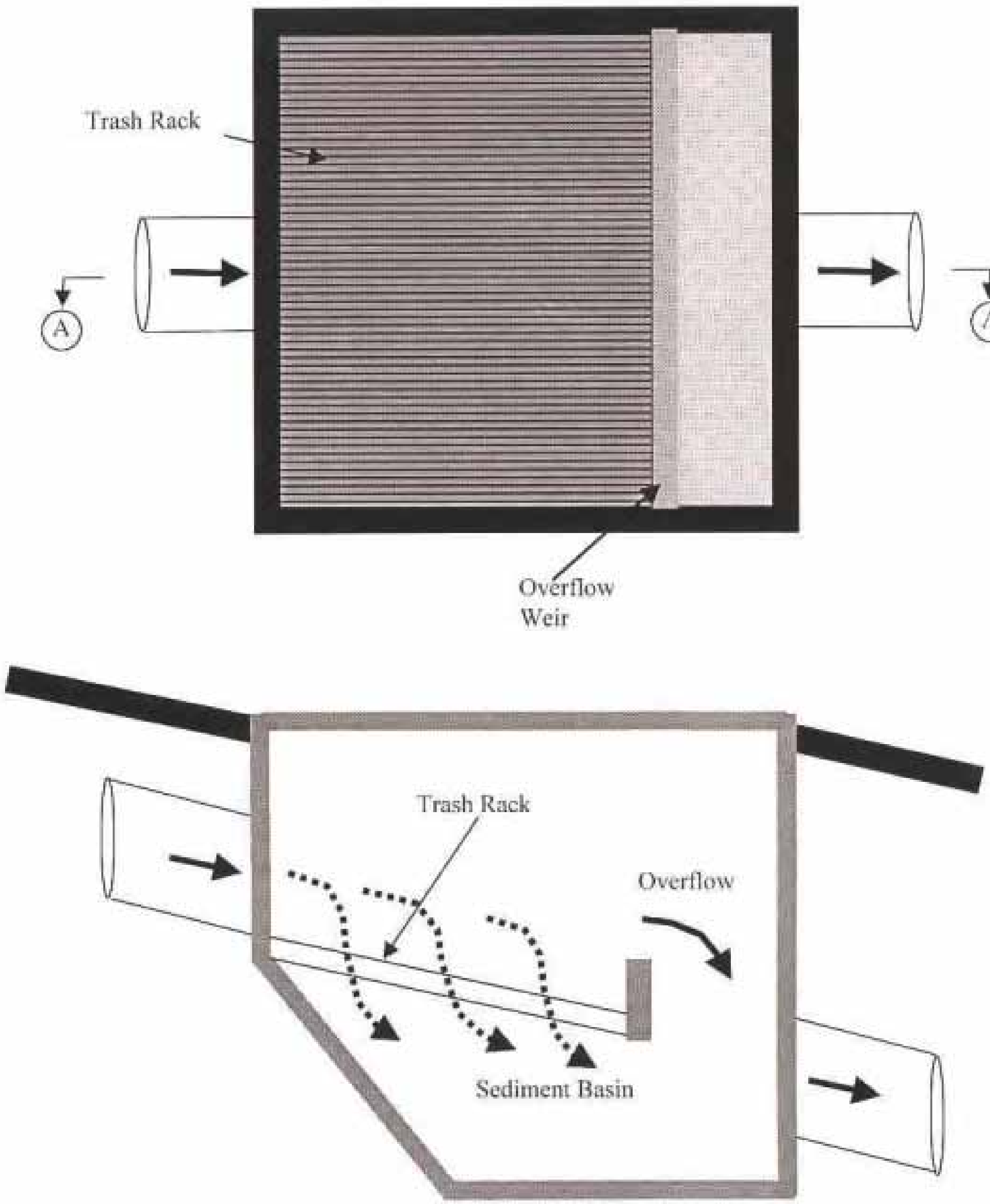


Figure 6.18 Putrajaya GPT Type I

6.4.2 Trash Rack Sill (or overflow weir)

6.4.2.1 The function of the trash rack sill is to enable trapped debris to be retained on the trash rack. A notional height of the sill would be the greater of the depth of flow in the inlet pipe under the design flow or half the diameter of the inlet pipe. In the case of an open drain system, the height of the sill should not be lower than the depth of flow under design flow conditions; ie.

$$H_{sill} = d_{inflow} \text{ or } 0.5D\text{, whichever is the greater} \qquad - \qquad 6.1$$

where H_{sill} is the height of the sill, d_{inflow} is the depth of the inflow under the design flow conditions and D is the diameter of the pipe.

6.4.2.2 The distance from the overflow weir to the downstream wall of the GPT should be such that the opening for overflow from the trash rack would not be blocked by flow debris. For an underground stormwater pipe system, the distance between the sill and the downstream wall should not be less than the diameter of the inlet pipe. For open drain systems, the corresponding distance should not be less than the width of the inlet drain, ie.

$$L_{of} = D \text{ or } B \qquad - \qquad 6.2$$

where L_{of} is the distance from the overflow weir to the downstream wall, D is the diameter of the inlet pipe and B is the width of the inlet drain.

6.4.3 Width of the Trash Rack and GPT

6.4.3.1 The width of the trash rack and the GPT is essentially defined by the required width of the overflow weir such that above-design discharges are conveyed over the concrete weir with minimum impact on the tailwater conditions of the inlet pipe or drain. The suggested weir coefficient for the overflow weir is 1.5. The depth of water (H_{of}) over the crest of the weir can be computed according to equation 6.3:-

$$H_{of} = 1.5\left(\frac{Q_{minor}}{W}\right)^{0.67} \qquad - \qquad 6.3$$

where H_{of} is the depth of water above the overflow weir and W is the width of the overflow weir, which is also the width of the trash rack and GPT.

The computed water surface elevation upstream of the overflow weir under Q_{minor} is given as follows:-

$$TW_{minor} = EL_{sill} + H_{of} \qquad - \qquad 6.4$$

TW_{minor} should not be higher than the obvert of the inlet pipe.

6.4.3.2 In the case of an open drain system, the water level upstream of the overflow weir should not exceed the normal depth of the open channel under Q_{minor}. Normal depth can be calculated using Manning's equation.

6.4.4 Length of the Trash Rack

The length of the trash rack is determined from considerations of the expected volume of litter captured by the trash rack, the dimension of the inlet pipe or open channel and the expected trajectory of material discharged from the inlet pipe or drain. The appropriate length needs to satisfy three conditions associated with these factors.

6.4.4.1 *Condition 1:-* As a general rule, minimum length of the trash rack is the diameter of the inlet pipe or the normal depth of the inlet drain at Q_{minor}.

6.4.4.2 *Condition 2:-* The available storage volume of trapped litter on the trash rack can be approximated by multiplying the area of the trash rack with the height of the sill. The volume should not be less than the expected volume of litter generated over a six month period, ie. 0.15 m^3/ha. The following equation applies:-

$$\left(W \times L_{rack} \times H_{sill}\right) \geq 0.15A_{catchment} \qquad - \qquad 6.5$$

where L_{rack} is the length of the trash rack, H_{sill} is the height of the sill and $A_{catchment}$ is the catchment area.

6.4.4.3 *Condition 3:-* The length of the trash rack needs to be longer than the trajectory distance of the incoming gross pollutant under the design flow conditions. The expected trajectory of floating material is expected to be the same as that of the water surface profile and will be dependent on the depth of flow and velocity of the incoming flow. The relationship for a typical nappe profile of a free overfall conditions can be use to estimate the horizontal trajectory distance of floating material conveyed by the incoming flow. There are a number of empirical equations for the lower and upper nappe profiles of a free overflow surface (eg. Rand, 1955, USBR, 1973, Donnelly and Blaisdell, 1965). Non of them are strictly applicable in this case for defining the appropriate length of the trash rack. The upper nappe profile is dependent on the discharge and the chart provided by Donnelly and Blaisdell (1955) is presented in Figure 3.18 for use in the Putrajaya project.

Figure 6.19 relates the ratio (y/d_c) to (x_u/d_c) for a family of tailwater conditions as defined by the ratio (y_t/d_c). The minimum length of the trash rack can thus be computed by reading the value of (x_u/d_c) for the relevent structure configuration and multiplying the ratio bythe critical depth. In the case of the trash rack, the ratio (y/d_c) will be zero.

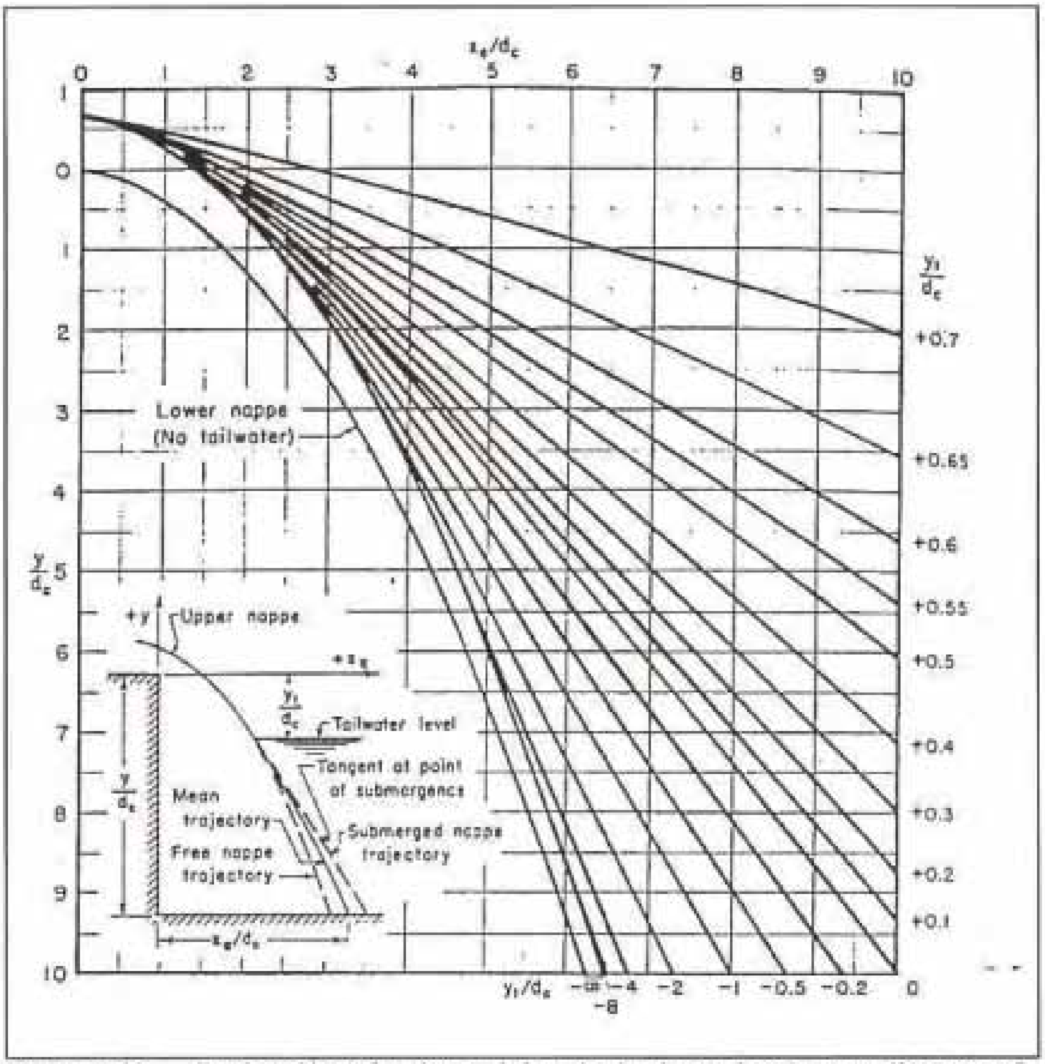


Figure 6.19 Design Chart for determining the horizontal trajectory distance of a free overflow water surface (Donnelly and Blaisdell, 1965)

The length of the trash rack should be longer than L_{traj}, ie.

$$L_{rack} > L_{traj} \qquad 6.6$$

6.4.5 Depth of Sediment Basin

The depth of the sediment basin is based on achieving three conditions:-

- maximum depth for ease of maintenance
- a maximum velocity that would not scour deposited sediment under design flow (Q_{GPT}) conditions. The suggested maximum velocity under design flow conditions is 0.5 m/s.
- sufficient storage capacity to keep maintenance of the basin to between 3 to 4 times annually. The suggested volumetric rate of sediment inflow is 5 m^3/ha/yr.

6.4.5.1 For ease of maintenance, the depth of the sediment basin from the base of the trash rack should ideally not exceed 2.0 m.

6.4.5.2 The computation of the velocity in the sediment basin requires the computation of the water elevation in the sediment basin for the design flow Q_{GPT}. Once this is determined, the minimum depth of the sediment basin to achieve a flow velocity of 0.5 m/s can be computed, ie.

$$(WSL - INV_{basin}) \times W \geq 2Q_{GPT} \qquad - \qquad 6.7$$

where WSL is the water surface elevation in the sediment basin, INV_{basin} is the invert elevation of the sediment basin, W is the width of the sediment basin computed from Section 6.4.3.

The water surface elevation within the sediment basin (WSL) is controlled by the outflow hydraulic characteristics. Two possible flow conditions are possible in the pipe or drain downstream of the GPT depending on the slope of the conveyance system, ie. supercritical flow conditions or subcritical flow conditions.

If flow conditions within the downstream pipe or drain is super-critical, the critical depth of the conveyance system defines the water surface elevation in the sediment basin, ie,

$$WSL = INV_{out} + d_{crit} \qquad - \qquad 6.8$$

where WSL is the water surface elevation within the sediment basin, INV_{out} is the invert elevation of the outlet pipe or drain and d_{crit} is the critical depth of the outlet pipe or drain. Figure 6.20 shows a standard chart for computing critical depths for pipes and trapezoidal channels (Hemderson, 1966).

If flow conditions within the downstream pipe or drain is sub-critical, the normal depth in the conveyance system defines the water surface elevation within the sediment basin, ie.

$$WSL = INV_{out} + y \qquad - \qquad 6.9$$

where WSL is the water surface elevation within the sediment basin, INV_{out} is the invert elevation of the outlet pipe or drain and y is the normal depth of the outlet pipe or drain.

The depth of the sediment basin is expressed as follows:-

$$d_{basin} = INV_{out} - INV_{basin} \qquad - \qquad 6.10$$

where dbasin is the depth of the sediment basin, INVout is the elevation of the outlet pipe or drain and INVbasin is the elevation of the sediment basin.

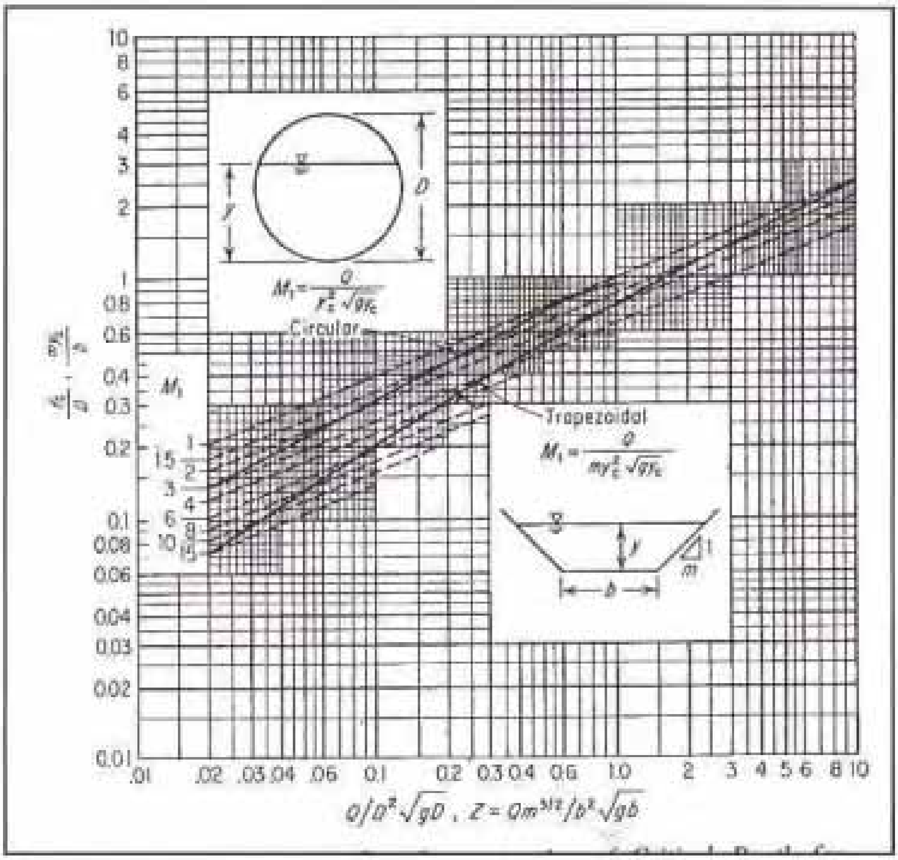


Figure 6.20 **Standard Charts for computing Critical Depths (Henderson, 1966)**

The water level in the sediment basin under design flow condiions should not be at or higher than the trash rack.

The vertical drop between the invert of the inlet pipe and the invert of the outlet pipe should therefore satisfy the following equations:-

For sub-critical flow conditions:-

$$H_{drop} > d + L_{rack}S_o \qquad - \qquad 6.11a$$

For super-critical flow conditions:-

$$H_{drop} > d_{crit} + L_{rack}S_o \qquad - \qquad 6.11b$$

where H_{drop} is the vertical distance between the inverts of the inlet and outlet pipes
d is the normal depth in the downstream pipe
d_{crit} is the critical depth in the downstream pipe
L_{rack} is the length of the trash rack
S_o is the slope of the trash rack (assumed to be the same as the slope of the inlet pipe)

6.4.6 Length of the GPT

The length of the GPT can be computed as the sum of the length of the trash rack L_{trash} (Section 6.4.4), the distance between the trash rack sill and the downstream wall L_{cr} (Section 6.4.2) and the thickness of the trash rack sill. This length must be checked against the required length to allow dissipation of the incoming energy according to standard design of drop structures charts as shown in Figure 6.19).

6.4.6.1 The finally adopted lengh of the GPT is the larger fof the two values given by equations 6.12a and 6.12b and (from Figure 6.19):-

$$L_{GPT} = L_{trash} + L_{cr} + 0.1 \qquad - \qquad 6.12a$$

or

$$\frac{L_{GPT}}{d_c} = \frac{x_a}{d_c} \text{ (from Fig. 6.19 for } \frac{y}{d_c} = \frac{d_{basin} + H_{drop}}{d_c}\text{)} \qquad - \qquad 6.12b$$

where d_{basin} is the depth of the sediment basin, H_{drop} is the vertical difference between the inverts of the inlet and outlet pipe or drain, d_c is the critical depth in the pipe or drain for the design flow (Q_{GPT}).

6.4.7 Frequency of clean-outs

The frequency of sediment clean-out each year should desirably not exceed four (4) times assuming that clean-out is required once the sediment basin is half full, ie.

for pipe system:-

$$\frac{10A_{cat}}{d_{basin} \times W \times L_{GPT}} \leq 4 \qquad - \qquad 6.13a$$

for open drain system:-

$$\frac{10A_{cat}}{d_{basin} \times W \times L_{GPT}} \leq 4 \qquad - \qquad 6.13b$$

where
A_{cat} = catchment area in ha;
d_{basin} = depth of the sediment basin (Eqn. 6.11)
W = width of the GPT (Section 6.4.3);
L_{GPT} = length of the GPT
D = diameter of the inlet pipe (Section 6.4.2); and
B = width of the inlet open drain (Section 6.4.2).

6.4.8 Structural Loading on Trash Rack

6.4.8.1 It is important that the trash rack maintains its structural integrity and will need to be checked to ensure that it can withstand impact loads from flood debris. (1989) suggest that the trash rack of minor GPTs should be designed to withstand a piece of debris weighing 250 kg and travelling at 2.0 m/s.

6.5 GPT TYPE II

6.5.1 General

The Type II trap is modelled after the minor GPTs commonly used in Canberra as discussed in Section 6.2.1 and shown in Figure 6.1. These units are generally bigger than the Type I GPT and are intended for installation at the interface of the minor stormwater drainage system and the receiving waters which is generally an open water system (ie. floodways, creeks or lakes). Suitable site conditions for Type II GPT covers a wide range of terrain and need not be confined to steep terrains as is the case for Type I GPTs.

Figure 6.21 shows a schematic diagram of the trap for an underground pipeline system discharging to receiving waters. The mechanism of gross pollutant capture is based on dissipation of inflow energy against the wall of the GPT before passing the flow through a trash rack.

The configuration of the Type II trap is typical of conventional GPT and the specifications for the size of this type of GPT are based primarily on experience gained from the design and installation of the Canberra minor GPTs.

6.5.2 Height of the Trash Rack

6.5.2.1 The height of the trash rack is required to be not lower than the water level in the sediment basin during design flow conditions. The dimension of the trash rack to satisfy this design flow operating criterion is dependent on two conditions, the tailwater level and the magnitude of the design discharge (Q_{GPT}). The depth of the water upstream of the trash rack is computed by one of the following two equations:-

$$d_{rack} = 1.22\left(\frac{Q_{GPT}}{W}\right)^{0.67} \qquad - \qquad 6.14a$$

or

$$d_{rack} = d_{TW} + \frac{0.3}{2g}\left(\frac{Q_{GPT}}{W \times d_{TW}}\right)^{2} \qquad - \qquad 6.14b$$

where d_{rack} is the depth of upstream water above the base of the trash rack, d_{TW} is the depth of the tailwater above the base of the trash rack and W is the width of the trash rack. The above two equations estimate the depth of water above the base of the trash rack under two possible flow conditions.

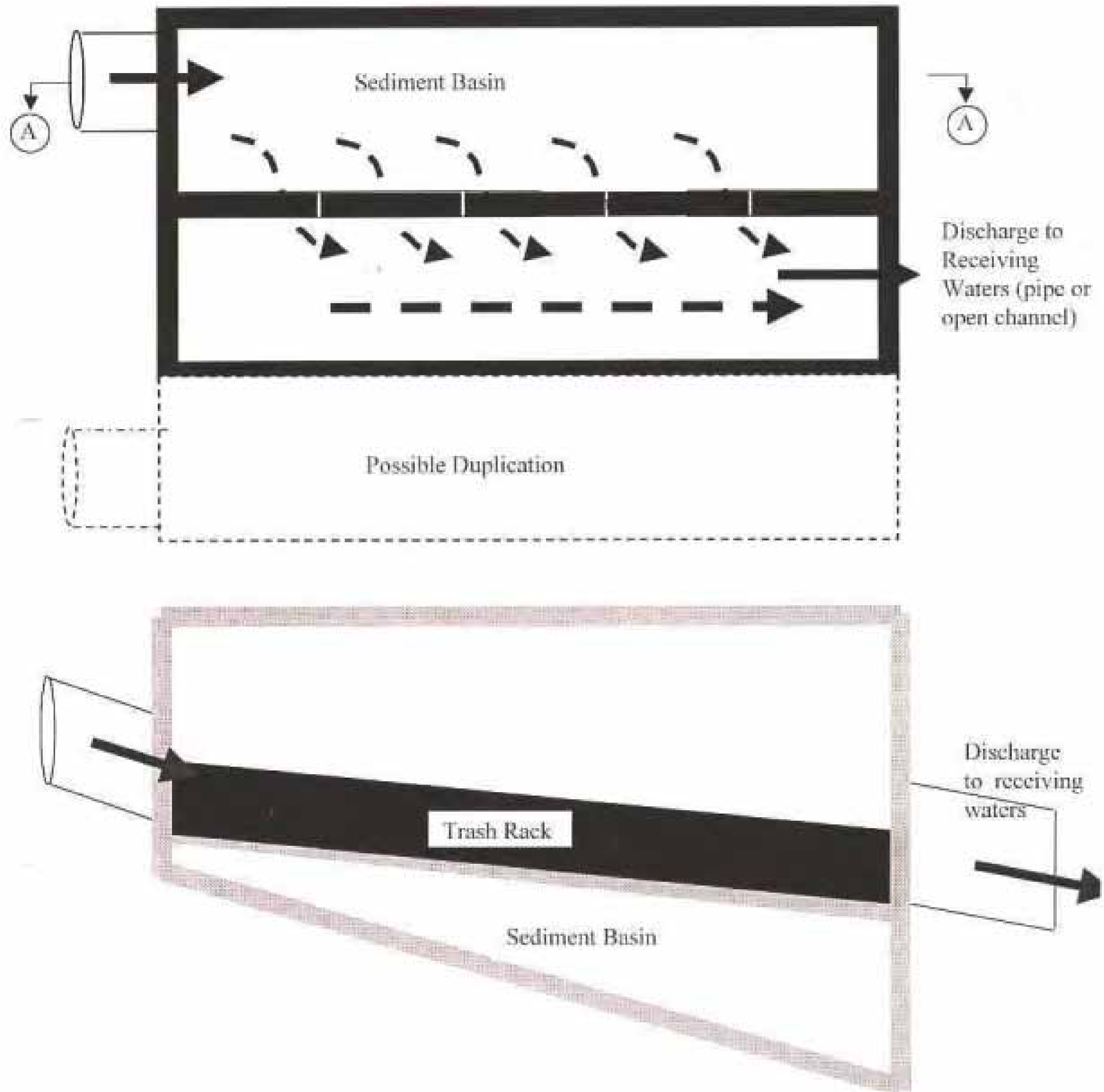


Figure 6.21 Putrajaya GPT Type II

Equation 6.14a is applicable for conditions when the downstream tailwater level is below the critical depth of the trash rack cross section. A weir equation is used to compute the water level upstream of the trash rack and a weir coefficient of 0.74 is adopted (ref. Willing and Partners, 1992).

Equation 6.14b is applicable for conditions when the downstream tailwater level is above the critical depth of the trash rack cross section. The depth of water upstream of the trash rack is computed by addition of the energy loss associated with flow through the trash rack to the depth of the tailwater above the base of the trash rack. A loss coefficient of 0.3 has been adopted from laboratory tests.

The height of the trash rack should not be less than the depth of the water (above the base of the trash rack) upstream of the trash rack, ie.

$$H_{rack} \geq d_{rack} \qquad - \qquad 6.15$$

It is immediately apparent from the above equations that the required height of the trash rack is dependent on the selected width of the trash rack. The width of the trash rack needs to satify a further design criterion, which is related to the satisfactory operation of the GPT under above-design conditions as discussed in Section 6.5.3.

6.5.2.2 *Minimum trash rack height* -Willing and Partners (1989) have suggested a number of "standard" trash rack heights for major GPTs although the basis for selection is unclear. The recommended minimum height of the trash rack is 0.5 m. Where the trash rack is easily accessible, the height of the trash rack shall be at least 1.2 m for public safety.

6.5.3 Width of the Trash Rack

6.5.3.1 The width of the trash rack is to be such that satisfactory hydraulic performance of the GPT is achieved under above-design flow conditions for discharges up to the discharge capacity of the stormwater conveyance system, ie. Q_{minor}. Checking of hydraulic performance for discharge can be carried out by two methods, ie. by ensuring that the tailwater conditions for the inlet pipe to the GPT is not higher than the obvert of the pipe or by computing the hydraulic grade line in the conveyance system upstream of the GPT assuming that the trash rack is completely blocked by gross pollutants and flood debris.

6.5.3.2 If the trash rack is completely blocked by flood debris, the trash rack operates essentially as a weir with weir coefficients of between 1.2 and 1.5. The suggested weir coefficient for the overflow weir is 1.35 and checks will need to be undertaken to ensure that submergence factors are taken into account. The depth of water over the top of the trash rack can be computed according to equation 6.16:-

$$H_{of} = 1.35\phi\left(\frac{Q_{minor}}{W}\right)^{0.67} \qquad - \qquad 6.16$$

where H_{cr} is the depth of water over the top of the trash rack, ϕ is the reduction factor for weir submergence (see Figure 6.17) and W is the width of the trash rack.

6.5.3.3 The computed water surface elevation upstream of the overflow weir under Q_{minor} should ideally not be higher than the obvert of the inlet pipe. If this was not the case, a more detailed hydraulic grade line calculation will need to be carried out.

A resulting hydraulic grade line can then be compared against the design hydraulic grade line of the system. A simplified means of checking the adequate hydraulic performance of the GPT under above design flow conditions can be carried out by ensuring that the water surface elevation at the inlet to the GPT is below the obvert of the inlet pipe.

Adjustments to the height or the width of the trash rack will be necessary when above-design operation conditions have led to a reduction in the discharge capacity of the drainage system.

6.5.4 Depth of the Sediment Basin

The depth of the sediment basin is based on achieving two conditions:-

- a maximum velocity that would not scour deposited sediment under design flow (Q_{GPT}) conditions. The suggested maximum velocity under design flow conditions and assuming that the sediment basin is half full is 0.5 m/s.
- sufficient storage capacity to keep maintenance of the basin to between 3 to 4 times annually. The suggested volumetric rate of sediment inflow is 5 m^3/ha/yr.

6.5.4.1 The flow conditions within the sediment basin of this type of GPT are not one-dimensional and thus it can be difficult to determine the representative velocity within the basin. This is due to the fact that the flow path within the sediment basin of the GPT is a right angle as shown in Figure 6.16. A conservative approach in checking that flow velocity within the sediment basin is below 0.5 m/s is to adopt a flow width (B) as that distance measured perpendicular to the longitudinal axis of the incoming pipe. This sets the minimum flow area in the sediment basin as follows:-

$$(0.5d_{basin} + H_t)B \geq 2Q_{des} \qquad - \qquad 6.17$$

where d_{basin} is the depth of the sediment basin (and sediment accumulation conditions assumed to be half full), B is the width of the sediment basin measured perpendicular to the longitudinal axis of the incoming pipe and H_t is the height of the trash rack as computed by equation 6.14. In the case of minor GPTs, the width B will be less than the width of the trash rack while for major GPTs, the width of the sediment basin is essentially the same as that of the trash rack.

6.5.4.2 Provision for maintenance vehicle access to clean out the sediment basins requires an access ramp and a minimum width of the sediment basin of 3.5 m, ie.

$$B \geq 3.5\text{m} \qquad - \qquad 6.18$$

6.5.4.3 The sediment basin needs to be of sufficient capacity to reduce the frequency of clean-out to a maximum of four times annually assuming that clean-out is required once the basin is half full. To achieve, the following equation applies:-

$$\frac{10A_{cat}}{d_{basin} \times B \times W} \leq 4 \qquad - \qquad 6.19$$

where A_{cat} = catchment area in ha;
d_{basin} = depth of the sediment basin
W = width of the GPT (Section 6.5.3);
B = width of the sediment basin

6.5.4.4 *Maximum depth of sediment basin* - The depth of the sediment basin from the base of the trash rack should ideally not exceed 2.0 m.

6.5.5 Structural Loading on Trash Rack

6.5.5.1 *Structural strength of trash rack* - It is important that the trash rack maintains its structural integrity during its operation throughout the full flow range (ie. up to the discharge capacity of the stormwater conveyance system). This will require the trash rack to withstand full water pressure in the event of it being fully blocked by flood debris. The trash rack will also need to be checked to ensure that it can withstand impact loads from flood debris. Willing and Partners (1989) suggest that the trash rack of minor GPTs should be designed to withstand a piece of debris weighing 250 kg and travelling at 2.0 m/s.

6.6 GPT Type III

6.6.1 General

The Type III trap is modelled after the major GPTs commonly used in Canberra as discussed in Section 6.2.1 and shown in Figure 6.2. These units are intended for installation on the major drainage system such as floodways or at the interface of a group of large culverts and the receiving waters. Suitable site conditions for Type III GPT covers a wide range of open watercourse terrain but will require a significant area of land to accommodate the sediment basin.

Figure 6.22 shows a schematic diagram of the trap for a grassed floodway. Unlike the Type I and II GPTs, the main mechanism promoting sedimentation in this type of gross pollutant traps is by the retardation of flow through an expanded flow section. Flood debris is captured by the use of a trash rack across the expanded section of the floodway.

The configuration of the Type III trap is typical of conventional GPT on open channel systems and the specifications for the size of this type of GPTs are based primarily on experience gained from the design and installation of the Canberra minor GPTs. The shape of the sediment basin can vary significantly depending on the site conditions as long as the designer is satisfied that the width of the basin is sufficient to promote effective sedimentation. Similarly, the trash rack need not be restricted to a single chord placed perpendicular across the sediment basin and may be aligned to increase the available overflow length as shown in Figure 6.2.

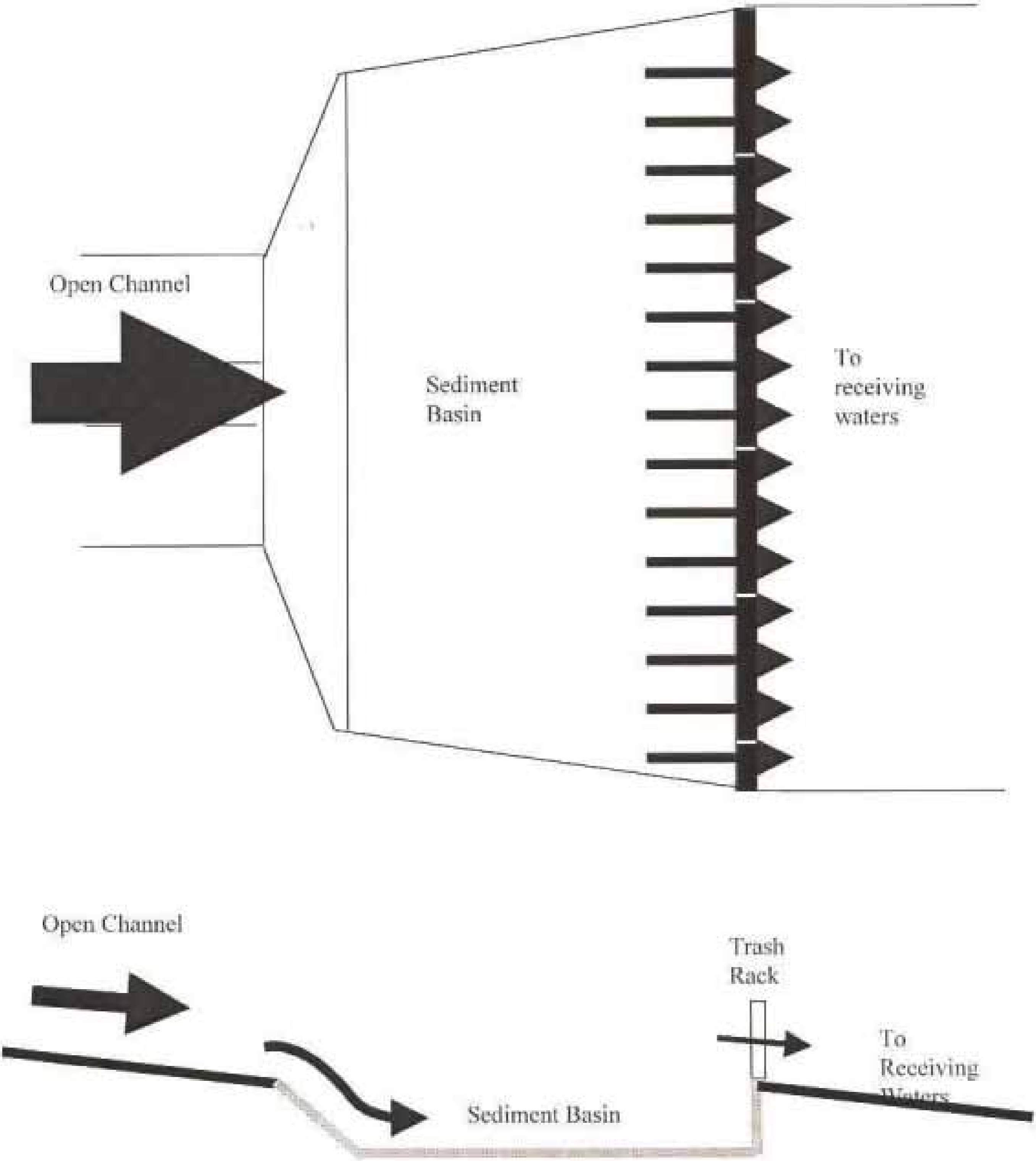


Figure 6.22 Putrajaya GPT Type III

6.6.2 Width and Length of the Sediment Basin

6.6.2.1 The area of the sedimentation basin is required to be sufficiently large to promote effective sedimentation. Standard sedimentation basin design procedures can be used to determine the required area. Willing and Partners (1989) adopted the method of Pemberton and Lara (1971) in estimating the capture efficiency of sedimentation basins according to the following equation:-

$$P = 100\left[1 - e^{\frac{-1.0548 A_{basin} U_s}{Q_{GPT}}}\right] \qquad - \qquad 6.20$$

where P is the percentage of sediment deposited, A_{basin} is the area of the sediment basin, U_s is the settling velocity (in m/s) of the minimum particle size required to be settled. This gives the expression for the required area of the sediment basin as follows:-

$$A_{basin} = \frac{2.176 Q_{GPT}}{U_s} \qquad - \qquad 6.21$$

Figure 6.23 shows the theoretical settling velocity of sediment particles.

6.6.2.2 For the Putrajaya project, it is recommended that a retention efficiency of 90% for a minimum particle size of 0.25 mm be selected and the setting velocity corresponding to this particle size is approximately 0.026 m/s.

6.6.2.3 The recommended ratio of basin width to length is between 1(w):2(L) and 1(w):3(L). However, this ratio may need to be increased to accommodate the requirements of maximum flow velocity in the basin as well as the minimum overflow length of the trash rack.

6.6.3 Height of the trash rack

6.6.3.1 The height of the trash rack is required to be not lower than the water level in the sediment basin during design flow conditions. The dimension of the trash rack to satisfy this design flow operating criterion is dependent on the tailwater levels and the magnitude of the design discharge (Q_{GPT}). The depth of water in the sediment basin is determined as follows:-

$$d_{rack} = d_{TW} + \frac{0.3}{2g}\left(\frac{Q_{GPT}}{B \times d_{TW}}\right)^2 \leq H_{rack} \qquad - \qquad 6.22$$

where H_{rack} is the height of the trash rack, d_{rack} is the depth of upstream water above the base of the trash rack, d_{TW} is the depth of the tailwater above the base of the trash rack and B is the width of the sediment basin. The depth of water upstream of the trash rack is computed by addition of the energy loss associated with flow through the trash rack to the depth of the tailwater above the base of the trash rack. A loss coefficient of 0.3 has been adopted from laboratory tests.

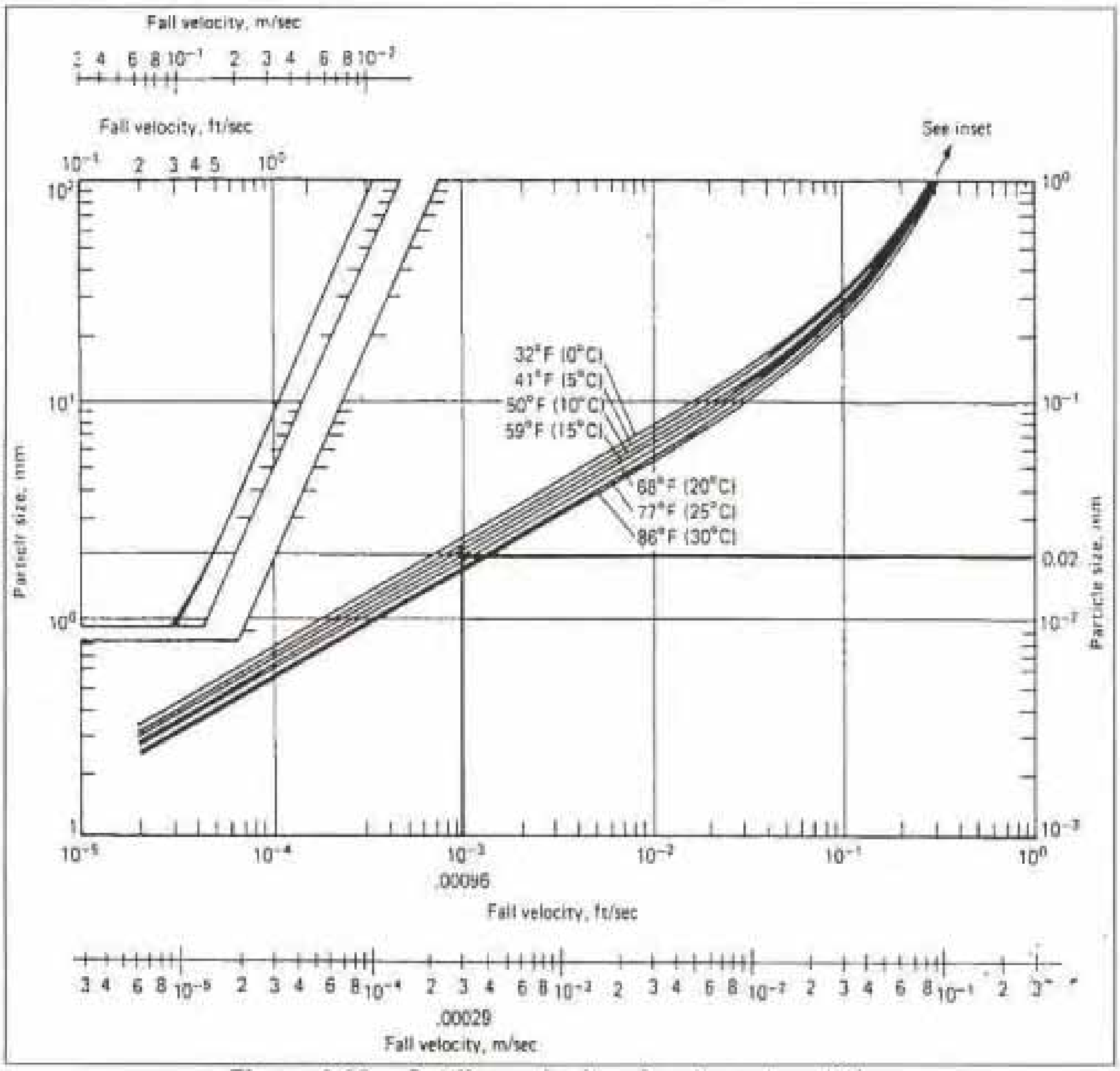


Figure 6.23 Settling velocity of sediment particles

The required height of the trash rack is dependent on the selected width of the trash rack. The length of the trash rack will need to satisfy a further design criterion, which is related to the satisfactory operation of the GPT under above-design conditions as discussed in Section 6.6.5.

6.6.3.2 *Minimum trash rack height* -Willing and Partners (1989) have suggested a number of "standard" trash rack heights for minor GPTs although the basis for selection is unclear. The recommended minimum height of the trash rack is 0.7 m. Where the trash rack is easily accessible, the height of the trash rack shall be at least 1.2 m for public safety.

6.6.4 Depth of Sediment Basin

Conditions requiring considerations in determining of the suitable depth of the basin is the same as those for the Type II GPT. The depth of the sediment basin is based on achieving two conditions:-

- a maximum velocity that will not scour deposited sediment under design flow (Q_{GPT}) conditions. The suggested maximum velocity under design flow conditions and assuming that the sediment basin is half full is 0.5 m/s.
- sufficient storage capacity to keep maintenance of the basin to between 3 to 4 times annually. The suggested volumetric rate of sediment inflow is 5 m^3/ha/yr.

6.6.4.1 The flow velocity is required to be below 0.5 m/s within the sediment basin to prevent excessive resuspension of deposited sediment, ie.

$$(0.5d_{sed} + H_t)B \geq 2Q_{des} \qquad - \qquad 6.23$$

where d_{basin} is the depth of the sediment basin (and sediment accumulation conditions assumed to be half full) and B is the width of the sediment basin.

6.6.4.2 It is desirable that the sediment basin have sufficient capacity to enable the frequency of clean-out to be three times yearly, assuming that clean-out is required once the sediment basin is half full. To achieve, the following equation applies:-

$$\frac{10A_{cat}}{d_{basin} \times A_{basin}} \leq 3 \qquad - \qquad 6.24$$

where A_{cat} = catchment area in ha;
d_{basin} = depth of the sediment basin
A_{basin} = the area of the sediment basin (Section 6.6.2);

6.6.4.3 *Maximum depth of sediment basin* - The depth of the sediment basin from the base of the trash rack should ideally not exceed 2.0 m.

6.6.5 Above Design Flow Conditions

6.6.5.1 Checking the satisfactory performance of the GPT under above-design flow conditions can be carried out by computing the water surface profile along the waterway for the pre- and post-GPT scenarios. The trash rack is assumed to be completely blocked by gross pollutants and flood debris. Under such circumstances, the trash rack operates essentially as a weir with typical weir coefficients of between 1.2 and 1.5. A weir coefficient of 1.5 is suggested for trash racks that are aligned perpendicular to the direction of flow.

6.6.5.2 If the downstream water level during above-design flow condition is such that the trash rack will be submerged, adjustments will need to be made to the weir coefficient according to the factor given in Figure 6.17.

6.6.6 Structural Loading on Trash Rack

6.6.6.1 It is important that the trash rack maintains its structural integrity during its operation throughout the full flow range (ie. up to the discharge capacity of the stormwater conveyance system). This will require the trash rack to withstand the water pressure in the event of it being full blocked by flood debris. The trash rack will also need to be checked to ensure that it can withstand impact loads from flood debris. Willing and Partners (1989) suggest that the trash rack of minor GPTs should be designed to withstand a piece of debris weighing 500 kg and travelling at 3.0 m/s.

6.6.7 Other Design Considerations

6.6.7.1 *Vehicle Access* – Provision for maintenance vehicle access to clean out sediment basin requires an access ramp with a minimum width of 3.5 m.

6.6.7.2 A low flow channel to by-pass the sediment basin is necessary to allow the sediment basin to be dried out under dry weather conditions.

6.6.7.3 A transition channel is required downstream of the trash rack to converge flows back to the waterway if the GPT is located as an in-line trap on an open waterway.

6.7 WORKED EXAMPLES

6.7.1 Introduction

According to the Drainage Masterplan for Putrajaya, the discharge standard for the minor drainage system for commercial precincts is to be the 100 year ARI event while the minor stormwater system for residential precincts is to be designed for the 5 year ARI event. Three worked examples are presented in this section to demonstrate the typical procedure to be followed in designing the three types of Gross Pollutant Traps (GPTs) for the area.

6.7.2 Worked Example I – GPT Type I

This worked example involves the design of a GPT Type I to cater for a 750 diameter stormwater pipe draining a catchment area of 8 ha. The landuse is residential and the design standard for the minor drainage system is the 5 year ARI event. The GPT is to be installed in-line and the pipeline is to be laid at a slope of 3%. There is sufficient slope in the terrain at the proposed location of the GPT to accommodate a drop structure of up to 3 m.

The design discharges for the 1 year and 5 year ARI events are as follows:-

$$Q_1 = Q_{GPT} \quad = \quad 1\ m^3/s$$
$$Q_5 = Q_{minor} \quad = \quad 1.7\ m^3/s$$

6.7.2.1 Trash Rack Design

6.7.2.1.1 The sill height of the trash rack is to be the higher of half the inlet pipe diameter or the depth of water in the inflow pipe, d_{inflow}.

The depth of water in the inflow pipe can be determined using standard pipe flow charts such as that shown in the figure below. To use the chart, the ratio of the design flow to the pipe full discharge needs to be determined.

The pipe full discharge can be computed using Manning's equation with a Manning's roughness coefficient of 0.014 and a pipe slope of 3%, ie.

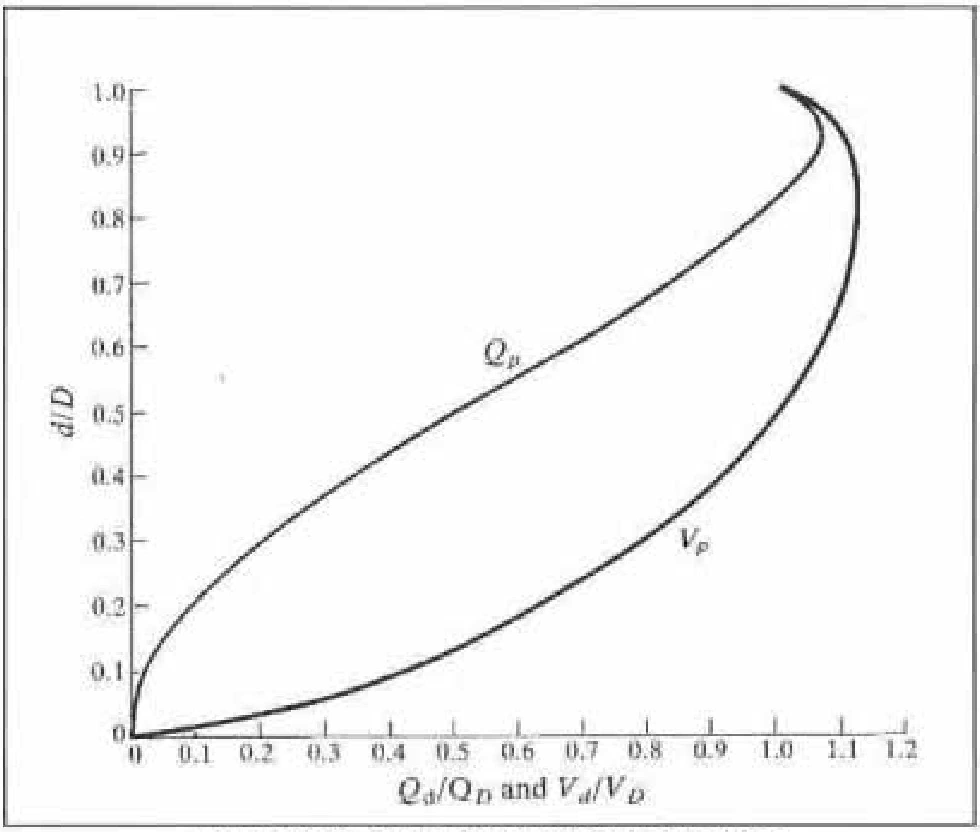


Figure 6.24 Pipe Discharge Relationships

$$Q_{full} = \frac{A_{pipe}^{1.667} S_o^{0.5}}{P^{0.667} n} = \frac{(0.442)^{1.667}(0.03)^{0.5}}{(2.356)^{0.667} \cdot 0.014} = 1.79 \text{ m}^3/\text{s}$$

$$\frac{Q_{GPT}}{Q_{full}} = \frac{1.0}{1.79} = 0.56\text{; refering to Figure 6.24 gives } \frac{d}{D} = 0.54\text{; } \frac{v}{V} = 1.02$$

$$v_{inflow} = 1.02\left(\frac{Q_{full}}{A_{pipe}}\right) = 4.05 \text{ m/s}$$

$d_{inflow} = 0.54D = 0.41$ m > 0.5D; therefore adopt $H_{sill} = 0.4$ m

From Figure 6.20; the critical depth of the pipe is 0.6 m. Flow conditons in the pipe is thus super-critical.

The height of trash rack sill (H_{sill}) is 0.4 m
The distance from the overflow weir to the downstream wall (L_{od}) is 0.75 m

6.7.2.1.2 The width of the trash rack is defined by the required width of the overflow weir such that the water level upstream of the weir for Q_{minor} is at or below the obvert of the inlet pipe.

Assuming the length of the GPT to be 2.5 m and an incline of the trash rack of 3%, the maximum water level over the trash rack sill for Q_{minor} is given as follows:-

$D-H_{sill}+L_{GPT}*0.03 = 0.75 - 0.4 + 2.5(0.03) = 0.425$ m.

The required width of the overflow weir can be computed as follows:-

$$W = \frac{Q_{minor}}{1.5H_{sill}} = \frac{1.7}{1.5 \cdot 0.425} = 2.7\text{m}$$

The width of the trash rack is 2.7 m.

6.7.2.1.3 The length of the trash rack needs to satisfy three conditions, ie.

Condition 1: $L_{rack} \geq 0.75$ m

Condition 2: The available storage volume for litter should not be less than the expected litter load over a six month period, ie. 0.15 m^3/ha.

$$L_{rack} \geq \frac{0.15 \cdot A_{catchment}}{W \cdot H_{sill}} = 1.15\text{m}$$

Condition 3: The horizontal trajectory of the water under design flow conditions should be less than the length of the trash rack. Assuming that the trash rack will be placed at the invert of the pipe outlet (ie. y = 0), the trajectory distance can be computed by reading the value of (x_s/d_c) for an assumed tailwater condition of $(y_t/d_c = 0.2)$ from Figure 6.19 as follows:-

From Figure 6.19 for $y_t/d_c = 0.2$; $y/d_c = 0 \Rightarrow x_s/d_c = 1.5$

Critical depth (6.7.2.1.1) is 0.6 m

The minimum length of the trash rack = 1.5 x 0.6 = 0.9 m

Therefore litter storage criterion (ie. Condition 2) is critical, $L_{rack} > 1.15$ m

The length of the trash rack is 1.15 m

6.7.2.1.4 The length of the GPT is the greater of two criterion, ie. (i) the sum of the length of the trash rack (6.7.2.1.3), the width of the overflow sill (assumed to be 100 mm) and the opening from the sill to the downstream wall (6.7.2.1.1), or (ii) the required length for a standard drop structure (from Figure 6.19). The notional value of L_{GPT} is first calculated:-

$L_{GPT} = L_{rack} + 0.1 + L_{st} = 1.15 + 0.1 + 0.75 = 2.0$ m

The notional length of the GPT, L_{GPT} is 2.0 m

6.7.2.2 Sediment Basin Design

6.7.2.2.1 The velocity in the sediment basin is to be limited to 0.5 m/s for Q_{GPT}. The water surface elevation in the sediment basin is defined bythe flow conditions in the downstream pipe. In this case, the flow conditions in the 750 mm diameter pipe for the design flow is super-critical with a critical depth of 0.6 m (6.7.2.1.1).

The depth of the sediment basin necessary to ensure that the flow velocity is kept at or below 0.5 m/s is computed as follows:-

$$(d_{basin} + d_{crit}) \cdot W \geq 2Q_{GPT}$$

therefore

$$d_{basin} \geq \frac{2Q_{GPT}}{W} + d_{crit} = 1.34 \text{ m; say } 1.5$$

The minimum depth of the sediment basin is 1.5 m

6.7.2.2.2 The veritcal drop between the invert of the inlet pipe and the invert of the outlet pipe is to be such that the water level in the sediment basin is below the trash rack. As flow conditions in the pipe is super-critical, the following equation applies

$H_{drop} > d_{crit} + L_{rack}S_o = 0.6 + 0.06 = 0.66$; adopt 0.75 m allowing a freeboard of approximately 0.1 m

The difference in elevation between the invert of the inlet pipe and the invert of the outlet pipe H_{drop} is 0.75 m.

6.7.2.2.3 The required volume of the sediment basin is such that the clean-out frequency does not exceed 4 times a year (assuming that clean-out is carried out once the basin is half full), ie

$$\frac{10A_{cat}}{d_{basin} \times W \times L_{GPT}} \approx 10 \text{ which is greater than the desired clean-out frequency.}$$

Adjust the depth of the basin to the maximum (Section 6.4.5.1) of 2.0 m, the width of the GPT to 3.0 m and the length of the GPT to 3.5 m satisfy this criterion , ie.

$$\frac{10A_{cat}}{d_{basin} \times W \times L_{GPT}} = \frac{10 \cdot 8}{2.0 \cdot 3.0 \cdot 3.5} = 3.8 < 4 \text{ .....OK}$$

6.7.2.2.4 The L_{GPT} will now need to be checked for compliance with the required length for a standard drop structure by refering to Figure 6.19.

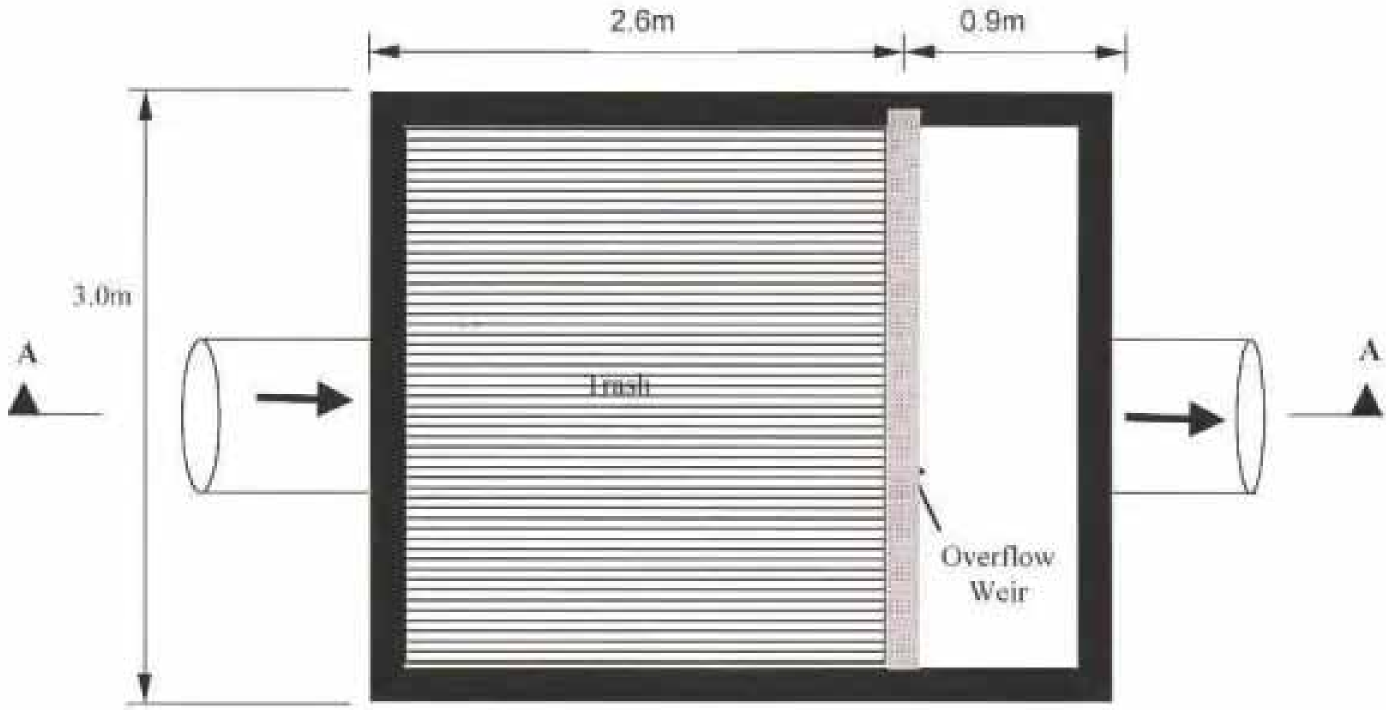
$$\frac{y}{d_c} = \frac{d_{basin} + H_{drop}}{d_c} = \frac{2.0 + 0.75}{0.6} = 4.6$$

$$\frac{y_1}{d_c} = \frac{H_{drop} - d_c}{d_c} = \frac{0.75 - 0.6}{0.6} = 0.25$$

From Figure 6.19; $x_a/d_c = 6$, giving $x_a = 3.6$ m > 3.5 m from 6.7.2.2.3.

Adopt $L_{GPT} = 3.6$ m; adjust L_{rack} and L_{af} to suit.

Final dimensions are as follows:- $\boldsymbol{L_{GPT} = 3.6\ m;\ W_{GPT} = 3.0\ m}$
$\boldsymbol{L_{rack} = 2.6\ m;\ L_{af} = 0.9\ m}$
$\boldsymbol{H_{sill} = 0.4\ m;\ H_{drop} = 0.75\ m}$



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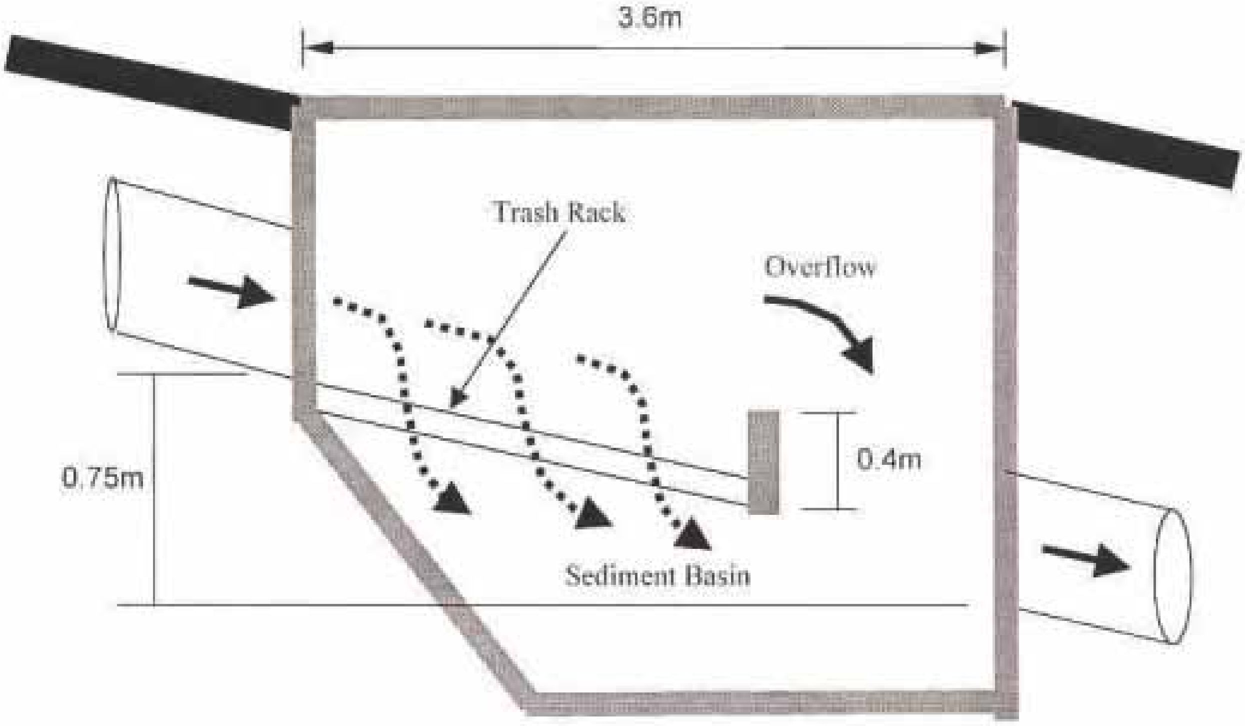


Section A-A

6.7.3 Worked Example 2 – GPT Type II

This worked example involves the design of a GPT Type II to cater for two 1.5 m diameter stormwater pipes draining a catchment area of 28 ha. The two pipes are located on either side of the boulevard.

The two pipes will be connected to a single GPT unit, which consists of two sediment basins and a common outlet chamber. Stormwater discharge from these two pipes will flow into their respective sediment basins and through their respective trash racks. A common outlet chamber for the two pipes will enable a single outlet point into the lake. The invert level of the inlet pipes is RL 21.5m. The lake level corresponding to the 100 year ARI event is assumed to be RL 21.5m. Normal lake level is assumed to be RL 21.0 m.

The design discharges for the 1 year and 100 year ARI events are as follows:-

$$Q_1 = Q_{GPT} \quad = \quad 6\ m^3/s \text{ or } 3\ m^3/s/pipe$$
$$Q_{100} = Q_{minor} \quad = \quad 16\ m^3/s \text{ or } 8\ m^3/s/pipe$$

6.7.3.1 Trash Rack Design

6.7.3.1.1 Trash racks should ideally not be submerged under dry weather/normal lake level conditions to enable ease of access for maintenance.

$\therefore$ base of trash rack > RL 21.0 m

The trash rack is not to be submerged under design conditions (ie. for the 1 year ARI event). However, owing to the significantly different catchment area of the GPT and the lake, it is possible for the 100 year ARI lake level to occur simultaneously with the I year ARI storm event in the 28 ha catchment. It is therefore necessary to ensure that the top of the trash rack is higher than the 100 year ARI lake level of RL 21.5m. This would avoid any likelihood of trapped litter being washed out into the lake when catchment flow is low but the lake level is high.

Select floor level of outlet chamber to range from RL 21.5 m (upstream) to RL 21.3 m (downstream)

6.7.3.1.2 The height of the trash rack should be such that if completely blocked, weir flow conditions over the rack should not result in higher tailwater conditions to the drainage system, ie. no reduction in the discharge capacity of the pipe drainage system. The design discharge selected to set the height of the trash rack should therefore correspond to the design standard of the minor drainage system, ie. Q_{minor}. In the case of the Boulevard Area of the Putrajaya Project, this design standard corresponds to the 100 year ARI event. The tailwater level at the pipe outfall to the GPT is not to exceed the obvert level of the pipe, ie. at RL 23.0 m.

Maximum water level in GPT for Q_{100} under conditions of full blockage of the trash rack is RL23.0 m.

6.7.3.1.3 Owing to the low tailwater level (ie. between 0m and 0.2m), the appropriate equation for computation of the depth of water upstream of the trash rack is Equation 6.13a, ie.

$$d_{rack} = 1.22\left(\frac{Q_{GPT}}{W}\right)^{0.67}$$

Assume a trash rack width (W) of 8 m give d_{rack} = 0.63. The elevation of the top of the trash rack is thus RL 22.13.

∴ select top of trash rack at RL 22.2 m.

Top of trash rack = RL 22.2 m
Height of rack = 0.7 m – 0.9 m.

6.7.3.1.4 The width of the trash rack is determined to satisfy design condition B.2.

Available head over top of trash rack ΔH = RL 23.0 – RL 22.2 = 0.8 m

Q_{p100} (per pipe) = 8 m^3/s

The width (W) of the trash rack is computed using the weir formula, ie.

$$W = \frac{Q}{C_w \cdot (\Delta H)^{1.5}}$$

where C_w is the weir coefficient. No weir submergence adjustment is required as the water level downstream is not expected to be higher than the top of the trash rack (check for this is contained in 6.7.4.3.1)

Adopting a weir coefficient of 1.35 gives the trash rack width as follows:-

$$W = \frac{8.0}{1.35 \cdot (0.8)^{1.5}} = 8.3\text{m}$$

Adopt width of trash rack of 9.0 m for each pipe

6.7.3.2 Outlet Pipes or Culverts

6.7.3.2.1 The outfall pipes or culverts are to be submerged of all times with a maximum lake level of RL 21.5 m and a minimum lake level of RL 20.0 m.

The maximum upstream or headwater level should not exceed the prescribed normal operating tailwater level of RL 23.0 m to satisfy design condition B.2. The critical lake level for this computation is RL 21.5 m.

Try 3 Nos. 1.5 m diameter pipes; L ~ 80 m to outfall into lake

Select Manning's n = 0.014

$$\Delta H = S_f \cdot L + 1.5\frac{v^2}{2g}$$

$$= \left(\frac{QnP^{0.67}}{A^{1.67}}\right)^2 L + 1.5\frac{(Q/A)^2}{2g}$$

$$= 0.53 + 0.70$$

$= 1.23$ $\therefore$ HW level = 21.5 + 1.23 = RL 22.7 m OK

It is desirable for the upstream or headwater level to be less than the top of the trash rack if at all possible. This would enable all stormwater to flow through the trash rack under "ideal" trash rack operating conditions for all events up to the discharge capacity of the pipe network (ie. 100 year ARI discharge in this case). This can often be a difficult design objective to satisfy but a check should be carried out to compare against the above outlet size.

Try 3 Nos. 2.0m (w) x 1.5m (h) box culverts; L ~ 80 m to outfall into lake

Select Manning's n = 0.014

$$\Delta H = S_f \cdot L + 1.5\frac{v^2}{2g}$$

$$= \left(\frac{QnP^{0.67}}{A^{1.67}}\right)^2 L + 1.5\frac{(Q/A)^2}{2g}$$

$$= 0.15 + 0.24$$

$= 0.39$ $\therefore$ HW level = 21.5 + 0.39 = RL 21.9 m OK

Adopt 3 Nos 2.0 m (W) x 1.5 m (H) box culverts if possible

An alternative should the above be too expensive is to adopt 3 Nos. 1.5 m diameter pipes

6.7.3.2.2 Invert Elevations

inlet invert level = RL 21.3m.
outlet invert level = RL 18.5m.

6.7.3.2.3 Outlet Works

Maximum velocities at outlet

For 3 Nos 2.0 m (W) x 1.5 m (H) box culverts (preferred)

100 year ARI discharge = $\frac{Q}{A} = \frac{16.0}{9.0} = 1.8\text{m/s}$

1 year ARI discharge = $\frac{Q}{A} = \frac{6.0}{9.0} = 0.67 m/s$

For 3 Nos 1.5 m diameter pipes

100 year ARI discharge = $\frac{Q}{A} = \frac{16.0}{5.3} = 3.0 m/s$

1 year ARI discharge = $\frac{Q}{A} = \frac{6.0}{5.3} = 1.13 m/s$

No special energy dissipation structure required at outfall to the lake Provide rip-rap on foreshore extending to 4 m downstream of outfall

6.7.3.3 Sediment Basin Design

6.7.3.3.1 Width of basin to be 3.5 m or greater to allow access by maintenance vehicle.

Select Minimum Sediment Basin width = 3.5 m

6.7.3.3.2 Velocity in sediment basin is to be limited to 0.5 m/s for Q_1. The critical downstream condition is when the lake level is low and normal depth occurs in the outfall culverts.

Q_1 for each pipe = 3 m^3/s.

Minimum depth of outfall culverts (3 Nos 2 m x 1.5 m) occurs under minimum lake level conditions. Flow conditions at inlet is weir flow conditions.

Flow depth (y) in outlet chamber is computed by applying the weir flow formula, ie

$$y = \left(\frac{Q}{L \cdot C_w}\right)^{0.67}$$

adopting Q = 6 m^3/s; L = 6 m; C_w = 1.4 $m^{0.5}/s$; gives y = 0.80 m.

Water level in the outlet chamber during the 1 year ARI event is

$$WS_1 = 21.3 + 0.8$$
$$= RL\ 22.1\ m \quad < \quad \text{top of trash rack of RL 22.2 m}$$

Water level in Outlet Chamber for the 1 year ARI event is below the top of trash rack (Design Condition B.3.)

Velocity in the sediment basin is computed as follows:-

$$V = \frac{0.5Q}{w \cdot (0.5d + y)} \leq 0.5$$

where w is the width of the sediment basin and d is the depth of the basin.

According to design condition 6.7.3.3.1, the minimum sediment basin width (w) is 3.5 m

For y = 0.8 m, d = 1.83

It is possible to reduce the depth of the sediment basin by increasing the width of the basin to satisfy the maximum velocity criterion. The mean width of the basin can be increased to 7.0 m in this case as the width of the road is approximately 40 m and the distance between the two 1.5 m diameter pipe would be typically that as well. In so doing, the depth of the sediment basin can be reduced to 0.9 m.

Allow the width of the sediment basin to taper from a width of 3.5 m to a width of 10.5 m such that the mean width is 7.0 m.

Adopt the depth of the sediment basin to be 0.9 m

6.7.3.3.3 Provision for maintenance access to the sediment basins is an important consideration in the design of these basins. For the present case, access ramps are to be provided at 1 in 4 slope at the downstream end of the sediment basin.

Provide access ramps at 1(v) to 4(h) slope at the downstream end of the sediment basin.

6.7.3.3.4 Check for storage volume in sediment basin

The sediment basin may be divided into two sections, ie. the 6 m long section with a depth of 0.9 m and a mean width of 5.8 m, and the 3 m long section with a sloping bed of mean depth of 0.45 m and a mean width of 9.3 m. The storage volume is computed as follows:-

$$S = 2 \text{ of } \{(6 \times 5.8 \times 0.9) + (3 \times 9.3 \times 0.45)\} = 87.75 \text{ m}^3 \qquad \text{Say } 88 \text{ m}^3$$

Expected sediment inflow 5 m^3/ha x 28ha = 140 m^3

Clean-out frequency assuming clean-out is required once the basin is half full is approximately 3 times/year

Clean-out frequency of sediment basin has been sized to a maximum of 3 times per year based on an annual sediment volume load of 5 m^3/ha

6.7.3.4 Operating Conditions

6.7.3.4.1 The water level in the sediment basin during the 100 year ARI event can be calculated by computing the head loss through the trash rack. Head loss through the trash rack can calculated using a head loss coefficient of 0.3, ie.

From 6.7.3.2.1, the water level in the outlet chamber for the 100 year ARI event is RL 21.9 m. The depth of water in the outlet chamber ranges from 0.4 m to 0.6 m. The open area of the trash rack is 83% of the trash rack area.

Mean velocity through the trash rack = $\dfrac{8}{0.83\times 9\times 0.5}=2.14\,\mathrm{m/s}$

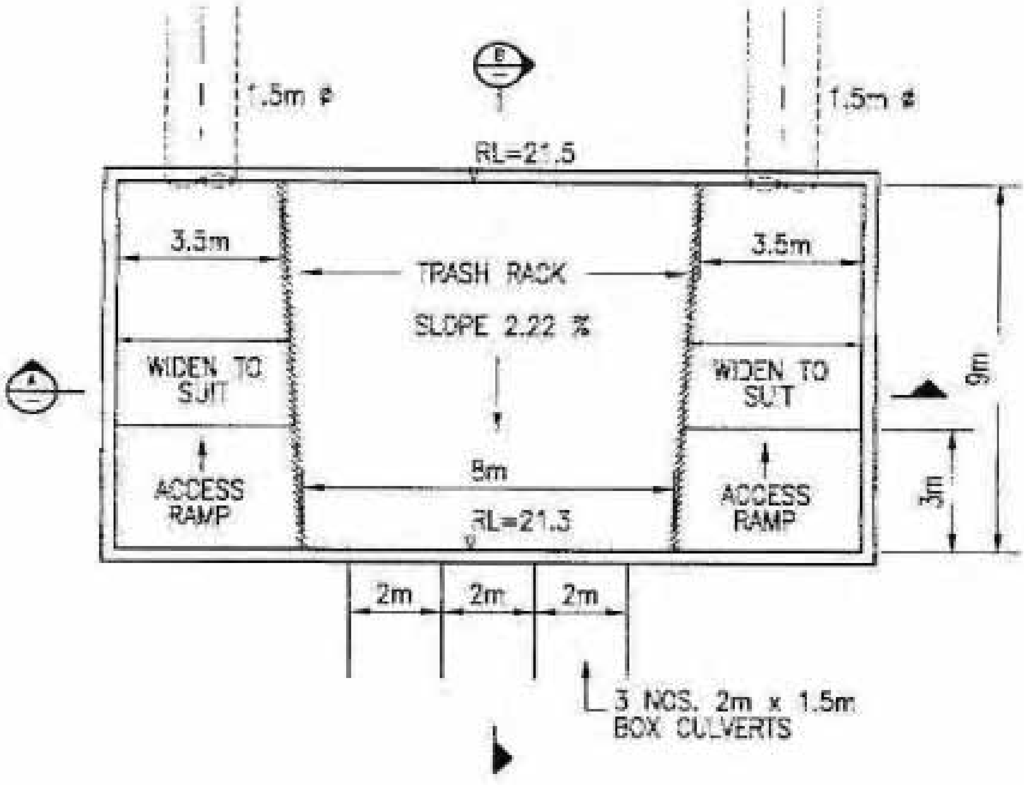
Head loss computed as $0.3\dfrac{(2.14)^2}{2g}$ gives a head loss of 0.07m.

For a head loss of 0.3 m, the velocity through the trash rack will have to be 4.43 m/s. This corresponds to a trash rack blockage of approximately 50%.

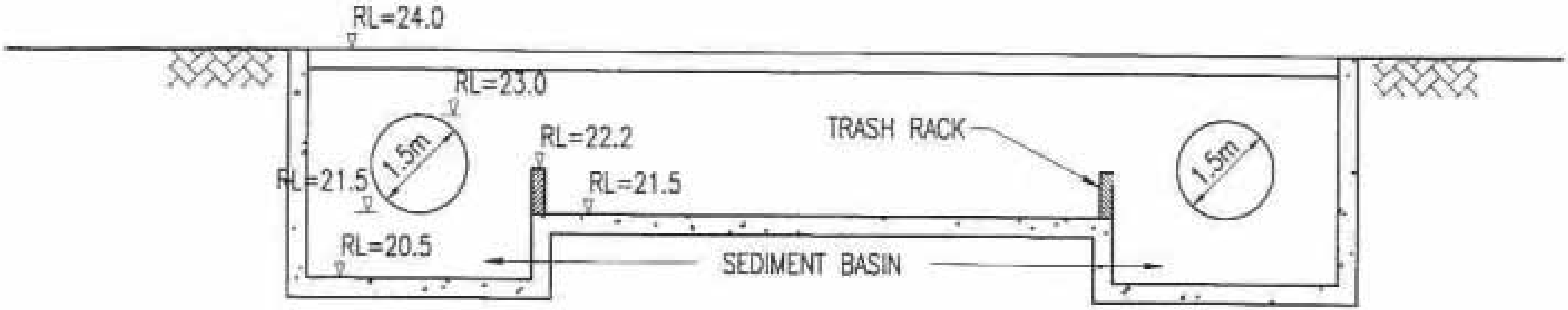
GPT has been designed to operate under the 100 year ARI condition without overtopping of the trash rack for blockage of up to 50%

6.7.3.4.2 The width of the GPT to allow both pipes to share a common outlet chamber is thus approximately 40 m. Under such circumstances, it is advisable to taper the trash rack towards from the edge of the GPT towards the centre of the GPT as shown in the diagram. This is expected to facilitate the movement of debris towards the downstream end of the sediment basin.

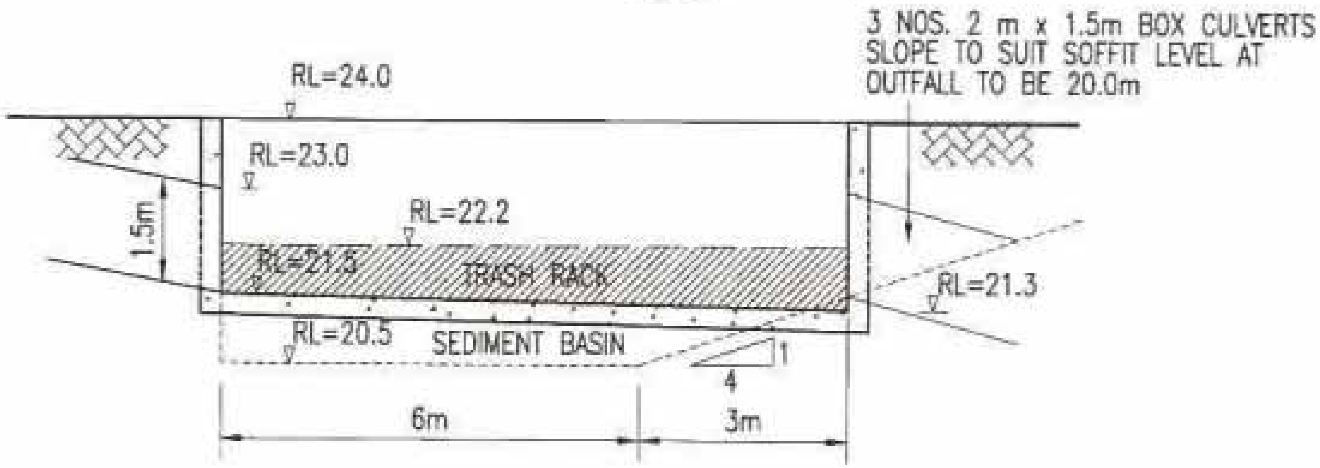
Taper trash rack in plan towards the centre of the GPT to allow movement of debris trapped towards the downstream end of the GPT



PLAN



SECTION A
SCALE A : 1 : 100



SECTION B
SCALE A : 1 : 100

6.7.4 Worked Example 3 – GPT Type III

This worked example involves the design of a GPT Type III built on a grassed trapezoidal floodway draining a catchment area of 100 ha. The landuse is predominantly residential with small areas of commercial and the design flood standard of the trapezoidal floodway is the 100 year ARI flood with a peak discharge of 50 m^3/s. The corresponding 1 year ARI peak discharge is 15 m^3/s. The trapezoidal floodway has a base width of 15 m, a side slope of 1(h) in 4(w) and a bed slope of 0.002.

Stormwater discharge from the waterway into a tributary of the river which flows into the Putrajaya Lake. The GPT is thus not subjected to lake tailwater conditions. The design tailwater level is taken as the normal depth of the fllodway.

During the 100 year ARI peak discharge, the estimated flow depth in the floodway is 1.5 m and the width of the water surface is approximately 27 m.

6.7.4.1 Tailwater Levels

6.7.4.1.1 The tailwater levels downstream of the GPT may be computed using the Manning's Equation with Manning's n value of 0.035. The following tailwater levels have been computed:-

1 year ARI flow depth	=	0.82 m
100 year ARI flow depth	=	1.6 m

6.7.4.2 Sediment Basin Dimensions

6.7.4.2.1 In determining the required area of the sediment basin, it is first necessary to select the target sediment size. According to Section 6.6.2.2, the recommended particle size for 90% retention efficiency for the Putrajaya Project is 0.25 mm. The corresponding settling velocity is approximately 0.026 m/s.

From equation (6.21), the required area for the sediment basin is computed as follows:-

$$A_{basin} = \frac{2.176 Q_{GPT}}{U_s} = \frac{2.176 \times 15.0}{0.026} = 1255 \text{ m}^2$$

The width of the floodway during the 100 year ARI peak discharge is 27.0 m. The width of the sediment basin should therefore not be less than 27.0 m.

The minimum area of the Sediment Basin is 1255 m².
The width of the Sediment Basin should not be less than 27.0 m.

6.7.4.2.2 A general recommended basin width to length ratio is between 1(w):2(L) and 1(w):3(L) as discussed in Section 6.6.2.3. The criteria of maximum design flow velocity of 0.5 m/s (6.6.4.1) and the maximum depth of 2.0 m (6.6.4.3) influence the width of the basin. Based on these two criteria, the minimum width of the basin may be computed as follows:-

$$B \text{ (minimum)} \sim \frac{Q_{des}}{V_{max} \cdot (d_{basin} + d_{TW})} = \frac{15.0}{0.5 \cdot (2.0 + 0.82)} = 10.6\text{m}$$

The minimum width specified in 6.7.4.2.1 of 27 m is the dominant criterion. Adopt basin width of 30 m. The required basin length is 42 m.

Sediment Basin dimension is 30 m (width) x 42 m (length)

6.7.4.3 Height of the trash rack

6.7.4.3.1 The height of the trash rack should be higher than the depth of water immediately upstream of the trash rack (d_{rack}) as computed using Equation 6.22, ie.

$$d_{rack} = d_{TW} + \frac{0.3}{2g}\left(\frac{Q_{GPT}}{B \times d_{TW}}\right)^2 = 0.82 + \frac{0.3}{2g}\left(\frac{15.0}{27.0 \times 0.82}\right)^2 = 0.83\text{m}$$

Notionally select height of trash rack H_{rack} of 1.0 m

6.7.4.3.2 Check the adequacy of the selected width of the trash rack under above-design flow conditions, ie. 100 year ARI discharge of 50 m^3/s.

Flow condition is that of a submerged weir. The downstream water level above the top of the trash rack (H_{ds}) is $d_{TW} - H_{rack}$ = 1.6 – 1.0 = 0.6 m.

Adopting a maximum afflux of 0.3 m gives the upstream water level above the top of the trash rack (H_{us}) of 0.9 m. The percent submergence (H_{ds}/H_{us}) is 67% and from Figure 6.17, the discharge reduction factor is close to 1.0. Therefore, no adjustment for weir submergence is necessary. The discharge over the trash rack under these conditions is computed using a weir coefficient of 1.5 is computed as follows:-

$Q(H_{us}=0.9m) = C_w \, B \, H_{us}^{1.5} = 1.5 \times 27 \times 0.9^{1.5} = 34.6$ m^3/s < 50 m^3/s.

There is thus insufficient discharge capacity to pass the 100 year ARI event with a maximum afflux of 0.3 m. Reduce H_{rack} to 0.9 m and lengthen the overflow section to 42.3 m by placing the trash rack at 45° angle; H_{us} = 1.0 m; C_w reduced to 1.35 for angled overflow section.

$Q(H_{us}=1.0m) = 1.35 \times 42.3 \times 1.0^{1.5} = 57.1$ m^3/s > 50 m^3/s.....OK

Align Trash Rack at 45°
Height of Trash Rack to be 0.9 m

6.7.4.4 Depth of Sediment Basin

6.7.4.4.1 The depth of the basin needs to be such that the design velocity is less than 0.5 m/s as outlined in 6.6.4.1. The minimum depth to satisfy this criterion may be computed from equation 6.2.3 as follows:-

$$d_{ba\sin} = \frac{1}{0.5}\left(\frac{2Q_{de}}{B} - H_{rack}\right) = \frac{1}{0.5}\left(\frac{2\times 15}{30.0} - 0.9\right) = 0.2\text{m}$$

6.7.4.4.2 The depth of the sediment basin needs to satisfy the criterion of adequate storage of deposited sediment to reduce the frequency of clean-outs to three times yearly. The minimum depth required to satisfy this criterion may be computed using equation 6.24.

$$d_{ba\sin} \rangle \frac{10\times A_{cat}}{3\times A_{ba\sin}} = \frac{10\times 100}{3\times 1255} = 0.27\text{m}$$

Adopt a sediment basin depth of 0.3 m

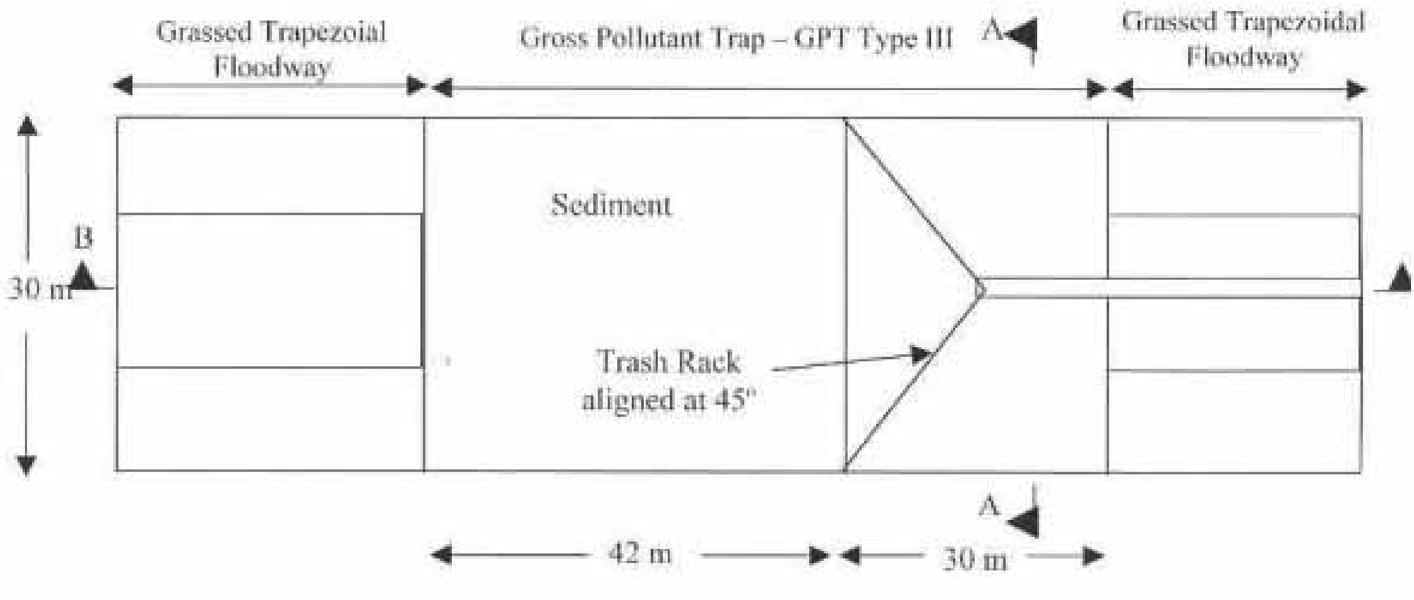
6.7.4.5 Downstream Transition

6.7.4.5.1 A concrete-lined transition channel is required downstream of the trash rack to converge flows back to the grassed trapezoidal floodway. The length of this transition is to be 15 m from the nearest point of the trash rack.

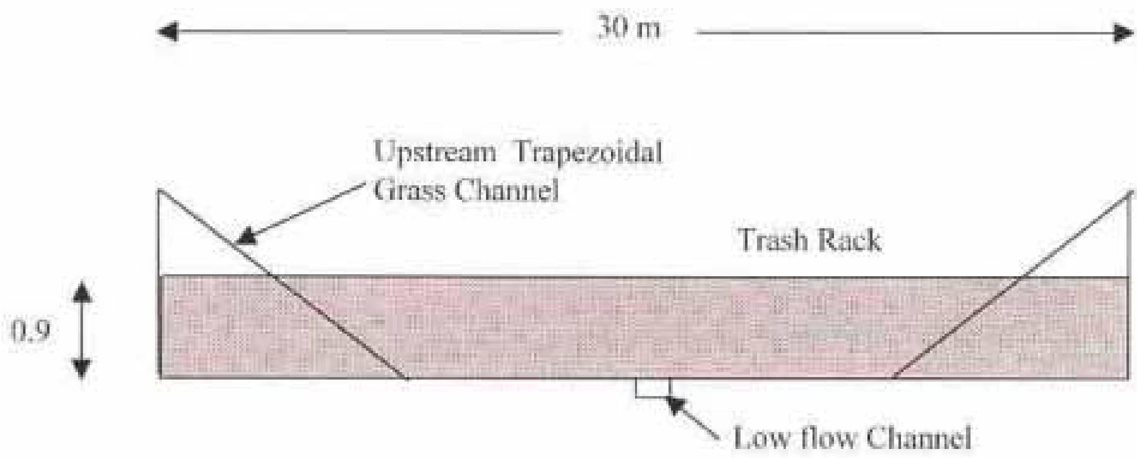
Adopt a concrete-line transition channel of 15 m from the trash rack

6.7.4.5.2 A low flow channel is to be provided on the donwstream transition channel. The depth of this low flow channel should be rectangular channel of 0.3 m wide and 0.06 m deep.

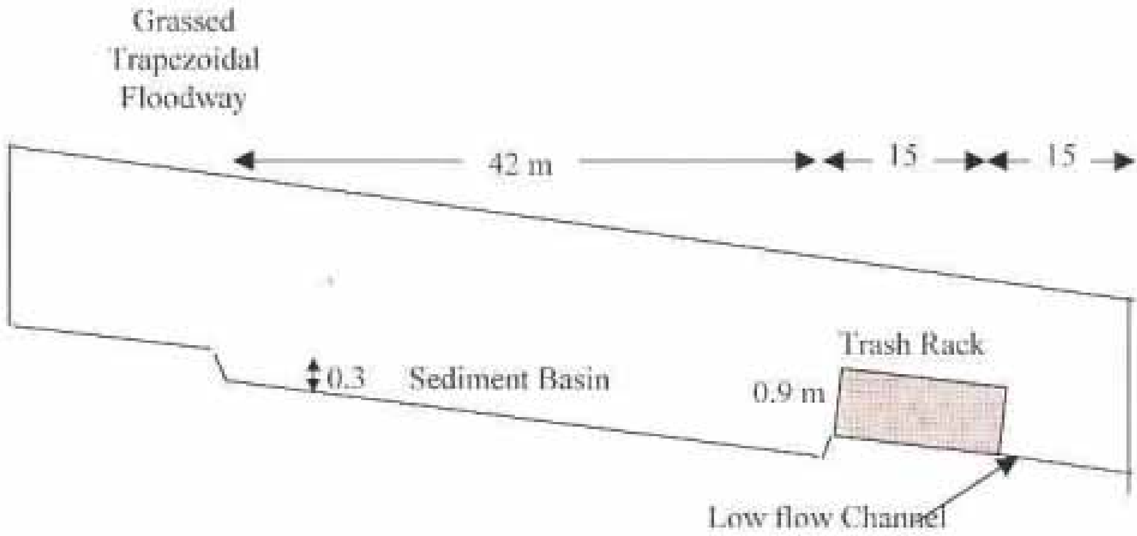
Construct a low flow channel of 0.3 m(W) x 0.06 m(D) on the transition channel downstream of the sediment basin



Plan



Section A-A


Grassed
Trapezoidal
Floodway
42 m
15
15
0.3
Sediment Basin
Trash Rack
0.9 m
Low flow Channel

Section B-B