Government of Malaysia
Department of Irrigation and Drainage

Urban Stormwater Management Manual *for* Malaysia

MSMA *2nd* Edition

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Foreword

MSMA (Manual Saliran Mesra Alam), an abbreviation from Malay Language translation of Urban Stormwater Management Manual, has been widely accepted term and since become trade mark in the stormwater industry in Malaysia. The first edition of the Manual, published in 2000, has served as invaluable references for both authority and private professionals. The version included the latest standards and practices, technologies, best engineering practices that were generally based from foreign countries. The first edition was also quite voluminous and relatively difficult for engineers and professionals to use. Recognising all these and after ten (10) years time lapse, the Department decided that it is timely for the first edition be improved. This improved version is called MSMA 2nd Edition.

The MSMA 2nd Edition is developed through contributions from the Government as well as private sectors and foreign experts. The Manual has been simplified and updated to serve as a source of information and to provide guidance pertaining to the latest stormwater best management practices (BMPs). This is one of the many initiatives undertaken by the DID to further enhance its services parallel with ongoing transformations taking place in Government Department and private sectors.

There are just too many to name and congratulate individually, all those involved in preparing this Manual. Most of them are my fellow professionals who are well-respected within their fields. I wish to record my sincere thanks and appreciation to all of them and I am confident that their contributions will be truly appreciated by the users for many years to come.

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This edition could not have been completed without the guidance and assistance of key staff members from DID's Stormwater Management Division recognized herein:
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The contribution of Technical Committee members are also acknowledge herein:
Ministry of Natural Resources and Environment, Ministry of Housing and Local Government, Department of Environment, National Landscape Department, Town and Country Planning Department, Real Estate and Housing Developers' Association Malaysia (REHDA), Association of Consulting Engineers Malaysia (ACEM), Malaysian Institute of Architects (PAM), Institution of Engineers Malaysia (IEM), Construction Industry Development Board (CIDB), Master Builders Association of Malaysia and all other agencies that has contributes directly and indirectly to the completed manual

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Last but not least, special recognition is expressed for our consultant, PWM Associates Sdn. Bhd. and his key team members who had contributed invaluable part of the process that led to the completed manual. The names of the many individual contributors and reviewers who helped in the development of this manual are listed in the List of Contributors.

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Abbreviations

AARY	Average Annual Rainwater Yield
AASHTO	American Association of State Highway and Transportation Officials
ACT	Australian Capital Territory
AEP	Annual Exceedance Probability
AN	Ammoniacal Nitrogen
ANCOLD	Australian National Committee on Large Dam
API	American Petroleum Institute
AR&R	Australian Rainfall and Runoff
ARC	Atlanta Regional Commission
ARI	Average Recurrence Interval
AS	Australian Standard
ASCE	American Society of Civil Engineers
BIOECODS	Bio-Ecological Drainage System
BMPs	Best Management Practices
BOD	Biochemical Oxygen Demand
CAD	Computer Aided Design
CDM	Camp Dresser & McKee
CFWP	Centre for Watershed Protection, Australia
CIRIA	Construction Industry Research and Information Association, UK
CMP	Corrugated Metal Pipe
COD	Chemical Oxygen Demand
CWA	Concrete Washout Area
DC	Design Chart
DCP	Discharge Control Pit
DID	Department of Irrigation and Drainage Malaysia
DOE	Department of Environment Malaysia
DOP	Drainage Outlet Protection
ECB	Erosion Control Blanket
ED	Extended Detention
EGL	Energy Grade Line
EL	Elevation
EMC	Event Mean Concentration
ESC	Erosion and Sediment Control
ESCP	Erosion and Sediment Control Plan
FHWA	Federal Highway Administration, USA
FOS	Factor of Safety
GIS	Geographic Information System
GPTs	Gross Pollutant Traps
GWL	Ground Water Level
GWT	Ground Water Table

HAT	Highest Astronomical Tide
HGL	Hydraulic Grade Line
HW	Head Water
HWL	High Water Level
IDF	Intensity Duration Frequency
JKR	Jabatan Kerja Raya (Public Works Department) Malaysia
LAT	Lowest Astronomical Tide
LOC	Limits of Construction
LSD	Land Survey Datum
LWL	Low Water Level
MAR	Mean Annual Rainfall
MDE	Maryland Department of the Environment, USA
MHHW	Mean Higher High Water
MHWS	Mean Higher Water Spring
MLLW	Mean Lower Low Water
MPCA	Minnesota Pollution Control Agency
MSA	Material Storage Area
MSL	Mean Sea Level
MSMA	Manual Saliran Mesra Alam (Urban Stormwater Management Manual for Malaysia)
MUSLE	Modified Universal Soil Loss Equation
NPS	Non-point Source
NRW	Natural Resources and Water
NSW	New South Wales
NTU	Nephelomatric Turbidity Unit
NZS	New Zealand Standard
O&G	Oil and Grease
OGI	Oil and Grease Interceptor
OSD	On-site Detention
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSD	Permissible Site Discharge
QUDM	Queensland Urban Drainage Manual
RCD	Reinforced Check Dam
RMHM	Rational Method Hydrograph Method
RWHS	Rainwater Harvesting System
SB	Sediment Basin
SBB	Sand Bag Barrier
SBTR	Sedimentation Basin Trash Rack
SCADA	Supervisory Control and Data Acquisition
SCL	Sediment Control Log
SCS	Soil Conservation Services, USDA
SIRIM	Standards and Industrial Research Institute of Malaysia

SOP	Standard Operation Procedure
SR	Surface Roughening
SSA	Stabilized Staging Area
SSR	Site Storage Requirement
ST	Sediment Trap
SUDS	Sustainable Urban Drainage Systems
TDH	Total Dynamic Head
TH	Total Head
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorus
TPF	Temporal Pattern in Fraction
TRM	Turf Reinforcement Mats
TSC	Temporary Stream Crossing
TSD	Temporary Slope Drain
TSS	Total Suspended Solids
TW	Tail Water
UDFCD	Urban Drainage and Flood Control District, Denver
UPVC	Unplasticised Polyvinyl Chloride
USBR	United States Bureau of Reclamation
USDA	United State Department of Agriculture
USDOT	United States Department of Transportation
USEPA	United State Environmental Protection Agency
USLE	Universal Soil Loss Equation
VTC	Vehicle Tracking Control
WQV	Water Quality Volume
WSE	Water Surface Elevation
WSUD	Water Sensitive Urban Design
YAS	Yield After Spillage
YBS	Yield Before Spillage

INTRODUCTION TO THE MANUAL

This Urban Stormwater Management Manual for Malaysia (MSMA 2nd Edition) is an improved version of the MSMA 1st Edition that provides planning and design guidance to all those involved in the management of stormwater.

Users are advised to read this section before start using the Manual. Chapters 1, 2 and 3 serve as the driver of the Manual while the rest, Chapter 4 to 20, detail the necessary design methods and procedures on relevant stormwater facilities. This edition is supplemented and ended with Annexures; on ecological plants and maintenance.

Stormwater management design requires a multi-skills and multi-disciplinary approach and it should be expected that some Chapters are interrelated. However, each Chapter is simplified, concised and complete in the coverage of its own subject material.

1. GENERAL

1.1 Goal and Objectives

The goal of this Manual is to provide easy guidance to all regulators, planners and designers who are involved in stormwater management implementation, which is often undertaken by a number of organisations. The challenge is to ensure that the administration of the planning, design and maintenance of stormwater management systems is consistent across the relevant Local, State and Federal Authorities and the professions of urban development, environmental, water resources, civil engineering and landscape architecture.

Under this direction, stormwater management will have multiple green and hazards-free objectives within and downstream of development area

- Ensure the safety of the public;
- Control nuisance flooding and provide for the safe passage of less frequent or larger flood events;
- Stabilise the landform and control erosion;
- Minimise the environmental impact of runoff; and
- Enhance the urban landscape and ecology.

1.2 Scope

This Manual covers most of the important aspects and requirements of stormwater management practices for new and existing urban areas.

1.3 Required Knowledge

Engineers, architects, planners and others who are involved in applying the guidelines set out in this Manual should have undertaken an appropriate course of study in their subject. For example, design engineers are expected primarily to have undertaken a course in hydrology and hydraulics, within tertiary civil engineering curriculum or equivalent experience, in order to apply the subject matter in the Manual.

1.4 Related Stormwater Management Documents

The related document "Design Guides for Erosion and Sediment Control in Malaysia (DID, 2010)" should be considered when planning urban development and/or designing stormwater management infrastructure.

2. ENHANCED DESIGN SKILLS

The Manual explains the design methods of each stormwater management control components in subsequent chapters. Users should not limit themselves only to the material available within this Manual but also to have initiatives in research to enhance their design and to continuously build up knowledge in this aspect which can subsequently be added on to enhance their design skills. Users should explore maximum combinations of these components as are practicable to meet their design objectives.

3. CONTENTS

The Chapters were prepared covering mainly administration, quantity control design, quality control design and conveyance design. They are accompanied by Annexures on planting and maintenance. In each design chapter, background information, analysis and simplified design procedures are presented. Where appropriate, supporting basic theory and worked examples are also provided to assist the users.

3.1 Administration and Requirement

These early sections are the key that sets requirement and direction to enable users to start and finish the facility design process of a stormwater facility project. Chapter 1 – Design Acceptance Criteria provides mainly design Average Recurrence Intervals (ARIs) for both quantity control and conveyance system as well as prescribed Water Quality Volume (WQV) for quality control system.

Before proceeding to subsequent design Chapters, design fundamentals for quantity and quality management facilities are provided in Chapters 2 and 3, respectively. They present hydrologic, hydraulic and water quality principles, methods and procedures that are inherent in the stormwater system design.

These three (3) Chapters 1, 2, and 3 are the pre-requisites to the rest of the chapters in the Manual.

3.2 Quantity Control System Design

Quantity control facilities covered in the Manual basically deal with control at premise level; Roof Drainage (Chapter 4), On-site Detention (Chapter 5) and Rainwater Harvesting (Chapter 6) while at community level using Detention Pond (Chapter 7). OSD, combined with rainwater tanks, would be preferred as they reduce more runoff peak at small scale.

Detention pond is regarded as the most cost-efficient mean of reducing peak flood runoff. A step-by-step procedure is detailed out in text and worked example involving pond routing, based on storage-indication curve, in each Chapter 2 and 7.

3.3 Quality Control System Design

Quality control or best management practices (BMPs) design covered in the Manual are for permanent facilities; Infiltration (Chapter 8), Bioretention (Chapter 9), Swales (Chapter 14), Gross Pollutant Traps (Chapter 10), Water Quality Pond and Wetlands (Chapter 11) as well for construction Erosion and Sediment control (Chapter 12).

The main parameters of concern are sediment, total suspended sediment (TSS), total phosphorus (TP) and total nitrogen (TN). TSS is known to have been the most important pollutant for treatment as it is more readily settled out and removed. Attached with it in water column are some heavy metals and oil and grease.

3.4 Conveyance System Design

Design procedures for conveyance system, minor and major, are found in Chapter 13 (Pavement Drainage), Chapter 14 (Drain and Swales), Chapter 15 (Pipe Drain), Chapter 16 (Engineered Channel), Chapter 17 (Bioengineered Channel), Chapter 18 (Culvert) and Chapter 19 (Gate and Pump). Chapter 20 contains various hydraulic structures as integral components of stormwater facilities. These facilities convey runoff from premise level to receiving waters, lakes, rivers and seas, connecting both quantity and BMPs structures.

Swales are recommended in most areas while lined drain or pipe drain are suitable in highly urbanised zones. Bioengineered systems deals more with visual and ecological objectives of development. Culvert, gate and pump are common practices and their design procedures are found also in most hydraulic documents elsewhere.

Design procedures for gate and pump are provided to guide users in solving stormwater disposal difficulties in high tailwater boundaries normally experienced at lowland areas closed to rivers and shorelines.

3.5 Facility Planting, Maintenance and Care

Each stormwater facility shall involve planting to enhance its ecological, environmental and visual quality purposes. Annexure 1 provide lists and guidance of various ecological plants, obtained locally, for possible application at various sites, primarily Detention Pond (Chapter 7), Infiltration (Chapter 8), Bioretention (Chapter 9), Swales (Chapter 14), Water Quality Pond and Wetlands (Chapter 11) and Bioengineered Stream (Chapter 17).

Annexure 2 independently provides the required inspection, maintenance and caring procedures for most of the stormwater facilities found in the Manual.

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DESIGN ACCEPTANCE CRITERIA 1

CHAPTER 1 DESIGN ACCEPTANCE CRITERIA

1.1 INTRODUCTION

Along with the increase in water quantity, urbanization results in the increase in non-point pollutants from various municipal landuses and activities which can all end up in receiving waters. Their damaging effects are not always immediately apparent.

Stormwater management (SWM) is the mechanism for controlling stormwater runoff for the purposes of minimizing the catchment flow rates, runoff volumes, frequency of flooding and degradation of surface water quality through implementation of construction erosion and sediment control, quantity control and treatment best management practices (BMPs) to diminish the effects of landuse changes. In response to the change, systems consisting of curb, gutter, drain and lined channels are developed to safely convey the runoff through the catchment. Although such effects from a small individual site may seem inconsequential the collective effects of numerous sites throughout the catchment can have substantial impact on the environment, especially in a catchment's lower reaches.

1.1.1 Stormwater System

Stormwater systems are divided into two categories: major and minor. The minor system consists of swales, gutters, pipes, on-site detention, bioretention and the various types of inlets and BMPs that collect, store, treat and convey runoff to a discharge area or impoundment. Components in the minor quantity system are sized to manage runoff generated by the more frequent short-duration storm events. The major system includes natural streams, channels, ponds, lakes, wetlands, large pipes and culverts (Figure 1.1). Design criteria for the major quantity system are typically based on significant amounts of rainfall produced by the less frequent long-duration storms. BMPs, all sizes, are designed based on the same selected storm but from much more frequent events.

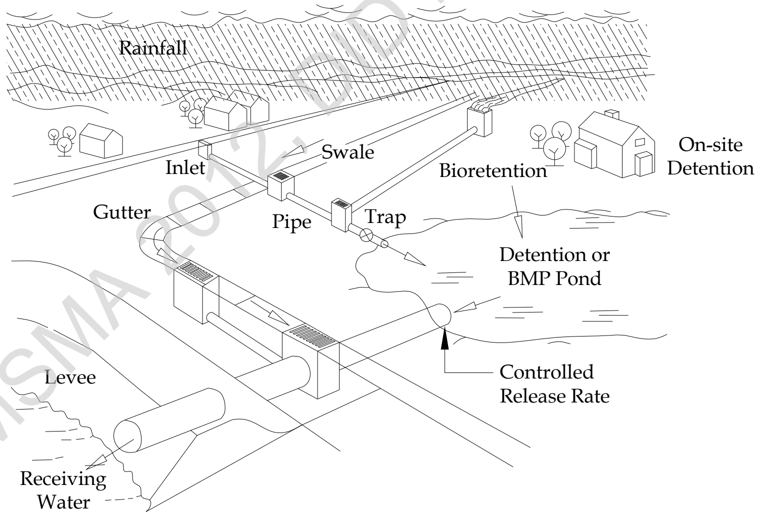

Figure 1.1: Typical Urban Drainage System (Adapted from Kibler, 1982)

1.1.2 Design Goal and Basis

The goal of this Manual is to provide the various design concepts, criteria and procedures that serve as the foundation for developing stormwater management facilities. They should set limits on development; provide guidance and methods of design; provide details of key components of drainage and control systems; and ensure longevity, safety, aesthetics and maintainability of the system.

The design acceptance criteria are mandatory requirements for the planning and design of new and/or upgrading of existing stormwater management systems in urban areas.

The criteria provided in this Chapter apply to *all* urban stormwater systems, while subsequent Chapters in the Manual give more detailed requirements for designing individual system components, quantity and quality facilities. The criteria are set based on the type of landuse, level of protection required, economy, risks of failure, public safety, ecology, aesthetics, etc. One of the most common criteria used in the facility design is the average recurrence interval (ARI), which is set based on whole life economy of the facility, the level of protection required and the hazard potentials to the downstream areas.

1.2 STORMWATER QUANTITY DESIGN CRITERIA

The minor and major systems are closely interrelated, and the design of each component must be done in conjunction with the overall stormwater management standards set by the authorities (Knox County, 2008).

Design storm ARIs to be adopted for the planning and design of minor and major storm runoff quantity systems shall be in accordance with Table 1.1. The storm runoff quantity design fundamentals are given in Chapter 2 of this Manual.

Table 1.1: Quantity Design Storm ARIs

Type of Development (See Note 1)	Minimum ARI (year) (See Note 2)	
	Minor System (See Note 3)	Major System (See Note 3)
Residential		
Bungalow and semi-detached dwellings	5	50
Link house/apartment	10	100
Commercial and business center	10	100
Industry	10	100
Sport field, park and agricultural land	2	20
Infrastructure/utility	5	100
Institutional building/complex	10	100

Notes:
1. For mixed developments, the highest of the applicable storm ARIs from the Table shall be adopted.
2. In the case where designing to the higher ARI would be impractical, the selection of appropriate ARI should be adjusted to optimise the cost to benefit ratio or social factors. If justified, a lower ARI might be adopted for the major system, with consultation and approval from the Department of Irrigation and Drainage (DID). Even if the stormwater system for the existing developed condition is designed for a lower ARI storm, sufficient land should be reserved for higher ARI flow rates, so that the system can be upgraded when the area is built up in the future.
3. All development projects shall be protected from both minor and major floods and, therefore, must have combination of minor and major systems. Habitable floor levels of the buildings (platform levels) shall be set above the 100 year ARI flood level based on the most recent data available. The drainage submission must show the minor and major system components in their drawings and plans.

The *minor system* is intended to collect, control and convey runoff from buildings, infrastructures and utilities in relatively frequent storm events (up to 10 year ARI) to minimise inconvenience and nuisance flooding. During any event larger than the minor storm ARI, the higher runoff will overspill the minor drainage components.

The *major system* is intended to safely convey and control runoff collected by the minor drainage system together with its possible overspill to the larger downstream systems and water bodies. The major system must protect the community from the consequences of large and reasonably rare storm events (generally up to 100 year ARI), which could cause severe property damage, injury or loss of life.

The main reason for adopting a higher standard for the minor system in large commercial, business and industrial areas is because of the much greater potential for damage and disruption in a flood, that exceeds the minor system capacity. At a minimum, habitable and human occupied floor levels of buildings shall be above the 100 year ARI flood level.

Restrictions on development of flood-liable land shall generally apply to land affected by flooding from stormwater drains, as well land affected by river flooding. However, it is recognised that the duration of stormwater flooding is shorter and therefore some short-duration disruptions (such as to road traffic) can be accepted, unless the loss due to traffic congestion is very high.

1.2.1 Peak Discharge Control

The level of runoff quantity control required is dependent on the type of development proposed, new development or redevelopment. Flow control requirements are stipulated as the following:

Runoff quantity control requirements for any size of development or re-development project is "Post development peak flow of any ARI at the project outlet must be less than or equal to the pre-development peak flow of the corresponding ARI** ($Q_{post} \leq Q_{pre}$)**".

The relevant local regulatory authority will decide the magnitude of the ARI and allowable discharge limit of the area depending on the existing receiving conveyance capacity, potential risk and vulnerability of the downstream resources and sensitivity of the catchment concern.

1.2.1.1 New Development

New development is defined as the conversion of natural or rural areas into urban, industrial infrastructure and/or utility development.

For new development proposals, the post-development peak flow from the outlet point(s) of the site to the downstream drainage system or receiving waters shall not exceed the corresponding ARI pre-development flow for both the minor and major system design storm ARIs. Pre-development peak flow shall be the estimated flow from the site based on known or estimated catchment conditions prior to the new development.

1.2.1.2 Redevelopment

Redevelopment is defined as the renewal and reconstruction of an existing residential, commercial, industrial or infrastructure areas. The degree of runoff control required will depend on the scale of the development and primarily the net increase in impervious area.

Flow control will be required for any redevelopment under the following conditions:

- The density of the redevelopment, measured as the total equivalent impervious area of the redevelopment, is greater than that of the existing development; and/or
- The capacity of the existing stormwater system does not meet the design storm ARIs given in Table 1.**Error! Bookmark not defined.**.

(a) Individual Lot

Lot redevelopment is defined as redevelopment of single or multiple adjacent lots where all of the stormwater systems are privately owned.

The post-redevelopment peak flow rate from a lot redevelopment shall not exceed the pre-redevelopment rate for the minor system design storm. This will generally require the provision of on-site stormwater control if the equivalent impervious area is increased.

(b) Subdivision

Subdivision redevelopment is defined as redevelopment where all or part of the stormwater system will be handed over to a government authority and become part of the municipal drainage system.

Post-redevelopment peak flows from the outlet point(s) of the redevelopment area to the existing downstream public conveyance system or receiving water shall not exceed the existing development flows for both the minor and major system design storm ARIs. Existing development peak flow *(the pre-redevelopment flow)* shall be the estimated flow from the site based on the developed catchment conditions prior to redevelopment.

The *minimum* responsibility of the developer is to ensure that the redevelopment does not create or worsen any capacity problems in the existing public drainage system up to the major system ARI, or higher in potentially hazardous situations. This will require the construction of detention pond, either on its own or in conjunction with on-site detention.

Notwithstanding the above, it may be advantageous in some areas (where upgrading of the existing municipal stormwater system is physically or economically impractical) for the developer to provide additional flow control.

1.2.2 Storage Facilities

Storage facilities are the core elements of achieving one of the major stormwater quantity control criteria which is the post-development peak discharge cannot be more than the pre-development peak discharge. Its achievable with proper locating and sizing of the storage facilities.

The recommended storage facilities are on-site detention (OSD) and detention pond. Depending on the land availability, these facilities can be located on-line or off-line to the conveyance system. Design of quantity control facilities should be done following the procedures given in Chapter 5 and 7.

1.2.2.1 On-site Detention

OSD may be provided as above-ground, below-ground, or a combination of both within a property boundary. The above-ground storages (basically as tanks) can be located at roof top, lawns, gardens, car park, driveway, etc. The below-ground storages are from tanks and pipe packages. For combined facilites, a proportion of the total storage is provided as below-ground, whilst the remainder of the storage is provided as above-ground.

Storage tanks to be used for OSD should be structurally sound and be constructed from durable materials that are not subjected to deterioration by corrosion or aggressive soil conditions. Tanks shall be designed to withstand the expected live and dead loads on the structure, including external and internal hydrostatic loadings. Buoyancy shall also be checked, especially for lightweight tanks, to ensure that the tank will not lift under high groundwater conditions.

To permit easy access to all parts of the storage for maintenance, the floor slope of the tank shall not be greater than 10%. The slope shall also be not less than 2%, to enable good drainage of the tank floor. The design storm for estimating the required storage volume shall be 10 year ARI.

1.2.2.2 Detention Pond

Detention pond is the most important component for stormwater quantity control. A detention pond can be dry or wet type. To meet the flow control objectives of a detention pond, it is necessary to consider the behaviour of the storage by examining the degree of reduction of flows from the catchment, water depth in the pond, pond empty time, etc.

The pond primary outlets can be designed as multi level riser to synchronise the flow from minor and major storms. Secondary outlets for all detention ponds shall be designed to safely pass a minimum design storm of 100 year ARI. The side slope of pond is recommended flatter than 1(V):6(H). Areas with slopes steeper than 1(V):4(H) may require a fence or rail for safety reason. Special attention should be paid to the outlets, to ensure that people are not drawn into them. Rails, fences, crib-walls, anti-vortex devices, and grates should be provided where necessary.

Provision should be provided in a dry detention pond to bypass small flows through or around the pond using low flow channel or pipe. This is necessary to ensure that the pond floor, particularly if it is grassed, is not inundated by small storms or continually wetted by dry weather baseflow. The minimum amount of bypass flow should be one half (1/2) the 1 month ARI flow.

1.2.3 Conveyance Facilities

Stormwater conveyance systems shall be planned, analysed, and designed in order to provide acceptable levels of safety for the general public and protection for private and public property. Design procedures of the respective conveyance facilities are given in Chapter 4 and Chapter 13 to 19.

1.2.3.1 Surface Flow Criteria

A range of surface flow criteria must be applied to minimise both nuisance runoff flow and hazards from runoff flooding of infrastructures, buildings, utilities and other areas that have regular public and vehicular access. The criteria apply to *both* major and minor systems.

The surface flow criteria to be adopted for conveyance design are provided in Table 1.**Error! Bookmark not defined.**.

Table 1.2: Surface Flow Criteria

Criteria	Recommended Limit
Overland flow velocity x depth for vehicle stability and pedestrian areas	< 0.5 m²/s
Flow width for street gutters	2 to 2.5 m or a half-lane
Flow velocity	
• soft lined waterways and overland flow paths	< 2.0 m/s
• hard lined channels	< 4.0 m/s
Ponding depth	< 0.20 m in unfenced areas < 1.2 m in fenced areas

1.2.3.2 Property Drainage

Property drainage refers to the systems which transfer runoff from roofs, paved areas and other surfaces to a suitable outlet or disposal facility. The system involves gutters, downpipes, drains, pipes, swales and storage and treatment facilities. It shall be the responsibility of the property owner to design infiltration or on-site detention structures or facilities, to limit the amount of stormwater that can be drained to streets or trunk drainage systems, in order to reduce flooding and pollution. Usually, 5 minute duration design rainfall of 20, 50 and 100 year ARIs shall be used for the sizing of property drainage components.

1.2.3.3 Pavement Drainage

Effective pavement drainage is essential to the maintenance of road service level and to traffic safety. The potential for hydroplaning at high speeds as well as the potential for vehicles to float or be washed off roads at lower speeds shall be given due considerations in designing pavement drainage. Recommended minimum values of roadway longitudinal slope shall be used for safe pavement drainage. Desirable gutter grades shall not be less than 0.5 percent for curbed pavements with an absolute minimum of 0.3 percent. To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent shall be maintained within 15 metres of the low point of the curve.

All roads in urban areas shall generally be provided with an integral curb and gutter. However, where the volume of gutter flow is negligible as in car parks and on the high side of single-crossfall roads, a curb without a gutter is acceptable. Inlet plays important role in pavement drainage. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in hazard to the travelling motorists.

1.2.3.4 Open Drain and Swale

Open drains are all drains other than pipe and box culverts used to convey runoff to its receiving waters. The most common types of open drains used in stormwater management are vegetated swales, grassed channels, concrete lined drains and composite drains. Determination of drainage types (earth/concrete/composite) should be based on space availability, site suitability, environment conditions (aesthetic, conservation values, etc.) and maintenance advantages and disadvantages. Each of the drains has its own design criteria that need to be followed.

1.2.3.5 Pipe Drain

Pipe drains are very common in urban areas, where the land is limited. Pipes can be used in privately owned property drains or open public spaces. A drainage reserve shall be wide enough to contain the service and provide working space on each side of the service for future maintenance activities. Minimum clearances between stormwater pipelines and other services, and other guidelines should be in accordance with the criteria given in the relevant Chapter. Stormwater pipelines should be located on the high side of road reserves to permit relatively short service tie connections to adjacent properties.

1.2.3.6 Engineered and Bio-engineered Waterways

The engineered waterways are generally constructed large open or composite channels. The design of engineered channels should, wherever practicable, mimic the natural stream forms in the immediate region. Various considerations should be planned for the successful management of aquatic, ephemeral and terrestrial environments along the engineered waterways. The greater the Froude number, the higher is the effect of tailwater on the jump. Therefore, for a Froude number as low as 8, the tailwater depth should be greater than the sequent depth downstream of the jump so that the jump will stay on the apron. When the Froude number is greater than 10, the common stilling basin dissipater may not be as cost-effective as a special bucket type dissipater.

On the other hand, bioengineered waterways use the combination of biological, mechanical, and ecological concepts to control erosion through the use of vegetation or a combination of vegetation and construction materials. Various criteria are to be adopted, which are related to scour, attrition, slumping, undermining, etc. The design of bioengineered channels involves the creation of channels with the attributes of natural watercourses pertinent to the location within the catchment and should be based on a sound understanding of fluvial geomorphic principles.

1.3 STORMWATER QUALITY DESIGN CRITERIA

The pollution from stormwater is termed as non-point source (NPS) due to its diffusive nature of generation from varied landuses. Various types of BMPs can be used to reduce the environmental impact due to NPS pollutants from development and re-development projects.

Stormwater quality control facilities, temporary or permanent best management practices (BMPs), shall be planned, analysed, and designed for all types of developments, in accordance with the criteria in Table 1.3. Additional criteria, specifically related to the BMPs are described in the following sections. The quality design principles for the temporary BMPs are given in Chapter 12 while the permanent BMPs in Chapter 3.

Table 1.3: Quality Control Design Criteria

Variables	Criteria
Water Quality Volume	Temporary BMPs - 50mm of rainfall applied to catchments draining to the BMPs. Permanent BMPs - 40mm of rainfall applied to catchments draining to the BMPs.
Primary Outlet Sizing	Based on the peak flow calculated from the 3 month ARI event
Secondary Outlet (Spillway) Sizing	As per the ARIs recommended in the respective chapters of the individual BMPs.

1.3.1 Pollution Control

The target for minimum retention (new development) or for reduction in annual pollutant loads (redevelopment) for the four (4) common pollutants shall be in accordance with Table 1.4.

Table 1.4: Pollutant Reduction Targets

Pollutant	Reduction Targets (%)
Floatables/Litters	90
Total Suspended Solids (TSS)	80
Total Nitrogen (TN)	50
Total Phosphorus (TP)	50

Note: Relevant local regulatory authorities may set higher (stringent) targets depending on the sensitivity and level of pollution in the surrounding areas.

The form of the minimum criteria in Table 1.4 are different for new development, and for redevelopment. For new development a minimum overall *percentage removal efficiency* is specified. For redevelopment or drainage system upgrading, the criteria are set in terms of a *reduction in average annual pollutant load* compared with the load under existing conditions.

1.3.2 Temporary BMPs Facilities

The main purpose of temporary BMPs, erosion, runoff and sediment control BMPs is to minimize erosion and soil delivery away from the developing/construction site resulting from land clearing and grading or other land-disturbing activities. The BMPs shall be provided on all land development and infrastructure projects of all sizes, minor or major. The BMPs shall be maintained in good working order at all times until disturbed areas have been permanently stabilised. Details on the design and preparation of temporary control BMPs are given in Chapter 12.

1.3.2.1 Erosion and Sediment Control Plan

The first step to minimize sediment accumulation from any site is to prepare an effective erosion and sediment control plan (ESCP), providing details of construction activities and practices, timing and staging of works, and the range of erosion and sediment BMPs practices and measures to be implemented. The next step would be to implement the ESCP at the site to minimise the erosion.

Various criteria are to be followed in order to reduce the erosion of soils. For hilly areas (slopes greater than 12° or 20%) terracing shall be built and maintained. Cover plants shall be established on the slopes of the platforms and walls of the terrace immediately after commencement of earthworks. For the slopes greater than 35° or 70% no works are allowed and should instead be maintained as natural.

1.3.2.2 Sediment Control Measures

Surface water collected from disturbed areas shall be routed through a sediment basin or sediment trap before release from the site. Sediment retention facilities shall be installed prior to the clearing, grading or disturbance of any contributing area.

Sediment basin shall be sized to retain a minimum of 90% of total suspended solid (TSS) for all storms that produce rainfall up to 50mm, for all sites. Such condition will not be applicable, for the compliance purpose, to any storm that produces event rainfall greater than 50mm.

1.3.3 Permanent BMPs Facilities

One of the aims of an ecologically-based stormwater management and planning approach is to identify the sustainable pollutant exports from a site to protect the environmental values of the receiving water. Ideally the identification of sustainable pollutant loads on a receiving water is based on the magnitude of overall catchment exports, the contribution of each landuse to the overall levels of pollutant export and the reduction in overall annual pollutant loads are required in order to achieve the water quality objectives that are linked to the environmental values. Chapter 8, 9, 10, 11 and 14 provide relevant design procedures for the permanent control BMPs.

1.3.3.1 Treatment Measures

The followings are requirements for source and treatment control BMPs facilities:

(c) Infiltration

Infiltration facilities (sumps, trenches, porous pavements and basins) are designed to capture a volume of stormwater runoff, retain it, and infiltrate all or part of that volume into the ground and the excess will overflow to conveyance system. A properly designed and constructed infiltration system can sustain longer from clogging

The limits of the maximum contributing catchment areas for infiltration sump, trench & porous pavement, and basin are recommended as 500m², 4 ha and 15 ha, respectively. Saturated soil infiltration rate of the proposed infiltration facility site should have a minimum value of 13mm/hr. The base of all facilities should be located at least 1.5m above the seasonal high ground water level and/or any impermeable layer. Other guidelines related to the design of infiltration facilities are given in Chapter 8.

(b) Bioretention System

Bioretention systems use infiltration and vegetation to remove pollutants from stormwater. Usually the systems consist of excavated basins or trenches that are filled with porous media and planted with vegetation. If the infiltration rate and nutrient content of the in-situ soil are suitable, it is not necessary to use imported soil.

These systems should usually be used to capture and treat runoff from small catchments, limited to less than 1.0 ha of impervious area. Larger areas should be divided into smaller sub-areas with individual bioretention systems. Specific design of a bioretention facility may vary considerably, depending on site constraints or preferences of the designer or community. However, pretreatment, treatment, conveyance, maintenance reduction, and landscaping components should be incorporated into all bioretention systems. These design features are given in Chapter 9 of this Manual.

(c) Swales

Swales are wide but shallow channels designed to store and/or convey runoff at a non-erosive velocity, as well as to enhance stormwater quality through infiltration, sedimentation and filtration. Swale can be located within open space areas, parklands and along roadways. A swale should have the capacity to convey the peak flows from the design storm without exceeding the maximum permissible velocity. If this is not practical or the space is limited, designer should consider dividing the flow into surface and subsurface components where underground pipeline or drainage blocks are to be provided as subsurface drains.

The longitudinal terrain slope should not exceed 2% as low runoff velocities are required for pollutant removal and to prevent erosion. Side slope shall not be steeper than 2H:1V while side slope 4H:1V or flatter is recommended for safety reason. Other criteria related to the design of swales are given in Chapter 14.

(d) Gross Pollutant Traps

Gross pollutant traps (GPTs) remove litters, debris, coarse sediment and hydrocarbon from stormwater. GPTs should be used as pre-treatment BMPs for detention pond, water quality pond and/or wetlands.

The GPTs shall be sized to retain 90% of gross pollutants with a minimum of 70% of coarse sediments and oil droplet. The GPTs must be designed so as to prevent any additional overflows in the stormwater system in the event of partial or complete blockage. Tidal influence and backwater effects must be considered. The pollutant reduction performance must be maintained up to the design ARI discharge. If design flows are exceeded, the GPTs shall not allow any significant remobilization of trapped material. Detail criteria related to the design of the GPTs are given in Chapter 10.

(e) Water Quality Pond and Wetlands

Water quality control ponds (wet ponds) or constructed wetlands must be protected from excessive sediment loads by installing upstream GPTs. Water quality ponds and wetlands shall be sized to achieve the pollutant capture above the permanent pool’s water surface as set out in Table 1.4 and to minimise the remobilization of deposited pollutants. Ponds shall be fitted with spillways able to discharge at least the 100-year ARI flow.

The designer must be guided by the frequency of the storm events and their peak discharge rates when determining the risk that high velocities will wash out the epiphytes and biofilms, which are very important for the proper function of a wetlands. Velocities > 0.1 m/s may cause washout. Occasional washout (say 1 event per year or less) may be acceptable provided that the epiphytes and biofilms can re-establish over time. Design criteria related to the water quality pond and wetlands are given in Chapter 11.

1.3.3.2 Treatment Trains

Various types of pollutants are generated due to storm runoff. Each BMPs is not suitable for treating all NPS pollutants. Proper combinations of stormwater BMPs usually provide best cost-effective solutions. Such a combination, which provides a sequence of treatment through various BMPs, is called a ***treatment train***. Typically, the NPS pollution control plan would set performance requirements for the BMPs which are part of the treatment train. However, if no plan is available, the designer should still investigate options for providing treatment trains in order to achieve the project objectives.

Often, the components of a treatment train are intended to treat different pollutants. An example is a GPT to treat coarse sediment and litter, combined with a water quality pond to treat fine sediment and dissolved nutrients. In this case, each device can be sized according to its own sizing guidelines and ignoring any pollutant removal provided by the other device. The resulting design is likely to be conservative.

Treatment trains shall be provided in all new development, redevelopment, or stormwater system upgrading. Each component may be designed for a different annual pollutant retention or reduction target. However, the overall annual pollutant reduction of the whole treatment train must fulfil the targets given in Table 1.4.

1.3.3.3 Housekeeping and Education

Housekeeping measures are those measures that keep out pollutants from being introduced to the runoff. These includes covering chemical storage and handling areas, sweeping up chemical spills, berming (bunding) chemical storage areas; mitigating the dumping of household and yard trash into the drainage system. Education is included as a housekeeping practice, although it is not actually a practice in itself.

Many of these measures do, however, relate to and work in conjunction with source control and treatment controls. Control of urban stormwater quality should not rely only on end-of-pipe solutions. Improved housekeeping practices will be of great benefit by preventing pollutant loads from entering the stormwater runoff. Therefore, housekeeping and community education or non-structural source control BMPs should be implemented at all relevant levels of a municipality.

1.3.3.4 General BMPs Selection Guidance

For large developments or where the expected impacts are particularly severe, local regulatory authorities should set standards in terms of annual pollutant removal targets. These targets should ideally be based on ecological and other water quality indicators, for which site-specific detailed study might be required.

Although it is desirable to tie in the selection of treatment devices to predicted outcomes in the receiving waterway, this is difficult to achieve in practice. In any case the required studies will take a long time to complete. Most regulatory agencies throughout the world have adopted a similar approach. This Manual provides guidance to developers in selecting, installing and operating suitable measures to achieve those targets. Guidelines on the BMPs selection are given in Chapter 3.

1.4 UNIFIED DESIGN CRITERIA

1.4.1 Natural Drainage Paths

Major systems shall be planned and designed to conform to natural drainage patterns and discharge to natural drainage paths within a catchment. As much as possible, this also should be done for minor system, which is often modified to conform to road and lot layouts.

In general, runoff from development sites within a catchment should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater systems.

Diverting runoff from other catchments or sub-catchments can cause adverse impacts on downstream properties and stormwater systems due to greater runoff volumes than would otherwise occur from the natural drainage catchment. Therefore, the diversion of runoff to or from other catchments or sub-catchments is not permitted without authority approval.

1.4.2 Property Drainage

If drainage is provided within a development, each lot to be serviced by a system (open or pipe drain) shall have an individual stormwater service tie to provide for the connection of drainage from buildings to the public stormwater conveyance system.

A public stormwater conveyance shall only be located within a lot where it is intended solely for the purpose of providing drainage for the lot or adjacent lots. The conveyance shall be located such that access can be readily achieved and restrictions imposed on the use of the land due to the presence of the service are minimised.

1.4.3 Drainage Reserves

A drainage reserve shall be provided for stormwater conveyances located within private lots to provide access for maintenance. As drainage reserves can restrict flexibility in locating structures on a lot, conveyance system alignments which minimise the need for such reserves shall be considered wherever possible.

To meet the intent of the major/minor stormwater drainage philosophy, only minor system conveyances may be contained within drainage reserves in private lots. The major drainage system shall be contained within separate drainage reserves located completely outside private land.

1.4.4 Rights of Other Authorities

Where a stormwater conveyance is proposed to be located within close proximity to another service, the designer shall ensure that the requirements of the authority responsible for that service are met.

Where there is significant advantage in placing a stormwater conveyance on an alignment reserved for another authority, it may be so placed provided that both the local regulatory authority responsible for maintenance of the stormwater conveyance and the other authority concerned agree in writing to release the reservation.

1.4.5 Extreme Events

Design of stormwater systems to pass or safely contain a runoff of a given ARI implies that an overflow will occur during a larger event. There is also a risk that overflows will occur during a smaller storm due to blockage of some drains, culverts or inlets. All hydraulic works sized by a flood estimate are designed on a risk basis.

If failure of conveyance systems and/or major structures such as detention pond occurs during an extreme storm event, the risk to life and property could be significantly increased. This risk must be balanced against the probability of the extreme event. In most urban stormwater drainage systems the provision for a major storm will provide adequate security. However in unusual situations, such as possible failure of levees and detention pond outlets, the designer must consider the element of risk in a flood larger than 100 year ARI.

1.5 SITE DEVELOPMENT

1.5.3 Aesthetics and Functionality

The stormwater drainage system shall be designed so that it enhances the appearance of the area, and maximises its use by the community (Arthington et al., 1993) without compromising the functionality of the drainage component or system. However, the functionality must be given priority over the aesthetics, to make sure that the objective of the component is met. Blending good aesthetics with the drainage component would make the overall system easily acceptable to the community and people.

1.5.4 Landscaping

Landscaping is intended to ensure that a stormwater drainage system will enhance visual quality an area while providing water quality improvement. The landscaping should increase the aesthetics of the surroundings without compromising the functionality of the stormwater drainage objectives. The landscape design shall take into account and be part of the overall stormwater system design.

The design shall consider the followings:

- Allow for landscaping or future changes in landscaping to enhance the visual appeal of the system;
- Enhance open space links through development areas;
- Retain existing flora and fauna, as much as possible;
- Respect the functional use of the space; and
- Form part of and be sympathetic with the landscape character of the surrounding neighbourhood.

1.5.5 Site Clearing

Natural vegetation shall be retained wherever possible to minimise erosion within a site. This will also reduce the requirement for erosion and sediment controls during construction.

1.5.6 Land Grading

Wherever practical, the natural slope of the land within the development site shall be retained to ensure development lots and roadways are free draining. Where possible minimise development sites with platform that result in poor runoff disposal primarily at hillside areas. The conveyance system may then be excessively deep at the site outlet and possibly below the tailwater level. The existing topography of some sites may naturally be very flat that grade creation shall be considered.

1.5.7 Project Layouts

Project layouts must be planned in conjunction with drainage engineers, ecologists, environmentalists, etc. to avoid potential problems. Attention to the layout at this stage can significantly reduce drainage costs. Issues to consider include,

- Avoiding trapped low points;
- Providing suitable flow paths for the major design flood; and
- Providing suitable areas for stormwater quantity and quality control facilities.

For all kinds of lot developments it is important that layouts do not result in the concentration and discharge of runoff from upstream lots to adjacent downstream lots in sufficient quantity to cause nuisance conditions.

1.5.8 Hillside Drainage

Planning and achieving sustainable development in hilly areas is particularly important in regard to drainage, flash flood, erosion and sediment and slope stability management. Of all the techniques considered for the correction or prevention of slope instability on hillside development, proper drainage is recognised as the most important element. Proper drainage reduces soil moisture content and the destabilising hydrostatic and seepage forces on a slope, as well as the risk of surface erosion and piping. Poor site planning and design in a hillside development area causes large quantities of rainwater collected in the drainage system and from direct rainfall, to infiltrate into the ground. Design and construction of stormwater facilities in the hilly areas are to be done with proper caution, failure of which may cause slope instability. Some special drainage requirements are stipulated for a few types of development on hillside areas.

1.6 DESIGN FOR MAINTENANCE

The design of a stormwater drainage system needs to take into account the maintenance requirements of the system after it has been constructed. The drainage system should include facilities for ease of maintenance. Consequently, designers will need to familiarise themselves with the capacity and capabilities of the authority responsible for maintaining the stormwater infrastructure in order to provide facilities, that can be readily and economically maintained.

The purchase of special maintenance equipment and plant requires considerable lead-time by the maintenance authority for approvals and funding. As a consequence, any design incorporating the need for special or unusual equipment shall not be prepared without the prior written approval of the maintenance authority. This approval also extends to the use of special techniques and the hire of special equipment.

A stormwater drainage system must also be designed such that maintenance activities can be performed without the risk of inadvertent damage to the assets of other authorities. Other authorities include those responsible for gas, electricity, telecommunications, water supply, sewerage and other related services.

1.7 HEALTH, SAFETY AND ENVIRONMENT

Many of the requirements for the planning and design of stormwater systems presented in this manual have either directly or indirectly considered the need to protect public safety. Notwithstanding these requirements, stormwater managers and designers must consider the need or otherwise to implement additional measures to further protect public safety.

Typical measures to improve public safety include the followings:

- Railings on crossings, headwalls, steep slope or other locations where the public could fall into drains or waterbodies;
- Grates over open drains and manholes;
- Limiting the depth of open drains and ponds;
- Gentle side slopes on engineered waterways and on the sides of ponds, wetlands and lakes;
- Maximum flow velocity criteria for engineered waterways;
- Maximum velocity-depth criteria for flow on or across roads; and
- Landgrading criteria for different storm water structures.

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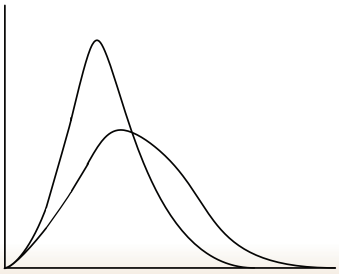


QUANTITY DESIGN FUNDAMENTALS 2

CHAPTER 2 QUANTITY DESIGN FUNDAMENTALS

2.1 INTRODUCTION

This chapter provides the fundamentals and appropriate methods required for designing stormwater quantity facilities. They apply to primarily detention pond as well as conveyance. The design ARIs for various types of facilities vary while the critical storm duration might be different due to differing facility operational concepts and mechanisms.

2.2 RAINFALL ESTIMATION

Rainfall data and characteristics are the driving force behind all stormwater studies and designs. Adequacy and significance of the rainfall design is a necessary pre-requisite for preparing satisfactory urban drainage and stormwater management projects. The estimation involves frequency, duration and intensity analyses of rainfall data.

2.2.1 Average Recurrence Interval

Rainfall and subsequent discharge estimate is based on the selected value of frequency or return period, termed as the Average Recurrence Interval (ARI) which is used throughout this Manual. ARI is the average length of time between rain events that exceeds the same magnitude, volume or duration (Chow, 1964), and is expressed as:

$$T_r = \frac{1}{P} \cdot 100 \tag{2.1}$$

where,

T_r = Average Recurrence Interval, ARI (year); and
P = Annual Exceedance Probability, AEP (%).

As an example, using Equation 2.1, 1% AEP of storm has an ARI of 100 years. According to the definition, a 100 year ARI storm can occur in any year with a probability of 1/100 or 0.01.

The design ARI of a stormwater facility is selected on the basis of economy and level of protection (risk) that the facility offers. ARIs to be used for the design of minor and major stormwater quantity systems are provided in Table 1.1. It is assumed that the design flow of a given ARI is produced by a design storm rainfall of the same ARI. Design rainfall intensity (mm/hr) depends on duration (minute or hour) and ARI (month or year). It is strongly recommended that performance of the designed drainage system must be examined for a range of ARIs and storm durations to ensure that the system(s) will perform satisfactorily.

2.2.2 Time of Concentration

Time of concentration (t_c) is the travel time of runoff flows from the most hydraulically remote point upstream in the contributing catchment area to the point under consideration downstream. The concept of time of concentration is important in all methods of peak flow estimation as it can be assumed that the rainfall occurring during the time of concentration is directly related to peak flow rate. The practice is to select the design storm duration as equal to or greater than the time of concentration (t_c)

In the design of stormwater drainage systems, t_c is the sum of the overland flow time (t_o) and the time of travel in street gutters (t_g), or roadside swales, drains, channels and small streams (t_d). The relevant equations necessary to calculate the t_c is given in Table 2.1 (QUDM, 2007). Calculation of t_c is subject to the catchment properties, particularly length, slope and roughness of the drainage path. The overland flow time t_o can be estimated with proper judgment of the land surface condition due to the fact that the length of sheet flow is short for steep slopes and long for mild slopes. This equation shall be applied only for distances (L) recommended in Table 2.1. Catchment roughness, length and slope affect the flow velocity and subsequently overland flow time t_o. Typical values of Horton's roughness n^* for various land surfaces are given in Table 2.2 (QUDM, 2007). Alternatively, the overland flow time can easily be estimated using the Design Chart 2.A1.

The drain flow time equation should be used to estimate t_d for the remaining length of the flow paths downstream. Care should be given to obtain the values of hydraulic radius and friction slope for use in the drain flow time equation. Note that recommended minimum time of concentration for a catchment is 5 minutes which applies to roof drainage.

Table 2.1: Equations to Estimate Time of Concentration (QUDM, 2007)

Travel Path	Travel Time	Remark
Overland Flow	$t_o = \frac{107.n^*.L^{1/3}}{S^{1/5}}$	t_o = Overland sheet flow travel time (minutes) L = Overland sheet flow path length (m) *for Steep Slope (>10%), L ≤ 50 m* *for Moderate Slope (<5%), L ≤ 100 m* *for Mild Slope (<1%), L ≤ 200 m* n^* = Horton's roughness value for the surface (Table 2.2) S = Slope of overland surface (%)
Curb Gutter Flow	$t_g = \frac{L}{40\sqrt{S}}$	t_g = Curb gutter flow time (minutes) L = Length of curb gutter flow (m) S = Longitudinal slope of the curb gutter (%)
Drain Flow	$t_d = \frac{n.L}{60R^{2/3}S^{1/2}}$	n = Manning's roughness coefficient (Table 2.3) R = Hydraulic radius (m) S = Friction slope (m/m) L = Length of reach (m) t_d = Travel time in the drain (minutes)

Table 2.2: Values of Horton's Roughness n^* (QUDM, 2007)

Land Surface	Horton's Roughness n^*
Paved	0.015
Bare Soil	0.0275
Poorly Grassed	0.035
Average Grassed	0.045
Densely Grassed	0.060

2.2.3 Design Rainfall Estimate

2.2.3.1 Intensity-Duration-Frequency Curves Development

The most common form of design rainfall data required for use in peak discharge estimation is from relationship represented by the intensity-duration-frequency (IDF) curves. The IDF can be developed from the historical rainfall data and they are available for most geographical areas in Malaysia.

Recognising that the rainfall data used to derive IDF are subjected to some interpolation and smoothing, it is desirable to develop IDF curves directly from local raingauge records, if these records are sufficiently long and reliable. The IDF development procedures involve the steps shown in Figure 2.1 while a typical developed curves are shown in Figure 2.2.

Table 2.3: Values of Manning's Roughness Coefficient (n) for Open Drains and Pipes

(Chow, 1959; DID, 2000 and French, 1985)

Drain/Pipe	Manning Roughness *n*
Grassed Drain	
Short Grass Cover (< 150 mm)	0.035
Tall Grass Cover (≥ 150 mm)	0.050
Lined Drain	
Concrete	
Smooth Finish	0.015
Rough Finish	0.018
Stone Pitching	
Dressed Stone in Mortar	0.017
Random Stones in Mortar or Rubble Masonry	0.035
Rock Riprap	0.030
Brickwork	0.020
Pipe Material	
Vitrified Clay	0.012
Spun Precast Concrete	0.013
Fibre Reinforced Cement	0.013
UPVC	0.011

2.2.3.2 Empirical IDF Curves

Empirical equation can be used to minimise error in estimating the rainfall intensity values from the IDF curves. It is expressed as

$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} \tag{2.2}$$

where,

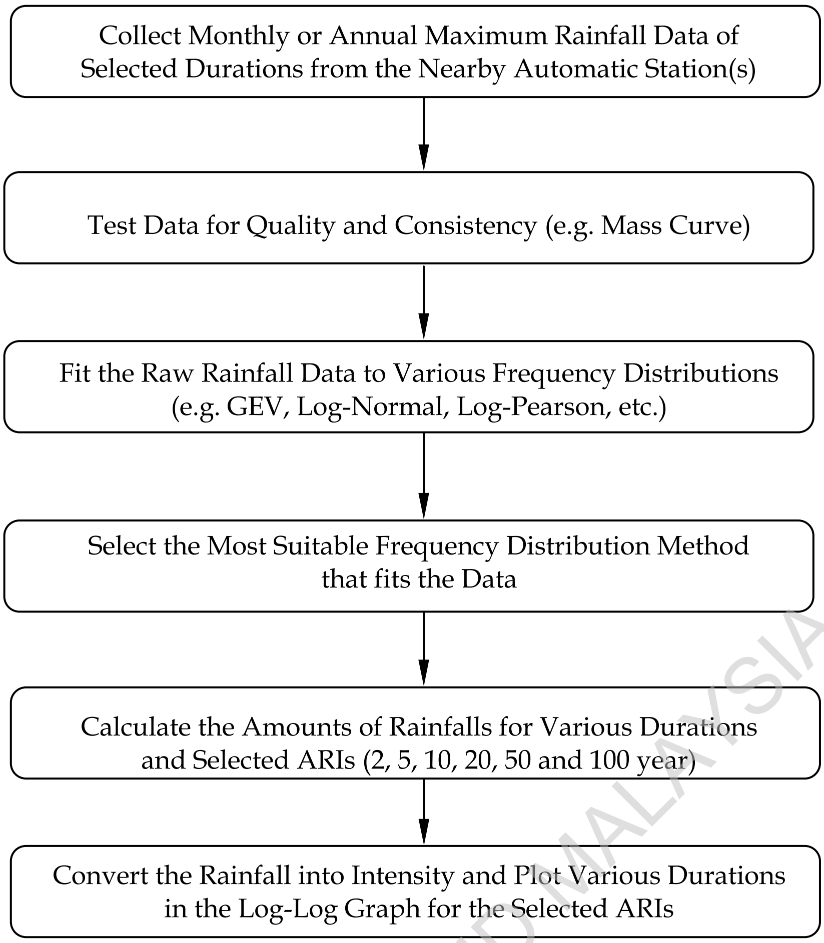
i = Average rainfall intensity (mm/hr);
T = Average recurrence interval - ARI ($0.5 \le T \le 12$ month and $2 \le T \le 100$ year);
d = Storm duration (hours), $0.0833 \le d \le 72$; and
λ, κ, θ and η = Fitting constants dependent on the raingauge location (Table 2.B1 in Appendix 2.B).

The equation application is simple when analysis is prepared by spreadsheet. Alternatively designers can manually use the IDF curves provided in Annexure 3.

2.2.4 Temporal Patterns

It is important to emphasise that the rainfall temporal patterns are intended for use in hydrograph generation *design* storms. They should not be confused with the real rainfall data in historical storms, which is usually required to calibrate and validate hydrological and hydraulic simulation results.

The standard time intervals recommended for urban stormwater modelling are listed in Table 2.4. The design temporal patterns to be used for a set of durations are given in Appendix 2.C.



Collect Monthly or Annual Maximum Rainfall Data of Selected Durations from the Nearby Automatic Station(s)

Test Data for Quality and Consistency (e.g. Mass Curve)

Fit the Raw Rainfall Data to Various Frequency Distributions (e.g. GEV, Log-Normal, Log-Pearson, etc.)

Select the Most Suitable Frequency Distribution Method that fits the Data

Calculate the Amounts of Rainfalls for Various Durations and Selected ARIs (2, 5, 10, 20, 50 and 100 year)

Convert the Rainfall into Intensity and Plot Various Durations in the Log-Log Graph for the Selected ARIs

Figure 2.1: Typical Steps to Develop IDF Curves

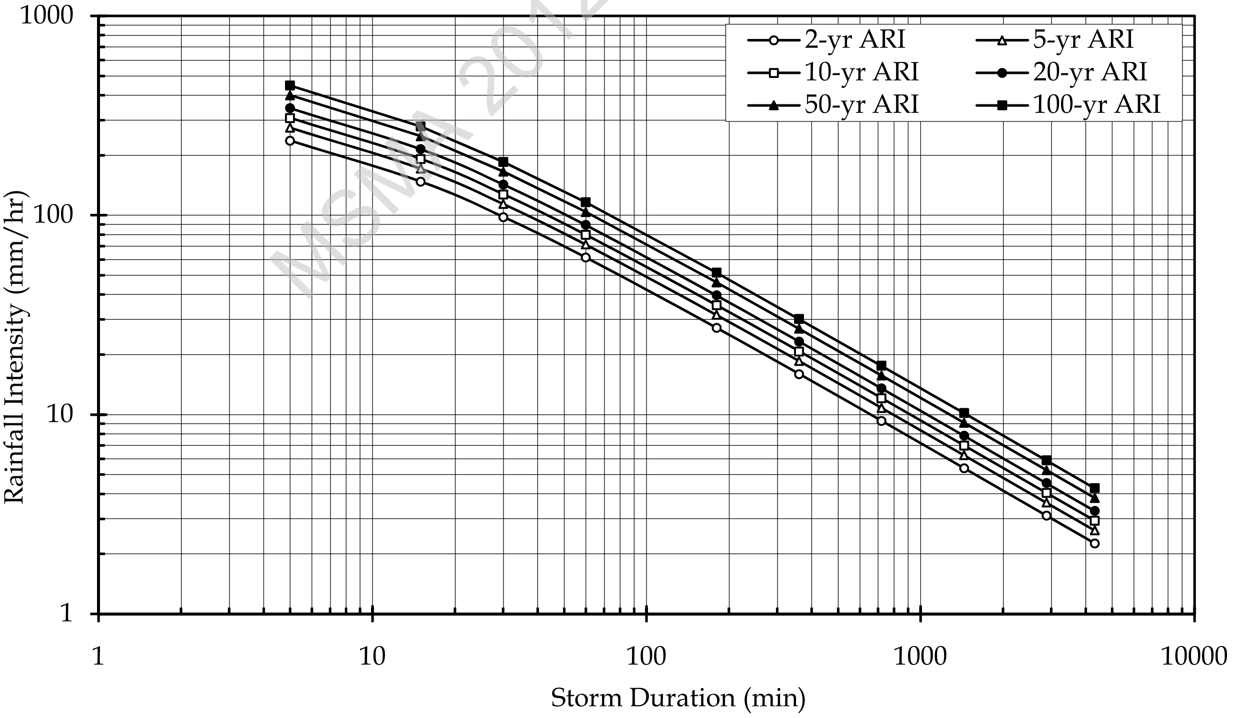


Figure 2.2: Typical IDF Curve

If data available, it is recommended to derive the temporal patterns using the local data following the example given in Appendix 2.D. For other durations, the temporal pattern for the nearest standard duration should be adopted. It is *NOT* correct to average the temporal patterns for different durations.

Table 2.4: Recommended Intervals for Design Rainfall Temporal Pattern

Storm Duration (minutes)	Time Interval (minutes)
Less than 60	5
60 - 120	10
121 - 360	15
Greater than 360	30

Various methods can be used to develop design rainfall temporal pattern. However, it is most important to note that design patterns are not derived from complete storms, but from intense bursts of recorded rainfall data for the selected durations. The method described herein incorporates the average variability of recorded intense rainfalls and also the most likely sequence of intensities. The highest rainfall bursts of selected design storm durations are collected from the rainfall record. It is desirable to have a large number of samples. The duration is then divided into a number of equal time intervals, as given in Table 2.4. The intervals for each rainfall burst are ranked and the average rank is determined for the intervals having same rainfall amount. The percentage of rainfall is determined for each rank for each rainfall burst, and the average percentage per rank is calculated. This procedure is then repeated for other durations. The procedure involves the steps as shown in Figure 2.3.

2.3 PEAK DISCHARGE ESTIMATION

This Section presents the methods and procedures required for runoff estimation. The recommended methods are the Rational Method and Hydrograph Methods. Each method has its own merits. *A simple Rational Hydrograph Method (RHM) is recommended for the design of small storage facilities.*

2.3.1 Rational Method

The Rational Method is the most frequently used technique for runoff peak estimation in Malaysia and many parts of the world. It gives satisfactory results for small drainage catchments and is expressed as:

$$Q = \frac{C.i.A}{360} \tag{2.3}$$

where,

Q = Peak flow (m^3/s);
C = Runoff coefficient (Table 2.5);
i = Average rainfall intensity (mm/hr); and
A = Drainage area (ha).

The primary attraction of the Rational Method has been its simplicity. However, now that computerised procedures for hydrograph generation are readily available, making computation/design by computerised method or software is also simple.

The most critical part of using the Rational Method is to make a good estimate of the runoff coefficient *C*. In general, the values of *C* depend mainly on landuse of the catchment and is very close to its imperviousness (in decimal form). The value of *C* also varies with soil type, soil moisture condition, rainfall intensity, etc. The user should evaluate the actual catchment condition for a logical value of *C* to be used. For larger area with high spatial variabilities in landuse and other parameters, this can easily be done by the use of AutoCAD, GIS or other computer softwares.

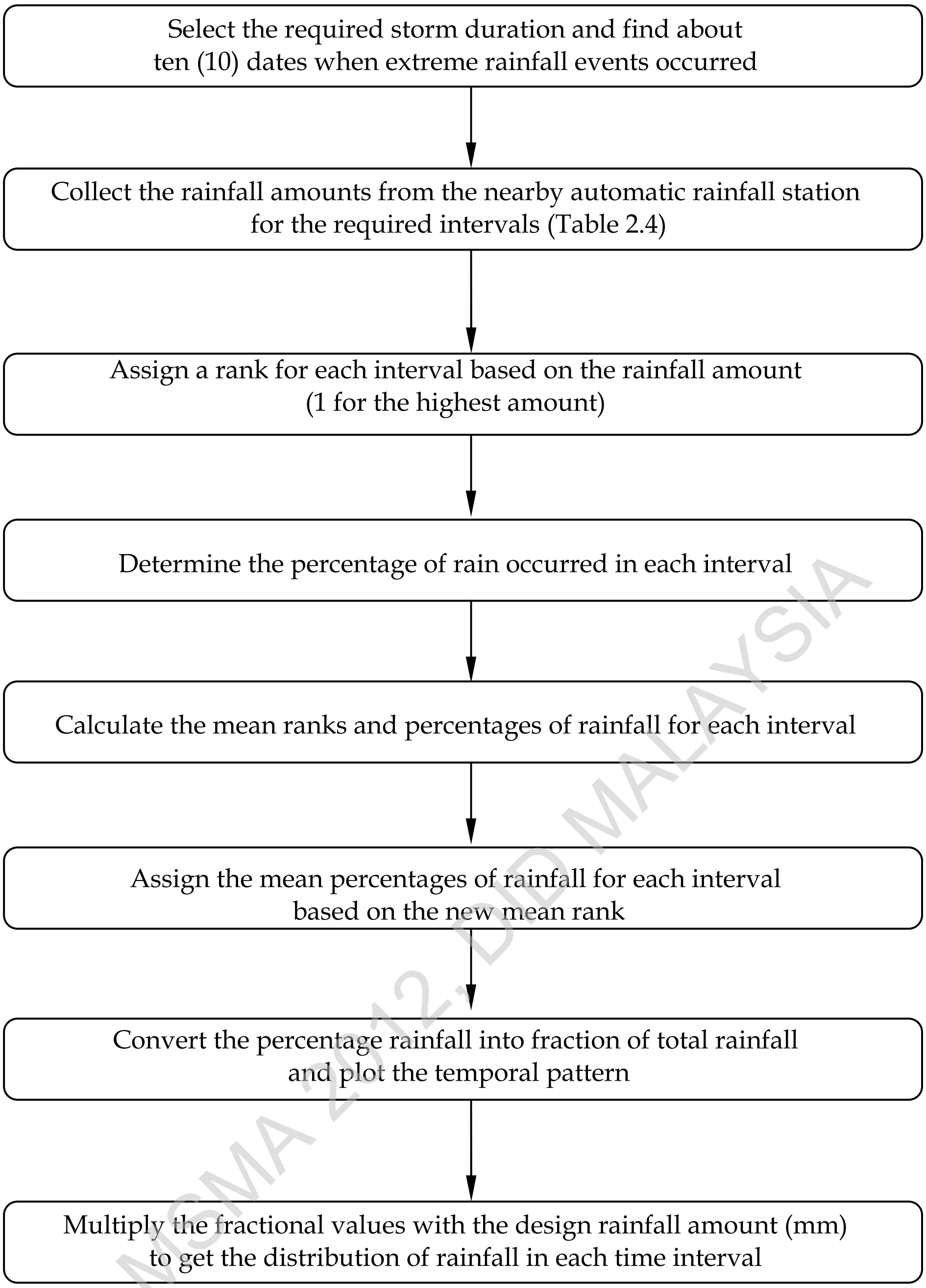


Figure 2.3: Typical Steps for the Development of Design Rainfall Temporal Pattern

2.3.1.1 Runoff Coefficient for Mixed Development

Segments of different landuse within a sub-catchment can be combined to produce an average runoff coefficient (Equation 2.4). For example, if a sub-catchment consists of segments with different landuse denoted by j = 1, 2,……, m; the average runoff coefficient is estimated, C, by:

$$C_{avg} = \frac{\sum_{j=1}^{m} C_j A_j}{\sum_{j=1}^{m} A_j} \tag{2.4}$$

where,

C_{avg} = Average runoff coefficient;
C_j = Runoff coefficient of segment *i;*
A_j = Area of segment *i* (ha); and
m = Total number of segments.

Table 2.5: Recommended Runoff Coefficients for Various Landuses
(DID, 1980; Chow et al., 1988; QUDM, 2007 and Darwin Harbour, 2009)

Landuse	Runoff Coefficient (C)	
	For Minor System (≤10 year ARI)	For Major System (> 10 year ARI)
Residential		
Bungalow	0.65	0.70
Semi-detached Bungalow	0.70	0.75
Link and Terrace House	0.80	0.90
Flat and Apartment	0.80	0.85
Condominium	0.75	0.80
Commercial and Business Centres	0.90	0.95
Industrial	0.90	0.95
Sport Fields, Park and Agriculture	0.30	0.40
Open Spaces		
Bare Soil (No Cover)	0.50	0.60
Grass Cover	0.40	0.50
Bush Cover	0.35	0.45
Forest Cover	0.30	0.40
Roads and Highways	0.95	0.95
Water Body (Pond)		
Detention Pond (with outlet)	0.95	0.95
Retention Pond (no outlet)	0.00	0.00

Note: The runoff coefficients in this table are given as a guide for designers. The near-field runoff coefficient for any single or mixed landuse should be determined based on the imperviousness of the area.

2.3.1.2 Assumptions

Assumptions used in the Rational Method are as follows:

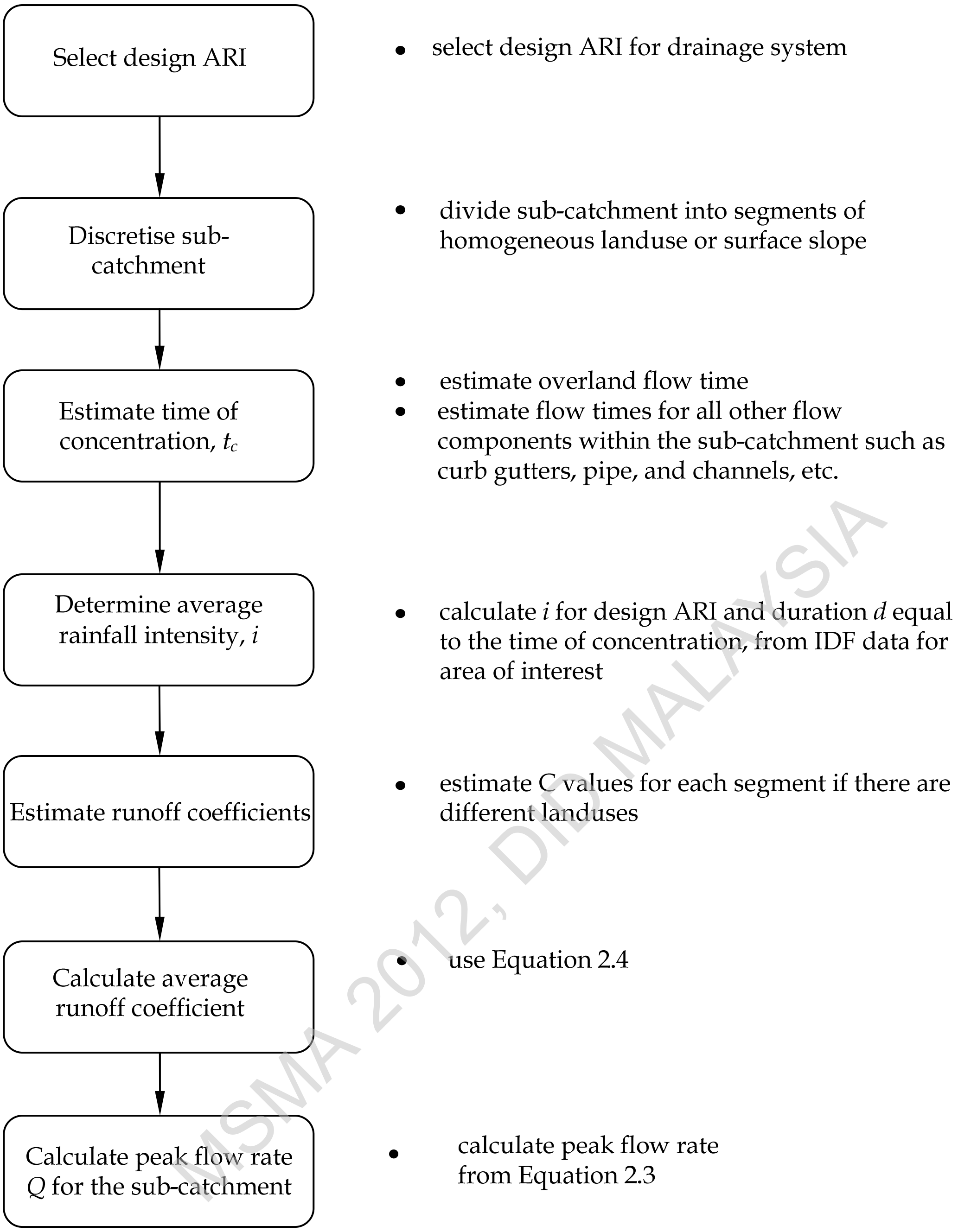
- The peak flow occurs when the entire catchment is contributing to the flow;
- The rainfall intensity is the uniform over the entire catchment area; and
- The rainfall intensity is uniform over a time duration equal to the time of concentration, t_c.

The Rational Method is *not recommended* for use where:

- The catchment area is greater than 80 ha (TxDOT, 2009);
- Ponding of stormwater in the catchment might affect peak discharge; and
- The design and operation of large and more costly drainage facilities are to be undertaken, particularly if they involve storage.

2.3.1.3 Calculation Steps

Steps for estimating a peak flow from a single sub-catchment for a particular ARI using the Rational Method are outlined in Figure 2.4.



- select design ARI for drainage system
- divide sub-catchment into segments of homogeneous landuse or surface slope
- estimate overland flow time
- estimate flow times for all other flow components within the sub-catchment such as curb gutters, pipe, and channels, etc.
- calculate i for design ARI and duration d equal to the time of concentration, from IDF data for area of interest
- estimate C values for each segment if there are different landuses
- use Equation 2.4
- calculate peak flow rate from Equation 2.3

Figure 2.4: General Procedure for Estimating Peak Flow Using the Rational Method (DID, 2000)

2.3.2 Rational Hydrograph Method

This procedure, Rational Hydrograph Method (RHM), extends the Rational Method to the development of runoff hydrographs. For simplicity, this method is recommended for the deriving inflow hydrograph on-site detention (OSD) and small detention pond. However, for complex drainage system and high risk areas, the Time Area Method in Section 2.2.3 or computer models should be used for obtaining the inflow hydrograph.

As illustrated in Figure 2.5, two types of hydrographs are to be used for the sub-catchment using the RHM procedure. Each hydrograph type is a function of the length of the rainfall averaging time, d, with respect to the sub-catchment time of concentration, t_c.

Type 1 (*d* is greater than t_c): The resulting trapezoidal hydrograph has a uniform maximum discharge *Q*, as determined from the Rational Method. The linear rising and falling limbs each has a duration of t_c.

Type 2 (*d* is equal to t_c,): The resulting triangular hydrograph has a peak discharge *Q*. The linear rising and falling limbs each have a duration of t_c.

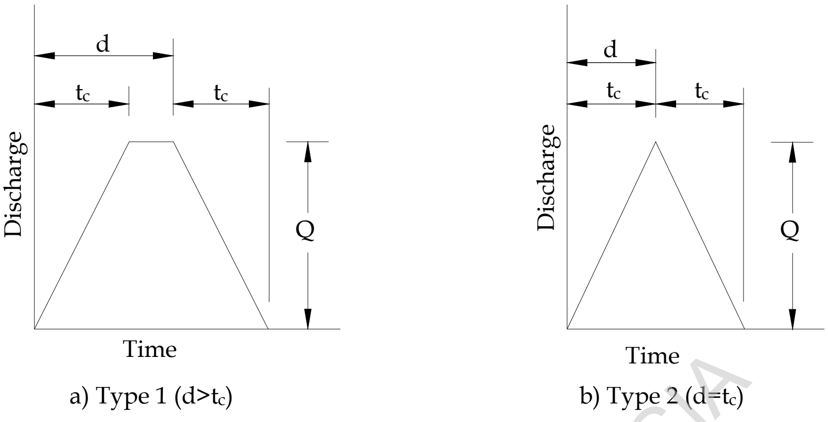


Figure 2.5: Hydrograph Types of the RHM

In summary, hydrograph type in the RHM is determined by the relationship between rainfall duration and the time of concentration of the sub-catchment. Given the hydrograph type, the peak discharge is determined using the Rational Method (Equation 2.3).

2.3.3 Time Area Hydrograph Method

2.3.3.1 Concept

This method assumes that the outflow hydrograph for any storm is characterised by separable subcatchment translation and storage effects. Pure translation of the direct runoff to the outlet via the drainage network is described using the channel travel time, resulting in an outflow hydrograph that ignores storage effects.

To apply the method, the catchment is first divided into a number of isochrones or lines of equal travel time to the outlet (Figure 2.6b). The areas between isochrones are then determined and plotted against the travel time as shown in Figure 2.6c. Derivation of isochrones is crucial and is illustrated in a worked example in Appendix 2.E2. The translated inflow hydrograph ordinates q_i (Figure 2.6d) for any selected design hyetograph can now be determined. Each block of storm, Figure 2.6a, should be applied (after deducting losses) to the entire catchment; the runoff from each sub-area reaches the outflow at lagged intervals defined by the time-area histogram. The simultaneous arrival of the runoff from areas A_1, A_2,…for storms I_1, I_2,…should be determined by properly lagging and adding contributions, or generally expressed as:

$$q_j = I_j . A_1 + I_{j-1} . A_2 + ... + I_1 . A_j \tag{2.5}$$

where,

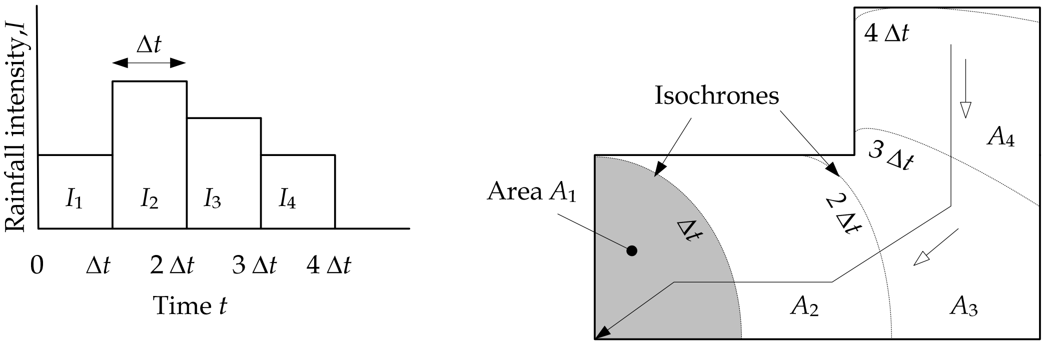
q_j = Flow hydrograph ordinates (m³/s);

I_j = Rainfall excess hyetograph ordinates (m/s);

A_j = Time-area histogram ordinates (m²); and

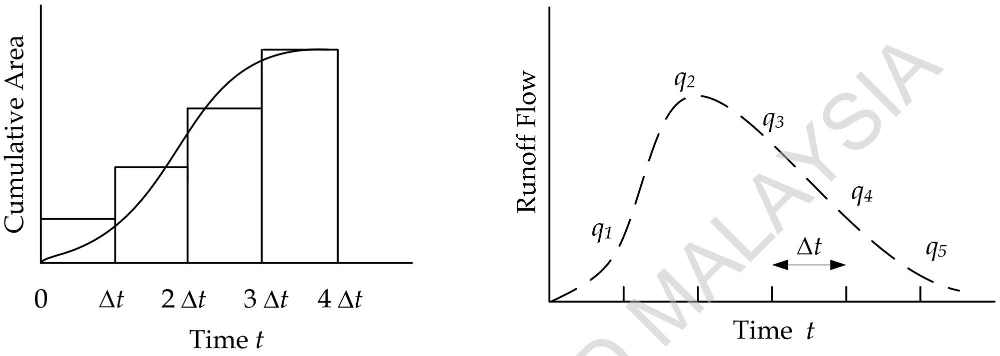
j = Number of isochrone contributing to the outlet.

As an example for j = 3, the runoff from storms I_1 on A_3, I_2 on A_2 and I_3 on A_1 arrive at the outlet simultaneously, and q_3 is the total flow. The inflow hydrograph (Figure 2.6d) at the outlet can be obtained using Equation 2.5.



(a) Rainfall Histogram

(b) Catchment Isochrones



(c) Time-Area Curve

(d) Runoff Hydrograph

Figure 2.6: Time–Area Hydrograph Method

2.3.3.2 Rainfall Excess

Total Rainfall should be deducted by losses, initial or continuous, to calculate the rainfall excess (RE), which will result in the surface runoff hydrograph. The rainfall losses can be assumed constant (for simplicity) or decaying (to be more practical), as shown in Figure 2.7. The parameter values are given in Table 2.6.

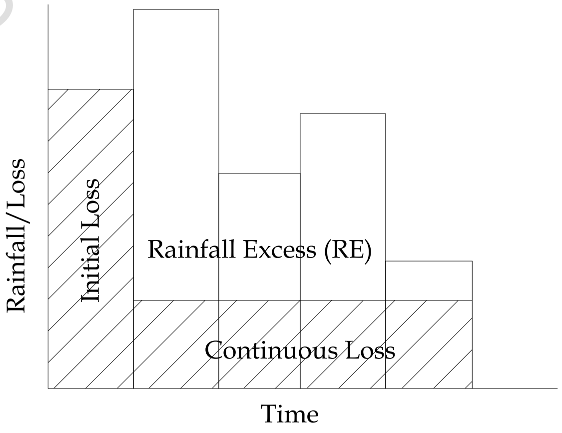


Figure 2.7: Initial and Continuous Loss Concept for Runoff Estimation

Table 2.6: Recommended Loss Values for Rainfall Excess Estimation (Chow et al., 1988)

Catchment Condition	Initial Loss (mm)	Continuous Loss (mm/hr)
Impervious	1.5	0
Pervious	10	(i) Sandy Soil: 10 - 25 mm/hr (ii) Loam Soil: 3 - 10 mm/hr (iii) Clay Soil: 0.5 - 3 mm/hr

2.3.4 Computer Software Application

Various types of simple and complex computer software (models) are available to simulate the runoff peak flow or hydrograph. Prudent use of such softwares can provide more flexibility and opportunity to estimate the runoff hydrograph and volume taking consideration of the variability in rainfall and catchment properties. Wherever and whenever possible, designers should use computer softwares to design and analyse stormwater management component or the whole system train, for more scenarios and reliability at reasonable cost.

Three types of computer methods might be considered, they are:

- Spreadsheets that can be used to implement all of the methods described in this chapter;
- Public domain softwares, such as SWMM-5, RORB and HEC-RAS; and
- Commercial softwares.

All runoff estimation methods will give different peak flow rates. The most practical way to minimise the variations is by calibrating and validating against the recorded rainfall and runoff data.

2.4 OUTFLOW CONTROL

Orifices and weirs outlet are typically used as outlet control structures for ponds and their characteristics must be specified when performing reservoir routing calculations. The relevant equations are given in the following sections.

2.4.1 Orifices

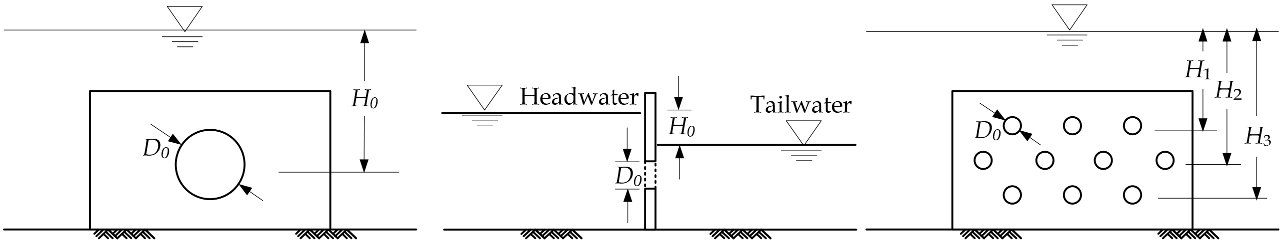
For a single orifice as illustrated in Figure 2.8, orifice flow can be determined using Equation 2.6.

$$Q = C_o A_o (2gH_o)^{0.5} \qquad (2.6)$$

where ,

Q = Orifice flow rate (m^3/s);
C_o = Discharge coefficient (0.60);
A_o = X-sectional area of orifice (m^2);
H_o = Effective head of the orifice measured from the centroid of the opening (m); and
g = Gravitational acceleration (9.81m/s^2).

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 2.8(b).



(a) Free Fall (b) Single (Submerged) (c) Multiple

Figure 2.8: Definition Sketch for Orifice Flow (FHWA, 1996)

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used. For circular orifices with C_o set equal to 0.6, the following equation results:

$$Q = K_{or}\, D^2\, H_o^{0.50} \qquad (2.7)$$

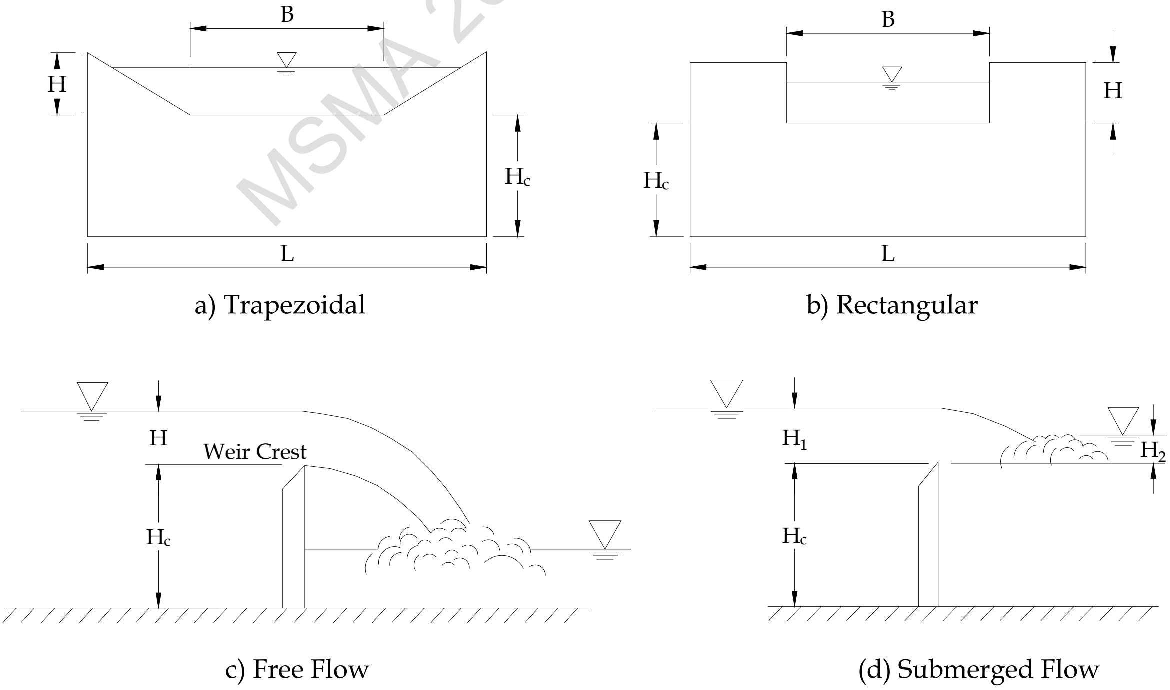
where,

K_{or} = 2.09 in S.I. units;
D = Orifice diameter (m); and
H_o = Height – $D/2$ for free fall and difference in head and tailwater for submerged orifice.

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

2.4.2 Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 2.9. Equation 2.8 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure 2.9a).



a) Trapezoidal b) Rectangular

c) Free Flow (d) Submerged Flow

Figure 2.9: Sharp Crested Weirs (FHWA, 1996)

$$Q = C_{SCW} . B . H^{1.5} \tag{2.8}$$

where,

Q = Discharge (m^3/s);
B = Horizontal weir width (m);
H = Head above weir crest excluding velocity head (m); and
C_{SCW} = Weir discharge coefficient = 1.81 + 0.22 (H/H_c).

As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c. Equation 2.9 provides the discharge equation for sharp-crested weirs with end contractions. For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 can be used.

$$Q = C_{SCW} (B - 0.2 H) H^{1.5} \tag{2.9}$$

2.4.3 Broad Crested Weirs

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated in Figure 2.10.

Equation 2.10 presents a relationship for computing the flow through a broad-crested emergency spillway. The dimensional terms used in the equation are illustrated in Figure 2.10.

$$Q = C_{SP} \, B H_p^{1.5} \tag{2.10}$$

where,

Q = Emergency spillway discharge (m^3/s);
C_{SP} = Spillway discharge coefficient ($m^{0.5}/s$);
B = Spillway base width (m); and
H_p = Effective head on the spillway weir crest (m).

The discharge coefficient C_{SP} in Equation 2.10 varies as a function of spillway base width and effective head (Table 2.7). Equations 2.11 and 2.12 can be used to compute the critical velocity V_c and critical slope S_c at the control section of an emergency spillway:

$$V_c = 2.14 \left(\frac{Q}{B} \right)^{0.33} \tag{2.11}$$

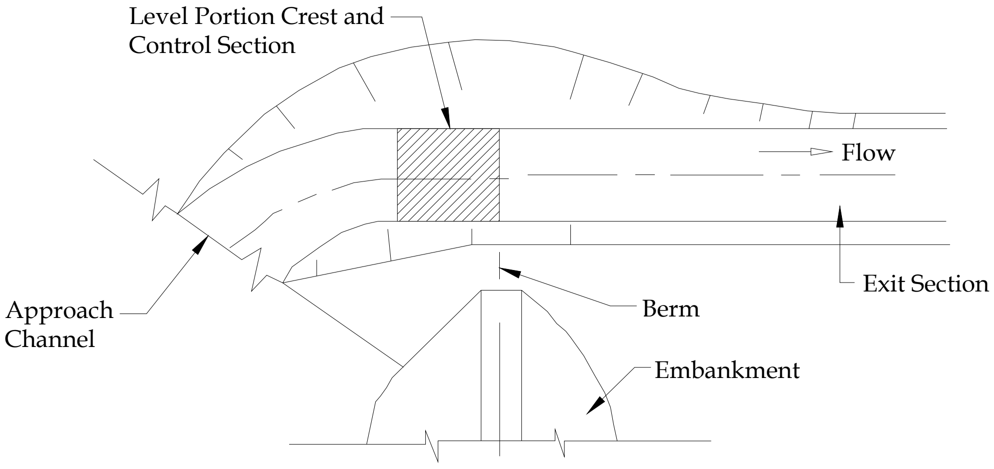
$$S_c = 9.84 \, n^2 \left(\frac{V_c \, B}{Q} \right)^{0.33} \tag{2.12}$$

where,

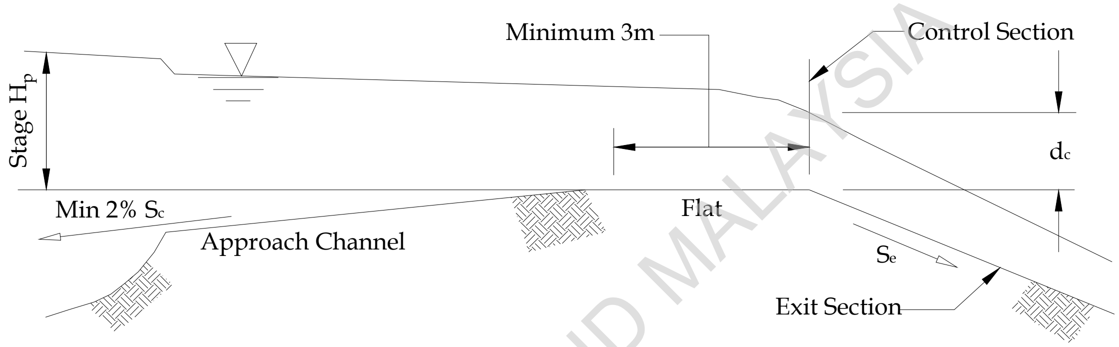
n = Manning's roughness coefficient;
V_c = Critical velocity (m/s); and
S_c = Critical slope (%).

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

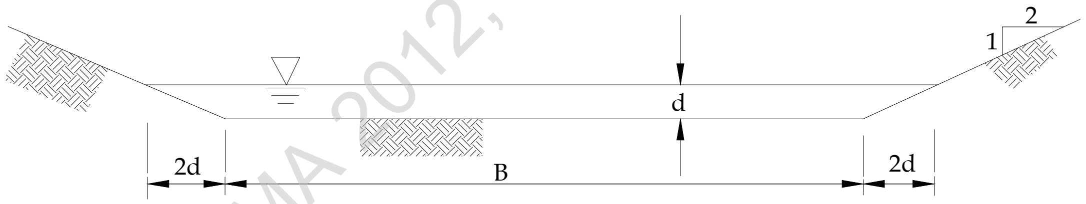
$$V_e = V_c \left(\frac{S_e}{S_c} \right)^{0.3} \tag{2.13}$$



a) Plan View of Excavated Emergency Spillway



b) Profile Along Centerline



c) Cross-Section at Control Section

Figure 2.10: Spillway Design Schematic (FHWA, 1996)

2.4.4 Drawdown Time

It is sometimes necessary to estimate the time it would take to drain a known stored water volume of a pond through an orifice system. The following equation may be used to check that the storage does not take too long time to empty the pond or to return to the normal water (pool) level, after the storm ends:

$$t = -\frac{1}{C_d \, A_o \sqrt{2g}} \int_{H_1}^{H_2} \left(\frac{A_s}{\sqrt{y}} \right) dy \qquad (2.14)$$

where,

t = Time to empty (seconds);

y = Depth of water above the centreline in the storage (m);

A_s = Storage water surface area at depth y (m^2); and

$H_{1,2}$ = Effective heads on the orifice measured from the centroid of the opening (m).

Where the water surface area is constant (i.e. vertical walls in the pond), Equation 2.14 reduces to:

$$t = \frac{2\,A_s}{C_d\,A_o\sqrt{2g}}\left(\sqrt{y_1}-\sqrt{y_2}\right) \tag{2.15}$$

Table 2.7: Broad-Crested Weir Coefficient C_{sp} Values as a Function of Weir Base Width and Head (FHWA, 1996)

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

(1) Measured at least 2.5H upstream of the weir

2.5 HYDROLOGIC POND ROUTING

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or modified Puls method. This method begins with the continuity equation which states that the inflow minus the outflow equals the change in storage (I-0=ΔS). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 2.16. This relationship is illustrated graphically in Figure 2.11.

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \tag{2.16}$$

where:

ΔS = Change in storage (m^3);

Δt = Time interval (min);

I = Inflow (m^3); and

O = Outflow (m^3).

In Equation 2.16, subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

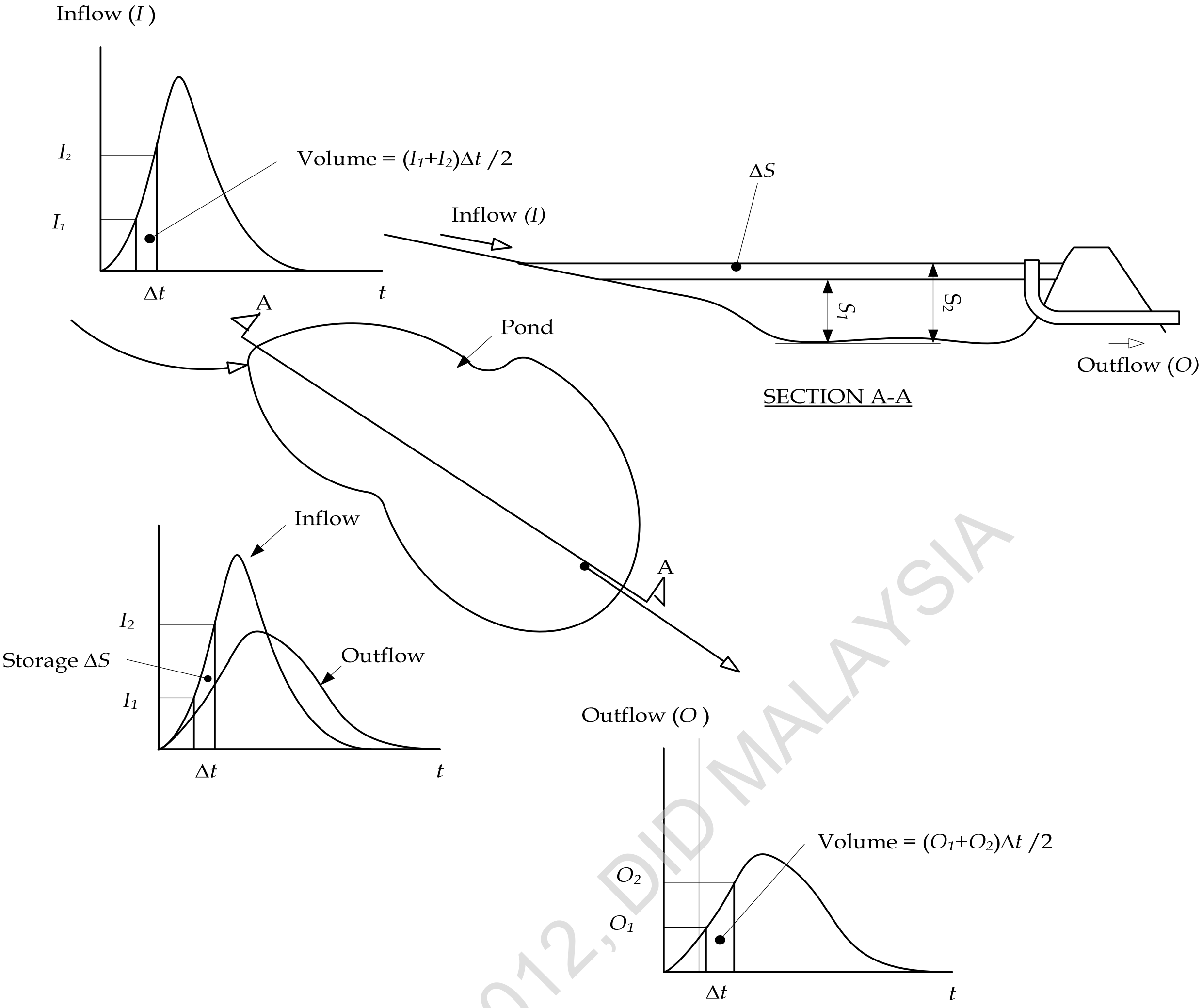


Figure 2.11: Development of the Storage-Discharge Function for Hydrologic Pond Routing

Equation 2.16 can be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right hand side of the equation, as shown in Equation 2.17. Now the equation with two unknowns, S_2 and O_2, can be solved with one equation. The following procedure can be used to perform routing through a reservoir or storage facility using Equation 2.17.

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \tag{2.17}$$

Step 1: Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

Step 2: Select a routing time period, Δt, to provide a minimum of five points on the rising limb of the inflow hydrograph.

Step 3: Use the stage-storage and stage-discharge data from Step 1 to develop a storage indicator numbers table that provides storage indicator values, S/(Δt) + O/2, versus stage. A typical storage indicator numbers table contains the following column headings:

1 Stage (m)	2 Discharge (O_2) (m^3/s)	3 Storage (S_2) (m^3)	4 $O_2/2$ (m^3/s)	5 $S_2/\Delta t$ (m^3/s)	6 $S_2/\Delta t + O_2/2$ (Storage Indicator Number)

- Discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.
- Subscript 2 is arbitrarily assigned at this time.
- Time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Step 4: Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column 6. An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed (Figure 2.12).

Step 5: A supplementary curve of storage (column 3) vs. $S_2/\Delta t + O_2/2$ (column 6) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of storage vs. time can be developed from this curve.

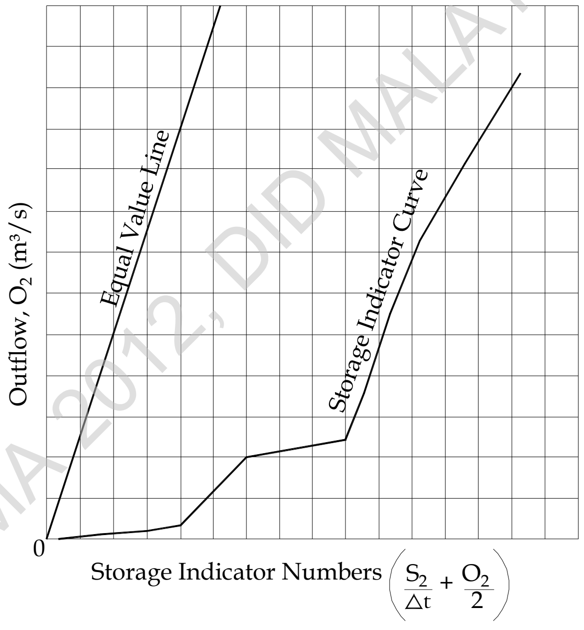


Figure 2.12: Storage Indicator Curve

Step 6: Routing can now be performed by developing a routing table for the solution of Equation 2.17 as follows:

1 Time (hr)	2 Inflow (m^3/s)	3 $(I_1+I_2)/2$ (m^3/s)	4 $(S_1/\Delta t+O_1/2)$ (m^3/s)	5 O_1 (m^3/s)	6 $S_2/\Delta t + O_2/2)$ (m^3/s)	7 O_2 (m^3/s)

- Columns (1) and (2) are obtained from the inflow hydrograph.

- Column (3) is the average inflow over the time interval.

- The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.

- The left side of Equation 2.17 is determined algebraically as columns (3) + (4) -(5). This value equals the right side of Equation 2.17 or $S_2/\Delta t + O_2/2$ and is placed in column (6).

- Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (column 7).

- Column (6) ($S_2/\Delta t + O_2/2$) and column (7) (O_2) are transported to the next line andbecome $S_1/\Delta t + O_1/2$ and O1 in columns (4) and (5), respectively. Because ($S_2/\Delta t$+ $O_2/2$) and O2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.

- Columns (3), (4), and (5) are again combined and the process is continued until the storm is routeh. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.

- Designer needs to make sure that the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

Step 7: Plot O_2 (column 7) versus time (column 1) to obtain the outflow hydrograph.

The above procedure is illustrated in Figure 2.13.

2.6 CRITICAL STORM DURATION

Determination of critical storm duration is important to make the stormwater management facilities safe. Critical storm duration is a function of rainfall intensity, antecedent moisture condition, rainfall temporal pattern, etc. Therefore, it is strongly recommended that the engineer or authority should look into various scenarios that can produce critical storm duration.

Determination of critical storm duration, the one that produces the highest runoff flow rate in the conveyance (pipe or open drain) system, or the highest water level in the storage facility, is required for the design of drainage systems.

2.6.1 Conveyance System

The critical storm duration of a conveyance system is usually close the value of time of concentration (t_c). However, depending on the antecedent moisture condition, variation in the temporal pattern, storm and wind direction, land development distribution of impervious surfaces in the subcatchment, etc. the critical storm duration might be significantly different from that of the t_c. Therefore, rainfall events of various durations and possible runoff contributing areas need to be analysed to determine the critical storm duration for the conveyance system.

Two options can be used to determine the critical storm duration for conveyance. Those are:

- Simple Calculation for catchment < 80 ha: Critical Storm Duration = t_c with possible checks for partial area effects; and

- Computer Model for catchment ≥ 80 ha: Run model for various storm durations and plotting the calculated peak flow rates for various durations to find the critical storm duration, as shown in Figure 2.14.

In order to develop the critical storm duration for a conveyance system, the designer has to select the design ARI and simulate hydrologic and hydraulic calculations for various storm durations together with the rainfall temporal patterns, antecedent moisture condition, etc. to get the peak flow values. The designer must then plot the design peak flow values against the storm durations, as shown in Figure 2.14 to find the critical storm duration for the drain or drainage system

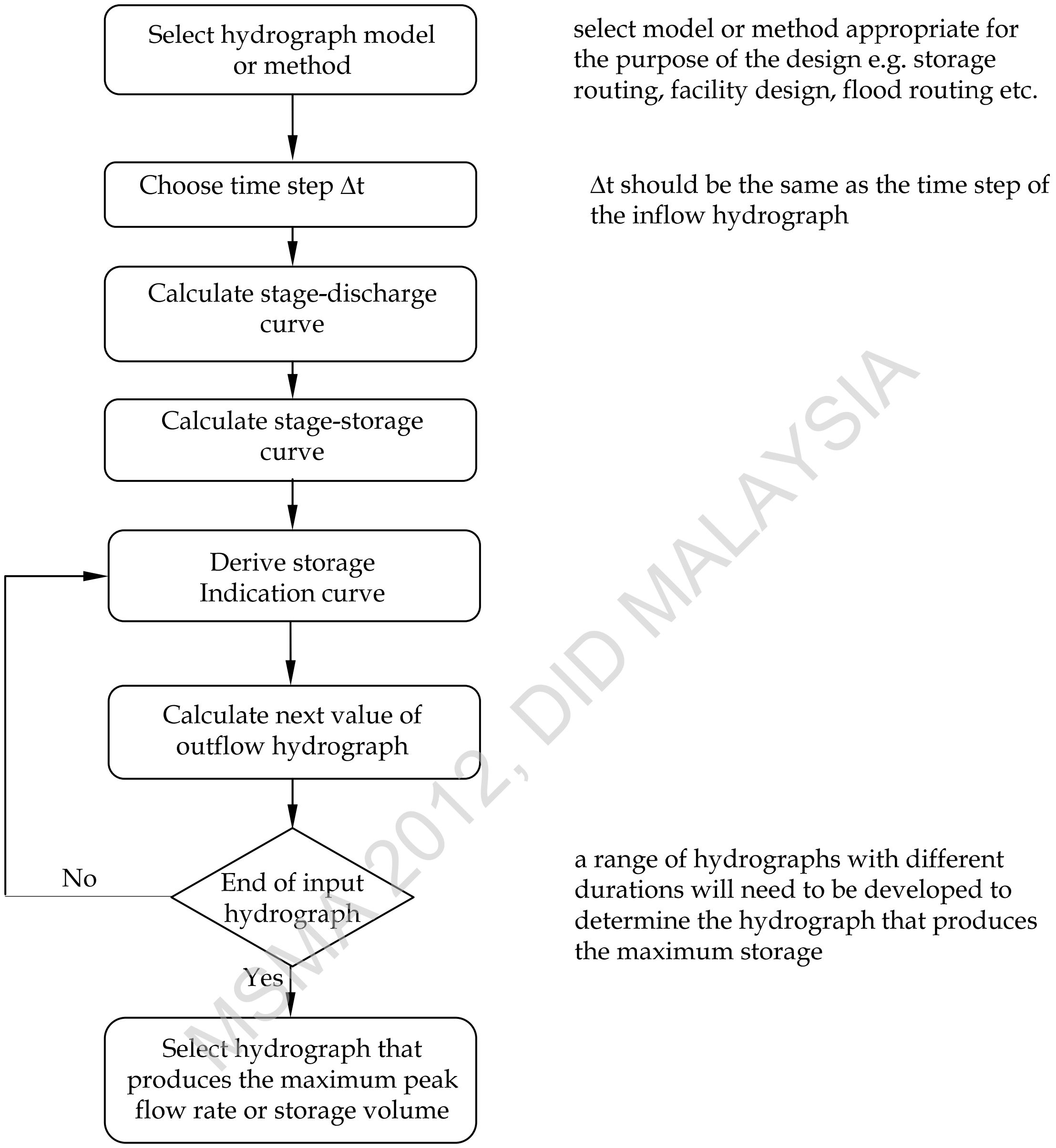


Figure 2.13: General Analysis Procedure for Pond Routing (DID, 2000)

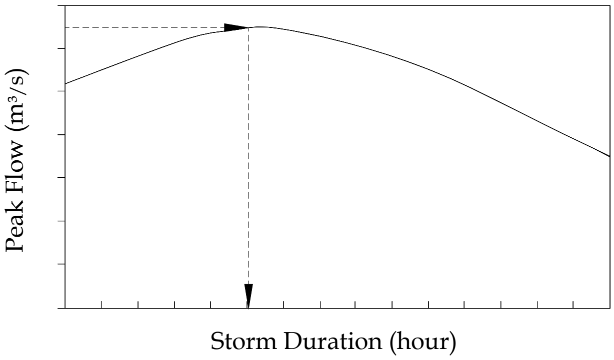


Figure 2.14: Determination of Critical Storm Duration for Conveyance

2.6.2 Storage System

On the other hand, the critical storm duration of any storage facility (OSD, Detention Pond, Wetland, etc.) mainly depends on the event runoff volume, inflow-outflow relationship, initial water level in the system, etc. In short, runoff volume is more critical, instead of just the intensity of the rainfall. Hydrologic and hydraulic routing of various storm durations for various rainfall temporal patterns, antecedent moisture conditions, etc. Is required to define the maximum water level in the storage facility. The designer must then plot the simulated highest water level in the pond, wetland or detention facility against the storm durations, as shown in Figure 2.15, to find the critical storm duration for the storage facilities.

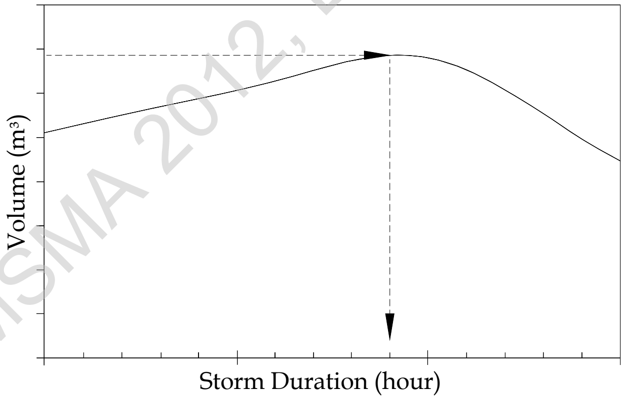


Figure 2.15: Determination of Critical Storm Duration for a Storage Facility

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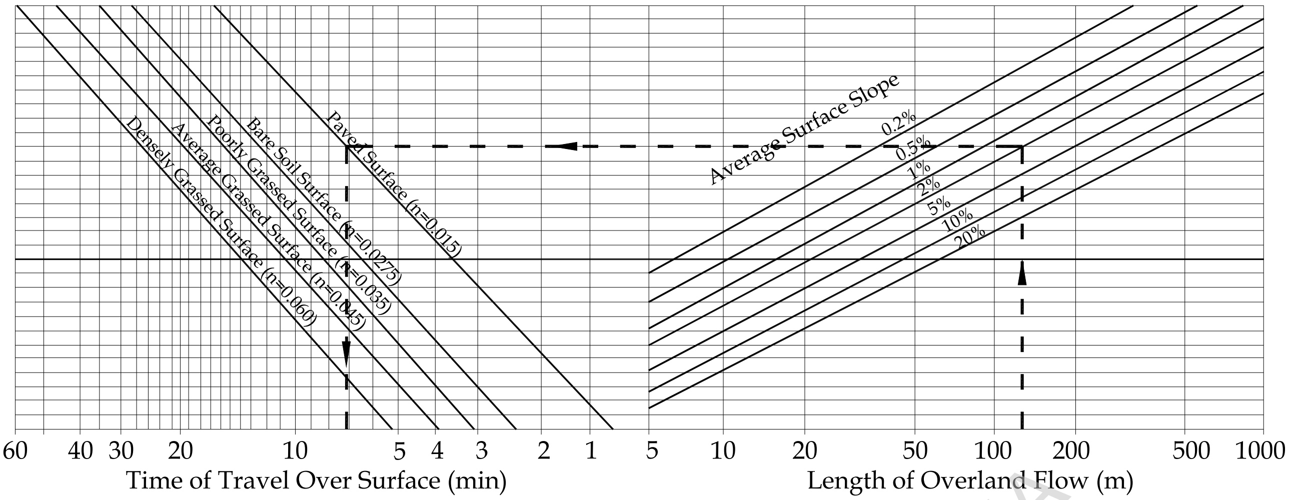
APPENDIX 2.A DESIGN CHART – OVERLAND FLOW TIME



Design Chart 2.A1: Nomograph for the Estimation of Overland Flow Time (t_o) for Sheet Flow (IEA, 1977)

APPENDIX 2.B IDF CONSTANTS

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Johor	1	1437116	Stor JPS Johor Bahru	59.972	0.163	0.121	0.793
	2	1534002	Pusat Kem. Pekan Nenas	54.265	0.179	0.100	0.756
	3	1541139	Johor Silica	59.060	0.202	0.128	0.660
	4	1636001	Balai Polis Kg Seelong	50.115	0.191	0.099	0.763
	5	1737001	SM Bukit Besar	50.554	0.193	0.117	0.722
	6	1829002	Setor JPS Batu Pahat	64.099	0.174	0.201	0.826
	7	1834124	Ladang Ulu Remis	55.864	0.166	0.174	0.810
	8	1839196	Simpang Masai K. Sedili	61.562	0.191	0.103	0.701
	9	1931003	Emp. Semberong	60.568	0.163	0.159	0.821
	10	2025001	Pintu Kaw. Tg. Agas	80.936	0.187	0.258	0.890
	11	2033001	JPS Kluang	54.428	0.192	0.108	0.740
	12	2231001	Ladang Chan Wing	57.188	0.186	0.093	0.777
	13	2232001	Ladang Kekayaan	53.457	0.180	0.094	0.735
	14	2235163	Ibu Bekalan Kahang	52.177	0.186	0.055	0.652
	15	2237164	Jalan Kluang-Mersing	56.966	0.190	0.144	0.637
	16	2330009	Ladang Labis	45.808	0.222	0.012	0.713
	17	2528012	Rmh. Tapis Segamat	45.212	0.224	0.039	0.711
	18	2534160	Kg Peta Hulu Sg Endau	59.500	0.185	0.129	0.623
	19	2636170	Setor JPS Endau	62.040	0.215	0.103	0.592
Kedah	1	5507076	Bt. 27, Jalan Baling	52.398	0.172	0.104	0.788
	2	5704055	Kedah Peak	81.579	0.200	0.437	0.719
	3	5806066	Klinik Jeniang	59.786	0.165	0.203	0.791
	4	5808001	Bt. 61, Jalang Baling	47.496	0.183	0.079	0.752
	5	6103047	Setor JPS Alor Setar	64.832	0.168	0.346	0.800
	6	6108001	Kompleks Rumah Muda	52.341	0.173	0.120	0.792
	7	6206035	Kuala Nerang	54.849	0.174	0.250	0.810
	8	6107032	AmpangPadu	66.103	0.177	0.284	0.842
	9	6306031	Padang Senai	60.331	0.193	0.249	0.829

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Kelantan	1	4614001	Brook	49.623	0.159	0.242	0.795
	2	4726001	Gunung Gagau	43.024	0.220	0.004	0.527
	3	4819027	Gua Musang	57.132	0.155	0.119	0.795
	4	4915001	Chabai	47.932	0.169	0.108	0.794
	5	4923001	Kg Aring	47.620	0.187	0.020	0.637
	6	5120025	Balai Polis Bertam	61.338	0.168	0.193	0.811
	7	5216001	Gob	41.783	0.175	0.122	0.720
	8	5320038	Dabong	51.442	0.189	0.077	0.710
	9	5322044	Kg Lalok	53.766	0.197	0.121	0.705
	10	5522047	JPS Kuala Krai	39.669	0.231	0.000	0.563
	11	5718033	Kg Jeli, Tanah Merah	72.173	0.196	0.360	0.703
	12	5719001	Kg Durian Daun Lawang	51.161	0.193	0.063	0.745
	13	5722057	JPS Machang	48.433	0.219	0.000	0.601
	14	5824079	Sg Rasau Pasir Putih	51.919	0.216	0.062	0.560
	15	6019004	Rumah Kastam Rantau Pjg	49.315	0.228	0.000	0.609
	16	6122064	Setor JPS Kota Bharu	60.988	0.214	0.148	0.616
Kuala Lumpur	1	3015001	Puchong Drop, K Lumpur	69.650	0.151	0.223	0.880
	2	3116003	Ibu Pejabat JPS	61.976	0.145	0.122	0.818
	3	3116004	Ibu Pejabat JPS1	64.689	0.149	0.174	0.837
	4	3116005	SK Taman Maluri	62.765	0.132	0.147	0.820
	5	3116006	Ladang Edinburgh	63.483	0.146	0.210	0.830
	6	3216001	Kg. Sungai Tua	64.203	0.152	0.250	0.844
	7	3216004	SK Jenis Keb. Kepong	73.602	0.164	0.330	0.874
	8	3217001	Ibu Bek. KM16, Gombak	66.328	0.144	0.230	0.859
	9	3217002	Emp. Genting Kelang	70.200	0.165	0.290	0.854
	10	3217003	Ibu Bek. KM11, Gombak	62.609	0.152	0.221	0.804
	11	3217004	Kg. Kuala Seleh, H. Klg	61.516	0.139	0.183	0.837
	12	3217005	Kg. Kerdas, Gombak	63.241	0.162	0.137	0.856
	13	3317001	Air Terjun Sg. Batu	72.992	0.162	0.171	0.871
	14	3317004	Genting Sempah	61.335	0.157	0.292	0.868

(Continued)

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Malacca	1	2222001	Bukit Sebukor	95.823	0.169	0.660	0.947
	2	2224038	Chin Chin Tepi Jalan	54.241	0.161	0.114	0.846
	3	2321006	Ladang Lendu	72.163	0.184	0.376	0.900
Negeri Sembilan	1	2719001	Setor JPS Sikamat	52.823	0.167	0.159	0.811
	2	2722202	Kg Sawah Lebar K Pilah	44.811	0.181	0.137	0.811
	3	2723002	Sungai Kepis	54.400	0.176	0.134	0.842
	4	2725083	Ladang New Rompin	57.616	0.191	0.224	0.817
	5	2920012	Petaling K Kelawang	50.749	0.173	0.235	0.854
Pahang	1	2630001	Sungai Pukim	46.577	0.232	0.169	0.687
	2	2634193	Sungai Anak Endau	66.179	0.182	0.081	0.589
	3	2828173	Kg Gambir	47.701	0.182	0.096	0.715
	4	3026156	Pos Iskandar	47.452	0.184	0.071	0.780
	5	3121143	Simpang Pelangai	57.109	0.165	0.190	0.867
	6	3134165	Dispensari Nenasi	61.697	0.152	0.120	0.593
	7	3231163	Kg Unchang	55.568	0.179	0.096	0.649
	8	3424081	JPS Temerloh	73.141	0.173	0.577	0.896
	9	3533102	Rumah Pam Pahang Tua	58.483	0.212	0.197	0.586
	10	3628001	Pintu Kaw. Pulau Kertam	50.024	0.211	0.089	0.716
	11	3818054	Setor JPS Raub	53.115	0.168	0.191	0.833
	12	3924072	Rmh Pam Paya Kangsar	62.301	0.167	0.363	0.868
	13	3930012	Sungai Lembing PCC Mill	45.999	0.210	0.074	0.590
	14	4023001	Kg Sungai Yap	65.914	0.195	0.252	0.817
	15	4127001	Hulu Tekai Kwsn."B"	59.861	0.226	0.213	0.762
	16	4219001	Bukit Bentong	73.676	0.165	0.384	0.879
	17	4223115	Kg Merting	52.731	0.184	0.096	0.805
	18	4513033	Gunung Brinchang	42.004	0.164	0.046	0.802
Penang	1	5204048	Sg Simpang Ampat	62.089	0.220	0.402	0.785
	2	5302001	Tangki Air Besar Sg Pinang	67.949	0.181	0.299	0.736
	3	5302003	Kolam Tkgn Air Hitam	52.459	0.191	0.106	0.729
	4	5303001	Rmh Kebajikan P Pinang	57.326	0.203	0.325	0.791
	5	5303053	Komplek Prai	52.771	0.203	0.095	0.717
	6	5402001	Klinik Bkt Bendera P Pinang	64.504	0.196	0.149	0.723
	7	5402002	Kolam Bersih P Pinang	53.785	0.181	0.125	0.706
	8	5404043	Ibu Bekalan Sg Kulim	57.832	0.188	0.245	0.751
	9	5504035	Lahar Ikan Mati Kepala Batas	48.415	0.221	0.068	0.692

(Continued)

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Perak	1	4010001	JPS Teluk Intan	54.017	0.198	0.084	0.790
	2	4207048	JPS Setiawan	56.121	0.174	0.211	0.854
	3	4311001	Pejabat Daerah Kampar	69.926	0.148	0.149	0.813
	4	4409091	Rumah Pam Kubang Haji	52.343	0.164	0.177	0.840
	5	4511111	Politeknik Ungku Umar	70.238	0.164	0.288	0.872
	6	4807016	Bukit Larut Taiping	87.236	0.165	0.258	0.842
	7	4811075	Rancangan Belia Perlop	58.234	0.198	0.247	0.856
	8	5005003	Jln. Mtg. Buloh Bgn Serai	52.752	0.163	0.179	0.795
	9	5207001	Kolam Air JKR Selama	59.567	0.176	0.062	0.807
	10	5210069	Stesen Pem. Hutan Lawin	52.803	0.169	0.219	0.838
	11	5411066	Kuala Kenderong	85.943	0.223	0.248	0.909
	12	5710061	Dispensari Keroh	53.116	0.168	0.112	0.820
Perlis	1	6401002	Padang Katong, Kangar	57.645	0.179	0.254	0.826
Selangor	1	2815001	JPS Sungai Manggis	56.052	0.152	0.194	0.857
	2	2913001	Pusat Kwln. JPS T Gong	63.493	0.170	0.254	0.872
	3	2917001	Setor JPS Kajang	59.153	0.161	0.118	0.812
	4	3117070	JPS Ampang	65.809	0.148	0.156	0.837
	5	3118102	SK Sungai Lui	63.155	0.177	0.122	0.842
	6	3314001	Rumah Pam JPS P Setia	62.273	0.175	0.205	0.841
	7	3411017	Setor JPS Tj. Karang	68.290	0.175	0.243	0.894
	8	3416002	Kg Kalong Tengah	61.811	0.161	0.188	0.816
	9	3516022	Loji Air Kuala Kubu Baru	67.793	0.176	0.278	0.854
	10	3710006	Rmh Pam Bagan Terap	60.793	0.173	0.185	0.884
Terengganu	1	3933001	Hulu Jabor, Kemaman	103.519	0.228	0.756	0.707
	2	4131001	Kg, Ban Ho, Kemaman	65.158	0.164	0.092	0.660
	3	4234109	JPS Kemaman	55.899	0.201	0.000	0.580
	4	4332001	Jambatan Tebak, Kem.	61.703	0.185	0.088	0.637
	5	4529001	Rmh Pam Paya Kempian	53.693	0.194	0.000	0.607
	6	4529071	SK Pasir Raja	48.467	0.207	0.000	0.600
	7	4631001	Almuktafibillah Shah	66.029	0.199	0.165	0.629
	8	4734079	SM Sultan Omar, Dungun	51.935	0.213	0.020	0.587
	9	4832077	SK Jerangau	54.947	0.212	0.026	0.555
	10	4930038	Kg Menerong, Hulu Trg	60.436	0.204	0.063	0.588
	11	5029034	Kg Dura. Hulu Trg	60.510	0.220	0.087	0.617
	12	5128001	Sungai Gawi, Hulu Trg	48.101	0.215	0.027	0.566
	13	5226001	Sg Petualang, Hulu Trg	48.527	0.228	0.000	0.547
	14	5328044	Sungai Tong, Setiu	52.377	0.188	0.003	0.558
	15	5331048	Setor JPS K Terengganu	58.307	0.210	0.123	0.555
	16	5426001	Kg Seladang, Hulu Setiu	57.695	0.197	0.000	0.544
	17	5428001	Kg Bt. Hampar, Setiu	55.452	0.186	0.000	0.545
	18	5524002	SK Panchor, Setiu Klinik	53.430	0.206	0.000	0.524
	19	5725006	Kg Raja, Besut	52.521	0.225	0.041	0.560

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Johor	1	1437116	Stor JPS Johor Bahru	73.6792	0.2770	0.2927	0.8620
	2	1534002	Pusat Kem. Pekan Nenas	62.6514	0.3231	0.1557	0.8212
	3	1541139	Johor Silica	79.5355	0.3363	0.2947	0.8097
	4	1636001	Balai Polis Kg Seelong	61.2124	0.3373	0.2375	0.8427
	5	1737001	SM Bukit Besar	61.3513	0.3027	0.2029	0.8240
	6	1829002	Setor Daerah JPS Batu Pahat	62.1576	0.3055	0.1423	0.8253
	7	1834124	Ladang Ulu Remis	59.1713	0.2935	0.1847	0.8380
	8	1839196	Simpang Masai K. Sedili	71.7947	0.2683	0.1863	0.8071
	9	1931003	Emp. Semberong	66.8854	0.3549	0.2107	0.8384
	10	2025001	Pintu Kaw. Tg. Agas	77.7719	0.3102	0.2806	0.8789
	11	2231001	Ladang Chan Wing	66.1439	0.3236	0.1778	0.8489
	12	2232001	Ladang Kekayaan	66.7541	0.3076	0.2270	0.8381
	13	2235163	Ibu Bekalan Kahang	62.3394	0.2786	0.1626	0.7389
	14	2237164	Jalan Kluang-Mersing	73.2358	0.3431	0.2198	0.7733
	15	2330009	Ladang Labis	65.2220	0.3947	0.2353	0.8455
	16	2528012	Rmh. Tapis Segamat	63.6892	0.3817	0.2586	0.8711
	17	2534160	Kg Peta Hulu Sg Endau	69.9581	0.3499	0.1808	0.7064
	18	2636170	Setor JPS Endau	77.6302	0.3985	0.2497	0.6927
Kedah	1	5507076	Bt. 27, Jalan Baling	62.7610	0.2580	0.3040	0.8350
	2	5704055	Kedah Peak	58.5960	0.3390	0.0640	0.661
	3	5806066	Klinik Jeniang	67.1200	0.3820	0.2380	0.8230
	4	5808001	Bt. 61, Jalan Baling	56.3990	0.3880	0.2520	0.8030
	5	6103047	Setor JPS Alor Setar	67.6410	0.3340	0.2740	0.8280
	6	6108001	Kompleks Rumah Muda	58.4040	0.2780	0.2340	0.8290
	7	6206035	Kuala Nerang	62.9600	0.3080	0.3590	0.8590
	8	6207032	Ampang Padu	70.9970	0.2930	0.3820	0.8630
	9	6306031	Padang Sanai	63.6150	0.3130	0.3090	0.8520

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Kelantan	1	4614001	Brook	49.7311	0.3159	0.1978	0.7924
	2	4915001	Chabai	56.2957	0.2986	0.1965	0.8384
	3	4923001	Kg Aring	70.2651	0.3810	0.2416	0.8185
	4	5120025	Balai Polis Bertam	67.7195	0.3271	0.2430	0.8424
	5	5216001	Gob	47.4654	0.2829	0.1531	0.7850
	6	5320038	Dabong	67.7907	0.3777	0.2740	0.8115
	7	5322044	Kg Lalok	67.7660	0.3288	0.2367	0.8188
	8	5522047	JPS Kuala Krai	63.0690	0.4681	0.3096	0.7833
	9	5718033	Kg Jeli, Tanah Merah	73.8139	0.3878	0.1161	0.7600
	10	5719001	Kg Durian Daun Lawang	67.2398	0.3651	0.1822	0.7531
	11	5722057	JPS Machang	57.3756	0.3441	0.1742	0.7085
	12	5824079	Sg Rasau, Pasir Putih	68.5083	0.4079	0.2019	0.7003
	13	6019004	Rumah Kastam Rantau Pjg	65.3650	0.4433	0.1582	0.7527
Kuala Lumpur	1	3015001	Puchong Drop, K Lumpur	68.5873	0.3519	0.1697	0.8494
	2	3116004	Ibu Pejabat JPS	65.9923	0.2857	0.1604	0.8341
	3	3116005	SK Taman Maluri	74.4510	0.2663	0.3120	0.8608
	4	3116006	Ladang Edinburgh	64.5033	0.2751	0.1814	0.8329
	5	3216001	Kg. Sungai Tua	62.9398	0.2579	0.1989	0.8374
	6	3216004	SK Jenis Keb. Kepong	69.7878	0.2955	0.1672	0.8508
	7	3217001	Ibu Bek. KM16, Gombak	66.0685	0.2565	0.2293	0.8401
	8	3217002	Emp. Genting Kelang	66.2582	0.2624	0.2423	0.8446
	9	3217003	Ibu Bek. KM11, Gombak	73.9540	0.2984	0.3241	0.8238
	10	3217004	Kg. Kuala Seleh, H. Klang	64.3175	0.2340	0.1818	0.8645
	11	3217005	Kg. Kerdas, Gombak	68.8526	0.2979	0.2024	0.8820
	12	3317001	Air Terjun Sg. Batu	75.9351	0.2475	0.2664	0.8668
	13	3317004	Genting Sempah	55.3934	0.2822	0.1835	0.8345

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Malacca	1	2222001	Bukit Sebukor	78.1482	0.2690	0.3677	0.8968
	2	2224038	Chin Chin Tepi Jalan	66.0589	0.3363	0.3301	0.8905
	3	2321006	Ladang Lendu	64.7588	0.2975	0.2896	0.8787
Negeri Sembilan	1	2719001	Setor JPS Sikamat	60.4227	0.2793	0.2694	0.8540
	2	2722202	Kg Sawah Lebar K Pilah	49.3232	0.2716	0.2164	0.8503
	3	2723002	Sungai Kepis	61.3339	0.2536	0.3291	0.8717
	4	2725083	Ladang New Rompin	65.0249	0.3575	0.3546	0.8750
	5	2920012	Petaling K Kelawang	51.7343	0.2919	0.2643	0.8630
Pahang	1	2630001	Sungai Pukim Sungai	63.9783	0.3906	0.2556	0.8717
	2	2634193	Anak Endau	79.4310	0.3639	0.1431	0.7051
	3	2828173	Kg Gambir	61.1933	0.3857	0.1878	0.8237
	4	3026156	Pos Iskandar	59.9903	0.3488	0.2262	0.8769
	5	3121143	Simpang Pelangai	64.9653	0.3229	0.3003	0.8995
	6	3134165	Dispensari Nenasi	88.6484	0.3830	0.4040	0.7614
	7	3231163	Kg Unchang	71.6472	0.3521	0.1805	0.7886
	8	3424081	JPS Temerloh	62.2075	0.3528	0.3505	0.8368
	9	3533102	Rumah Pam Pahang Tua	80.8887	0.3611	0.4800	0.7578
	10	3628001	Pintu Kaw. Pulau Kertam	63.5073	0.3830	0.2881	0.8202
	11	3818054	Setor JPS Raub	61.3432	0.3692	0.3929	0.8445
	12	3924072	Rmh Pam Paya Kangsar	58.3761	0.3334	0.2421	0.8430
	13	3930012	Sungai Lembing PCC Mill	77.0004	0.4530	0.5701	0.8125
	14	4023001	Kg Sungai Yap	77.1488	0.3725	0.3439	0.8810
	15	4127001	Hulu Tekai Kwsn.”B”	60.2235	0.4650	0.1241	0.8020
	16	4219001	Bukit Bentong	67.6128	0.2706	0.2459	0.8656
	17	4223115	Kg Merting	62.7511	0.2843	0.3630	0.9024
	18	4513033	Gunung Brinchang	42.1757	0.2833	0.1468	0.7850
Penang	1	5204048	Sg Simpang Ampat	59.3122	0.3394	0.3350	0.8090
	2	5302001	Tangki Air Besar Sg Pinang	71.7482	0.2928	0.2934	0.7779
	3	5302003	Kolam Tkgn Air Hitam	56.1145	0.2975	0.1778	0.7626
	4	5303001	Rmh Kebajikan P Pinang	60.1084	0.3575	0.2745	0.8303
	5	5303053	Kompleks Prai P Pinang	49.4860	0.3314	0.0518	0.7116
	6	5402001	Klinik Bkt Bendera P Pinang	68.0999	0.3111	0.1904	0.7662
	7	5402002	Kolam Bersih P Pinang	62.7533	0.2688	0.2488	0.7757
	8	5504035	Lahar Ikan Mati Kepala Batas	60.8596	0.3369	0.2316	0.7981

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Perak	1	5005003	JPS Teluk Intan	65.1854	0.3681	0.2552	0.8458
	2	4010001	JPS Setiawan	56.2695	0.3434	0.2058	0.8465
	3	4207048	Pejabat Daerah Kampar	79.2706	0.1829	0.3048	0.8532
	4	4311001	Rumah Pam Kubang Haji	47.8316	0.3527	0.1038	0.8018
	5	4409091	Politeknik Ungku Umar	62.9315	0.3439	0.1703	0.8229
	6	4511111	Bukit Larut Taiping	83.3964	0.3189	0.1767	0.8166
	7	4807016	Rancangan Belia Perlop	57.4914	0.3199	0.2027	0.8696
	8	4811075	Jln. Mtg. Buloh Bgn Serai	63.2357	0.3176	0.3330	0.8462
	9	5207001	Kolam Air JKR Selama	67.0499	0.3164	0.2255	0.8080
	10	5210069	Stesen Pem. Hutan Lawin	53.7310	0.3372	0.2237	0.8347
	11	5411066	Kuala Kenderong	68.5357	0.4196	0.1558	0.8378
	12	5710061	Dispensari Keroh	59.2197	0.3265	0.1621	0.8522
Perlis	1	6401002	Padang Katong, Kangar	52.1510	0.3573	0.1584	0.7858
Selangor	1	2815001	JPS Sungai Manggis	57.3495	0.2758	0.1693	0.8672
	2	2913001	Pusat Kwln. JPS T Gong	65.3556	0.3279	0.3451	0.8634
	3	2917001	Setor JPS Kajang	62.9564	0.3293	0.1298	0.8273
	4	3117070	JPS Ampang	69.1727	0.2488	0.1918	0.8374
	5	3118102	SK Sungai Lui	68.4588	0.3035	0.2036	0.8726
	6	3314001	Rumah Pam JPS P Setia	65.1864	0.2816	0.2176	0.8704
	7	3411017	Setor JPS Tj. Karang	70.9914	0.2999	0.2929	0.9057
	8	3416002	Kg Kalong Tengah	59.9750	0.2444	0.1642	0.8072
	9	3516022	Loji Air Kuala Kubu Baru	66.8884	0.2798	0.3489	0.8334
	10	3710006	Rmh Pam Bagan Terap	62.2644	0.3168	0.2799	0.8665
Terengganu	1	3933001	Hulu Jabor, Kemaman	74.8046	0.2170	0.2527	0.7281
	2	4131001	Kg, Ban Ho, Kemaman	68.6659	0.3164	0.1157	0.6969
	3	4234109	JPS Kemaman Jambatan	75.8258	0.2385	0.3811	0.7303
	4	4332001	Tebak, Kem.	77.2826	0.3460	0.3036	0.7301
	5	4529001	Rmh Pam Paya Kempian	65.2791	0.3642	0.1477	0.6667
	6	4631001	Almuktafibillah Shah	81.8861	0.3400	0.2600	0.7459
	7	4734079	SM Sultan Omar, Dungun	66.4262	0.3288	0.2152	0.7015
	8	4832077	SK Jerangau	81.4981	0.3736	0.4226	0.7586
	9	4930038	Kg Menerong, Hulu Trg	80.9649	0.3782	0.2561	0.7158
	10	5029034	Kg Dura. Hulu Trg	62.7859	0.3495	0.1103	0.6638
	11	5128001	Sungai Gawi, Hulu Trg	59.3063	0.4001	0.1312	0.6796
	12	5226001	Sg Petualang, Hulu Trg	51.7862	0.2968	0.0704	0.6587
	13	5328044	Sungai Tong, Setiu	63.4136	0.3864	0.0995	0.6540
	14	5331048	Setor JPS K Terengganu	67.0267	0.2844	0.2633	0.6690
	15	5426001	Kg Seladang, Hulu Setiu	76.9088	0.4513	0.1636	0.6834
	16	5428001	Kg Bt. Hampar, Setiu	57.9456	0.2490	0.0380	0.6000
	17	5524002	SK Panchor, Setiu	75.1489	0.4147	0.2580	0.6760

APPENDIX 2.C NORMALISED DESIGN RAINFALL TEMPORAL PATTERN

2.C1 Region 1: Terengganu and Kelantan

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.316	0.133	0.060	0.060	0.059	0.070	0.019	0.027	0.021
2	0.368	0.193	0.062	0.061	0.067	0.073	0.022	0.028	0.029
3	0.316	0.211	0.084	0.071	0.071	0.083	0.027	0.029	0.030
4		0.202	0.087	0.080	0.082	0.084	0.036	0.033	0.033
5		0.161	0.097	0.110	0.119	0.097	0.042	0.037	0.037
6		0.100	0.120	0.132	0.130	0.106	0.044	0.040	0.038
7			0.115	0.120	0.123	0.099	0.048	0.046	0.042
8			0.091	0.100	0.086	0.086	0.049	0.048	0.048
9			0.087	0.078	0.073	0.084	0.050	0.049	0.053
10			0.082	0.069	0.069	0.083	0.056	0.054	0.055
11			0.061	0.060	0.063	0.070	0.058	0.058	0.058
12			0.054	0.059	0.057	0.064	0.068	0.065	0.067
13							0.058	0.060	0.059
14							0.057	0.055	0.056
15							0.050	0.053	0.053
16							0.050	0.048	0.052
17							0.048	0.046	0.047
18							0.046	0.044	0.041
19							0.043	0.038	0.038
20							0.039	0.034	0.036
21							0.028	0.030	0.033
22							0.025	0.029	0.030
23							0.022	0.028	0.022
24							0.016	0.019	0.020

2.C2 Region 2: Johor, Negeri Sembilan, Melaka, Selangor and Pahang

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.255	0.124	0.053	0.053	0.044	0.045	0.022	0.027	0.016
2	0.376	0.130	0.059	0.061	0.081	0.048	0.024	0.028	0.023
3	0.370	0.365	0.063	0.063	0.083	0.064	0.029	0.029	0.027
4		0.152	0.087	0.080	0.090	0.106	0.031	0.033	0.033
5		0.126	0.103	0.128	0.106	0.124	0.032	0.037	0.036
6		0.103	0.153	0.151	0.115	0.146	0.035	0.040	0.043
7			0.110	0.129	0.114	0.127	0.039	0.046	0.047
8			0.088	0.097	0.090	0.116	0.042	0.048	0.049
9			0.069	0.079	0.085	0.081	0.050	0.049	0.049
10			0.060	0.062	0.081	0.056	0.054	0.054	0.051
11			0.057	0.054	0.074	0.046	0.065	0.058	0.067
12			0.046	0.042	0.037	0.041	0.093	0.065	0.079
13							0.083	0.060	0.068
14							0.057	0.055	0.057
15							0.052	0.053	0.050
16							0.047	0.048	0.049
17							0.040	0.046	0.048
18							0.039	0.044	0.043
19							0.033	0.038	0.038
20							0.031	0.034	0.035
21							0.029	0.030	0.030
22							0.028	0.029	0.024
23							0.024	0.028	0.022
24							0.020	0.019	0.016

2.C3 Region 3: Perak, Kedah, Pulau Pinang and Perlis

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.215	0.158	0.068	0.060	0.045	0.040	0.027	0.015	0.021
2	0.395	0.161	0.074	0.085	0.070	0.060	0.031	0.020	0.023
3	0.390	0.210	0.077	0.086	0.078	0.066	0.033	0.026	0.024
4		0.173	0.087	0.087	0.099	0.092	0.034	0.028	0.025
5		0.158	0.099	0.100	0.113	0.114	0.035	0.038	0.028
6		0.141	0.106	0.100	0.129	0.166	0.036	0.039	0.031
7			0.104	0.100	0.121	0.119	0.039	0.045	0.044
8			0.098	0.088	0.099	0.113	0.042	0.046	0.049
9			0.078	0.087	0.081	0.081	0.044	0.052	0.058
10			0.075	0.085	0.076	0.066	0.053	0.057	0.063
11			0.072	0.063	0.047	0.046	0.056	0.069	0.074
12			0.064	0.059	0.041	0.036	0.080	0.086	0.081
13							0.076	0.073	0.078
14							0.055	0.060	0.070
15							0.048	0.056	0.058
16							0.044	0.046	0.050
17							0.041	0.045	0.044
18							0.039	0.044	0.044
19							0.036	0.039	0.030
20							0.034	0.035	0.026
21							0.033	0.028	0.025
22							0.032	0.021	0.024
23							0.031	0.017	0.022
24							0.023	0.014	0.008

2.C4 Region 4: Mountainous Area

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.146	0.117	0.028	0.019	0.019	0.041	0.000	0.002	0.005
2	0.677	0.130	0.052	0.019	0.040	0.052	0.002	0.007	0.006
3	0.177	0.374	0.064	0.055	0.045	0.056	0.007	0.018	0.011
4		0.152	0.073	0.098	0.060	0.059	0.009	0.024	0.014
5		0.121	0.106	0.164	0.082	0.120	0.023	0.027	0.018
6		0.107	0.280	0.197	0.390	0.253	0.026	0.033	0.027
7			0.119	0.169	0.171	0.157	0.027	0.037	0.028
8			0.079	0.132	0.062	0.065	0.040	0.043	0.035
9			0.066	0.095	0.054	0.058	0.049	0.053	0.056
10			0.058	0.027	0.041	0.052	0.055	0.062	0.065
11			0.042	0.019	0.020	0.048	0.112	0.080	0.116
12			0.028	0.006	0.016	0.038	0.227	0.204	0.171
13							0.142	0.081	0.127
14							0.060	0.066	0.096
15							0.050	0.057	0.060
16							0.048	0.047	0.039
17							0.034	0.037	0.034
18							0.027	0.036	0.028
19							0.026	0.031	0.023
20							0.023	0.026	0.016
21							0.008	0.018	0.011
22							0.007	0.007	0.009
23							0.001	0.003	0.005
24							0.000	0.000	0.000

2.C5 Region 5: Urban Area (Kuala Lumpur)

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.184	0.097	0.056	0.048	0.033	0.003	0.003	0.001	0.006
2	0.448	0.161	0.061	0.060	0.045	0.051	0.011	0.011	0.014
3	0.368	0.400	0.065	0.078	0.092	0.074	0.015	0.015	0.019
4		0.164	0.096	0.095	0.096	0.086	0.021	0.018	0.023
5		0.106	0.106	0.097	0.107	0.140	0.025	0.024	0.027
6		0.072	0.164	0.175	0.161	0.206	0.032	0.027	0.040
7			0.108	0.116	0.118	0.180	0.047	0.031	0.049
8			0.103	0.096	0.102	0.107	0.052	0.033	0.050
9			0.068	0.093	0.096	0.081	0.055	0.041	0.054
10			0.065	0.062	0.091	0.064	0.076	0.068	0.067
11			0.058	0.050	0.037	0.007	0.087	0.129	0.072
12			0.050	0.030	0.023	0.003	0.103	0.142	0.110
13							0.091	0.132	0.087
14							0.080	0.096	0.070
15							0.075	0.053	0.060
16							0.054	0.036	0.052
17							0.048	0.033	0.050
18							0.035	0.030	0.047
19							0.027	0.026	0.031
20							0.023	0.020	0.025
21							0.017	0.017	0.022
22							0.012	0.012	0.014
23							0.009	0.004	0.009
24							0.002	0.001	0.003

APPENDIX 2.D EXAMPLE – IDF CURVE DEVELOPMENT

Problem:

Develop IDF curves for 2, 5, 10, 20, 50 and 100 year ARI using the annual maximum rainfall data of 5, 10, 15, 30¸60, 180, 360, 540, 720, 900 and 1440 minutes durations for a raingauge station located at Ampang, Selangor. The required rainfall data is given in Table 2.D1.

Table 2.D1: Annual Maximum Rainfall Data at Ampang Station

Year	Annual Maximum Rainfall (mm) Data for Various Durations (minutes)												
	5	10	15	30	45	60	120	180	360	540	720	900	1440
1980	20.2	35.3	40.8	53.0	59.8	65.4	72.5	72.5	72.5	72.5	72.5	122.4	123.5
1981	34.3	41.0	45.2	49.5	62.6	65.2	76.1	87.2	97.5	113.0	113.0	113.0	114.5
1982	22.3	26.3	35.9	54.9	64.3	69.1	89.0	89.0	89.0	89.0	89.0	89.0	102.5
1983	12.5	15.7	23.5	46.0	65.7	84.7	111.0	111.0	111.0	113.0	113.0	113.0	119.5
1984	38.9	44.5	50.2	67.2	75.5	75.5	75.5	75.5	84.0	90.8	92.5	93.0	93.0
1985	50.1	50.4	50.7	51.6	52.8	58.1	74.8	83.5	84.0	84.0	84.5	89.5	118.5
1986	32.2	36.5	36.5	38.2	54.3	59.0	77.9	89.5	101.7	107.0	108.8	135.2	181.5
1987	8.4	11.0	12.7	25.4	35.0	46.7	64.0	64.0	64.0	64.0	64.0	64.0	74.0
1988	34.5	34.5	34.5	53.3	69.5	85.8	103.0	103.0	103.0	103.0	103.0	103.0	103.0
1989	11.9	23.7	31.5	31.5	38.6	42.0	56.8	64.5	78.5	83.1	91.5	111.0	115.5
1990	18.7	33.0	33.0	33.0	42.4	54.4	59.5	59.5	61.0	66.0	66.0	66.0	88.0
1991	5.2	10.4	15.6	31.2	40.0	47.7	63.2	68.0	86.0	86.0	86.5	86.5	95.2
1992	8.8	17.5	23.7	28.9	42.0	51.7	89.7	107.0	107.0	107.0	107.0	107.0	107.0
1993	10.1	20.3	30.1	54.8	71.4	86.3	105.0	119.0	121.5	121.5	121.5	122.4	124.5
1994	17.0	20.4	24.9	43.0	49.8	54.2	72.0	72.1	77.0	77.0	77.0	77.0	77.0
1995	27.5	30.2	34.0	42.5	50.9	57.4	65.5	66.0	66.5	66.5	67.0	67.0	82.5
1996	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0
1997	31.3	39.2	47.1	48.5	48.5	50.3	65.4	79.2	109.3	113.5	113.5	113.5	113.5
1998	25.5	29.5	31.8	41.8	51.1	56.0	59.0	59.5	59.7	59.8	60.0	60.1	61.6
1999	26.9	30.2	33.5	44.3	57.6	69.3	84.5	103.5	112.5	112.5	112.5	112.5	119.0
2000	21.1	26.9	35.3	49.8	58.4	65.6	97.3	104.6	111.6	111.7	111.9	112.0	116.1
2001	20.5	29.5	39.0	66.9	87.2	95.6	113.7	114.1	118.1	118.4	119.1	119.3	140.1
2002	19.6	33.4	41.1	61.7	81.3	94.0	115.8	117.5	118.4	118.7	119.1	138.6	139.0
2003	16.5	25.6	35.5	66.0	95.7	102.7	110.0	110.5	110.7	110.9	111.0	111.0	133.4
2004	58.5	58.5	58.5	58.5	67.8	77.9	89.7	91.9	92.3	92.5	92.5	92.5	128.2
2005	14.6	27.0	36.7	62.1	70.5	83.2	90.8	91.0	99.4	103.6	104.3	105.0	110.0
2006	15.1	28.2	39.2	68.7	87.6	111.6	140.7	142.9	144.3	144.6	144.7	144.9	145.0
2007	18.3	29.8	42.8	69.8	90.5	103.2	133.1	137.3	137.7	138.0	138.1	138.3	191.9
2008	18.2	27.6	34.0	61.0	81.0	87.6	90.0	90.1	90.3	90.5	98.1	98.3	98.5
2009	11.8	21.5	27.1	45.5	61.7	72.4	76.0	76.1	76.3	76.4	117.8	139.1	139.4

Solution:

Reference	Calculation	Output
Figure 2.1	Step 1: Collect annual maximum rainfall data of selected durations from the Ampang station, which is given in Table 2.D1.	
Figure 2.1	Step 2: Calculate the cumulative rainfall value for each duration to check the consistencies by mass curve method. A sample mass curve is shown in Figure 2.D1 for the storm duration of 24 hours.	Figure 2.D1
Figure 2.1	Step 3: Fit the Raw Rainfall Data to Various Frequency Distributions. For this purpose, calculate the mean and standard deviation for the annual maximum rainfall values of each duration, as shown in Table 2.D2.	Table 2.D2.
Figure 2.1	Step 4: Select the Most Suitable Frequency Distribution Method that fits the Data. Various statistical distribution should be used to determine the most suitable method that fits the data set best. This step is required to estimate the design rainfall of various ARIs. The Gumble distribution is used in this example, which used the following equation. $RF_T = RF_{mean} + \sigma K$ where, RF_T = The magnitude of the rainfall for a return period of T year; RF_{mean} = The arithmetic mean value of the annual rainfall values of various durations; σ = The standard deviation from the mean; K = The frequency factor for extreme values, which depends on the type of distribution used.	
Figure 2.1	Step 5: Calculate the Amounts of Rainfalls for Various Duration and Selected ARIs (2, 5, 10, 20, 50 and 100 year). Calculate the frequency factors for the required ARIs as given in Table 2.D3. Multiply the standard deviation values with the corresponding frequency factors of various ARIs and add to the mean annual maximum rainfall values to get the design rainfall as given in Table 2.D4.	Table 2.D3 and 2.D4
Figure 2.1	Step 6: Convert the Rainfall into Intensity (Table 2.D5) and Plot Various Durations in the Log-Log Graph for the Selected ARIs. Plot the data of Table 2.D4 to get the IDF curves, as shown in Figure 2.D2. If the graphs are not smooth based on the actual statistical data, adjust the data to produce smooth graphs.	Table 2.D5 and Figure 2.D2

Table 2.D2: Calculation of Mean and Standard Deviation for the Data given in Table 2.D1

Term	Annual Maximum Rainfall (mm) for Various Durations (minutes)												
	5	10	15	30	45	60	120	180	360	540	720	900	1440
Mean	24.6	31.5	37.1	51.2	63.4	72.0	87.0	91.2	95.7	97.4	99.6	104.4	114.8
Std. Dev.	16.9	14.9	13.6	14.2	16.9	18.9	21.9	22.2	21.7	21.8	21.5	23.5	28.7

Table 2.D3: Calculation of Frequency Factors for the Selected ARIs

ARI (Year)	Frequency Factor
2	-0.1681
5	0.7371
10	1.3379
20	1.9134
50	2.6585
100	3.2166

Table 2.D4: Calculation of Design Rainfall Depths for Various ARIs

ARI (Year)	Design Rainfall (mm) Data for Various Storm Durations (minute)										
	5	10	15	30	60	180	540	720	900	1080	1440
2	21.2	29.4	36.7	55.3	67.0	77.0	86.6	87.9	93.9	99.9	110.9
5	23.1	32.0	40.0	60.1	75.3	85.3	96.0	97.2	103.2	109.2	120.2
20	25.5	35.5	44.4	66.4	86.0	96.0	108.2	109.3	115.3	121.3	132.3
50	27.1	37.7	47.1	70.4	92.8	102.8	115.9	117.0	123.0	129.0	140.0
100	28.3	39.3	49.1	73.4	97.9	107.9	121.7	122.7	128.7	134.7	145.7

Table 2.D5: Calculation of Design Rainfall Intensities for Various ARIs

ARI (Year)	Design Rainfall Intensities (mm/hr) for Various Storm Durations (minute)										
	5	10	15	30	60	180	540	720	900	1080	1440
2	216.00	173.93	146.89	99.31	80.68	68.81	41.63	29.16	15.35	10.41	7.99
5	260.62	215.28	181.55	126.47	101.09	85.90	51.55	35.87	18.62	12.60	9.61
20	300.00	247.74	206.54	142.56	114.63	97.25	58.12	40.32	20.80	14.05	10.69
50	348.34	287.74	237.68	159.22	127.61	108.12	64.43	44.59	22.88	15.45	11.72
100	389.05	325.09	274.16	181.55	144.41	122.19	72.58	50.11	25.57	17.25	13.05

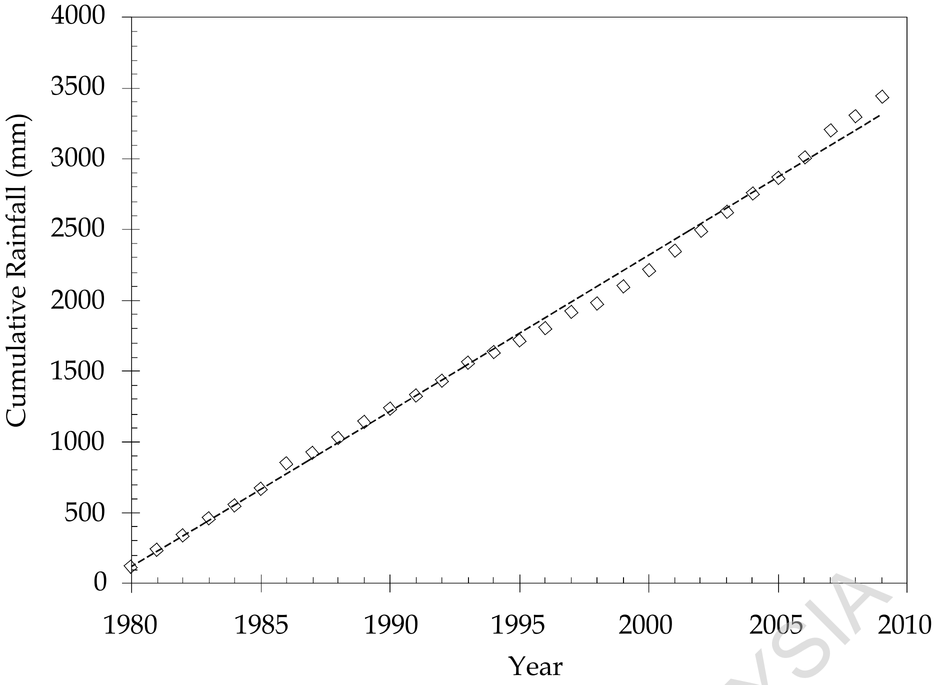


Figure 2.D1: Mass Curve to Check Consistency of the Raw Rainfall Data (24 hours Duration)

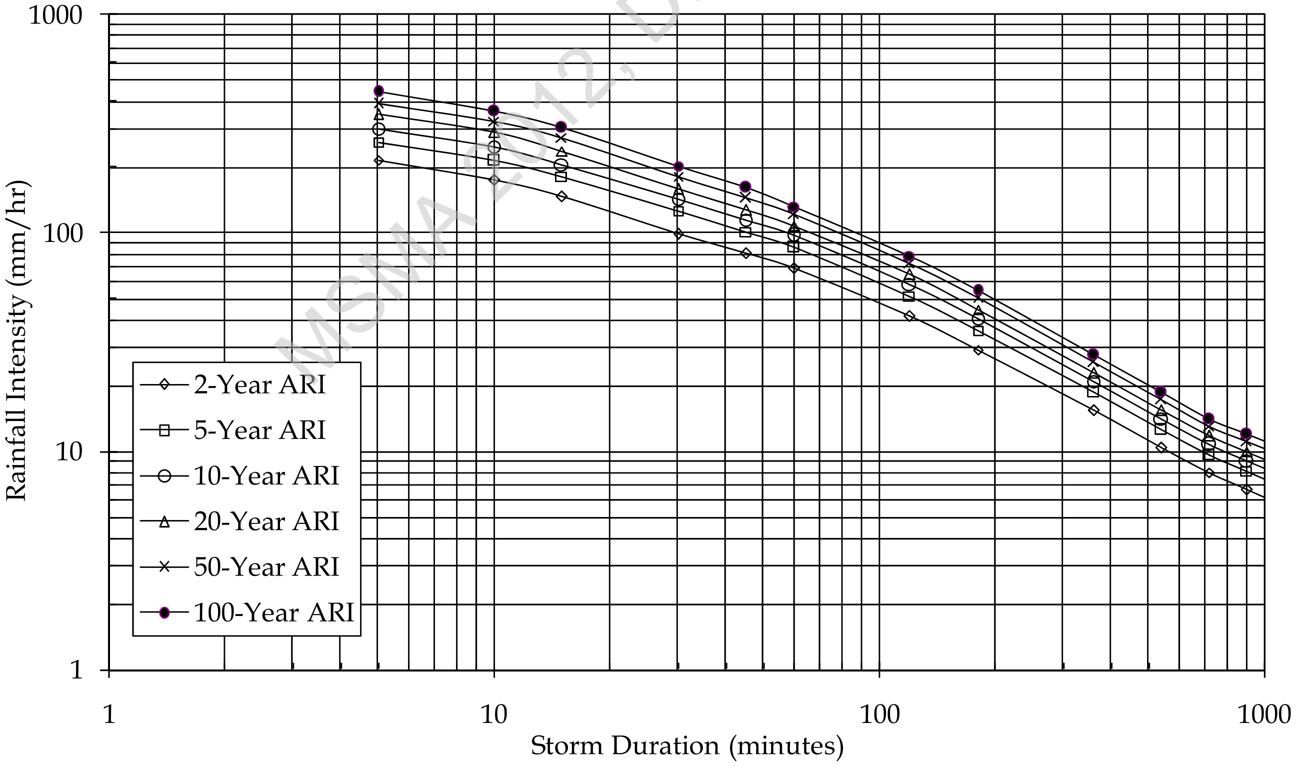


Figure 2.D2: Developed IDF Curves for Ampang Station

APPENDIX 2.E EXAMPLE - DESIGN TEMPORAL PATTERNS

Problem:

Determine the design rainfall temporal pattern for the raw data given in Table 2.E1.

Table 2.E1: Recorded Rainfall Data for Temporal Pattern

A	B	C	D	E	F	G	H
Storm Duration = 25 min Number of Intervals = 5			Rain (mm) at 5-minute Interval				
Date	Total Rain (mm)	Rank	1st	2nd	3rd	4th	5th
12.01.1972	58.7	1	9.2	12.8	12.8	12.8	11.1
07.12.1983	58.3	2	8.0	11.7	14.0	13.3	11.2
21.07.1992	54.0	3	9.5	12.3	11.2	10.9	10.2
03.12.1985	52.3	4	11.2	17.3	9.3	4.0	10.5
19.01.1999	51.1	5	4.8	13.3	12.0	10.7	10.3
27.04.2003	50.0	6	10.7	7.2	10.9	11.2	10.0
14.06.2005	46.7	7	9.3	10.3	9.7	9.0	8.3
30.06.1989	43.7	8	11.2	10.7	9.3	7.0	5.5
04.02.1990	40.9	9	8.1	8.9	9.9	6.9	7.1
17.11.2001	40.2	10	9.6	12.8	10.7	4.0	3.1

Solution:

Reference	Calculation	Output
	Step 1: Select the required storm duration and find about ten (10) dates of extreme most rainfall events, as given in Table 2.E1.	
	Step 2: Collect the rainfall amounts from the nearby automatic rainfall station for the required intervals as given in Table 2.5. The selected highest storm burst with dates and total rainfall amount are collected from the raw rainfall data of 5 minutes interval and listed in Table 2.E1. The Table also shows the distribution of raw rainfall data for the most extreme events in that area. Data of Column D to H are extracted from the five minute rainfall intervals.	
	Step 3: Assign rank for each interval based on the rainfall amount (1 for the highest amount and so on). This arrangement is given in columns I to M of Table 2.E2. For same rainfall amounts in the intervals, the average ranks should be used, as shown in the first event of Table 2.E2.	Table 2.E2
	Step 4: Determine percentage of rain occurred in each interval as given in columns N to P of Table 2.E2	Table 2.E2

Reference	Calculation	Output
	Step 5: Calculate the mean ranks (columns I to M) and percentages of rainfall (columns N to P) for each interval as given in the row (MV).	Table 2.E2
	Step 6: Assign the mean percentages of rainfall for each interval based on the new mean rank as given in the row (NR for columns I to M).	Table 2.E2
	Step 7: Convert the percentage rainfall into fraction of total rainfall and plot the temporal pattern (TPF for column I to M).	Table 2.E2
	Step 8: Multiply the TPF values with the design rainfall amount (mm) to get the distribution of rainfall in each time interval.	

Table 2.E2: Calculation for the Determination of Design Rainfall Temporal Pattern

A	B	C	D	E	F	G	H	I	J	K	L	M	N	M	N	O	P
Storm Duration = 25 min Number of Intervals = 5			Rain (mm) at 5-minute Interval					Rank of Each Rainfall Interval (Mean Rank for the Intervals with Same Rainfall Values)					Percentage of Rain for the Interval				
Date	Total Rain (mm)	Rank	1st	2nd	3rd	4th	5th	1st	2nd	3rd	4th	5th	1st	2nd	3rd	4th	5th
12.01.1972	58.7	1	9.2	12.8	12.8	12.8	11.1	5.0	2.0	2.0	2.0	4.0	16	22	22	22	19
07.12.1983	58.3	2	8.0	11.7	14.0	13.3	11.2	4.0	3.0	1.0	2.0	5.0	14	20	24	23	19
21.07.1992	54.0	3	9.5	12.3	11.2	10.9	10.2	5.0	1.0	2.0	3.0	4.0	18	23	21	20	19
03.12.1985	52.3	4	11.2	17.3	9.3	4.0	10.5	2.0	1.0	4.0	5.0	3.0	21	33	18	8	20
19.01.1999	51.1	5	4.8	13.3	12.0	10.7	10.3	5.0	1.0	2.0	3.0	4.0	9	26	23	21	20
27.04.2003	50.0	6	10.7	7.2	10.9	11.2	10.0	3.0	5.0	2.0	1.0	4.0	21	14	22	22	20
14.06.2005	46.7	7	9.3	10.3	9.7	9.0	8.3	3.0	1.0	2.0	4.0	5.0	20	22	21	19	18
30.06.1989	43.7	8	11.2	10.7	9.3	7.0	5.5	1.0	2.0	3.0	4.0	5.0	26	24	21	16	13
04.02.1990	40.9	9	8.1	8.9	9.9	6.9	7.1	3.0	2.0	1.0	5.0	4.0	20	22	24	17	17
17.11.2001	40.2	10	9.6	12.8	10.7	4.0	3.1	3.0	1.0	2.0	4.0	5.0	24	32	27	10	8
				Mean Value (MV)				3.4	1.9	2.1	3.3	4.3	19	24	22	18	17
				New Rank (NR)				4	1	2	3	5	3	1	2	4	5
				Rainfall Pattern (in % as per the New Rank)				18	24	22	19	17					
				Design Temporal Pattern, in fraction (TPF)				0.18	0.24	0.22	0.19	0.17					

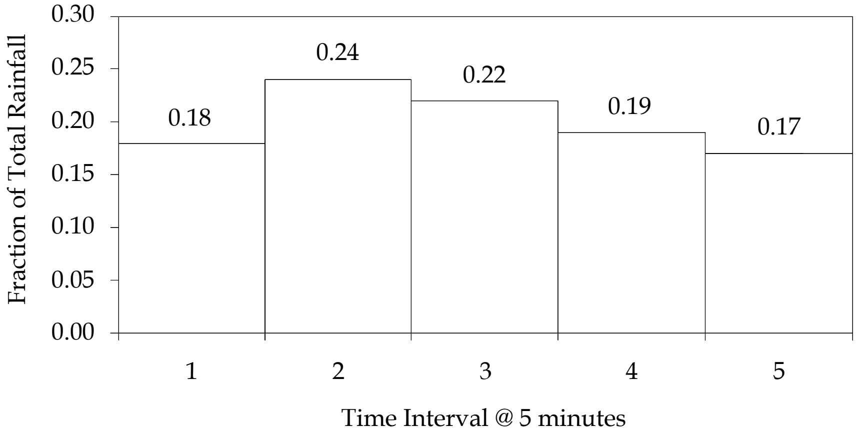


Figure 2.E1: Design Rainfall Temporal Pattern

For a 100 yr ARI rainfall with total 70 mm and 25 min duration, the distribution design rainfall, based on Figure 2.E1, is given in Table 2.E3 below.

Table 2.E3: Distribution of Design Rainfall according to the Temporal Pattern

Storm Duration = 25 min Number of Intervals = 5	Rain (mm) at 5-minute Interval				
Total Design Rainfall (mm) for 100 year ARI	1st	2nd	3rd	4th	5th
70.0	12.6	16.8	15.4	13.3	11.9

APPENDIX 2.F EXAMPLE – RUNOFF QUANTITY ESTIMATION

2.F1 Rational Method and RHM

Problem:

Using Rational Method procedure to calculate a 20 year ARI peak discharge from a subcatchment area of 40.7 ha in Wangsa Maju, Kuala Lumpur (Figure 2.F1). Also develop the runoff hydrograph using the RHM for drain AB based on 5 and 10 minute durations design storms.

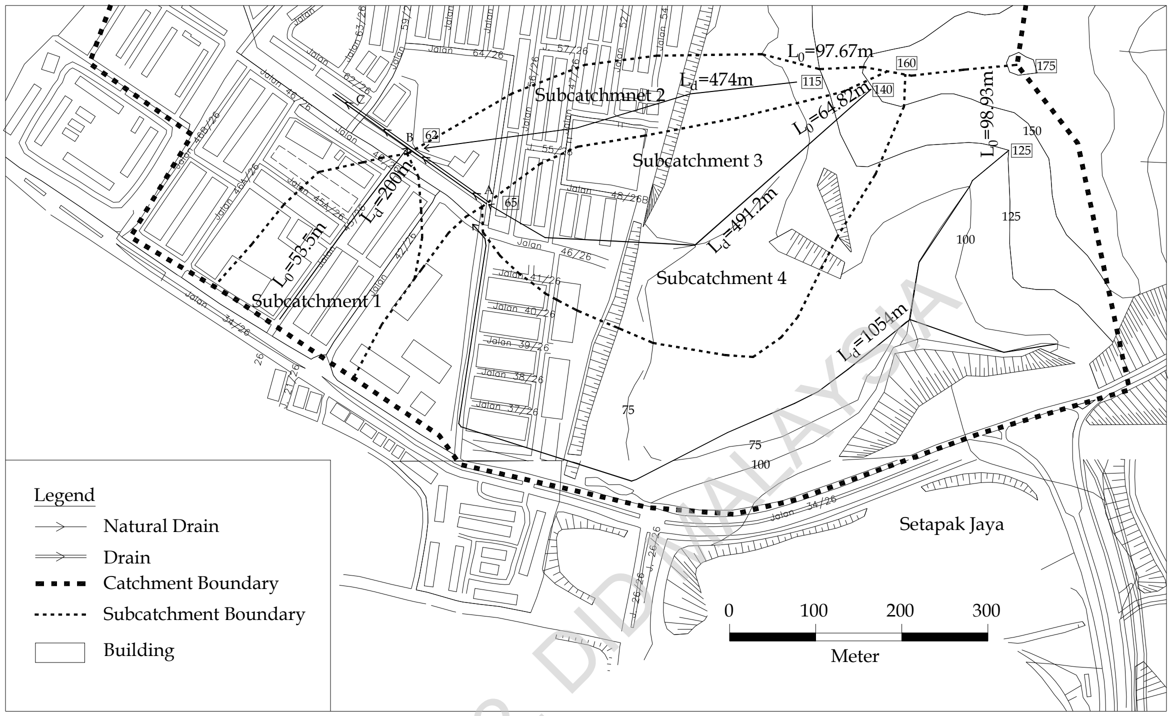


Figure: 2.F1: Drainage Subcatchment Wangsa Maju

Solution:

Reference	Calculation	Output
	Step 1:Delineate the subcatchments, as shown in Figure 2.F1. The subcatchments in this example are identified as 1, 2, 3 and 4 (Table 2.F1).	Table 2.F1
	Step 2: Calculate the subcatchment areas. The area for subcatchment 1 is 3.87 ha, subcatchment 2 is 4.95 ha, subcatchment 3 is 8.61 ha, and subcatchment 4 is 23.22ha.	
Table 2.5	Step 3: Select runoff coefficient (C). The C value for subcatchment 1 is 0.8 for flat and apartment area, 0.4 for open spaces (grass cover), for subcatchment 2 contain two types of landuse, which is 0.8 for flat and apartment area, 0.4 for open spaces (grass cover), for subcatchment 3 contain two type of landuse which is 0.75 for condominium area and 0.5 for open spaces (bare soil), and for subcatchment 4 also contain two types of landuse which is 0.9 for commercial and business centres and 0.4 for open spaces (grass cover).	

Reference	Calculation	Output
Equation 2.4	Step 4: Calculate C_{avg} values. C_{avg} = [(3.67 x 0.80) + (0.2 x 0.4)]/(3.67 + 0.2) =	0.78
	Step 5: Determine overland sheet flow path length, L_o for the flow paths in every subcatchment to calculate the time of concentration of each subcatchment. Follow the guideline in Table 2.F2 to estimate L_o.	Table 2.F2
	Step 6: Determine slope of overland surface in percent (%) *S* = [(68 – 66)/53.5] x 100% =	3.74%
Table 2.1 Table 2.2.	Step 7: Calculate t_o. Use the Horton's n^* Value (use n^*=0.015 from Table 2.2). $t_o = \frac{107.n^*.L^{1/3}}{S^{1/5}} = \frac{107x0.015\,x\,53.5^{1/3}}{3.74^{1/5}}$ =	4.6 min
Figure 2.F1	Step 8: Determine channel length, L_d for the channels in every subcatchment.	
	Step 9: Calculate area of the channel (triangular shape with slope = 1:2). From the site visit the depth of the channel is assumed at 0.3 m and width = 1.2 m	
	Step 10: Calculate wetted perimeter of the channel (P) $P = 2(0.6^2 + 0.3^2)^{1/2}$ =	1.34 m
	Step 11: Calculate hydraulic radius by, R = A/P *A* = 1/2 x 0.3 x 1.2 = 0.18 m^2 *R* = 0.18/1.34 =	0.134 m
	Step 12: Determine the friction slope of the channel, s (m/m) by dividing the different elevation by the length of channel. *S* = (66 – 62)/200 =	0.02 m/m
Table 2.1 Table 2.3	Step 13: Calculate travel time in channel, t_d (use *n*=0.015 from Table 2.3). $t_d = \frac{n.L}{60R^{2/3}\,S^{1/2}} = \frac{0.015\text{x}\,200}{60\text{x}0.134^{2/3}\,\text{x}0.02^{\,1/2}}$ =	1.4 min
	Step 14: Calculate time of concentration by using equation below. $t_c = t_o + t_d$ = 4.6 + 1.4 =	6.0 min
	Step 15 : Calculate Peak Discharge, *Q* (Table 2.F3)	Table 2.F3
	Drain AB: This drain discharge water from subcatchments 3 and 4. From Table 2.F1 A_3 = 8.61ha, C_3 = 0.57, and the t_c = 4.4 min, while A_4 = 23.22ha, C_4 = 0.51, and the t_c = 7.5 min. Hence, the total area drained by drain AB is 31.83ha and $\sum CA = C_3A_3 + C_4A_4 = (0.57x8.61) + (0.51x23.22)$ =	16.75ha
Equation 2.3	The time of concentration used is 7.5 min the larger of two drain times. The rainfall intensity, *i* is taken from Table 2.B1 for 20 year return period storm duration and 7.5min time of concentration at Ibu Pejabat JPS Station, 300.36mm/hr. Calculate peak flow, $Q_{peak.}$	

Reference	Calculation	Output
	Drain BC: This drain conveys flow from all 4 subcatchments. Subcatchment 3 and 4 through drain AB, while subcatchment 2 and 1 directly to point B. There are thus 3 possible paths for water to reach at point B. The time of concentration is the largest of the flow times. The flow time for flowing coming from drain AB is 7.5 min plus 1.18 min travel time = 8.7 min; the flow time from subcatchment 1 and 2 is 6.0 min and 4.8 min, respectively. Thus, the time of concentration for pipe BC is taken as 8.7 min. Then for rainfall intensity and Q_{peak}, use the same method for the previous drain.	

Table 2.F1: Characteristics of the Drainage Catchment

Subcatchment ID	Landuse	Area (ha)		Runoff Coefficient, C (Table 2.5)		Area Weighted C
		Developed Area	Undeveloped Area	Developed Area	Undeveloped Area	
1	Condo	3.67	0.20	0.80	0.40	0.78
2	Apartment	3.22	1.73	0.80	0.40	0.66
3	Terrace	2.37	6.24	0.75	0.50	0.57
4	Industry	5.10	18.12	0.90	0.40	0.51

Table 2.F2: Calculation of Time of Concentration (t_c)

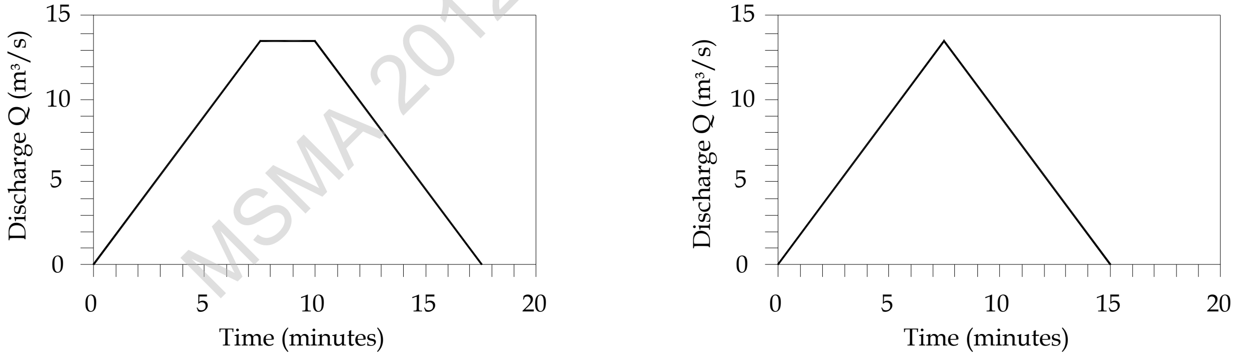
Subcatchment	L_o	S, %	t_o, min	L_d	A	P	R	S, m/m	t_d, min	t_c, min
1	53.50	3.74	4.6	200.00	0.18	1.34	0.134	0.02	1.4	6.0
2	97.67	46.10	3.4	474.00	0.18	1.34	0.134	0.11	1.4	4.8
3	64.82	30.85	3.2	491.20	0.18	1.34	0.134	0.15	1.2	4.4
4	98.93	50.54	3.4	1054.00	0.18	1.34	0.134	0.06	4.1	7.5

Table 2.F3: Calculation for Peak Discharge, Q

Drain	Total Area (ha)	∑CA	t_c (min)	I (mm/hr)	Flow, Q (m^3/s)
AB	31.83	16.750	7.5	300.36	13.98
BC	40.65	23.036	8.7	281.83	18.03

Hydrograph Development using RHM

Reference	Calculation	Output
Table 2.F3	Step 1: Select the drain AB for which the hydrograph need to be generated.	
Figure 2.F1 Figure 2.5	Step 2:Determine whether the storm duration (d) is shorter or longer than the time of concentration (t_c) of the drain. This information is necessary to determine the type of the hydrograph by RHM. In this case, when the d is 5 minutes (d < t_c) it will be type 2 hydrograph and when the d is 10 minutes (d > t_c) it will be type 1 hydrograph.	
Table 2.F3 Figure 2.5(a)	Step 3: Now for the storm duration (d) of 10 minutes which is longer than the t_c, follow similar procedure and construct a trapezoidal hydrograph with height as 13.98 m^3/s and base as d+t_c=10+7.5= 17.5 minutes, as shown in Figure 2.F2(a). The coordinates of the triangular hydrograph are A(0,0), B(7.5,13.98), C(10,13.98) and D(17.5,0).	Figure 2.F2(a)
	Step 4: When d=5 minutes, base of the hydrograph will be 2t_c=2x7.5=15 minutes.	
Figure 2.5(b)	Step 5: Construct a triangular hydrograph with height as 13.98 m^3/s and base as 15 minutes, as shown in Figure 2.F2(b). The coordinates of the triangular hydrograph are A(0,0), B(7.5, 13.98) and C(15,0).	Figure 2.F2(b)



a) d > t_c

b) d < t_c

Figure 2.F2 : Hydrographs by RHM

2.F2 Time-Area Hydrograph Method

Problem:

Using the Time-Area Hydrograph Method calculate a 20 year ARI runoff hydrograph from a 97 hectare mixed urban area located in Wangsa Maju, Kuala Lumpur. The study area is shown in Figure 2.F3.

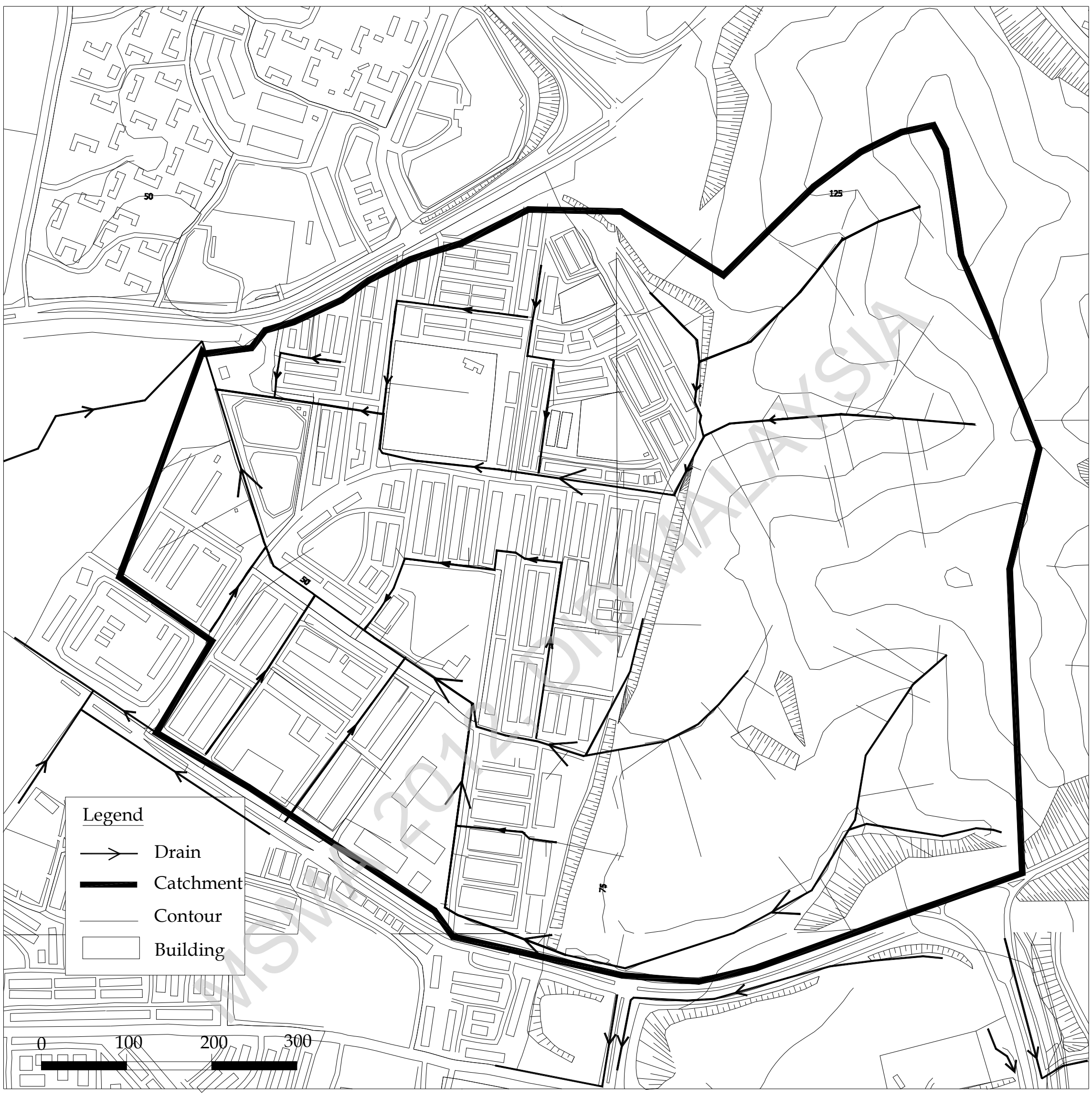


Figure 2.F3: Catchment Area in Wangsa Maju, Kuala Lumpur

Solution:

Reference	Calculation	Output
	A. Isochrone Development Step 1: Setting grid system	Figure 2.F4

Reference	Calculation	Output
Table 2.1	Step 2: Estimate overland flow time, t_o $t_o = \frac{107.n^*.L^{1/3}}{S^{1/5}}$	Table 2.F4
Table 2.1	Step 3: Estimate drain time, t_d $t_d = \frac{n.L}{60R^{2/3}S^{1/2}}$ However, flow velocity through the drain channel is assumed, in this example, 1m/s for the sake of simplicity.	Table 2.F4
	Step 4: Calculate t_c at each grid point	Table 2.F4
	Step 5: Plot contour of equal t_c for 5 minute interval to produce isochrones; 5, 10, 15, 20, and 25 min	Figure 2.F5
	Step 6: Estimate area between isochrones using AutoCAD	Table 2.F5
	Step 7: Calculate rainfall excess	
Table 2.B1	By using rainfall data of 20 year ARI and 30 minutes duration at station Ibu Pejabat JPS, Kuala Lumpur in Table 2.B1, rainfall intensity is:	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$ $= \frac{61.976 \times 20^{0.145}}{(0.5 + 0.122)^{0.818}}$ = 141.11mm/hr Total Rainfall = 141.11 x (30/60) = 70.56mm	
Appendix 2.C5	By using Appendix 2.C5, rainfall temporal pattern is obtained from fraction for 30-min storm duration as follows : 0-5 : 0.097 x 70.56 = 6.84mm 5-10 : 0.161 x 70.56 = 11.36mm 10-15 : 0.400 x 70.56 = 28.22mm 15-20 : 0.164 x 70.56 = 11.57mm 20-25 : 0.106 x 70.56 = 7.48mm 25-30 : 0.072 x 70.56 = 5.08mm	
Table 2.6	Using Table 2.6 to assume the losses, in decay form, as follows : 0-5 : 3.5mm 5-10 : 3.0mm 10-15 : 2.5mm	

	15-20 : 2.0mm 20-25 : 1.5mm 25-30 : 1.0mm	
	Rainfall Excess = Rainfall Temporal Pattern – Losses 0-5 : 6.84 – 3.5 = 3.34mm 5-10 : 11.36 – 3.0 = 8.36mm 10-15 : 28.22 – 2.5 = 25.72mm 15-20 : 11.57 – 2.0 = 9.57mm 20-25 : 7.48 – 1.5 = 5.98mm 25-30 : 5.08 – 1.0 = 4.08mm	Table 2.F6
	Step 8: Calculate hydrograph ordinate	Table 2.F7
	Step 9: Identify the peak discharge	Figure 2.F6
	From the Table 2.F7 or Figure 2.F6 the peak discharge is	32.81 m^3/s.

Table 2.F4: Calculation Time of Concentration

Grid no.	L_d (m)	L_o (m)	n*	S (%)	t_o (min)	V (m/s)	t_d (min)	t_c (min)
A6	140.44	101.83	0.015	0.5	8.6	1.0	2.3	11.0
A7	202	133.1	0.015	0.5	9.4	1.0	3.4	12.8
B5	78.58	-	-	-	-	1.0	1.3	1.3
B6	167.63	-	-	-	-	1.0	2.8	2.8
B7	258.38	37	0.015	0.5	6.1	1.0	4.3	10.4
B8	375.11	22.25	0.015	0.5	5.2	1.0	6.3	11.4
B9	547.27	8.44	0.015	0.5	3.8	1.0	9.1	12.9
C5	180.81	12.28	0.015	0.5	4.3	1.0	3.0	7.3
C6	191.56	88.22	0.015	0.5	8.2	1.0	3.2	11.4
C7	297.05	71.38	0.015	0.5	7.6	1.0	5.0	12.6
C8	397.1	19.2	0.015	0.5	4.9	1.0	6.6	11.6
C9	470.35	71.03	0.015	0.5	7.6	1.0	7.8	15.5
C10	698.4	-	-	-	-	1.0	11.6	11.6
D5	300.33	-	-	-	-	1.0	5.0	5.0
D6	329.84	20.51	0.018	0.5	6.1	1.0	5.5	11.6
D7	543.55	23.32	0.018	0.5	6.3	1.0	9.1	15.4
D8	457.39	25.15	0.018	0.5	6.5	1.0	7.6	14.1
D9	528.72	49.41	0.018	0.5	8.1	1.0	8.8	16.9
D10	633	78.56	0.018	0.5	9.5	1.0	10.6	20.0
D11	736.21	146.81	0.018	0.5	11.7	1.0	12.3	23.9
E4	514.4	-	-	-	-	1.0	8.6	8.6
E5	312.32	106.01	0.018	0.5	10.5	1.0	5.2	15.7
E6	534.59	-	-	-	-	1.0	8.9	8.9
E7	626.41	24.81	0.018	0.5	6.5	1.0	10.4	16.9
E8	513.82	87.91	0.018	0.5	9.8	1.0	8.6	18.4
E9	598.97	-	-	-	-	1.0	10.0	10.0
E10	700.41	-	-	-	-	1.0	11.7	11.7
E11	795.95	21.73	0.018	0.5	6.2	1.0	13.3	19.4
F3	827.07	35.29	0.018	0.5	7.3	1.0	13.8	21.0
F4	756.59	-	-	-	-	1.0	12.6	12.6
F5	631.7	-	-	-	-	1.0	10.5	10.5
F6	536.7	-	-	-	-	1.0	8.9	8.9
F7	748.69	17.25	0.018	0.5	5.7	1.0	12.5	18.2
F8	869.59	-	-	-	-	1.0	14.5	14.5
F9	954.2	-	-	-	-	1.0	15.9	15.9
F10	745.74	43.69	0.018	0.5	7.8	1.0	12.4	20.2
F11	828.38	19.46	0.018	0.5	6.0	1.0	13.8	19.8
F12	1009.78	15.88	0.018	0.5	5.6	1.0	16.8	22.4
G4	981.95	51.69	0.018	0.5	8.2	1.0	16.4	24.6
G5	624.4	99.51	0.018	0.5	10.3	1.0	10.4	20.7
G6	638.56	42.88	0.018	0.5	7.7	1.0	10.6	18.4
G7	775.02	152.36	0.018	0.5	11.8	1.0	12.9	24.7
G8	966.13	22.37	0.018	0.5	6.2	1.0	16.1	22.3
G9	863.39	-	-	-	-	1.0	14.4	14.4

Table 2.F4: Calculation Time of Concentration (Continued)

Grid no.	L_d (m)	L_o (m)	n	S (%)	t_o (min)	V (m/s)	t_d (min)	t_c (min)
G10	830.13	39.1	0.018	0.5	7.5	1.0	13.8	21.3
G11	899.7	88.56	0.018	0.5	9.9	1.0	15.0	24.9
G12	1108.96	42.6	0.018	0.5	7.7	1.0	18.5	26.2
H3	1000	95.92	0.018	3.43	6.9	1.0	16.7	23.6
H4	944.6	27.04	0.018	2.22	4.9	1.0	15.7	20.7
H5	852.6	-	-	-	-	1.0	14.2	14.2
H6	751.78	16.37	0.018	23	2.6	1.0	12.5	15.1
H7	710.12	76.6	0.018	17	4.6	1.0	11.8	16.5
H8	973.69	76.85	0.018	31.4	4.1	1.0	16.2	20.3
H9	908.11	100.11	0.018	25.4	4.7	1.0	15.1	19.8
H10	908.99	63.86	0.018	11.1	4.8	1.0	15.1	19.9
H11	902.89	157	0.018	3.38	8.1	1.0	15.0	23.2
H12	1225.4	-	-	-	-	1.0	20.4	20.4
I3	1087.2	61.9	0.018	0.9	7.8	1.0	18.1	25.9
I4	1011.6	-	-	-	-	1.0	16.9	16.9
I5	896.86	35.18	0.018	0.4	7.6	1.0	14.9	22.5
I6	880.33	65.93	0.018	34	3.8	1.0	14.7	18.5
I7	796.26	164.71	0.018	37.24	5.1	1.0	13.3	18.4
I8	1030.23	71.77	0.018	22.67	4.3	1.0	17.2	21.5
I9	1024.6	57.79	0.018	12.41	4.5	1.0	17.1	21.6
I10	958.8	123.63	0.018	4.9	7.0	1.0	16.0	23.0
I11	923.51	201.87	0.018	1.1	11.0	1.0	15.4	26.4
I12	1318.7	28.14	0.018	10.7	3.6	1.0	22.0	25.6
J4	1035.5	86.72	0.018	36.0	4.2	1.0	17.3	21.4
J5	995.64	29.53	0.018	37.3	2.9	1.0	16.6	19.5
J6	998.59	70.44	0.018	42.0	3.8	1.0	16.6	20.4
J7	931.68	179.89	0.018	38.1	5.2	1.0	15.5	20.8
J8	1030.23	151.48	0.018	29.0	5.2	1.0	17.2	22.4
J9	1622.78	47.58	0.018	32.6	3.5	1.0	27.0	30.5
J10	1540.87	-	-	-	-	1.0	25.7	25.7
J11	1443.19	26.86	0.018	22	3.1	1.0	24.1	27.2
J12	1385.47	80.71	0.018	7.31	5.6	1.0	23.1	28.7
K3	1169.14	92.02	0.018	60.9	3.8	1.0	19.5	23.3
K4	1056.37	139.61	0.018	64.1	4.3	1.0	17.6	22.0
K5	1083.1	37.39	0.018	54	2.9	1.0	18.1	21.0
K6	1078.29	65.58	0.018	63.5	3.4	1.0	18.0	21.4
K7	1040.3	174.25	0.018	46.8	5.0	1.0	17.3	22.3
K8	1742.31	42.35	0.018	70.8	2.9	1.0	29.0	31.9
K9	1690.2	36.24	0.018	41.4	3.0	1.0	28.2	31.2
K10	1494.02	111.44	0.018	22.43	5.0	1.0	24.9	29.9
K11	1498.53	96.72	0.018	30	4.5	1.0	25.0	29.5

Table 2.F5: Areas between the Isochrones

ID	Isochrones	Area (m²)
A_1	0 – 5	44449
A_2	5 – 10	79304
A_3	10 – 15	229404
A_4	15 – 20	213852
A_5	20 – 25	160342
A_6	25 >	45306

Table 2.F6: Rainfall Excess for 20 year ARI Design Rainfall

Time (min)	Total Rainfall (mm)	Losses (mm)	Rainfall Excess (mm)
5	6.84	3.5	3.34
10	11.36	3.0	8.36
15	28.22	2.5	25.72
20	11.57	2.0	9.57
25	7.48	1.5	5.98
30	5.08	1.0	4.08

Table 2.F7: Time-Area Hydrograph Method

Time (min)	Area (m²)	Rainfall Excess (mm)						Hydrograph (m³/s)
		3.34	8.36	25.72	9.57	5.98	4.08	
0	0	0						0.00
5	44449	0.49	0					0.49
10	79304	0.88	1.24	0				2.12
15	229404	2.55	2.21	3.81	0			8.57
20	213852	2.38	6.39	6.80	1.42	0		16.99
25	160342	1.79	5.96	19.67	2.53	0.89	0	30.83
30	45306	0.50	4.47	18.33	7.32	1.58	0.60	**32.81**
35			1.26	13.75	6.82	4.57	1.08	27.48
40				3.88	5.11	4.26	3.12	16.38
45					1.45	3.20	2.91	6.32
50						0.90	2.18	3.08
55							0.62	0.62

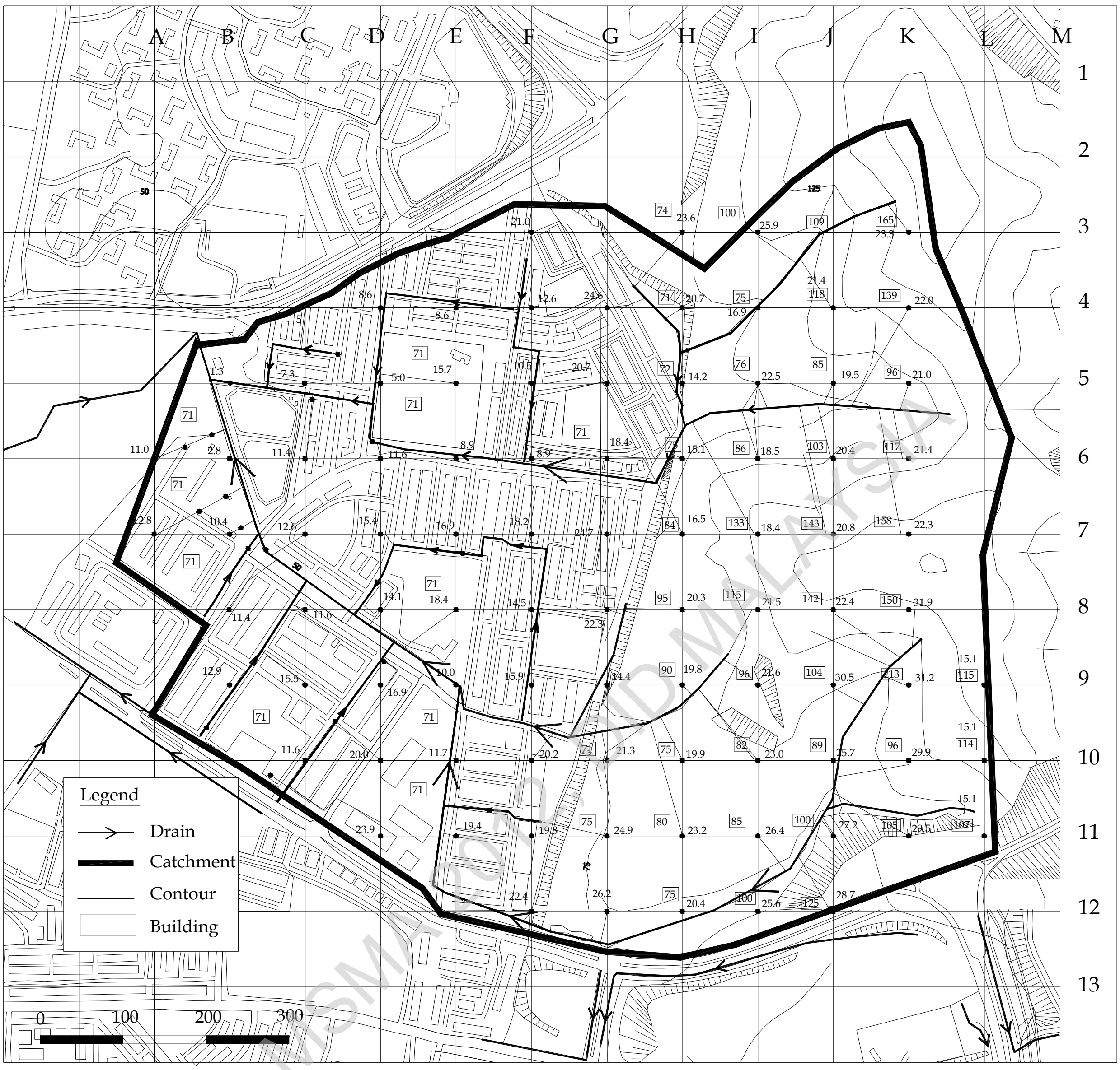


Figure 2.F4: Grid System to Calculate t_c

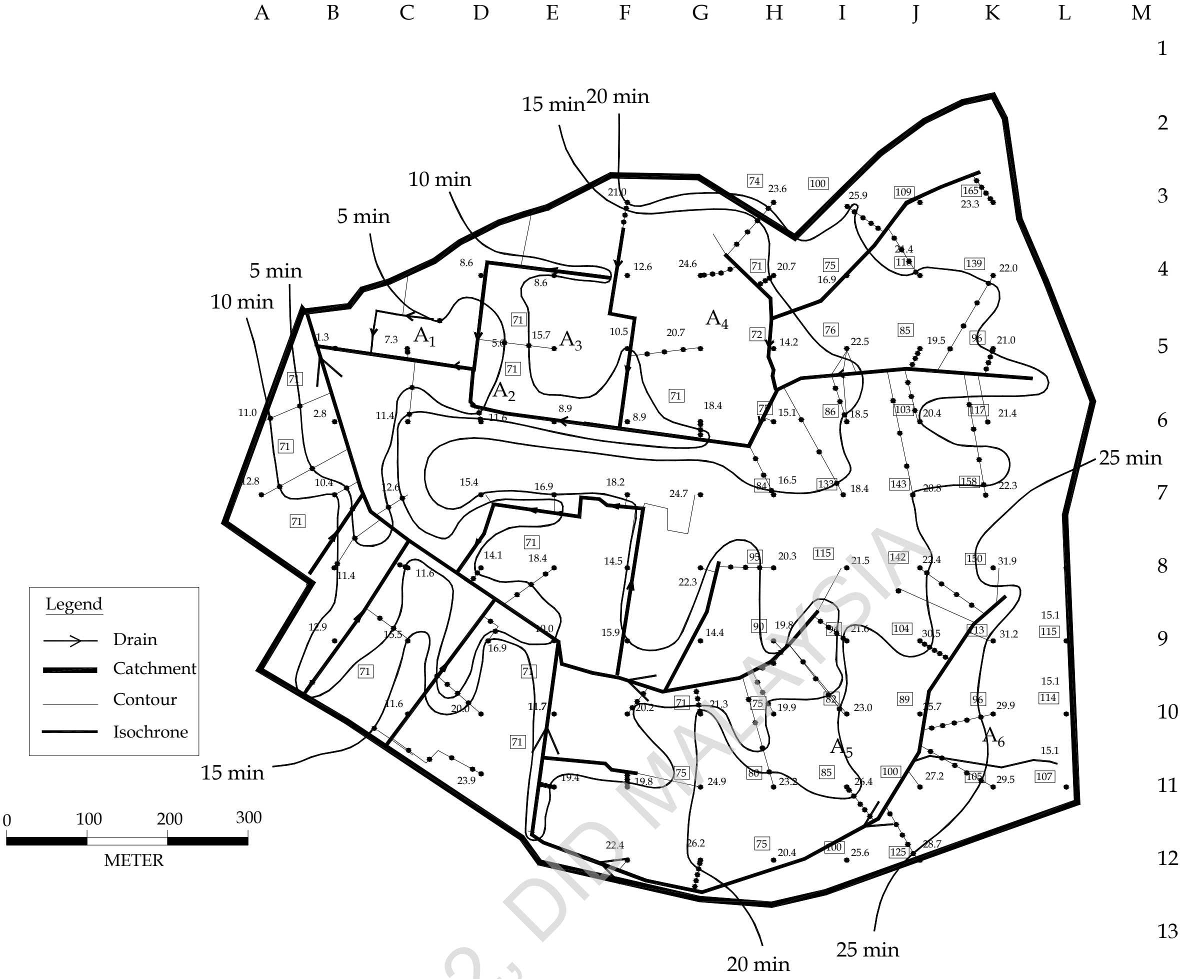


Figure 2.F5: Catchment Area with the Developed Isochrone Lines

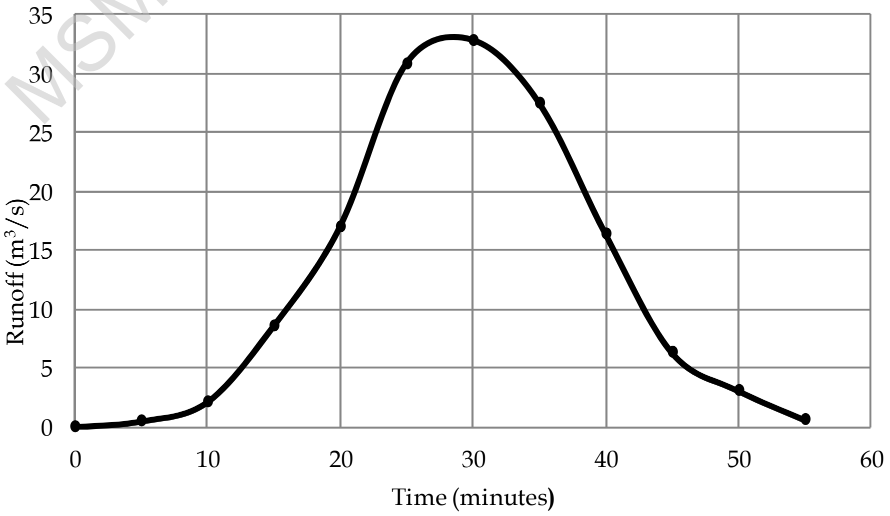


Figure 2.F6: Runoff Hydrograph

APPENDIX 2.G EXAMPLE - POND ROUTING

Problem:

Given is a triangular inflow hydrograph with Q_p = 10.93 m³/s at t_c = 11.65 min (Figure 2.G1). Determine the outflow hydrograph from a storage pond using the routing procedure in Section 2.5. Given are pond stage-storage curve (Figure 2.G2) and stage-discharge curve of the outlet structure, orifice and spillway combined (Figure 2.G3).

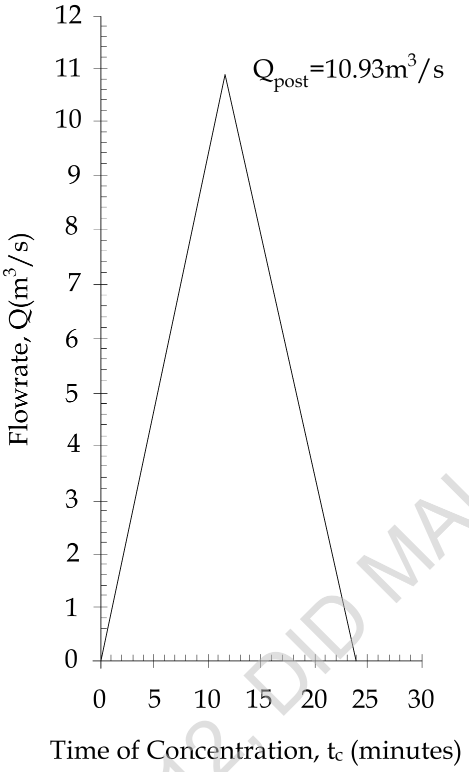


Figure 2.G1: Triangular Hydrograph

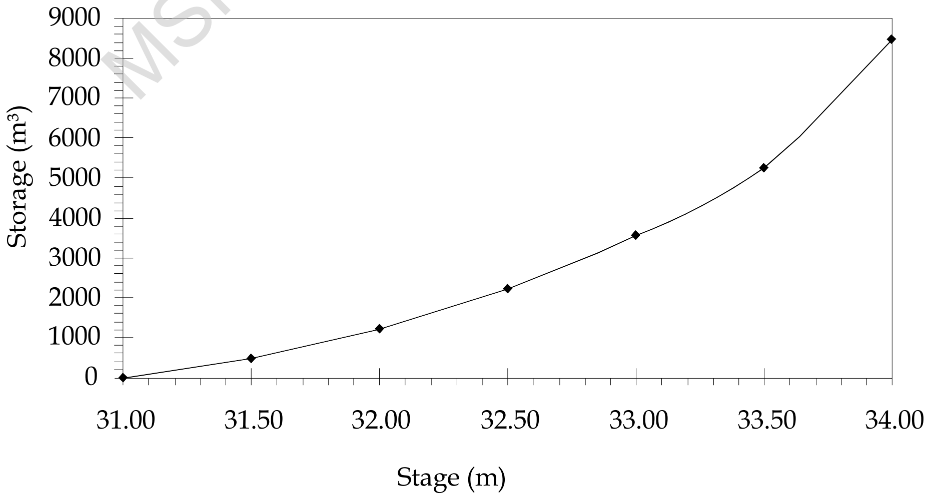


Figure 2.G2: Stage-storage Curve

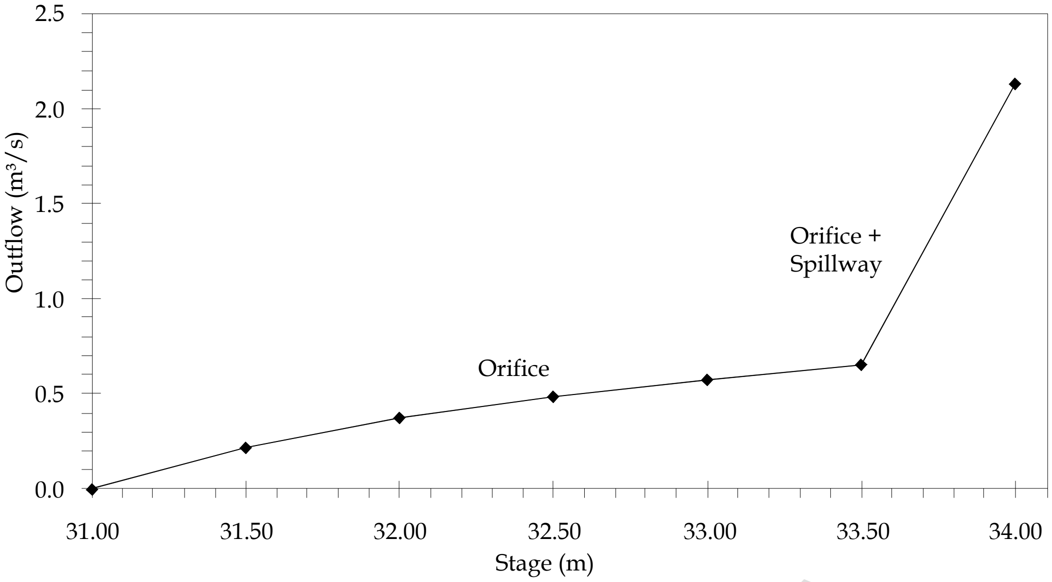


Figure 2.G3: Stage-discharge Curve (Composite)

Solution

Reference	Calculation	Output
	<u>Develop storage indicator curve</u> - For each stage point determine storage(S) and discharge (O) - For each discharge point determine storage indicator ($S_2/\Delta t + O_2/2$)	Table 2.G1 Figure 2.G4
Section 2.5	<u>Develop outflow hydrograph</u>	Table 2.G2 Figure 2.G5

Table 2.G1: Storage Indicator Numbers – Orifice (Δt = 2.5 min. or 150 sec)

Stage (m)	Discharge, O_2 (m^3/s)	Storage Volume, S_2 (m^3)	$O_2/2$ (m^3/s)	$S_2/\Delta t$ (m^3/s)	$S_2/\Delta t + O_2/2$ (m^3/s)
31.00	0.000	0.000	0.000	0.000	0.000
31.50	0.217	486.838	0.109	3.246	3.354
32.00	0.376	1216.161	0.188	8.108	8.296
32.50	0.485	2228.079	0.243	14.854	15.097
33.00	0.574	3562.700	0.287	23.751	24.038
33.50	0.651	5260.137	0.326	35.068	35.393
34.00	2.131	8489.051	1.065	56.594	57.659

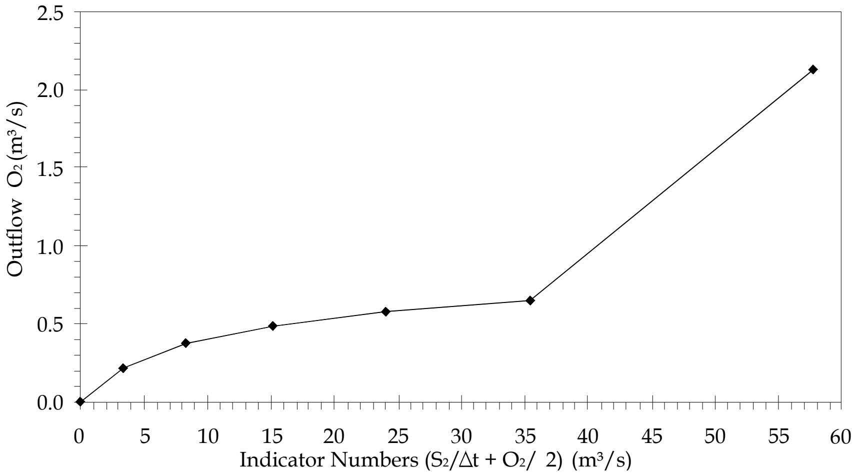


Figure 2.G4: Storage Indicator Curve (Composite)

Table 2.G2: Final Routing Table

Time (hr)	Inflow (I) (m^3/s)	($I_1 + I_2$)/2 (m^3/s)	$S_1/\Delta t + O_1/2$ (m^3/s)	Outflow (O_1) (m^3/s)	$S_2/\Delta t + O_2/2$ (m^3/s)	Outflow (O_2) (m^3/s)
0.00	0.000	0.000	0.000	0.000	0.000	0.000
0.04	2.350	1.173	0.000	0.000	1.173	0.078
0.08	4.690	3.520	1.173	0.078	4.614	0.463
0.13	7.040	5.866	4.614	0.463	10.017	0.715
0.17	9.390	8.212	10.017	0.715	17.515	0.904
0.21	10.140	9.761	17.515	0.904	26.371	1.040
0.25	7.790	8.963	26.371	1.040	34.295	1.142
0.29	5.440	6.617	34.295	1.142	39.770	1.669
0.33	3.100	4.270	39.770	1.669	42.372	1.971
0.38	0.750	1.924	42.372	1.971	42.324	1.966
0.42	0.000	0.375	42.324	1.966	40.734	1.781
0.46	0.000	0.000	40.734	1.781	38.953	1.574
0.50	0.000	0.000	38.953	1.574	37.379	1.391
0.54	0.000	0.000	37.379	1.391	35.988	1.229
0.58	0.000	0.000	35.988	1.229	34.759	1.148
0.63	0.000	0.000	34.759	1.148	33.611	1.133
0.67	0.000	0.000	33.611	1.133	32.479	1.118
0.71	0.000	0.000	32.479	1.118	31.360	1.104
0.75	0.000	0.000	31.360	1.104	30.257	1.090
0.79	0.000	0.000	30.257	1.090	29.167	1.076
0.83	0.000	0.000	29.167	1.076	28.092	1.062
0.88	0.000	0.000	28.092	1.062	27.030	1.048
0.92	0.000	0.000	27.030	1.048	25.982	1.035
0.96	0.000	0.000	25.982	1.035	24.947	1.022
1.00	0.000	0.000	24.947	1.022	23.926	1.008
1.04	0.000	0.000	23.926	1.008	22.918	0.991
1.08	0.000	0.000	22.918	0.991	21.927	0.975
1.13	0.000	0.000	21.927	0.975	20.951	0.960
1.17	0.000	0.000	20.951	0.960	19.992	0.944

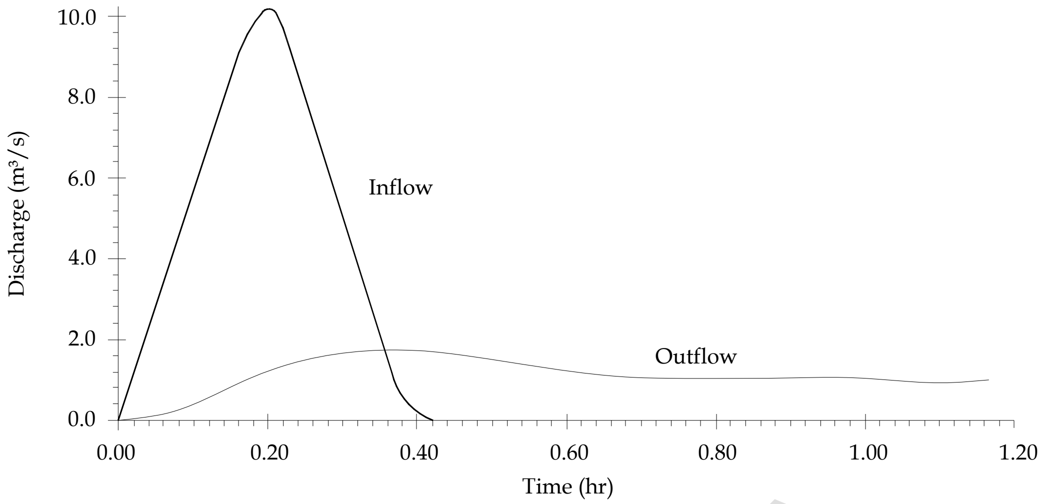
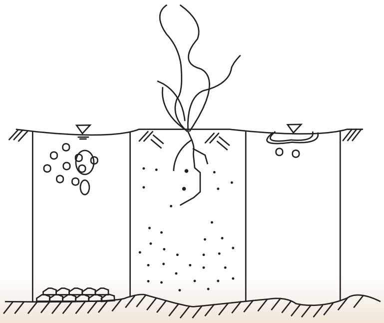


Figure 2.G5: Inflow and Outflow Hydrograph



QUALITY DESIGN FUNDAMENTALS 3

CHAPTER 3 QUALITY DESIGN FUNDAMENTALS

3.1 INTRODUCTION

This chapter provides fundamentals on the non-point source (NPS) pollutants from typical urban areas, estimation of annual pollutant load based on the event mean concentration (EMC) and simplified procedures required for preliminary sizing and designing stormwater quality control facilities or Best Management Practices (BMPs). The recommended approach is to adopt performance standards as set out in Chapter 1. For new development and redevelopment, a minimum overall *reduction in the average annual pollutant load* is specified (Table 1.4) compared with the load under existing condition. The targeted reduction is achievable applying the prescribed BMPs and treatment trains, as presented in the relevant design Chapters.

3.2 POLLUTANT ESTIMATION

3.2.1 Typical Runoff Pollutants

Although many different constituents can be found in urban runoff, the focus is primarily on certain pollutants that can be used as representative indicators of others. The following constituents are recommended as typical pollutants characterising urban runoff in Malaysia:-

- Gross pollutants;
- Total suspended solids (TSS);
- Biochemical oxygen demand (BOD);
- Chemical oxygen demand (COD);
- Total nitrogen (TN);
- Total phosphorus (TP);
- Copper (Cu);
- Lead (Pb);
- Zinc (Zn);
- Oil and grease (O&G); and
- Bacteria (*E. coli*).

However, in the current practice, BMPs are to be designed based on three types of selected pollutants; namely gross pollutants, total suspended solids (TSS), and nutrients (TN and TP). These stormwater pollutants are potentially generated from various landuses as given in Table 3.1.

Table 3.1: Selected Pollutants Generation Potentials (Adapted from Melbourne Water, 2005)

Landuse	Generation Potentials		
	Gross Pollutants	Total Suspended Solids (TSS)	Nutrients (TN & TP)
Roads and Highways	Low	High	Low
Residential	High	High	High
Commercial	High	Medium	Medium
Industrial	Medium	Medium	Low
Parks and Agricultural	Low	Medium	High

3.2.2 Water Quality Volume (WQV)

It is necessary to estimate runoff volumes before any assessment can be made of pollutant loads. Runoff estimation is necessary to size certain BMPs and also to design the outlet structures of the BMPs. Runoff volumes (expressed as rainfall depth multiplied by the catchment area and volumetric runoff coefficient, C_v) should be used to determine the BMPs volumes.

The required water quality volume (WQV) for BMPs can be calculated using Equation 3.1 as follows:

$$WQV = C_v.(P_d).A \tag{3.1}$$

where,

WQV = Water quality volume (m^3);
C_v = Area-weighted volumetric runoff coefficient for the landuse (Table 2.5);
P_d = Rainfall depth for water quality design storm (m); and
A = Contributing drainage area (m^2).

The runoff coefficient *C* for Rational Method (Chapter 2) is a function of rainfall intensity, and therefore of event ARI. In general, the value of average volumetric runoff coefficient C_v will be less than that of Rational Method runoff coefficient *C* for events of large ARI (say >2 years), and C_v will be greater than *C* for events of small ARI. However, for the simplicity and without compromising the accuracy, the values of C_v can be considered same as *C* given in Table 2.5, which would provide safe values as the BMPs are designed for small ARIs.

3.2.3 Load Estimate

Ideally, the identification of sustainable pollutant loads in receiving water should be based on the magnitude of overall catchment pollutant exports, and the reduction in pollutant loads required to achieve water quality objectives. Land activities and practices within a catchment contribute to the accumulation of pollutant loads as a result of stormwater wash off. Event mean concentrations (EMC) for different land activities are to be used to estimate pollutant loads.

Load estimation, based on the EMC method, is recommended for most practices. This is the simplest but most widely used method to estimate runoff pollutants from urban areas. However, the reliability of this method depends on the accuracy of the EMC values for the intended parameters and landuses. Recommended mean EMC values for various pollutants and landuses are set out in Table 3.2. They are related to level of housekeeping practices within the selected study areas, namely in Malacca, Damansara and Penang river catchments and Kajang Municipality.

Table 3.2: Mean EMC Values for Selected Landuses

Pollutants		Landuses			
Parameter	Unit	Residential	Commercial	Industrial	Highway
TSS	mg/L	128.00	122.00	166.00	80.00
Turbidity	NTU	122.00	96.00	147.00	69.00
TDS	mg/L	131.00	43.00	137.00	38.00
pH	-	6.46	6.77	6.66	6.57
BOD	mg/L	17.90	22.90	19.30	14.90
COD	mg/L	97.00	134.00	140.00	81.00
AN	mg/L	0.73	0.85	1.00	0.44
TKN	mg/L	2.38	2.53	4.25	1.43
TN	mg/L	4.21	4.84	5.00	2.25
TP	mg/L	0.34	0.32	0.49	0.16
O&G	mg/L	2.00	4.00	NA	3.00
Zn	mg/L	0.19	0.34	0.43	0.21
Pb	µg/L	6.00	22.00	12.00	20.00
Cu	µg/L	28.00	37.00	42.00	28.00
Cr	µg/L	4.00	32.00	31.00	11.00
Ni	µg/L	10.00	17.00	30.00	15.00
Cd	µg/L	6.00	26.00	5.00	10.00

Source: Local stormwater studies conducted by DID in Malacca, Damansara, Penang and Kajang

The load estimated by EMC method is

$$L = R.EMC \, . A.C_v \; / \; 100 \tag{3.2}$$

where,

L = Annual pollutant load (kg/year);
R = Mean annual rainfall-MAR (mm/year);
EMC = Event mean concentration (mg/L);
A = Catchment area (ha); and
C_v = Area-weighted volumetric runoff coefficient for the whole catchment (Table 2.5).

3.3 POLLUTANT CONTROL

Stormwater pollutants can be particulates, dissolved substances and floatables (litters and hydrocarbons). Physicochemical nature of each group of pollutants is different. Therefore, mechanism to remove them from the water is also different, which are briefly discussed below.

3.3.1 Particulate Pollutants

Particulates are the primary pollutants found in stormwater and are removed through sedimentation and filtration. Sedimentation occurs when particles have a greater density than the surrounding liquid. Under laboratory quiescent conditions, it is possible to settle out very small particles; the smallest practical settling size in the field is around 0.01 mm (Metcalf & Eddy, 1979). Sometimes, the smallest particles become electrically charged, which can further interfere with their ability to settle out. The Newton's and Stoke's laws are often used to quantify the sedimentation process. More information on sedimentation theory, including Stokes' and Newton's laws, can be found in other references (Hazen, 1904).

Particulates can also be removed by filtration through use of infiltration facilities. The mechanism is different from sedimentation process, where media is used for example sand, to increase the efficiency of the pollutant filtration process. Excellent removal of suspended solids in field bioretention systems has been reported elsewhere (Li and Davis, 2009).

3.3.2 Dissolved Pollutants

Dissolved pollutants are mainly removed through adsorption, microbial fixation, biochemical degradation and plant uptake (phytoremediation). However, the dominancy of the process mechanism depends on the type of BMPs. Soil or plant-based media is required to remove the dissolved pollutants (Metals, TN, TP, BOD, COD, pesticides, herbicides, etc.).

Two most common BMPs used to remove dissolved pollutant from stormwater are wetlands and bioretention facilities. Wetlands support various types of microphytes and macrophytes that either absorb or adsorb dissolved pollutants from the storm runoff. On the other hand removal of dissolved pollutants by bioretention facility is mainly done by media and plant uptakes. This is because due to the dry nature of the bioretention system, the number of microbial aquatic organism (e.g. algae) will be less, which plays important role in removing micronutrients of low concentrations.

3.3.3 Floatable Pollutants

Floatable pollutants in the storm runoff can be in the form of litters (refuse or trash) or oil and grease (floating hydrocarbon). The litters are floating or submerged particles large enough to be removed by screen, trash rack, nets and other mechanical devices. Opening of the litter trapping devices depend on the size of the particles to be trapped. However, fine screens pose high potential to choke or clog the drainage system. Therefore, captured floatable pollutants should be removed from the gross pollutant trap at the soonest possible time, preferably before the next storm occurs.

The removal of floating oil and grease (hydrocarbon) is based upon the rise rate velocity of oil droplet and rate of runoff. However, with the exception of stormwater from oil refineries there are no data describing the characteristics of petroleum products in urban stormwater that are relevant to design; either oil density and droplet size to calculate rise rate or direct measurement of rise rates. Further, it is known that a significant percentage of the petroleum products are attached to the fine suspended solids and therefore are removed by settling (API, 1990).

3.4 BMPs FACILITIES

This Section provides types, treatment objectives and guidance for the selection of a stormwater treatment system (BMPs). They should be carefully selected based on site-specific conditions and the overall management objectives of the catchment.

The selection process has the following aspects:

- Identification of problems and treatment objectives;
- Familiarisation with capabilities of the alternative treatment devices and treatment mechanisms;
- Familiarisation with constraints on use of various treatment devices; and
- Cost considerations.

The advantages or disadvantages of each BMPs facility, constraints on use and some factors likely to affect cost are summarised.

3.4.1 Types

Only the more universally accepted BMPs types are recommended for present use. They are infiltration facilities, bioretention facilities, swales, gross pollutant traps and water quality pond and wetlands. It is to be expected that more newer and improved BMPs facilities will be developed worldwide and the designer should keep up-to-date with these in the future.

(a) Infiltration Facilities

Infiltration devices can take a number of structural forms including pits, trenches, or basins. All of these devices work by storing stormwater flow and promoting infiltration into the soil. They are primarily for removing soluble and fine materials from stormwater.

(b) Bioretention Facilities

These devices use a filtering action to remove pollutants, mainly particulate material. Two types of filtration are used: biofiltration, using biological methods, and media filtration through porous media such as sand. Bioretention facilities are good for abstracting nutrients and fine colloidal particles, which are difficult to be retained in sediment basins and water quality ponds.

(c) Gross Pollutant Traps

Gross pollutant traps (GPTs) remove floating and submerged gross litter, hydrocarbons and coarse solids. They can be either built in-situ or pre-fabricated. The GPTs mainly help improve the visual quality of the storm runoff. However, these facilities require frequent maintenance for them to be performed to meet the designed efficiency. Choked GPTs in the drains may cause nuisance flooding in the locality.

(d) Swales

Swales are vegetated drains that can be used to convey and filter runoff. They take advantage of biological processes to improve pollutant removal from the storm runoff. Biological controls provided by swales are typically cheaper and have better aesthetic than structural controls, but may involve more maintenance and landtake.

(e) Water Quality Pond and Wetlands

Water quality ponds have a beneficial effect on stormwater quality treatment by controlling the volume of runoff and providing treatment by gravity settling, biological stabilisation of soluble pollutants such as nutrients, and adsorption and decomposition of biodegradable pollutants such as BOD and light oils. The function of wetlands is similar to that of ponds, except that areas of active vegetation growth are the main component of wetlands instead of open water. This promotes biological action in preference to sedimentation. In practice, most water quality ponds contain wetlands areas and vice versa so it is convenient to consider the two types together.

3.4.2 Treatment Objectives

The primary objective of the treatment BMPs in this manual (with the exception of oil separators) is to remove total suspended solids and sediment-bound compounds. Removal of TSS will result in the removal of many of the contaminants of concern, including:

- Particulate trace metals;
- Particulate nutrients; and
- Oil and grease on solids.

The overall objective of removing 80% of the TSS load (on a long-term average basis) in a catchment has been adopted in this Manual (refer to Chapter 1). A higher degree of removal would require a large increase in treatment device size and cost. In some retrofitting cases in older developed catchments, it may not be possible to meet the 80% removal objectives due to space limitations. For oil and grease removal, the objective of treating over 90% of the total annual runoff is proposed.

3.4.3 Selection

The permanent facilities such as infiltration basins, swales, GPTs, bioretention facilities, water quality ponds and wetlands are to be provided to take care of the runoff laden pollutants generated from the developed areas. Table 3.3 provides a rating from low to high for each treatment BMPs type against its removal efficiency and other factors such as maintenance and cost. Consideration should be given to the intended catchment area to be treated, as some systems such as wetlands take up considerable amounts of land.

Table 3.3: Selection of BMPs for Various Pollutants (Adapted from Melbourne Water, 2005)

BMPs Type	Pollutant Removal Efficiency			Other factors			
	Gross Pollutants	TSS	Nutrient (TN & TP)	Maintenance	Land Required	Treatable Catchment Area	Cost
Infiltration	Low	High	High	Medium	High	Medium	Low
Bioretention	Low	High	High	Low	Medium	Medium	Medium
Swale	Low	Medium	Medium	Low	Low	Low	Low
GPT	High	Low	No	Medium	Medium	Medium	Medium
Water Quality Pond	Medium	Medium	Medium	Medium	High	High	High
Wetlands	Medium	High	High	High	Medium	High	High

Note: In order to remove the desired contaminants for the application being considered, a composite system incorporating numerous design elements may be required. This is commonly referred to as a 'treatment train'. In this instance, an order of suitability can be obtained by referencing various combinations of design elements and their associated pollutant treatment ability and other factors as listed above. The higher the level, the more suitable that combination of design elements will be.

The definitions of low, medium and high capture or treatment target for each of the BMPs for TSS and nutrients is given in Table 3.4.

Table 3.4: Classification of Treatment Targets for Individual BMPs (Adapted from Melbourne Water, 2005)

Pollutant	Target of Treatment		
	Low	Medium	High
TSS	Less than 40% of particulates greater than 0.125mm retained.	40-70% of particulates greater than 0.125mm retained.	More than 70% of particulates greater than 0.125 mm retained.
Nutrients (TN & TP)	Less than 10% reduction	10-40% reduction	More than 40% reduction

Note: There is insufficient data available to give firm guidance on gross pollutants. The level of treatment chosen has an effect on the area required for the BMPs. However, in all situations, the overall pollutant reduction by any BMPs or treatment train (at least) ***must comply*** with the criteria given in Table 1.4.

Suitability of BMPs also depends on the type and size of the development and its associated landuse. Table 3.5 provides a guide to select the most appropriate BMPs for different development types while Table 3.6 provides commentary on benefits and siting considerations for different BMPs elements.

Table 3.5: Suitability of BMPs for Various Landuses (Adapted from City of Knox, 2002)

Development Types/Landuse	Bioretention	Swale	Wetlands	Water Quality Pond	Infiltration	GPT
New Roads and Highways						
- On slopes less than 4%	High	High	High	High	High	High
- On slopes greater than 4%	Medium	Medium	Medium	Medium	Medium	Medium
Old Roads and Highways						
- On slopes less than 4%	High	Medium	High	High	Medium	High
- On slopes greater than 4%	Medium	Low	High	Medium	Low	High
New Residential/Commercial/ Industrial Development	High	High	High	High	High	High
Old Residential/Commercial/ Industrial Development	Medium	Low	Low	Medium	High	High

3.5 PRELIMINARY SIZING OF BMPs

Removal curves are presented in this Section for the preliminary estimate of BMPs sizes. They are for swale, water quality pond and wetlands application only. The simplified procedure is used for the purpose of planning submission to a regulatory local authority for initial assessment. The "planning application process" usually entails an applicant submitting to a local regulatory authority the prepared plans and associated documentation that outline the intent and extent of the proposed development. Detailed design is not undertaken at this stage, as the authority is likely to provide recommendations or conditions during the initial assessment. Local regulatory authority can, therefore, also use the preliminary design outcomes to assess submissions based on the methodology proposed in this section.

This process is by no means intended to replace the more rigorous processes outlined in the design Chapters of the Manual, and as such should not be used to determine final treatment sizing or efficiency. Designers should

Table 3.6: Treatment Application Guide (Landcom, 2004)

BMPs	Potential Benefits	Suitable Site Condition	Unsuitable Conditions
GPT	Reduces litter and debris Can reduce sediments Pre-treatment for other measures	Conventional drainage systems	Sites Larger than 100 ha Natural channels
Infiltration	Reduce runoff Pollutant removal Passive irrigation	Sandy to sandy-clay soils (k more than 36mm/hr) Flat terrain (less than 2%) Deep groundwater table	Silty clay to clay soils Steep terrain Shallow groundwater table Highly polluted runoff
Swales	Medium and fine particulate removal Passive irrigation	Mild slopes (less than 4%)	Steep slopes (more than 4%)
Bioretention	Fine and soluble pollutants removal Small ARI Frequent flood retardation	Flat terrain	Steep terrain High groundwater table
Water Quality Pond	Storage for reuse Fine sediment removal Flood retardation Community and wildlife asset	Steep terrain with confined valleys	Proximity to airports, landfill High groundwater table
Wetlands	Community asset Medium to fine particulates and some soluble pollutant removal Wildlife habitat	Flat terrain	Steep terrain High groundwater table Acid sulphate soils

recognise that local factors may strongly influence the results from the process and take these into account in their final decision in sizing the BMPs.

The preliminary BMPs sizing procedure is given below:

Step 1: Identify the main non-point source (NPS) pollutants that would be generated by the proposed development (Table 3.3).

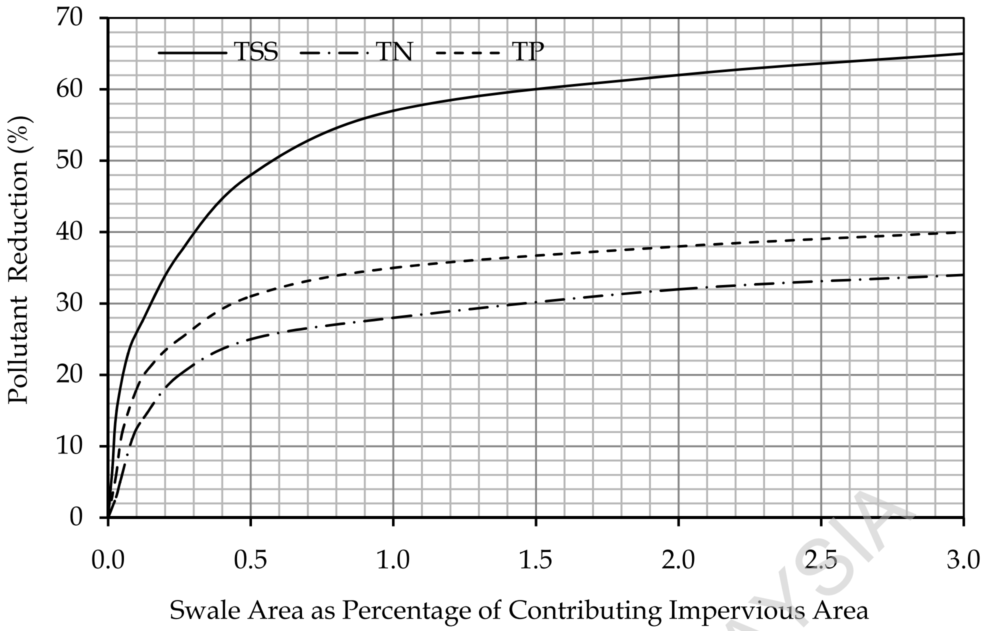
Step 2: Choose the BMPs best suited to the target the NPS pollutants (Step 1) and landuse (Table 3.5).

Step 3: Choose the level of treatment and reduction target required for the BMPs (Table 1.4).

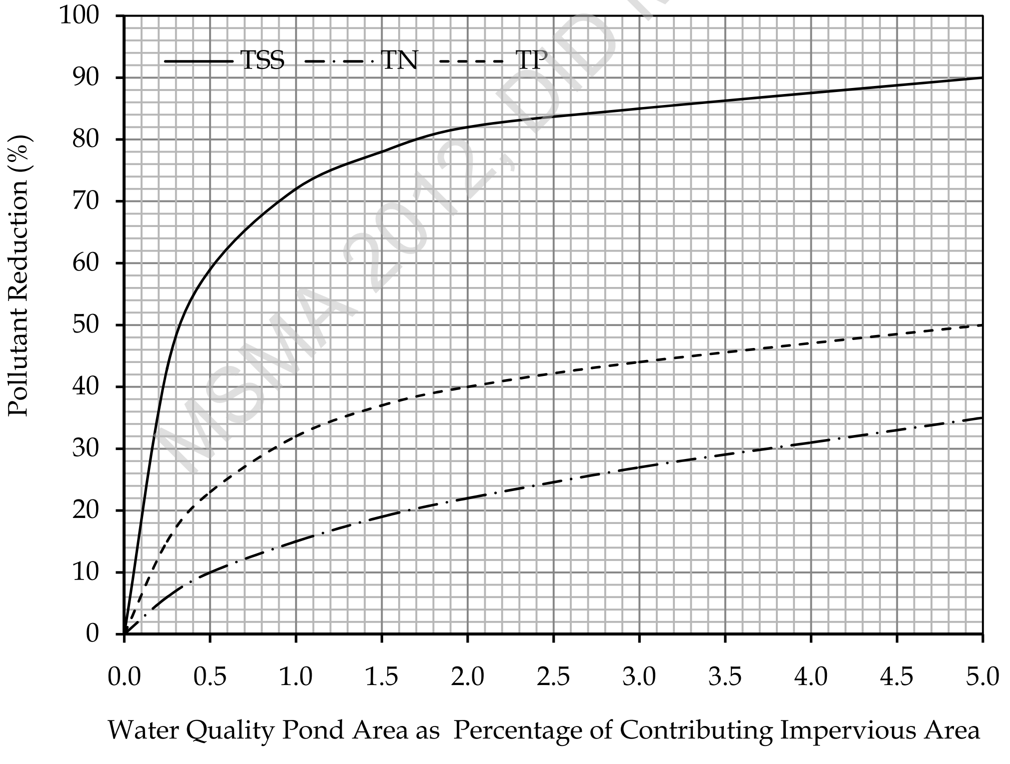
Step 4: Work out the BMPs area required by reading the target pollutant reduction value from the Y-axis of the relevant chart. Then move horizontally right and meet the curve of the target pollutant. Then move vertically down and read the BMPs/Catchment Impervious Area ratio in percentage (%) from the appropriate chart in Figure 3.1. Estimate the BMPs area by multiplying the BMPs/Catchment Impervious Area ratio by the estimated impervious area (in ha) of the catchment and dividing by 100. If the impervious area is not available, a relevant runoff coefficient (Table 2.6) appropriate for the target landuse can be used instead of the imperviousness (%).

Step 5: Choose the appropriate layout plans and guideline drawing(s) and for the proposed BMPs.

Step 6: Collate the results and submit the information to the local regulatory authority for initial approval.

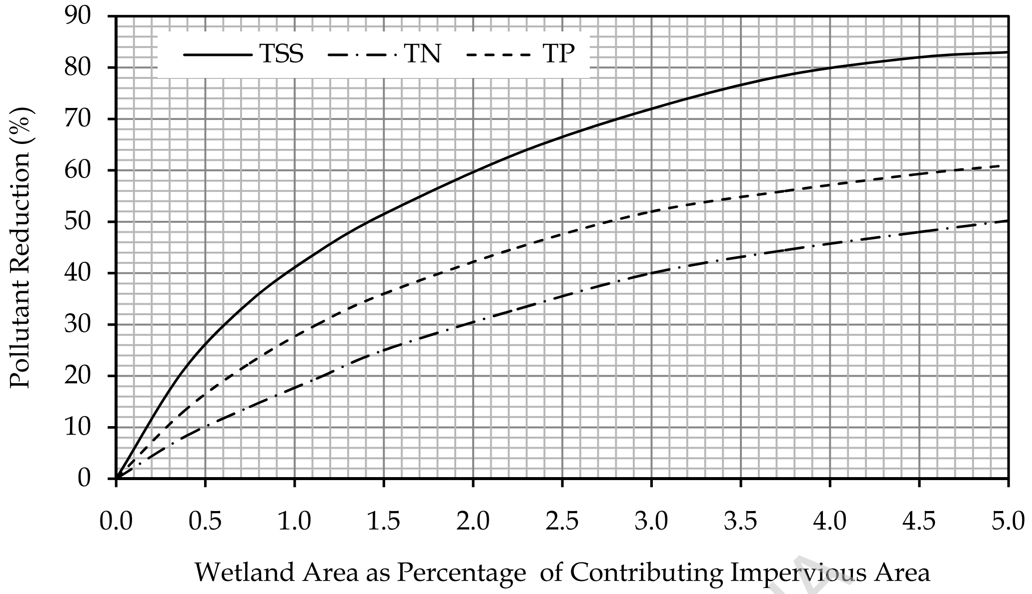


a) Swale



b) Water Quality Pond

Figure 3.1: Pollutant Reduction Curves (Adapted from Melbourne Water, 2005 and Darwin Harbour, 2009)



c) Wetlands

Figure 3.1: Pollutant Reduction Curves (Adapted from Melbourne Water, 2005 and Darwin Harbour, 2009)

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APPENDIX 3.A EXAMPLE - POLLUTION LOAD FROM MULTIPLE LANDUSES

Problem:

Determine the annual pollution loading (in tonnes/yr) generated from a 753.28 ha mixed development area located in upstream of Sg. Buloh catchment shown in Figure 3.A1. The mean annual rainfall for the catchment is 2850 mm.

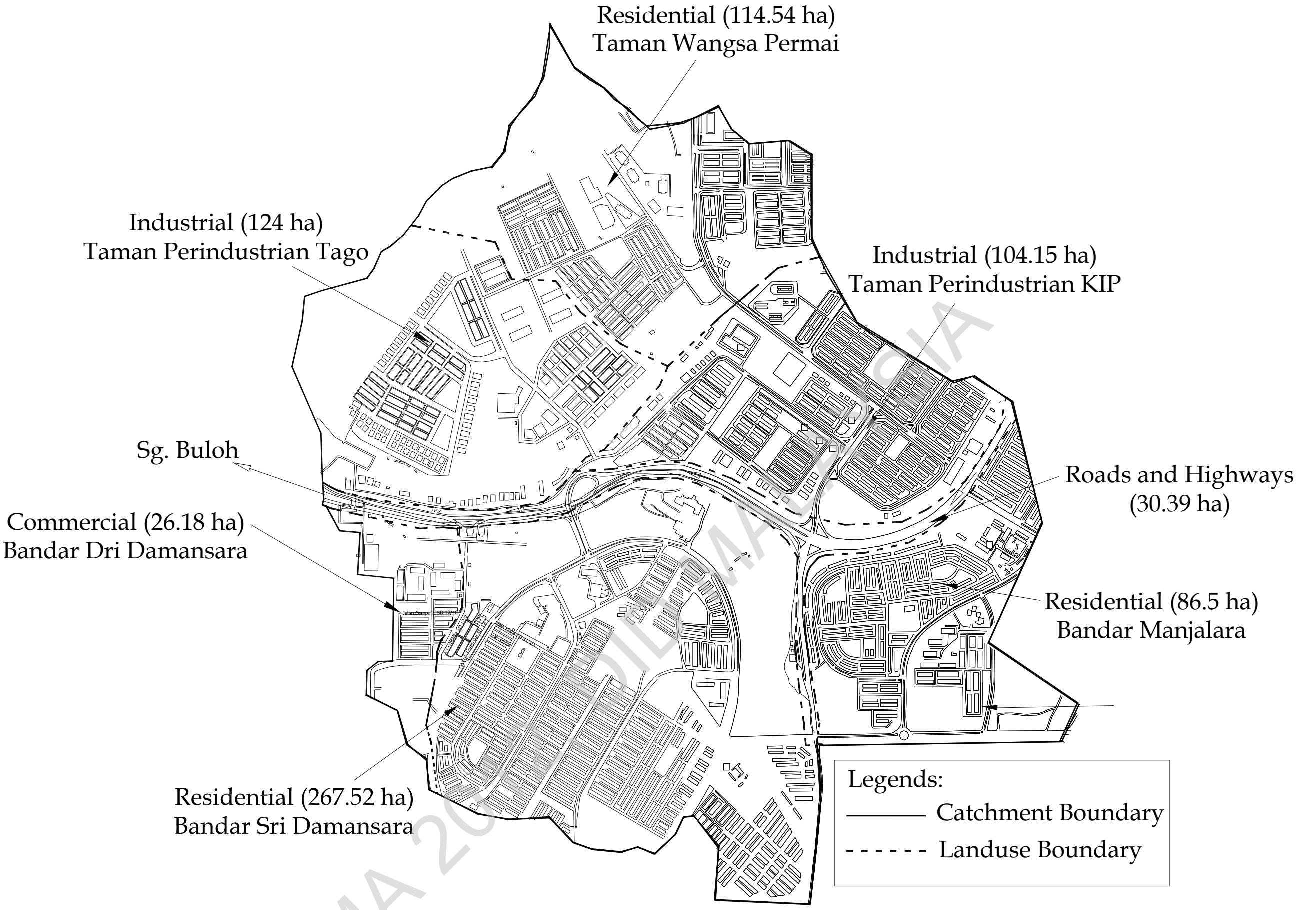


Figure 3.A1: Catchment Area for the Calculation of Pollution Load

Solution:

Reference	Calculation	Output
Table 2.5 Equation 2.4	Step 1: Get the mean annual rainfall (MAR) data for the catchment (in this case, 2850 mm) and multiply with the corresponding runoff coefficient to calculate the annual runoff from each landuse.	Table 3.A1
Table 3.2 Equation 3.2	Step 2: Pick the EMC values for each pollutant corresponding to the landuse and use Equation 3.2 to calculate the pollutant loading. The results are summarised in Table 3.A1.	Table 3.A1

Reference	Calculation	Output
	Step 3: Calculate the total annual pollution load for TSS, TP and TN in tonne per year. They are summed as follows: TSS = TSS from Residential + Industry + Commercial + Road and Highway = 1299.07 + 971.44 + 81.93 + 65.84 =	2418.28 tonne/year
	Similarly TP = 3.45 + 2.87 + 0.22 + 0.13 =	6.67 tonne/year
	TN = 42.73 + 29.26 + 3.25 + 1.85 =	77.09 tonne/year

Table 3.A1: Calculation of Pollution Loading from Various Catchments

Landuse	Area (ha)	Volumetric Runoff Coefficient (C_v)	Pollutant	EMC (mg/L)	Annual Runoff (mm)	Annual Loading	
						(kg/year)	(tonne/year)
Residential	468.56	0.76*	TSS	128	2166	1299073	1299.07
			TP	0.34		3451	3.45
			TN	4.21		42727	42.73
Industry	228.15	0.90	TSS	166	2565	971440	971.44
			TP	0.49		2868	2.87
			TN	5.00		29260	29.26
Commercial	26.18	0.90	TSS	122	2565	81925	81.93
			TP	0.32		215	0.22
			TN	4.84		3250	3.25
Roads and Highways	30.39	0.95	TSS	80	2708	65837	65.84
			TP	0.16		132	0.13
			TN	2.25		1852	1.85

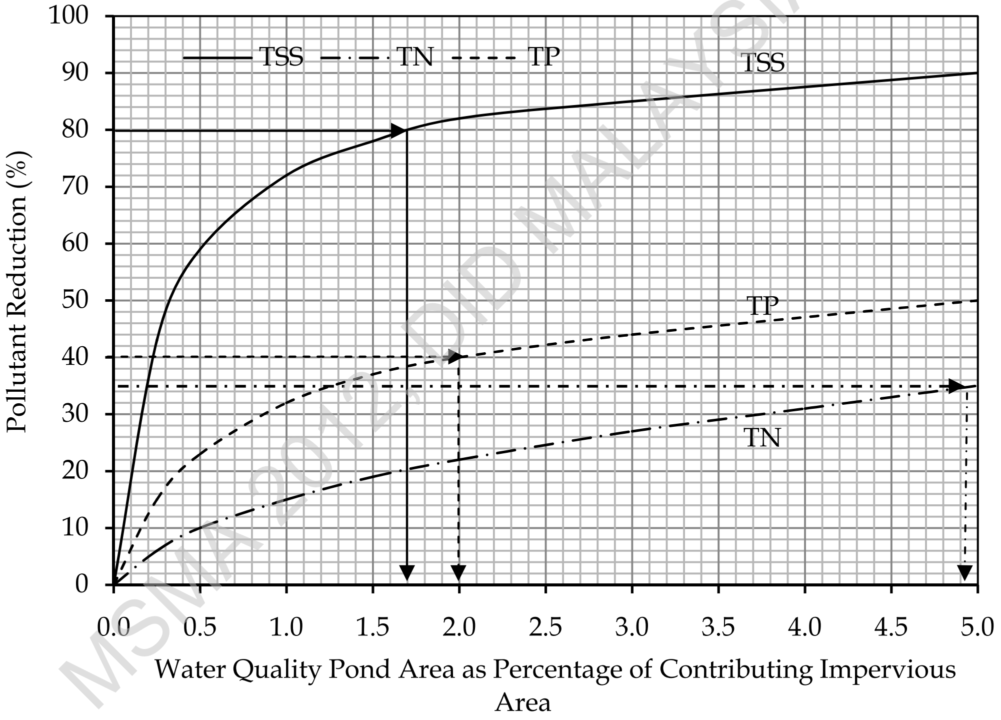
* Area-weighted average volumetric runoff coefficient for the residential areas (using Table 2.5 and Equation 2.4).

APPENDIX 3.B EXAMPLE - PRELIMINARY SIZING OF WATER QUALITY POND

Problem:

Estimate the preliminary size of a water quality pond required to reduce the TSS, TN and TP by 80%, 35% and 40%, respectively from the residential area (Taman Wangsa Permai, 114.54 ha), as given in Example 3.A (Figure 3.A1). The average runoff coefficient of the area is 0.55, which was calculated based on the actual imperviousness of the mixed development residential area (e.g. Bungalow, Link House and Open Spaces).

Solution:

Reference	Calculation	Output
Figure 3.1b	Step 1 - Calculate the contributing impervious area as 0.55 x 114.54 ha =	63 ha
	Step 2 - Identify the percentage of pollutant needed to be removed by the proposed pond and draw the horizontal lines as shown below:  	
	Step 3 - Determine the ratio of Pond Area/Contributing Impervious Area ratio in %, from the X-axis as shown above, which are:	
	For TSS =	1.7%
	For TN =	4.95%
	For TP =	2.0%
	Step 4 - Now the preliminary pond area must be determined based on the maximum area required which is 4.95% for the TN removal. Therefore, the water quality pond area should be,	
	(4.95 x 63 ha) /100 =	3.12 ha.

APPENDIX 3.C EXAMPLE - PRELIMINARY SIZING OF TREATMENT TRAIN

Problem:

Estimate the preliminary sizes of a BMPs treatment train using swale, wetlands and water quality pond shown in Figure 3.C1, to reduce TSS by 80% from a residential subcatchment, Taman Bukit Rahman Putra located in Upper Sg. Buloh Catchment, with an area of 14.46 ha (Figure 3.C1). The average contributing imperviousness of the residential area is 75%.

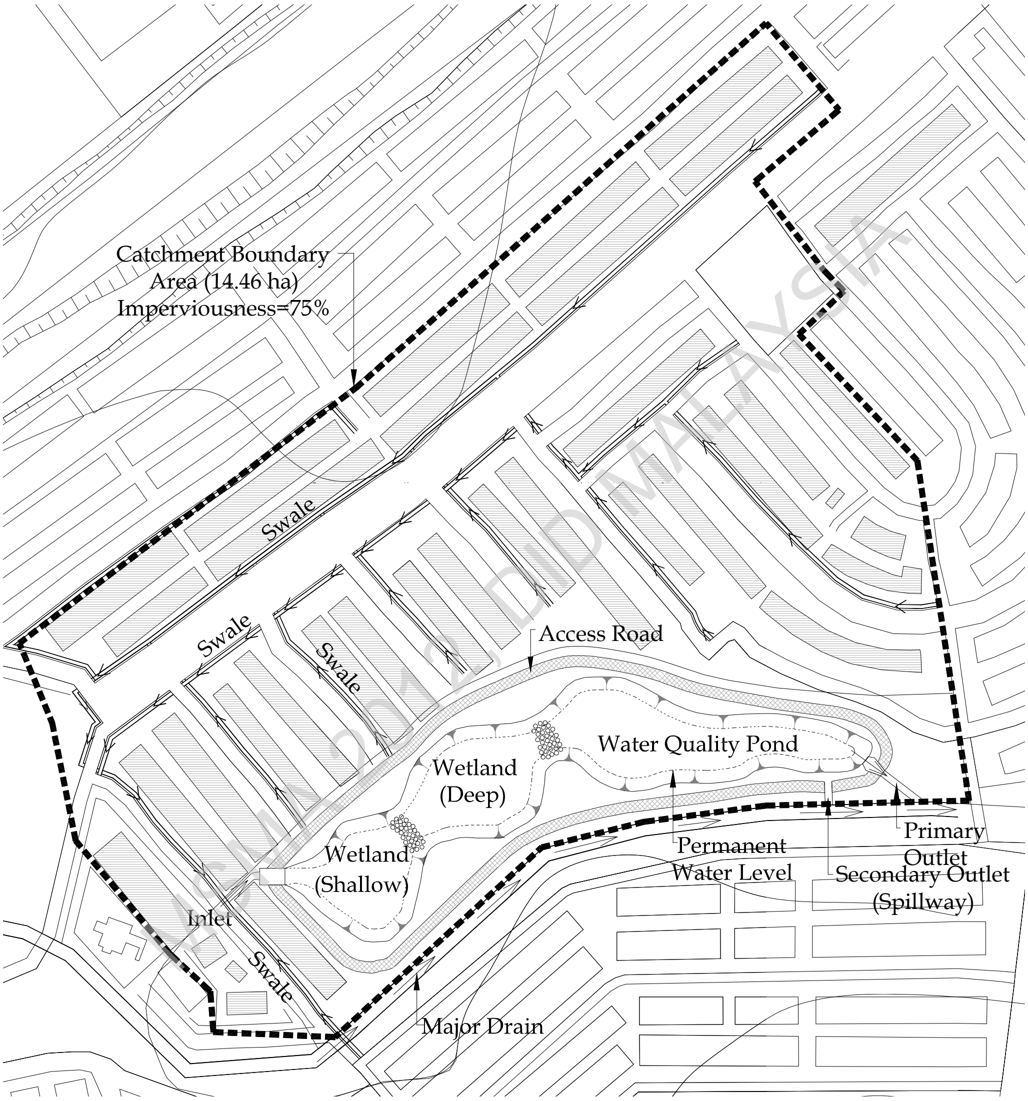
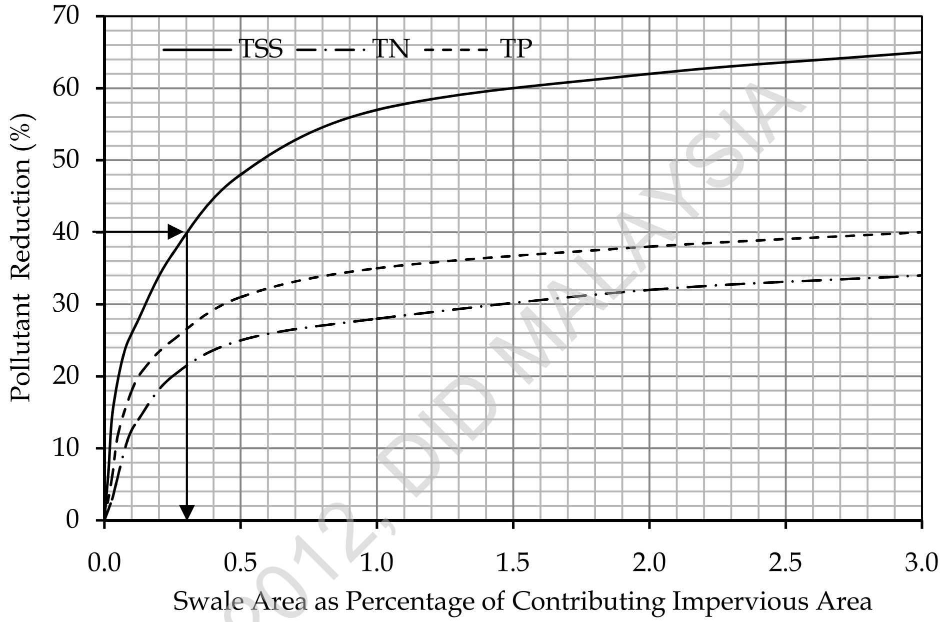
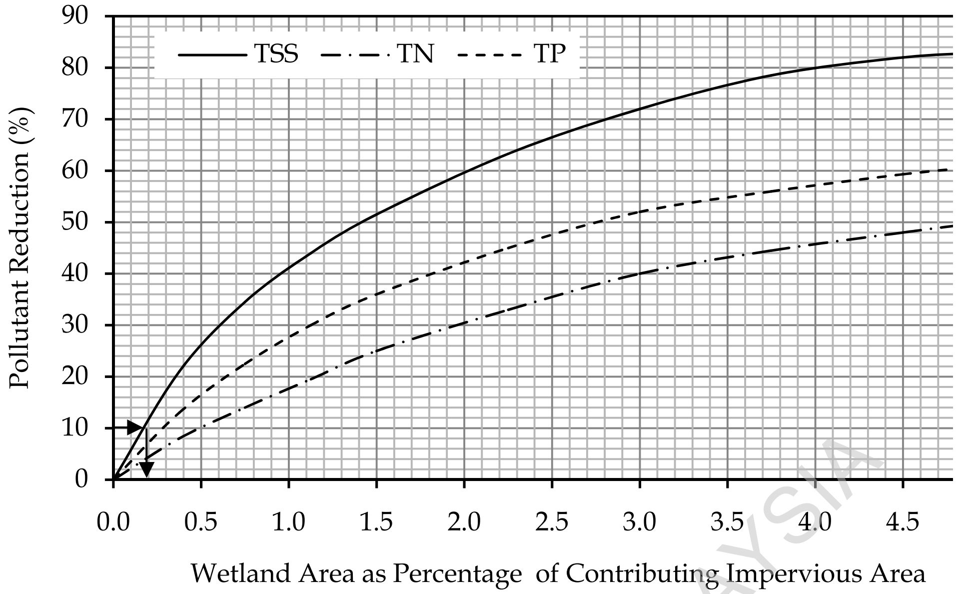
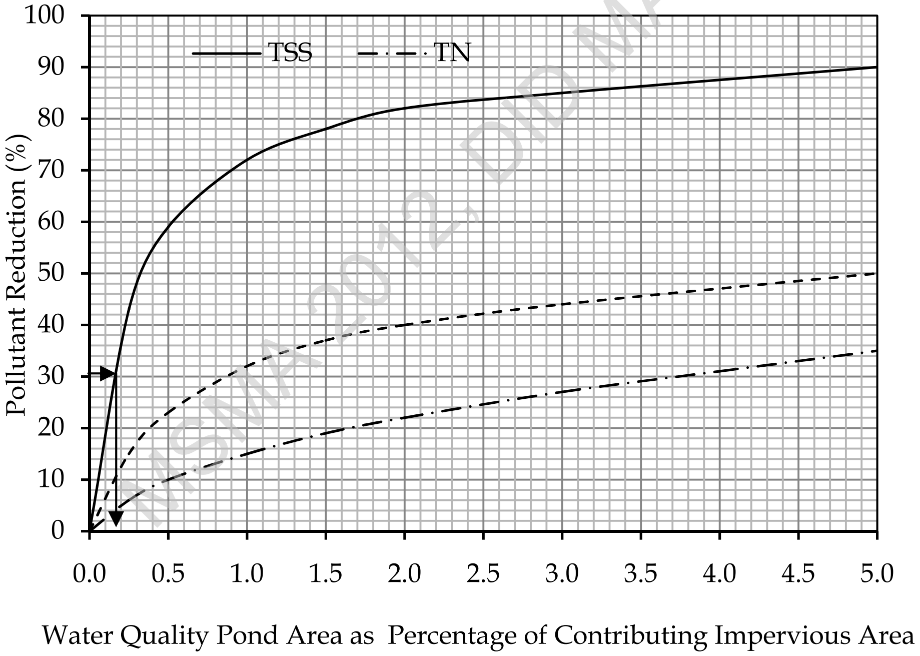
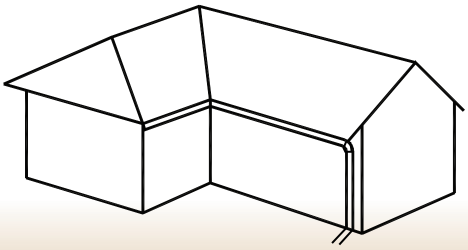


Figure 3.C1: Layout of the Proposed of BMPs Treatment Train

Solution:

Reference	Calculation	Output
Table 3.3 and 3.4	Step 1: Assuming TSS removal within the treatment train is distributed as follows : Swale (40%) ⟶ Wetlands (10%) ⟶ Water Quality Pond (30%)	
Figure 3.1a	Step 2: Top widths of the swales are assumed to be 3 m wide, including the drainage reserves that act as buffer strip. Now the total area of the swales required to remove 40% of the TSS can be determined using Figure 3.1a, which is shown below:  So, the total swale area required is 0.30% of the contributing impervious area,	
	= 0.30/100 x 14.46 x 0.75 = 0.033 ha =	330 m^2
	Swale length (for width 3m) =	110 m
Figure 3.1c	Step 3: The next step is to estimate the size of wetland to remove 10% of the TSS. A preliminary size of the wetland is estimated from the Figure 3.1 (c). The required wetland area to remove 10% of the TSS is 0.20%, i.e.	
	Wetlands area = 0.20/100 x 14.46 x 0.75 = 0.022 ha =	220 m^2
	The area is divided into two units of wetlands, as shown in Figure 3.C1. One is for shallow pool (100 m^2) and the other is for deep pool (120 m^2) with submerged macrophytes.	
Figure 3.1b	Step 4: The remaining 30% of the TSS needed to meet the removal target of 80% will be removed by the water quality pond, which is the last component of the proposed treatment train BMPs. Preliminary size of the pond is estimated from the Figure 3.1 (b). The required water quality pond area to remove 30% of the TSS is 0.18%, i.e.	
	Water quality pond area = 0.18/100 x 14.46 x 0.75 = 0.020 ha =	200m^2

Reference	Calculation	Output
	  Summary and flow of the estimated BMPs sizes are as follows: Swale (330m²) ⟶ Wetland 1 (100m²) ⟶ Wetland 2 (120m²) ⟶ Water Quality Pond (200m²)	



ROOF AND PROPERTY DRAINAGE 4

CHAPTER 4 ROOF AND PROPERTY DRAINAGE

4.1 INTRODUCTION

Property drainage refers to the systems that transfer runoff from roofs, paved areas and other surfaces of a premise to a suitable outlet or disposal facility. The system involves gutters, downpipes, drains, pipes, swales and storage and treatment facilities. Typical property drainage components for residential and industrial premises are shown in Figure 4.1.

Local authorities may place limitations on the amount of stormwater that can be drained to streets or trunk drainage systems, in order to reduce flooding and pollution. In these cases it is the responsibility of the property owner to provide infiltration or on-site detention facilities.

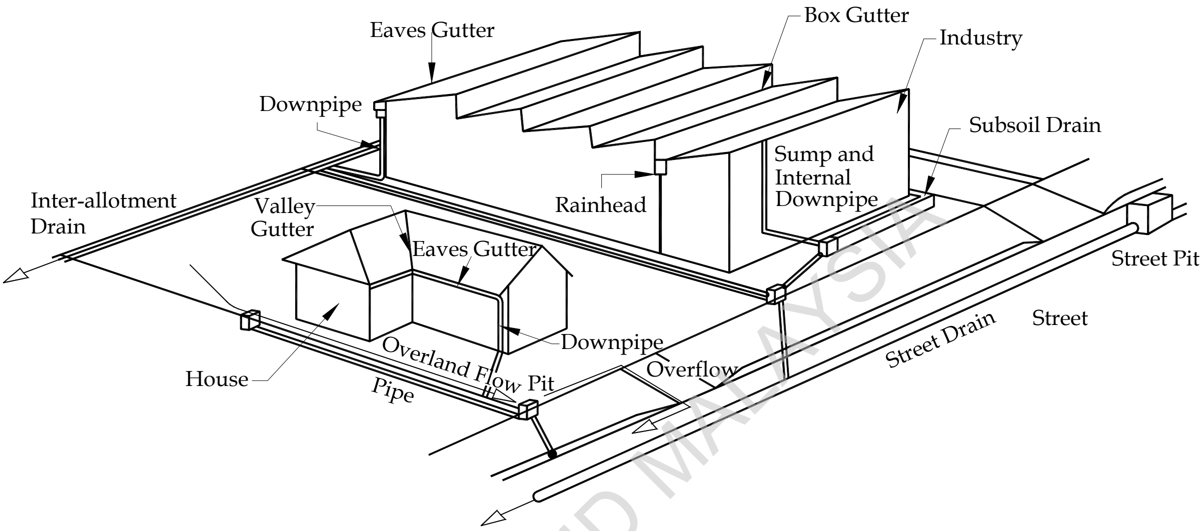


Figure 4.1: Typical Property Drainage Components

4.2 ROOF DRAINAGE SYSTEM

Roof drainage systems are located at the top of property drainage systems. Because the areas are usually small and there are fewer complications, roof drainage can be designed using simpler methods than those employed at larger scale drainage. This chapter applies to those buildings where roof drainage is specified for reasons of runoff conveyance and collection to storage/detention facilities as well as for comfort and safety of occupants and the protection of the building structure. The rules also apply to all inhabited buildings as well as industrial buildings and warehouses.

The methods given in this section are based on the Australian/ New Zealand Standard (AS/NZS 3500.3, 2003), adapted for Malaysian conditions. Eaves gutters are located on the outside of a building while box gutters and valley gutters are located within the plan area of the building and the intersecting sloping surfaces of a roof respectively (Figure 4.2).

a) Eaves

b) Box

Figure 4.2: Roof Gutters

4.3 ROOF DRAINAGE DESIGN PROCEDURE

4.3.1 Catchment Area

The design approach for drainage of roofs is to determine the layouts and sizes of components, and then to analyse their behaviour in one or more design storms that will test the adequacy of the system.

The design procedure herein follows AS/NZS 3500.3 in recognising that wind causes the rain to slope, creating a horizontal component of rainfall, which becomes significant on vertical walls or sloping roofs. The direction of wind that results in the maximum roof catchment area should be selected. It is not sufficient to consider only the direction of prevailing winds. A maximum rainfall slope of 2 vertical to 1 horizontal is assumed. The roof catchment area (A_c) estimates, based on Figure 4.3, are given below:

For single sloping roof fully exposed to wind

$$A_c = A_h + \frac{A_v}{2} \qquad (4.1)$$

where the meaning of terms is as shown in Figure 4.3. If the roof is partly shielded by another wall, the net vertical area A_v is the area seen by looking in the same direction as the wind. The following formula is used

$$A_c = A_h + \frac{1}{2}(A_{v_2} - A_{v_1}) \qquad (4.2)$$

For the two adjacent sloping roofs

$$A_c = A_{h_1} + A_{h_2} + \frac{1}{2}(A_{v_2} - A_{v_1}) \qquad (4.3)$$

Roofs of larger buildings may have complex arrangements of catchments and drainage systems. Where it is difficult to define catchments, a conservative approach should be adopted, assuming the largest possible catchments.

4.3.2 Design Average Recurrence Intervals

Roof drainage shall use the ARIs set out in Table 4.1. The critical storm duration of 5 minutes should be adopted for all roofs unless special circumstances justify a longer duration.

Table 4.1: Design ARIs for Roof Drainage

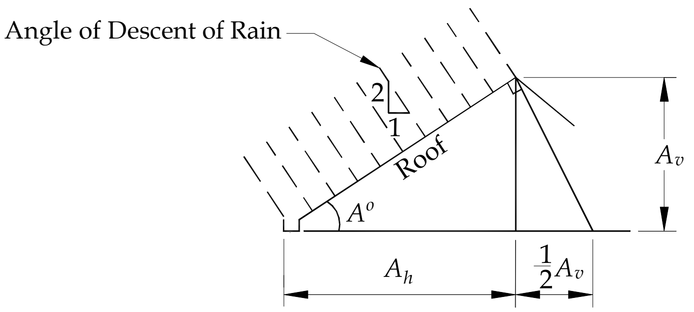
Property Type	Eaves Gutters	Valley and Box Gutters
All buildings	20 year ARI	100 year ARI

Notes :

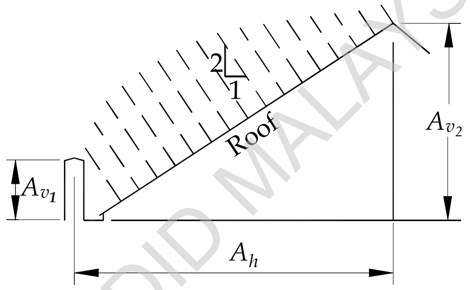
1. If water can flow back into the building, then overflow measures are required
2. A higher design ARI shall be adopted for buildings located on hillside area

4.3.3 Discharge Estimation

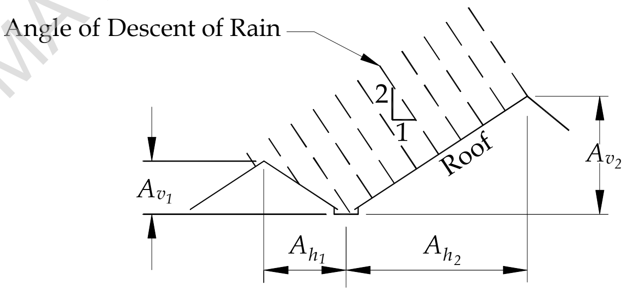
The 5 minute duration 20, 50 and 100 year ARI rainfall intensities for the particular location are obtained from the short-duration rainfall IDF method in Chapter 2, applied at the particular site. If special circumstances justify the use of a longer time of concentration, the rainfall intensity for this time of concentration shall be derived using the methods set out in Chapter 2.



(a) Single Sloping Roof Freely Exposed to the Wind



(b) Single Sloping Roof Partially Exposed to the Wind



(c) Two Adjacent Sloping Roofs

Figure 4.3: Sloped Roof Catchment Area Relationships

The roof flow produced by the design rainfall shall be calculated using the Rational Method, with runoff coefficient C = 1.0, which can be expressed by the following form:

$$Q = \frac{i.A_c}{3600} \tag{4.4}$$

where,

Q = Peak flow (L/s);

i = Rainfall intensity (mm/hr); and

A_c = Roof catchment area draining to a downpipe (m^2).

4.3.4 Design of Eaves Gutters and Downpipes

For a simple sloping (gabled) roof, the eaves gutter (Figure 4.4) should slope from one end to the downpipe location at the other end.

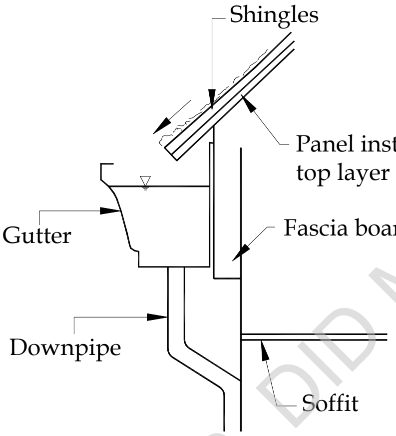


Figure 4.4: Cross Section of Eaves Gutter

The procedure for the design of an eaves gutter is as follows:

- Determine the catchment area to each downpipe;
- Determine the design 5 minute duration, 20 year ARI rainfall intensity; and
- Choose the gutter size from Design Chart 4.A1.

To provide adequate fall and minimise the risk of ponding, the minimum gradient of an eaves gutter shall be 1:500.

The minimum cross-sectional size of an eaves gutter shall be 4,000mm^2 while the normal maximum 22,000mm^2. If calculations indicate that a larger size is required, it is preferable to provide more downpipes rather than increasing the gutter size.

The required size of eaves gutter shall be determined from Design Chart 4.A1. This Chart is derived from Manning's formula with 'n' = 0.016 and S = 1/500. This is a simplified method because the effect of varying flow depth is neglected. When applying the design chart, A_c is the catchment area draining to a single downpipe.

Downpipe size is then determined from Table 4.2 to match the eaves gutter size. Downpipes may be either rectangular or circular. Note that for a given roof catchment area, the size of downpipe will be the same irrespective of the slope of the eaves gutter.

If the listed size is not available, an alternative downpipe with equal or greater cross-sectional area than that shown may be substituted.

Table 4.2: Required Size of Downpipe for Eaves Gutter (AS/NZS 3500.3, 2003)

Eaves Gutter Size (mm²)	Minimum Nominal Size of Downpipe (mm)	
	Circular	Rectangular
4,000	75	65 x 50
4,200		
4,600		75 x 50
4,800	85	
5,900		100 x 50
6,400	90	
6,600		75 x 70
6,700	100	
8,200		100 x 75
9,600	125	
12,800		100 x 100
16,000	150	125 x 100
18,400		150 x 100
19,200	*Not applicable*	
20,000		125 x 125
22,000		150 x 125

4.3.5 Design of Valley Gutters

Valley gutters are located between the sloping roof sections of a hipped roof (see Figure 4.1). The following points should be noted when designing systems incorporating valley gutters:

- Valley gutters should end at the high point of an eaves gutter; and
- The discharge from a valley gutter does not flow equally into both eaves gutters. Therefore the designer should allow at least 20% excess capacity in the sizing of the eaves gutters.

The profile of a valley gutter is shown in Figure 4.5.

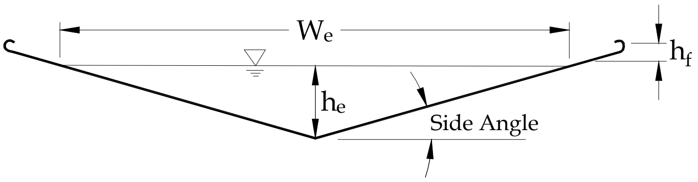


Figure 4.5: Profile of a Valley Gutter

The sizing guidelines in Table 4.3 are valid for the following conditions:

- Roof slope of not less than 12.5°;
- The nominal side angle of valley gutters is 16.5°; and

- The catchment area shall not exceed 20m².

The procedure for the design of a valley gutter is as follows:

- Select the ARI;
- Determine the design 5 minute duration, 100 year ARI rainfall intensity; and
- Choose the girth size and dimensions from Table 4.3.

Table 4.3: Minimum Dimensions for Valley Gutters

Design Rainfall Intensity (mm/hr)	Minimum Dimension (mm)		
	Sheet Width	Effective Depth (h_e)	Effective Width (W_e)
≤ 200	355	32	215
201 - 250	375	35	234
251 - 300	395	38	254
301 - 350	415	40	273
351 - 400	435	43	292
> 400	455	45	311

Notes:

1. Freeboard (h_f) = 15 mm
2. The sheet width from which the valley is to be formed has been calculated on the basis of h_f = 15 mm and an allowance for side rolls or bends of 25 mm.

4.3.6 Design of Box Gutters and Downpipes

Box gutters are located within a building plan area (Figure 4.6). Gutters adjacent to a wall or parapet shall be designed as box gutters. The main principle in the design of box gutters is to avoid the potential for blockages, which would prevent the free runoff of roof water, and possibly cause water to enter the building. The design criteria of box gutter are as the followings:

- Box gutters must be straight (no bends);
- Cross-section shape must have a constant base width and vertical sides;
- Longitudinal slope must be between 1:200 and 1:40. Changes in slope are not permitted; and
- The gutter must discharge directly into a rainhead or sump at the downstream end without change of direction.

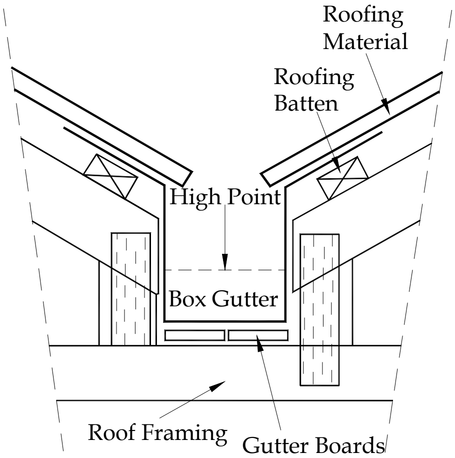


Figure 4.6: Box Gutter

Box gutters are connected to either rainheads or sumps with overflow devices. The minimum width of box gutters for commercial or industrial construction is 300mm. For residential construction, a minimum width of 200 mm is permitted but such gutters are more prone to blockage and should be subject to more frequent inspection and maintenance.

The procedure for design of a box gutter is as follows:

- Determine the catchment area draining to each downpipe (Equation 4.1);
- Determine the design 5 minute duration, 100 year ARI rainfall intensity;
- Select the width and slope of the box gutter to suit the building layout; and
- Read off the minimum depth of the box gutter from Design Chart 4.A3. This minimum depth must be used for the full length of the box gutter. If a sloping box gutter is built with a horizontal top edge for architectural reasons, the minimum depth requirement still applies. When applying the design chart, the catchment area obtained is for draining to a single downpipe.

4.3.6.1 Rainheads and Downpipes

Box gutters shall discharge via a rainhead or sump, to a downpipe. The required size of downpipe from a box gutter is determined from Design Chart 4.A3 (AS/NZS 3500.3, 2003). The sizing principle is to limit the maximum capacity of the downpipe in order to prevent slugs of unstable flow. The graph does not permit very deep, or very shallow rainheads. The minimum depth of water in the rainhead is limited to about half of the diameter of the downpipe. Above this depth, orifice flow conditions apply. A standard rainhead is shown in Figure 4.7. It includes an overflow to safely discharge flow from the box gutter even if the downpipe is blocked. The design flow of a rainhead shall not exceed 16 L/s.

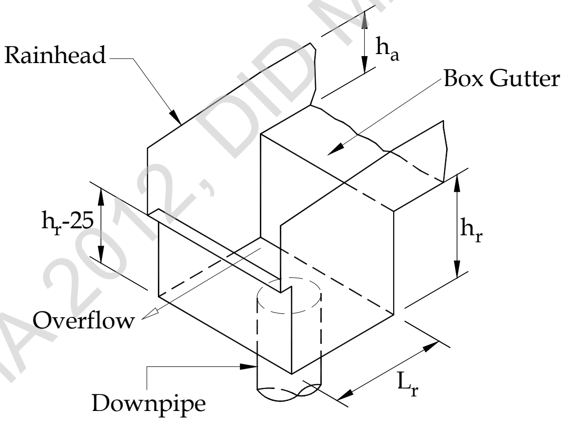


Figure 4.7: Typical Rainhead

Also designers are to observe the followings:-

- Width of rainhead is equal to the width of box gutter. The rainhead must be sealed to the box gutter;
- The depth of a rainhead, h_r must be at least 1.25 x equivalent diameter of a rectangular downpipe ($h_r \geq 1.25D_e$) or 1.25 x internal diameter of a circular downpipe ($h_r \geq 1.25D_i$).
- There is an overflow weir at a lower level than the sole of the box gutter; and
- The rainhead to be fully seated to the box gutter and the front of the rainhead left open above the overflow weir.

4.3.6.2 Sumps

Sumps are located at the low point of a box gutter, which slopes towards the sump in both directions. A standard sump is shown in Figure 4.8. All sumps must be provided with an overflow to prevent overtopping of the box gutter even if the downpipe is blocked. Two types of overflow devices are permitted for use *a) Side Overflow Device*

This device is shown in Figure 4.8. This design has been in use world-wide for many years. It is only suitable for box gutters, which run parallel and adjacent to a parapet wall.

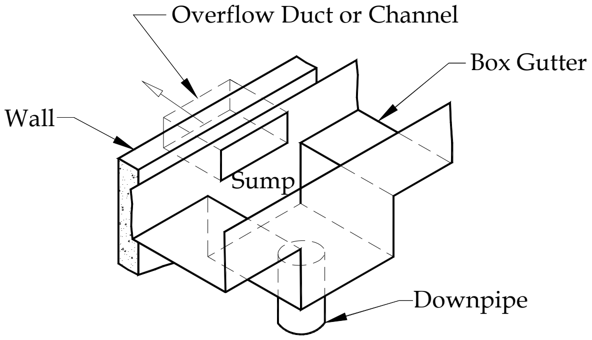


Figure 4.8: Typical Sump and Side Overflow

b) High Capacity Overflow Device

As shown in Figure 4.9 this device design is developed in Australia (Jones and Kloti, 1999). It is anticipated that further research will be conducted into developing overflow devices suitable for Malaysian conditions.

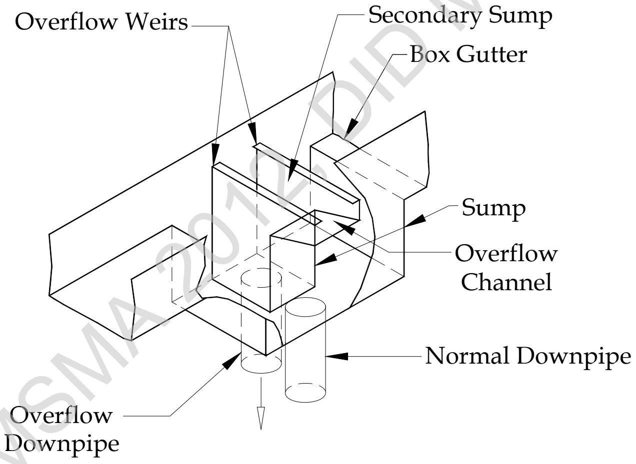


Figure 4.9: Sump with Alternative Overflow Devices

4.4 PROPERTY DRAINAGE

4.4.1 General

The drainage system proposed within allotments depends upon the topography, the importance of the development and the consequences of failure. The drainage systems collect water from roofs (via downpipes), surfaces of areas around buildings, and flows onto the property from adjacent allotments in major storm.

In areas with suitable soils and water table conditions, stormwater may be infiltrated directly into the soil, rather being directed to the street drainage system (Chapter 8).

4.4.2 Design Average Recurrence Intervals

Elements in the property drainage systems shall be designed to contain flows from minor storm events of ARI not less than that specified in Table 4.4.

The property drainage systems shall be designed to ensure that overflows in a major storm event do not present a hazard to people or cause significant damage to property.

Table 4.4: Minimum Design ARIs for Property Drainage

Effect of Surcharge and Overland Flow	ARI (years)
Small impact, in low density area	1
Normal impacts	2
Ponding in flat topography; or flooding of parking lots to depths greater than 150 mm	10
Impeded access to commercial and industrial building	10
Ponding against adjoining buildings; or impeded access to institutional or important buildings (e.g., hospitals, halls, school entrances)	20

4.4.3 Drainage to the Rear of Properties

Where the natural ground level does not permit drainage by gravity to the street drain or gutter, it will be necessary to either fill the site to obtain a fall to the street, or alternatively, to provide a piped drain through an adjoining private property or properties, to discharge the runoff from the site by gravity.

Requirements for piped drainage in privately owned lots are set out in Chapter 15. Open drains shall not be permitted at the rear of private lots because they are difficult to maintain.

Any piped drain in private property, which serves an adjoining property, shall be protected by a drainage easement. Such easements shall be free of any building encroachments, including eaves footing and shall contain a single pipe only. Full details of any proposed easement is to be submitted to the regulatory authority for approval and this easement shall be registered with the authority prior to release of the building plans.

4.4.4 Drainage on Hillside Area

In common practice, although the roofs of buildings are designed to collect storm water, there is no provision to effectively drain them to the perimeter drain surrounding the buildings. The concentrated runoff from the roof eaves is sometimes much higher than the direct impact due to rainwater and this can cause ground erosion.

Buildings in which roof gutters are omitted shall not be permitted in hillside areas. This type of roof drainage would have unacceptable consequences in term of concentrated runoff on potentially unstable hill slopes.

The 20 year ARI standard for roof eaves gutters (Table 4.1) should be increased to 50 year ARI in hillside areas. The standard for box gutters is governed by other factors, and does not change.

Properly designed gutters must be provided to collect stormwater from the roof and convey it to the formal property drainage system, either open drains or pipes. As a general principle, it is desirable to directly connect all significant impervious areas to the lined drainage system.

Property drainage shall be installed at or below ground level, to maximise the interception of surface runoff. The creation of ponding areas due to poor grading of property drainage is not permitted.

4.4.5 Drainage through Public Reserves

Where a low level property adjoins a public reserve, the construction of drainage line through the reserve generally will not be permitted and alternative methods of drainage should be investigated, including:

- construction of a pipeline through an adjoining private property; and
- a pump out system.

Construction of a drainage line through a public reserve may be permitted by the regulatory authority, only in situations where the applicant provides satisfactory proof that the alternatives have been investigated and found to be impractical.

Where drainage through a public reserve is permitted, the applicant is required to enter into a licence agreement with the regulatory authority, subject to the payment of a one off licence fee under the respective agreement covering any installation, legal or other costs associated with the preparation and execution of the licence agreements, together with an amount considered appropriate towards the improvement of the respective reserve.

Pipes on public streets and land that drain developments are permitted, provided that they are built to the approval authority's standards and ownership is transferred to the authority. It is not permitted where the proposed drainage system will cause a conflict with other drainage systems or services.

4.4.6 Rainwater Harvesting and Detention

Rainwater tanks may be provided to collect flow from roof and gutter systems. These tanks can be used to:

- provide water supplies; and/or
- provide on-site detention storage.

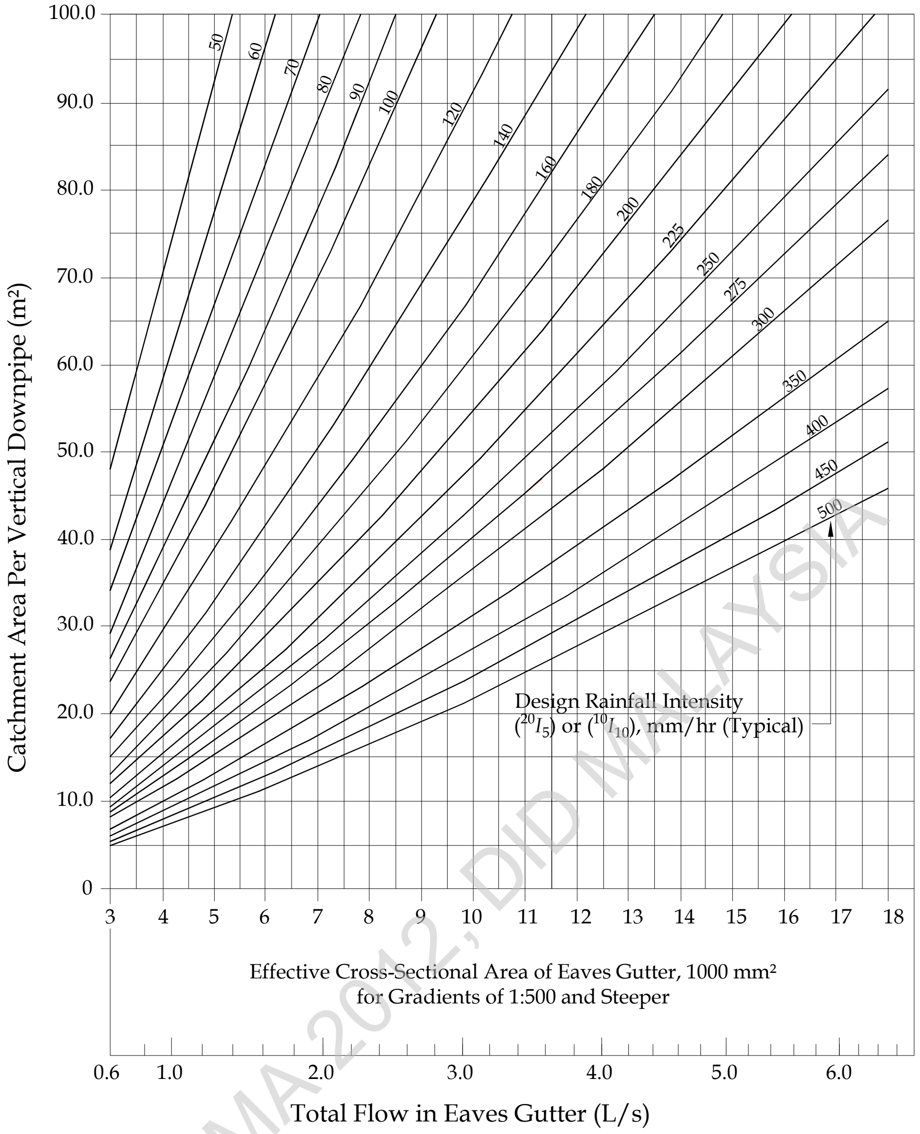
The design of rainwater tanks for water supply is covered in Chapter 6 while the design of on-site detention storage in Chapter 5. Note that any tank volume provided for detention is *additional* to that set aside for water supply storage.

REFERENCES

1. AS/NZS 3500.3 (2003). *National Plumbing and Drainage Code Part 3: Stormwater Drainage – Acceptable Solutions.* Published jointly by Standards Australia and Standards New Zealand.

2. Jones and Klöti (1999). *High Capacity Overflow Device for Internal Box Gutters of Roofs*. 8th International Conference on Urban Stormwater Drainage (ICUSD), Sydney, Australia.

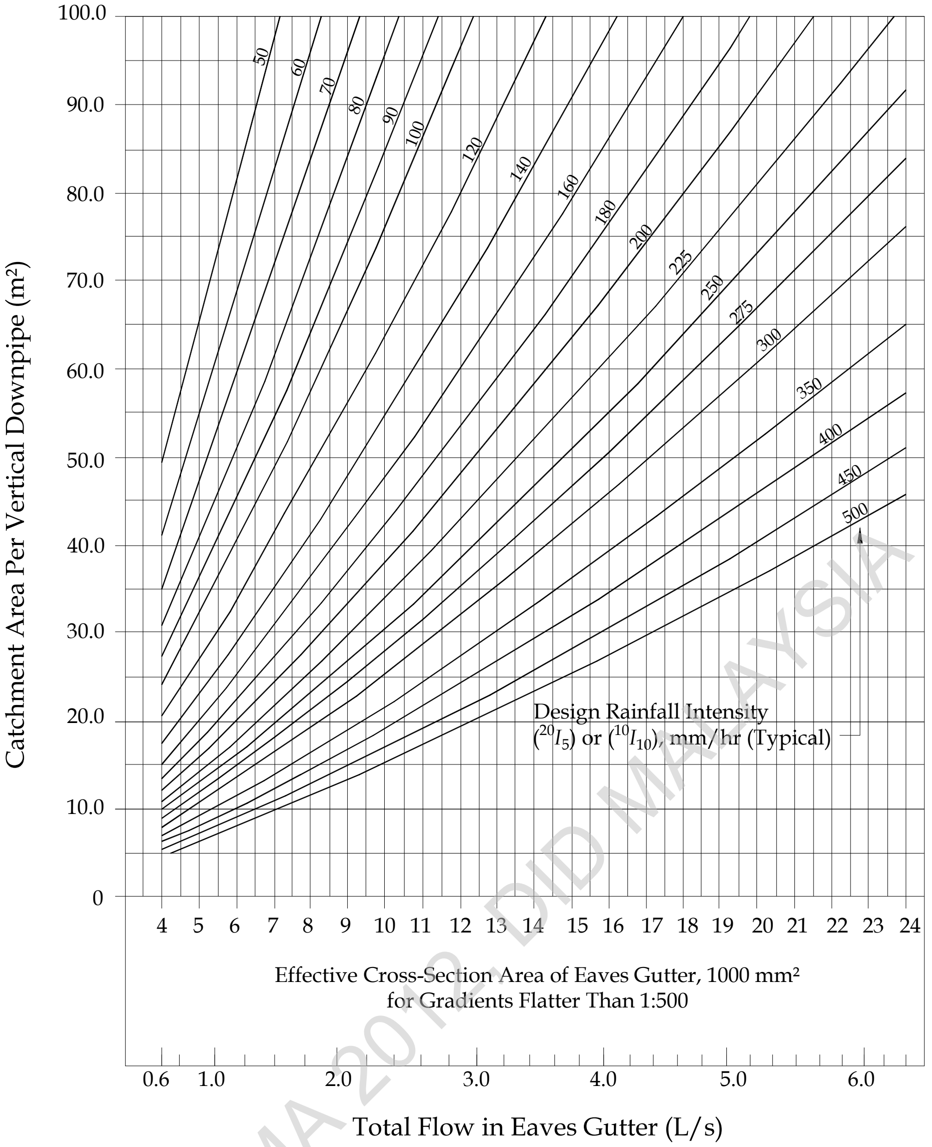
APPENDIX 4.A DESIGN CHARTS



Design Chart 4.A1: Sizing Eaves Gutters for Gradients 1:500 and Steeper

Notes:

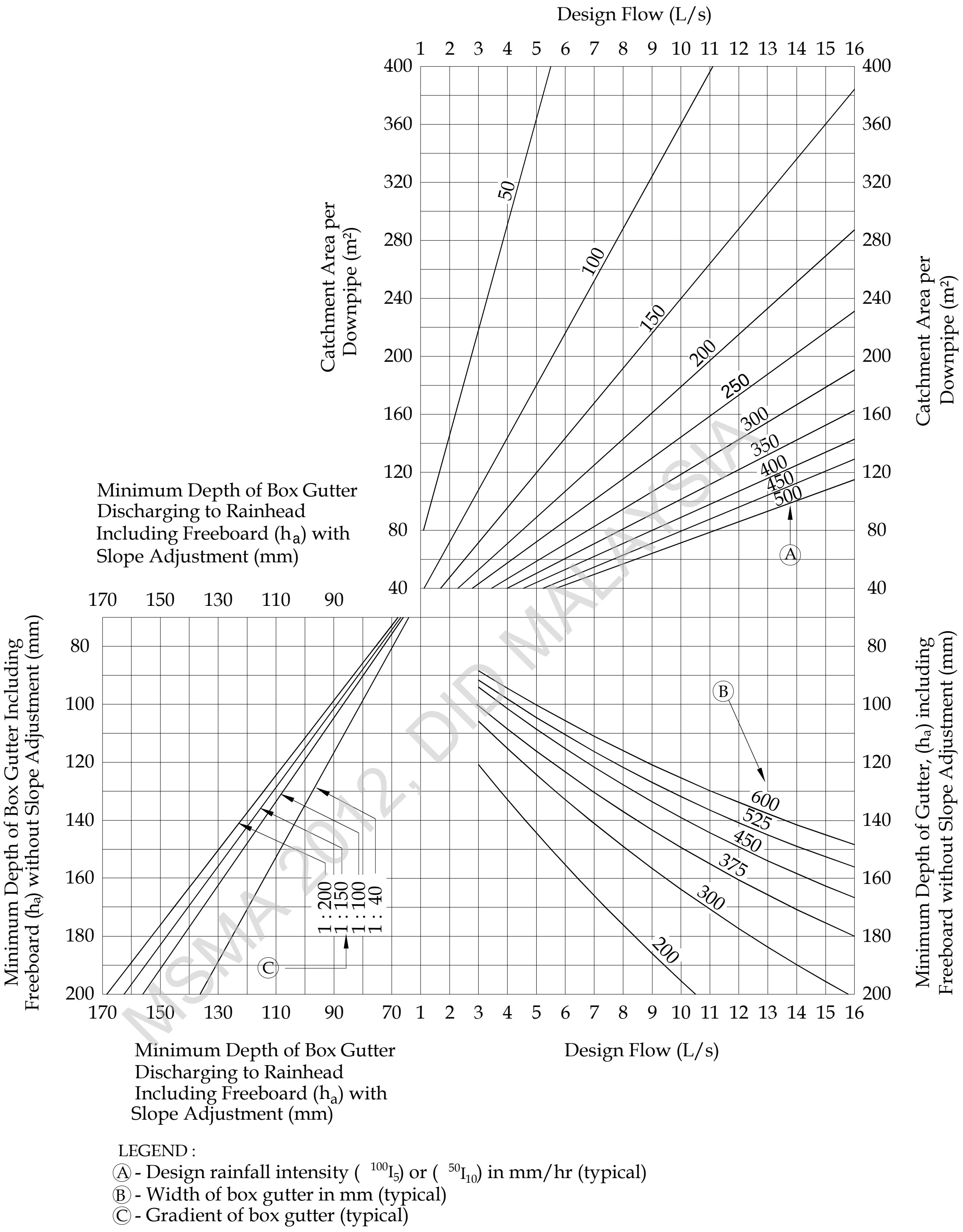
1. This graph assumes –
   (a) An effective width to depth ratio is about 2:1;
   (b) A gradient in the direction of flow is 1:500 or steeper;
   (c) The least favourable positioning of the downpipe and bends within the gutter length;
   (d) A cross –section of half round, quad, ogee or square; and
   (e) The outlet to a vertical downpipe is located centrally in the sole of the eaves gutter.
2. The required eaves gutter discharge areas do not allow for loss of waterway due to internal brackets.



Design Chart 4.A2: Sizing Eaves Gutters for Gradients Flatter than 1:500

Notes:

1. This graph assumes-
   - (a) An effective width to depth is a ratio of about 2:1;
   - (b) A gradient in the direction of flow flatter than 1:500;
   - (c) The least favourable positioning of the downpipe and bends within the gutter length;
   - (d) A cross –section or half round, quad, ogee or square; and
   - (e) The outlet to a vertical downpipe is located centrally in the sole of the eaves gutter.
2. The required eaves gutter discharge areas do not allow for loss of waterway due to internal brackets.



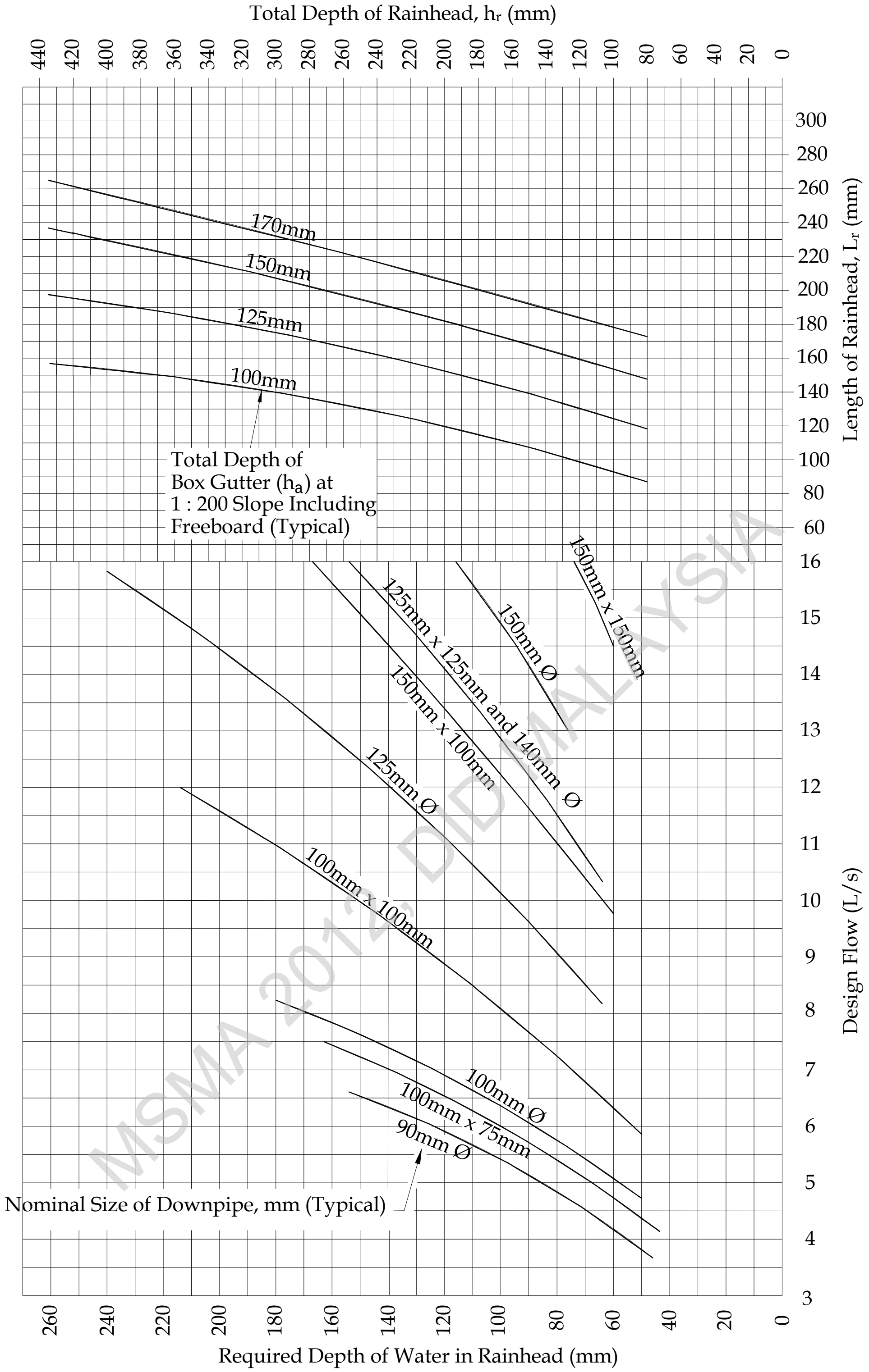
LEGEND :

Ⓐ - Design rainfall intensity ($^{100}I_5$) or ($^{50}I_{10}$) in mm/hr (typical)

Ⓑ - Width of box gutter in mm (typical)

Ⓒ - Gradient of box gutter (typical)

Design Chart 4.A3: Sizing a Freely Discharging Box Gutter



Design Chart 4.A4: Sizing Downpipes from Box Gutters

APPENDIX 4.B EXAMPLE – EAVES GUTTERS

Problem:

A house, with the gable roof shown in Figure 4.B1, is located in Kuala Lumpur. Determine the size of the required eaves gutters and downpipes.

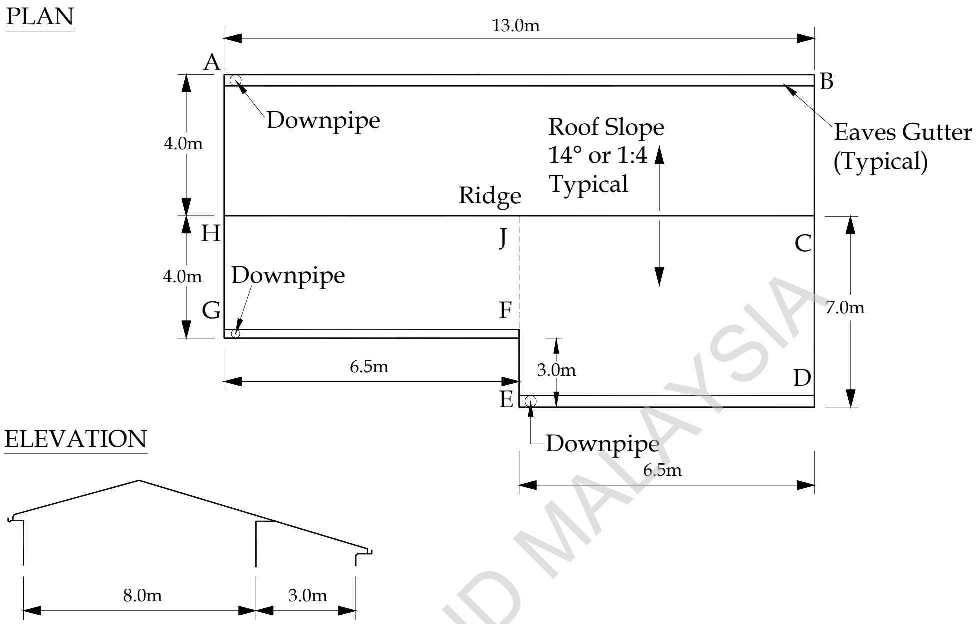


Figure 4.B1: Gable Roof of a House

Solution:

Reference	Calculation	Output
	a) Calculate rainfall intensity for 5 minutes storm duration and 20 year ARI for Kuala Lumpur.	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$	
Appendix 2.B1	λ = 69.650 K = 0.151 θ = 0.223 η = 0.880 d = 0.0883	
	$i = \frac{(69.650)(20)^{0.151}}{(0.0833+0.223)^{0.880}}$ =	304.4 mm/hr

Reference	Calculation	Output
	b) Calculation of Catchment Area	
Equation 4.1	*Roof A-B-C-H-A:*	
	Plan area A_h = 13 x 4 =	52m^2
	Rise is 1 m. Vertical area A_v = 1 x 13 =	13m^2
	Total catchment area = $A_h + A_v/2$	
	= 52 + (13)/2 =	58.5m^2
Equation 4.1	*Roof J-C-D-E-J:*	
	Plan area A_h = 7 x 6.5 =	45.5m^2
	Rise is 1.75 m. Vertical area A_v = 1.75 x 6.5 =	11.4m^2
	Total Catchment area = $A_h + A_v/2$	
	= 45.4 + (11.4)/2 =	51.2m^2
Equation 4.1	*Roof H-J-F-G-H:*	
	Plan area A_h = 4 x 6.5 =	26m^2
	Rise is 1m. Vertical area A_v = 1 x 6.5 =	6.5m^2
	Total catchment area = $A_h + A_v/2$	
	= 26+(6.5/2) =	29.3m^2
	c) Calculation of Gutter and Downpipe Size Assume gradient of eaves gutter is 1:600	
	Roof A-B-C-H-A:	
	Catchment area =	58.5m^2
Chart 4.A2	Required effective cross section area for gutter A-B =	19,500mm^2
	The value of effective cross section area for gutter A-B exceeded the maximum value of cross section for circular downpipe. Therefore, the square downpipe is chosen.	
Table 4.2	Required downpipe size =	125mm x 125mm
	Roof J-C-D-E-J:	
	Catchment area =	51.2m^2
Chart 4.A2	Required effective cross section area for gutter E-D =	17,250mm^2
Table 4.2	Required downpipe size =	150mm Ø
	Roof H-J-F-G-H:	
	Catchment area =	29.3m^2
Chart 4.A2	Required effective cross section area for gutter F-G =	11,000mm^2
Table 4.2	Required downpipe size =	125mm Ø

APPENDIX 4.C EXAMPLE – BOX GUTTERS

Problem:

A part of a factory as shown in Figure 4.C1 is to be constructed at Kota Damansara, Kuala Lumpur. Determine the size of the box gutter and associated vertical downpipes with rainheads that are to discharge to the site stormwater drains of the surface water drainage system.

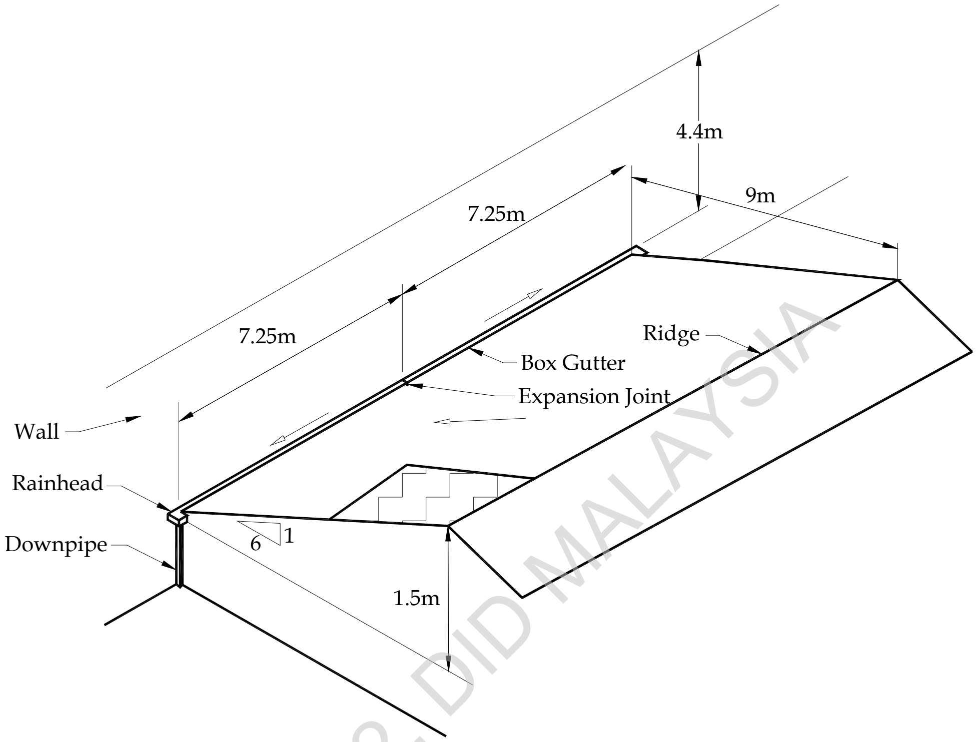
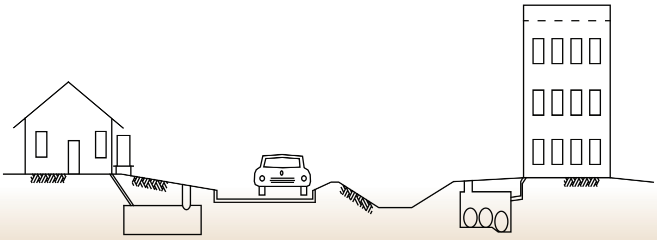


Figure 4.C1: Roof of a Factory

Solution:

Reference	Calculation	Output
Table 4.1 Equation 2.2 Appendix 2.B	Select 100 year ARI for Kuala Lumpur $i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$ $\lambda = 54.927$ K = 0.209 $\theta = 0.123$ $\eta = 0.544$ d = 0.083 $\mathrm{i} = \frac{(54.927)(100)^{0.209}}{(0.083+0.123)^{0.544}}$ = The dimension and other relevant data are shown in Figure 4.C1 Select position of expansion joint and rainheads as shown in Figure 4.C1	339.66 mm/hr

Reference	Calculation	Output
	Calculate catchment area	
	Roof A_h = 7.25m x 9m =	65.25m^2
	Roof Slope 1:6, Rise = 1/6 x 9 =	1.5m
	Roof A_{v1} = 7.25m x 1.5m =	10.88m^2
	Wall A_{v2} = 7.25m x 4.4m =	31.9m^2
Equation 4.2	$A_c = A_h + 1/2\ (A_{v2} – A_{v1})$	
	A_c = 65.25m^2 + 1/2 (31.9 – 10.88) m^2 =	75.76m^2
Chart 4.A3	Q = 7 L/s	
	Q < 16 L/s0	
Chart 4.A3	For Q = 7 L/s, select sole width of box gutter (W_{bg}) = 375mm and gradient = 1:200	
Chart 4.A3	For Q = 7 L/s, W_{bg} = 375mm and gradient = 1:200 , the actual minimum depth of box gutter including free board (ha) =	115mm
	As each box gutter discharges to a rainhead that is design to divert the design flow away from the building in the event of a total blockages of the downpipe, without increasing the depth of flow in the box gutter, this is the minimum depth required for the box gutter. The design flow in each box gutter is also the design flow in the rainhead, Q = 7 L/s.	Use box gutters 375 mm x 115 mm minimum with gradient 1:200
Chart 4.A4	Select 100 mm diameter downpipe.	
	Depth of water in rainhead =	125mm
	Total depth of rainhead h_r =	205mm
Chart 4.A4	Alternatively, select 100mm x 75mm downpipe.	
	Depth of water in rainhead =	140mm
	Total depth of rainhead =	235mm
	Use total depth of rainhead =	250mm
	Check if the total depth h_r needs to be adjusted as required by notes in Figure 4.7.	
Chart 4.A4	For 115 mm depth of box gutter, length of rainhead (L_r) =	150mm
	Depth of rainhead =	250mm
	Refer to Figure 4.7,	
	final dimension of rainhead, h_r = 250 mm, h_r – 25 =	225mm
	h_a =	115mm
	L_r =	150mm



5

ON-SITE DETENTION

CHAPTER 5 ON-SITE DETENTION

5.1 INTRODUCTION

This chapter provides simplified guidelines for the design of on-site detention (OSD) facilities. OSD may be provided as above-ground storages, below-ground storages, or a combination of both within a property boundary. Above-ground storages typically are located in lawns/gardens, car parking/driveway areas, rooftop and tanks. Below-ground storages can be tanks and pipe packages (Figure 5.1). The main advantages of above-ground storages are that they can generally be incorporated into the site by slight modification to the design of surface features and are relatively inexpensive compared to below-ground storages. Safety features such as sign board and fencing must be incorporated in the design of above-ground storage to prevent drowning, particularly of children and senior citizen. Below-ground storages however, are out of sight, occupy minimum land space. OSD facilities are designed to be dry most of the time, unless integrated with rainwater harvesting.

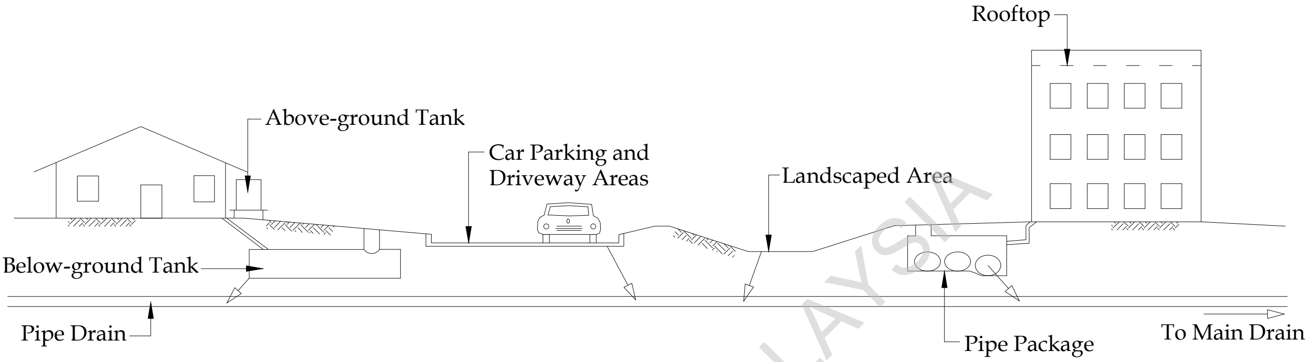


Figure 5.1: Typical OSD Storage Facilities

5.2 GENERAL DESIGN CONSIDERATIONS

5.2.1 Requirement of OSD

For the development area less than 0.1 ha, the individual OSD facilities is recommended meanwhile for the area more than 0.1 ha, the community OSD need to be provided.

5.2.2 Drainage System

The stormwater drainage system including gutters, pipes, open drains, and overland flow paths for the site must meet the followings:

- be able to convey all runoff to the OSD storage whenever possible; and
- ensure that the OSD storage is bypassed by all runoff from neighbouring properties and any part of the site not being directed to the OSD storage facility.

The outlet from the OSD facility must be designed to ensure that outflow discharges:

- do not cause adverse effects on downstream properties by concentrating flow; and
- can be achieved with low maintenance.

5.2.3 Multiple Storages

Sometimes OSD storages need to be designed as multiple unit with the separate parts of a property draining to each storage defined. When establishing the catchments draining to each storage, it is important to remember that all flows from the site, up to and including the storage design storm ARI, need to be directed to the storage. In addition to the property drainage system, surface gradings will need to be checked to ensure that surface flows and overflows from roof gutters, pipes, and open drains are directed to the appropriate OSD storage.

The outlet pipe from a storage needs to be connected downstream of the primary outlet structure of any other storage, i.e. storages should act independently of each other and not be connected in series.

5.2.4 Floor Levels

The site drainage system must ensure that:

- all habitable floor levels for new and existing dwellings are a minimum of 200mm above the storage maximum water surface level for the storage design storm ARI; and
- garage floor levels are a minimum of 100mm above the storage maximum water surface level.

A similar freeboard should be provided for flowpaths adjacent to habitable buildings and garages.

5.2.5 Aesthetics

The designer should try to ensure that OSD storages and discharge control structures blend in with the surrounding environment and enhance the overall aesthetic view of the site.

5.2.6 Signs

It is essential that current and future property owners are aware of the purpose of the OSD facilities provided. A permanent advisory sign for each OSD storage facility provided should be securely fixed at a pertinent and clearly visible location stating the intent of the facility. An example of an advisory sign is shown in Figure 5.2 (UPRCT,1999).

Signs:	Colours:
Triangle and "WARNING"	Red
Water	Blue
Figure and other Lettering	Black

Figure 5.2: Typical OSD Advisory Sign (UPRCT, 1999)

5.3 OPEN STORAGE

Typical open storages are shown in Figure 5.3 and Figure 5.4 while Table 5.1 recommends their allowable ponding depths. The guidelines given below allow the designer maximum flexibility when integrating the storage into the site layout.

5.3.1 Lawns, Car Parks and Tanks

Landscaped areas offer a wide range of possibilities for providing above-ground storage and can enhance the aesthetics of a site. The minimum design requirements for storage systems provided in landscaped areas are:

- storage volumes provided shall be 20% more than the calculated volumes to compensate for construction inaccuracies and the inevitable loss of storage due to vegetation growth over time;
- the minimum ground surface slope shall be 2% to promote free surface drainage and minimise the possibility of ponding; and
- side slopes should be a maximum of 4(H):1(V) where possible. If steep or vertical sides (e.g. retaining walls) are unavoidable, due consideration should be given to safety aspects, such as the need for fencing, both when the storage is full and empty.

Car parks, driveways, storage yards, and other paved surfaces may be used for stormwater detention. The minimum design requirements for storage systems provided in impervious areas shall be as follows:

- to avoid damage to vehicles, depths of ponding on driveways and car parks shall not exceed the limits recommended in Table 5.1 under design conditions; and
- transverse paving slopes within storages areas shall not be less than 0.7%.

Table 5.1: Maximum Depths for Different Open Site Storage

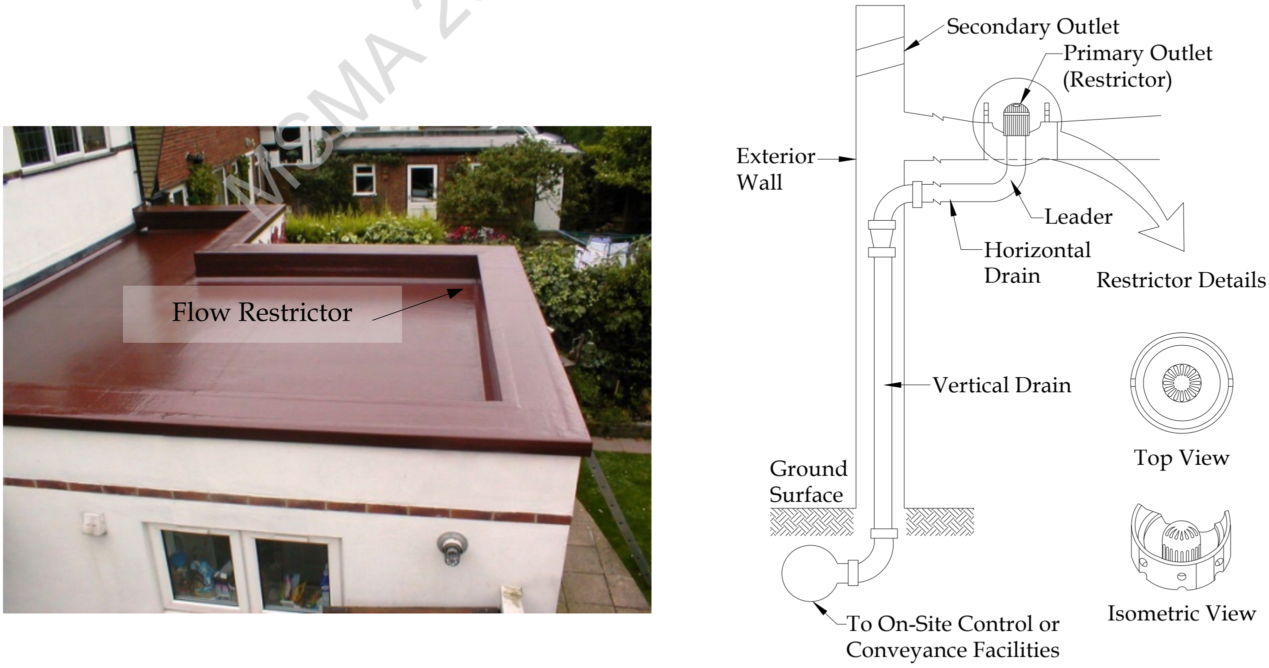
Site Classes	Maximum Storage Depth
Pedestrian Areas	50mm
Parking Areas and Driveways	150mm
Landscaped Areas	600mm
Rooftops	300mm
Paved Recreation Areas	100mm

a) Lawn b) Car Park c) Open Tank

Figure 5.3 Open Storage Features

5.3.2 Rooftops

Stormwater can be detained up to the maximum depth recommended in Table 5.1 by installing flow restrictors on roof drains. Flat roofs used for detention will have a substantial live load component. It is therefore essential that the structural design of the roof is adequate to sustain increased loadings from ponded stormwater. The latest structural codes and standards should be checked before finalising plans. Roofs must also be sealed to prevent leakage. A typical flow restrictor on a roof drain is shown in Figure 5.4. As can be seen, the outlet has a strainer that is surrounded by a flow restricting ring. The degree of flow control is determined by the size and number of holes in the ring.



a) Storage Area b) Flow Restrictor

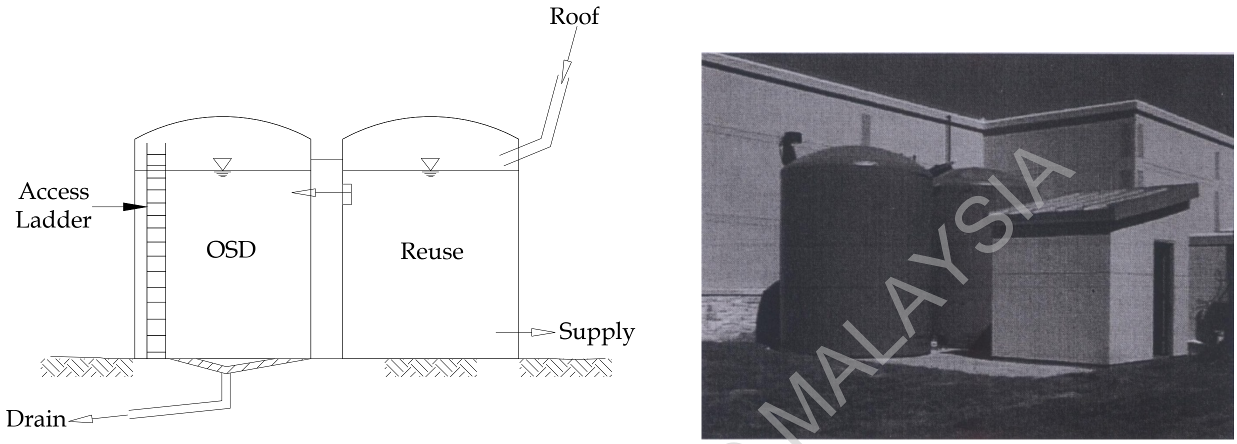
Figure 5.4: Typical Roof Storage System

5.4 CLOSED STORAGE

Closed storage is in tank and is located either above-and/or below-ground.

5.4.1 Above-ground Tank

Above-ground tanks may be used solely for on-site detention, or utilised in combination with storage provided for rainwater harvesting as illustrated in Figure 5.5. They provide a detention volume of runoff from building rooftops only. If a combined system is provided, the rainwater harvesting storage volume cannot be relied upon for detention purposes as this portion may be full or partly full at the onset of rain and therefore ineffective for detention. The storage volume that is required for on-site detention must therefore be in addition to the storage volume provided for rainwater harvesting.



a) Concept

b) Completion

Figure 5.5: Above-ground Storage Tank – Combined with Rainwater Harvesting

5.4.2 Below-ground Tank

Similarly a below-ground tank may be used for OSD only, or utilised in combination with storage provided for rainwater harvesting (Figure 5.6). When preparing a design for below-ground storage, designers should be aware of any statutory requirements for working in confined spaces. Access should be provided to allow routine inspection and maintenance. The safety aspect of the access should be considered in the design.

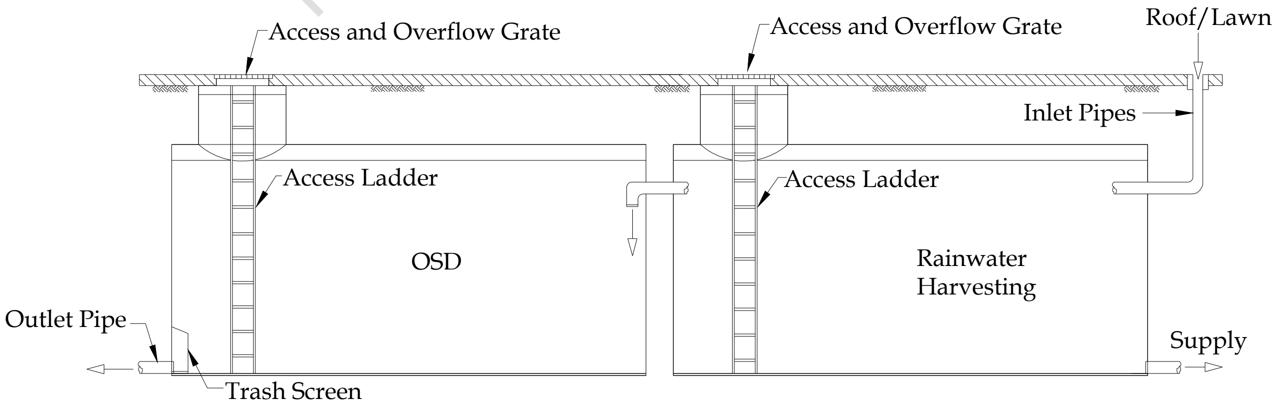


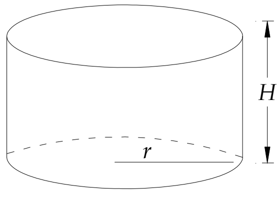
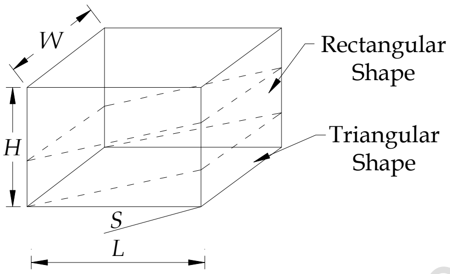
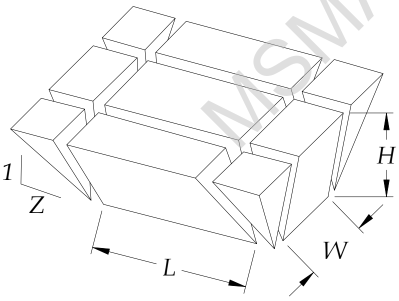
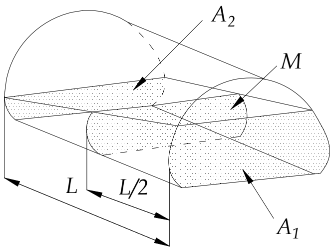
Figure 5.6: Below-ground Storage Tank - Combined with Rainwater Harvesting

5.5 STORAGE TANK CHARACTERISTICS

5.5.1 Basic Configuration

Typical storage tanks are either circular or rectangular in plan and/or cross-section but, due to their structural nature, can be configured into almost any geometrical plan shape (Table 5.2). For below–ground tank the configuration is largely determined by site conditions. For instance, the vertical fall in the stormwater system will determine if the storage can be drained by gravity or if pumping will be required.

Table 5.2: Typical Types of Tank and Storage Volume Formula

Type	Storage Volume Formula
1. Cylindrical  	$V = \pi r^2 H$ where, V = Volume at Specific Depth (m); π = 3.142; H = Depth of Ponding (m); and r = Radius of Basin (m).
2. Rectangular  	Rectangular : $V = LWH$ Triangle : $V = \frac{1}{2} W \frac{H^2}{S}$ where, V = Volume at a Specific Depth (m^3); H = Depth of Ponding for That Shape (m); W = Width of Basin at Base (m); L = Length of Basin at Base (m); and S = Slope of Basin (m/m).
3. Trapezoidal  	$V = LWH + LZH^2 + \frac{4}{3} Z^2 H^3$ where, V = Volume at a Specific Depth (m^3); H = Depth of Ponding for That Shape (m); L = Length of Basin at Base (m); W = Width of Basin at Base (m); R = Ratio of Width to Length of Basin at The Base; and Z = Side Slope Factor; Ratio of Horizontal to Vertical Components of Side Slope.
4. Pipe and Conduits  	$V = \frac{L}{6}(A_1 + 4M + A_2)$ where, V = Volume of Storage (m^3) ; L = Length of Section (m); A_1 = Cross-sectional Area of Flow at Base (m^2); A_2 = Cross-sectional Area of Flow at Top (m^2); and M = Cross-sectional Area of Flow at Midsection (m^2).

5.5.2 Structural Adequacy

Storage tanks must be structurally sound and be constructed from durable materials that are not subjected to deterioration by corrosion or aggressive soil conditions. The below-ground tanks must be designed to withstand the expected live and dead loads on the structure, including external and internal hydrostatic loadings. Buoyancy should also be checked, especially for lightweight tanks, to ensure that the tank will not be lifted under high groundwater conditions.

5.5.3 Layout Plan

Site layout will dictate how the installation is configured in plan. Obviously, the area that the storage facility will occupy will depend, among other things, on height and width limitations on the site. This can be especially critical in high-density developments where available site space may be very limited. A rectangular shape offers certain cost and maintenance advantages, but space availability will sometimes dictate a variation from a standard rectangular plan. It may be necessary on some sites to design irregular-shaped tanks.

5.5.4 Bottom Slope

To permit easy access to all parts of the below-ground tank for maintenance, the floor slope of the tank should not be greater than 10%. The lower limit for this slope is 2%, which is needed for good drainage of the tank floor.

5.5.5 Ventilation

It is very important to provide ventilation for below-ground storage systems to minimise odour problems. Ventilation may be provided through the tank access opening(s) or by separate ventilation pipe risers. Also, the ventilation openings should be designed to prevent air from being trapped between the roof of the storage and the water surface.

5.5.6 Overflow Provision

An overflow system must be provided to allow the tank to overflow in a controlled manner if the capacity of the tank is exceeded due to a blockage of the outlet pipe or a storm larger than the storage design ARI. As illustrated in Figure 5.6, an overflow can be provided by installing a grated access cover on the tank.

5.5.7 Access Openings

Below-ground storage tanks should be provided with openings to allow access by maintenance personnel and equipment. An access opening should be located directly above the outlet for cleaning when the storage tank is full and the outlet is clogged. A permanently installed ladder or step iron arrangement must be provided below each access opening if the tank is deeper than 1200 mm.

5.6 PRIMARY OUTLET

5.6.1 General Design Considerations

(a) Flow Regulator

Flow detention is provided by a storage volume from which water is released through a flow regulating device. It is the flow regulators that determine how efficiently the storage volume will be utilised. Obviously, the flow regulator has to be in balance with the available storage volume for the range of runoff events it is designed to control.

Flow regulator are often called upon to perform what may appear to be conflicting tasks, such as limiting flow rates, be free of clogging, and be relatively maintenance free.

(b) Location of the Flow Regulator

Flow regulating devices for above-ground storages are typically housed in an outlet structure, called a discharge control pit (DCP), which is an important component of the storage facility. It not only control the release rate, but also determine the maximum depth and volume within the storage.

Flow regulating devices for below-ground storages are typically located within the storage facility. In this type of arrangement, the flow regulator should be located at, or near, the bottom of the storage facility. In some cases, where the topography does not permit emptying of the storage facility by gravity, pumping will be required to regulate the flow rate. Figure 5.7 shows the indicative location of the primary outlet flow regulator in a typical above and below-ground storages.

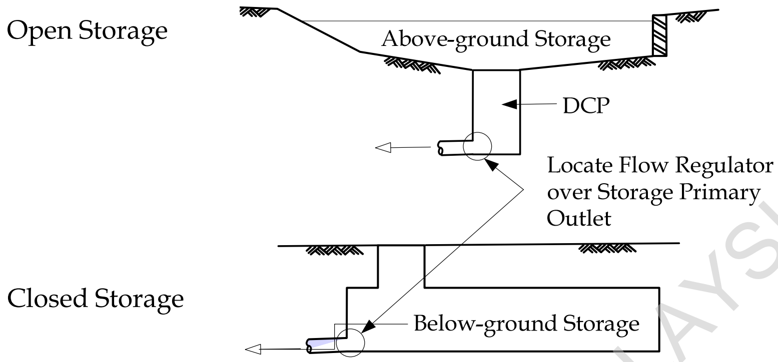


Figure 5.7: Primary Outlet Flow Regulator

5.6.2 Flow Restricting Pipe

The main advantage of using a flow restricting pipe as a storage outlet is that it is difficult to modify the hydraulic capacity of the pipe, unlike an orifice which can be easily removed. As illustrated in Figure 5.8, the net flow restricting effect of the pipe is mostly a function of the pipe length and pipe roughness characteristics.

Another advantage is that the required flow reduction may be achieved using a larger diameter opening than an orifice, which considerably reduces the possibility of blockage of the outlet. The pipe must be set at a slope less than the hydraulic friction slope, but steep enough to maintain a minimum velocity of 1.0 m/s under normal flow condition in the pipe in order to keep any silt carried by the water from settling out within the pipe.

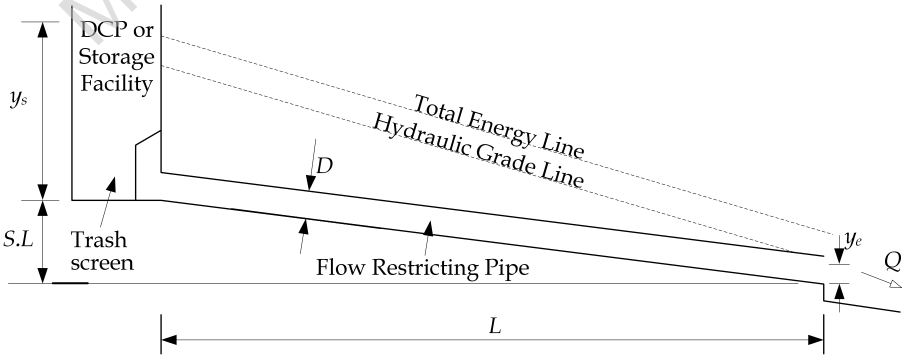


Figure 5.8: Flow Regulation with an Outlet Pipe (Stahre and Urbonas, 1990)

If the pipe is assumed to be flowing full, the outlet capacity can be calculated from Equation 5.1 which is based on the continuity equation. This equation is applicable to all outlet conditions, i.e. free outfall as well as submergence:

$$Q = A \cdot \sqrt{2g \cdot \frac{y_s + S.L - y_e}{K_L}} \tag{5.1}$$

where,

Q = Pipe Capacity (m³/s);
A = Cross-sectional Area of the Pipe (m²);
g = Acceleration due to Gravity (9.81 m/s²);
y_s = Water Depth at the Upstream Invert of the Pipe (m);
y_e = Water Depth at the Downstream Invert of the Pipe (m);
S = Pipe Longitudinal Slope (mm);
L = Pipe Length (m); and
K_L = Sum of Loss Factors for the Pipe System.

Figure 5.9, developed by Li and Patterson (1956), can be used to determine the discharge if the pipe is, in fact, entirely full. Although this figure is based on model tests using plastic pipe, it should provide a reasonable basis for checking the flow condition in other pipe types.

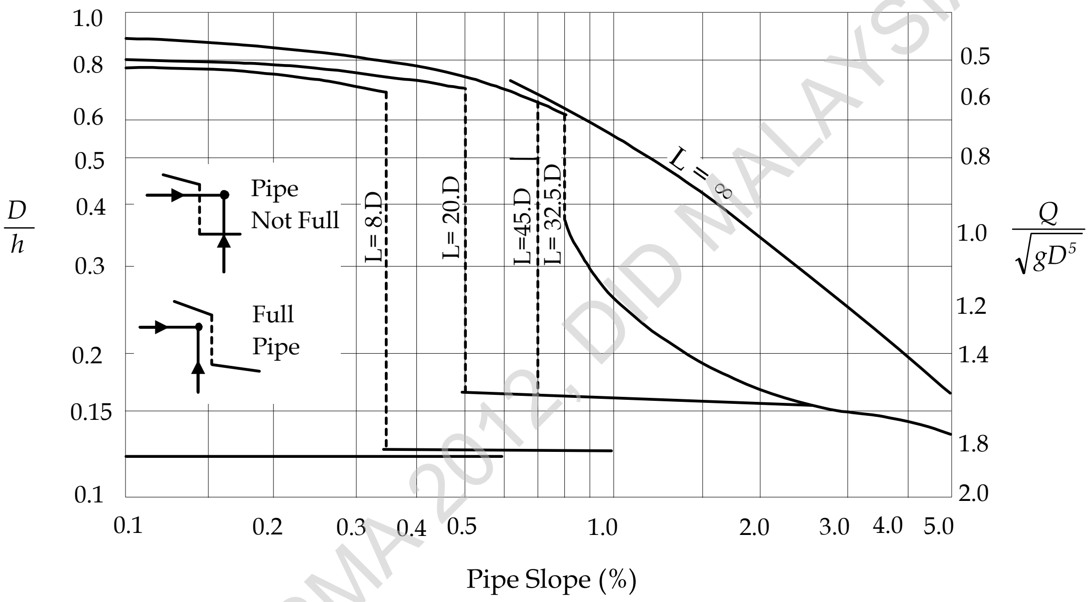


Figure 5.9: Length Upstream of Outlet Needed to Assure Full Pipe Flow (After Li and Patterson, 1956)

The sum of the loss factors will depend on the characteristics of the outlet and is expressed as:

$$K_L = K_t + K_e + K_f + K_b + K_o \tag{5.2}$$

where,

K_t = Trash Screen Loss Factor;
K_e = Entrance Loss Factor;
K_f = Friction Loss Factor;
K_b = Bend Loss Factor; and
K_o = Outlet Loss Factor.

a) Trash Screen Loss Factor

According to Creager and Justin (1950), the average loss factor of a trash screen can be approximated using the following equation:

$$K_t = 1.45 - 0.45\left(\frac{A_n}{A_g}\right) - \left(\frac{A_n}{A_g}\right)^2 \tag{5.3}$$

where,

A_n = Net Open Area between the Screen Bars (m^2); and
A_g = Gross Area of the Screen and Supports (m^2).

When estimating the maximum potential losses at the screen, assume that 50% of the screen area is blocked. However, the maximum outlet capacity should be calculated assuming no blockage. Minimum and maximum outlet capacities should be calculated to ensure that the installation will function adequately under both possible operating scenarios.

b) *Entrance Loss Factor*

Assuming orifice conditions at the pipe entrance, the pipe entrance loss factor may be expressed as:

$$K_e = \frac{1}{C_d^2} - 1 \tag{5.4}$$

where,

C_d = Orifice Discharge Coefficient.

c) *Friction Loss Factor*

The pipe friction loss factor for a pipe flowing full is expressed as:

$$K_f = f\,\frac{L}{D} \tag{5.5}$$

where,

f = Darcy-Weisbach Friction Loss Coefficient; and
D = Pipe Diameter (m).

The Darcy-Weisbach friction loss coefficient, under certain simplifying assumptions, can be expressed as a function of Manning's *n*, namely:

$$f = 125\,\frac{n^2}{D^{1/3}} \tag{5.6}$$

d) *Bend Loss Factor*

Bend losses in a closed conduit are a function of bend radius, pipe diameter, and the deflection angle at the bend. For 90° bends having a radius at least twice the pipe diameter, a value of K_{90} = 0.2 may be adopted. For bends having other than 90°, the bend loss factor can be calculated using the following equation:

$$K_b = F_b \cdot K_{90} \tag{5.7}$$

where,

F_b = Adjustment Factor provided in Table 5.3; and
K_{90} = Loss Factor for 90^o Bend.

e) *Outlet Loss Factor*

Virtually no recovery of velocity head occurs where the pipe outlet freely discharges into the atmosphere or is submerged under water. Therefore, unless a specially shaped flared outlet is provided, assume that $K_o = 1.0$. If the pipe outlet is submerged, assume $K_o = 0.5$.

Table 5.3: Adjustment Factors for other than 90° Bends

Angle of Bend (in degrees)	Adjustment Factor, F_b
0	0.00
20	0.37
40	0.63
60	0.82
80	0.90
90	1.00

5.6.3 Discharge Control Pit (DCP)

As previously stated, a DCP (see Figure 5.10) is typically used to house a flow regulator for an above-ground storage. The DCP provides a link between the storage and the connection to the municipal stormwater drainage system.

To facilitate access and ease of maintenance, the minimum internal dimensions (width and breadth) of a DCP shall be as follows:

- up to 600mm deep : 600mm x 600mm; and
- greater than 600mm deep : 900mm x 900mm.

These dimensions can be increased to allow greater screen sizes or improve access.

The following minimum dimensions will achieve predictable hydraulic characteristics:

- minimum design head = 2 D_o (from centre of orifice to top of overflow);
- minimum screen clearance = 1.5 D_o (from orifice to upstream face of screen); and
- minimum floor clearance = 1.5 D_o (from centreline of orifice to bottom of pit).

Note: D_o is the diameter of the orifice.

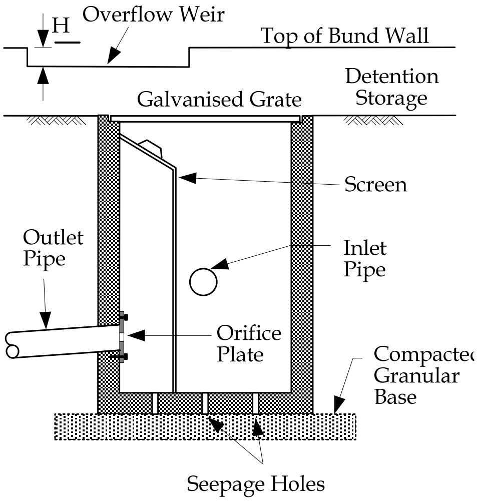


Figure 5.10: Typical DCP (After UPRCT,1999)

5.7 SECONDARY OUTLET

A suitable overflow arrangement must be provided to cater for rarer storms than the OSD facilities were designed for, or in the event of a blockage anywhere in the site drainage system.

The most commonly used arrangement for an above-ground storage is a broad-crested weir which, with most storages, can be designed to pass the entire overflow discharge with only a few centimetres depth of water over the weir. This is particularly desirable for car park storages to minimise the potential for water damage to parked vehicles. For a below-ground storage, it is common for the access chamber or manhole to be designed as the overflow system. If this is not practicable, an overflow pipe may be provided at the top of the storage to discharge to a safe point downstream.

It is essential that the access opening or overflow pipe has sufficient capacity to pass the storage design storm flow. An access point must be sized for the dimensions required to pass this flow or the dimensions required for ease of access, whichever is larger. Some typical examples of secondary outlets for above and below-ground storages are illustrated in Figure 5.11.

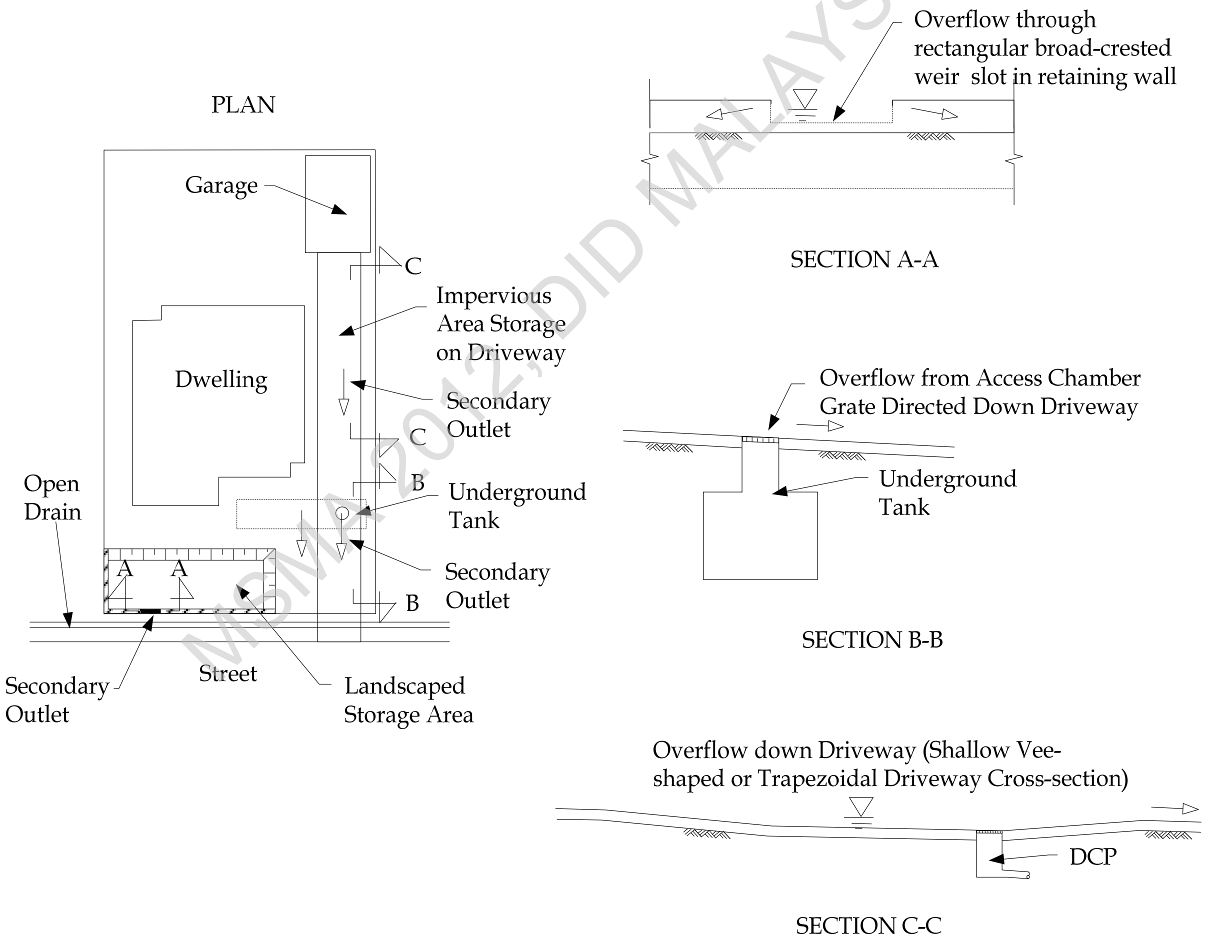


Figure 5.11: Examples of Secondary Outlets

5.8 DESIGN PROCEDURE

Instead of the computations specified in the previous edition of MSMA, the simplified procedure only involves the use of tables, figure and simple computations. Tables and figure that used in OSD design computations are as follows:

- Figure 5.A1: 5 (Five) Design Regions;
- Table 5.A1: Maximum Permissible Site Discharge (PSD) and Minimum Site Storage Requirement (SSR) Values in Accordance with The Five Regions in Peninsular Malaysia;
- Table 5.A2: Maximum Permissible Site Discharge (PSD), Minimum Site Storage Requirement (SSR) and Inlet Values in Accordance with The Major Towns in Peninsular Malaysia;
- Table 5.A3: OSD Volume, Inlet Size and Outlet Size for Five Different Regions in Peninsular Malaysia; and
- Table 5.A4: Discharge – Pipe Diameter.

5.8.1 Design Consideration

(a) Design Storm ARI

The design storm shall be 10 year ARI in accordance with the minor drainage system ARI provided in Table 1.1.

(b) Permissible Site Discharge (PSD)

The PSD is the maximum allowable post-development discharge from a site for the selected design storm and is estimated on the basis that flows within the downstream stormwater drainage system will not be increased.

(c) Site Storage Requirement (SSR)

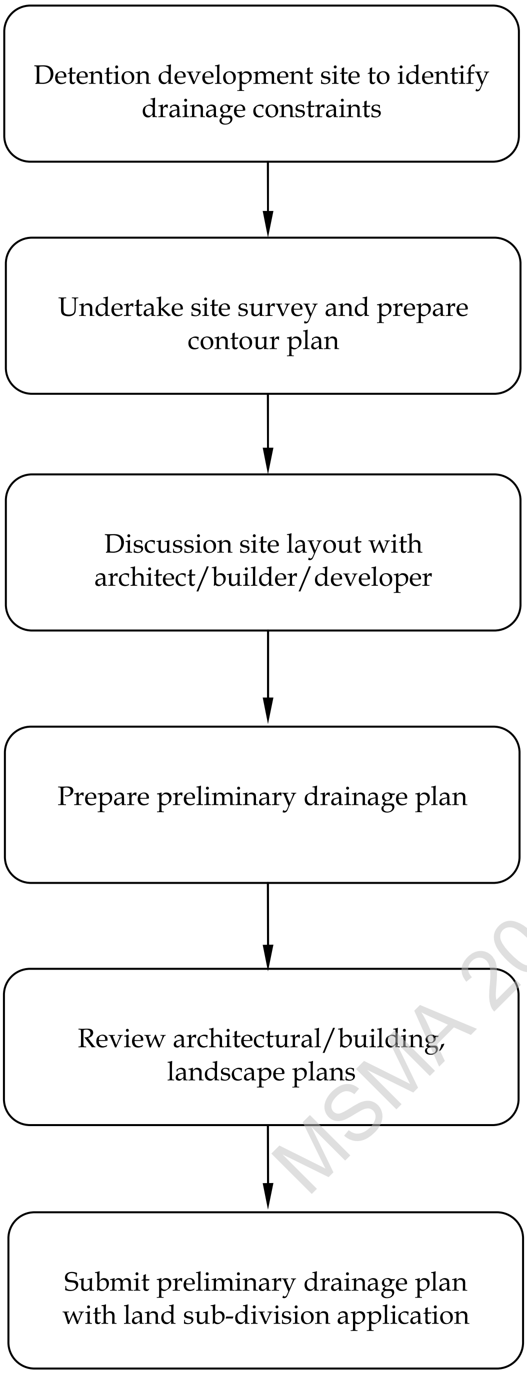
The SSR is the total amount of storage required to ensure that the required PSD is not exceeded and the OSD facility does not overflow based on the storage design storm ARI.

(d) Concept Plan Preparation

An OSD Concept Plan (OCP) is required to support all development applications except for single family dwellings, properties located near the end of a drainage system such as lakeside or seaside properties. The purpose of OCP is to identify the drainage constraints and to demonstrate that the OSD system can be integrated into the proposed site layout. In the preparation of OCP, preliminary design of OSD storage system needs to be carried out. The steps involved are as shown in Figure 5.12.

Developers should involve their OSD designer in developing the initial site layout. There are issues to be addressed at the conceptual stage, these include:

- simplifying the design by identifying adequate storage areas in the planning stage;
- allowing for the cost of development relating to OSD at the planning stage;
- reducing project costs by maximising the use of proposed landscape and architectural features as part of the OSD system;
- constructing in multi lot subdivisions, a common OSD system on one lot rather than a separate system on each individual lot;
- diverting overland flows from upstream past the OSD unless the storage volume is increased to cater for the upstream catchment; and
- grading the site to ensure that all flows are directed to the storage even if blockages or larger storm event occur.



- external flows entering the site;
- catchment area of any external flows;
- potential discharge points; and
- potential storage areas.

- location and levels of public drainage system;
- sufficient surface levels to characterize site;
- extending into adjoining lots if necessary; and
- any other constraints (e.g. services and drainage reserves).

- estimate storage volume required;
- estimate external flow entering the site; and
- establish building and site layout.

- select type and location of suitable storage(s);
- determine areas unable to drain to storages;
- estimate storage levels assess available discharge points;
- identify emergency spillway types and locations; and
- identify overland flow paths for external flows and storage overflows.

- check other plans prepared for the development for any anomalies or conflicts with the preliminary drainage plan.

Figure 5.12: Preliminary Design Procedure for OSD Storage Systems (DID, 2000)

5.8.2 Design Steps

The steps involved in a typical design required to be carried out are outlined in Figure 5.13.

A step by steps procedure for designing storage system of OSD are as follows:

Step 1: Determine the storage type(s) to be used within the site, i.e. separate above and/or below-ground storage(s), or a composite above and below- ground storage.

Step 2: Identify the region of the detention site from Figure 5.A1.

Step 3: Determine the catchment characteristics such as terrain type and percentage of impervious area.

Step 4: Determine Permissible Site Discharge (PSD) per hectares (PSD/ha) from Table 5.A1. Then multiply with project area to determine PSD.

Step 5: Determine Site Storage Requirement (SSR) per hectares (SSR/ha) from Table 5.A1. Then multiply with project area to determine SSR.

Step 6: Identify the major town of the detention site in Table 5.A2 and determine inlet flow per hectares from Table 5.A2 . Then multiply with project area to determine inlet flow.

Step 7: Determine PSD per hectares (PSD/ha) from Table 5.A2. Then multiply with detention area to determine PSD.

Step 8: Determine SSR per hectares (SSR/ha) from Table 5.A2. Then multiply with detention area to determine SSR.

Step 9: Compare the value of PSD from Step 4 and Step 7. The smaller PSD value is adopted for subsequent sizing of outlet pipe.

Step 10: Compare the value of SSR from Step 5 and Step 8. The larger SSR value is adopted for Selected Design Value.

Step 11: Determine the Inlet Pipe diameter from Table 5.A3.

Step 12: Determine the Outlet Pipe diameter from Table 5.A3.

Step 13: Determine the Inlet Pipe diameter from Table 5.A4 by using the Inlet Flow value from Step 6 as discharge.

Step 14: Determine the Outlet Pipe diameter from Table 5.A4 by using the PSD value from Step 9 as discharge.

Step 15: Compare the value of Inlet Pipe diameter and from Step 11 and Step 13. The smaller Inlet Pipe diameter is adopted for Selected Design Value.

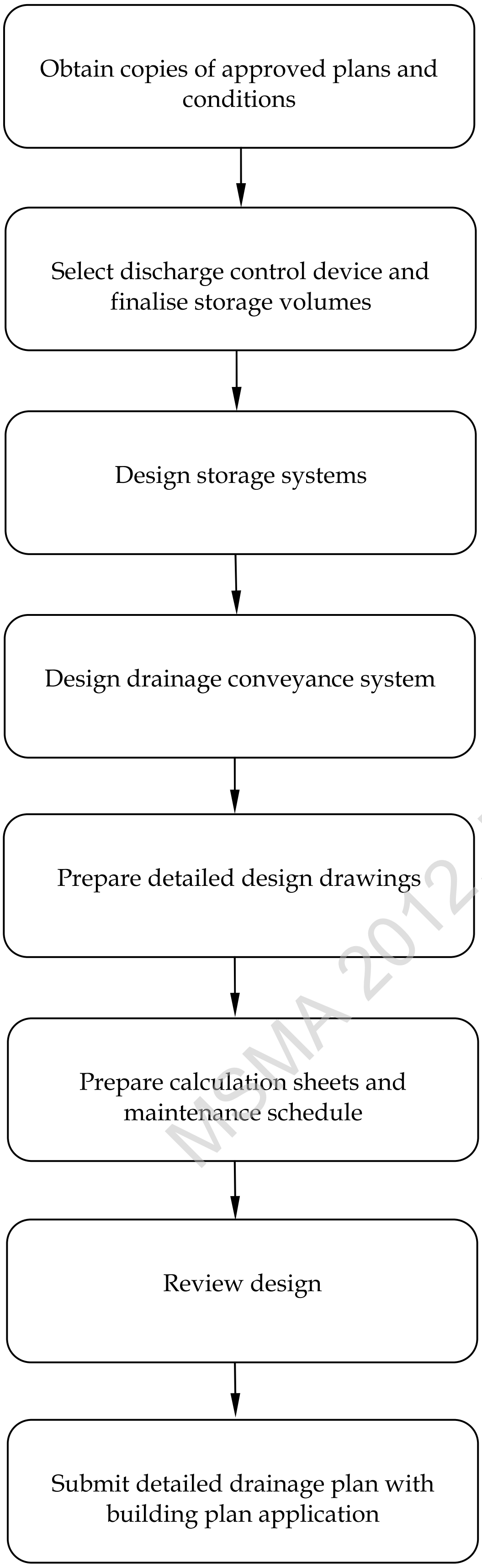
Step 16: Compare the value of Outlet Pipe diameter and from Step 12 and Step 14. The smaller Outlet Pipe diameter is adopted for Selected Design Value.

5.8.3 Simplified OSD Design Procedure

The main objective in the design of OSD facilities is to determine the required storage (SSR) to reduce the post development discharge to that of the pre-development level (PSD) . In order to achieve this the outlet structure needs to be sized appropriately. The OSD Design Manual simplifies most of the processes involved in the design of OSD facilities required for a proposed site, taking into consideration the design storm for short duration rainfall less than 1 hour, the critical time of concentration for various slopes and catchment areas, the PSD and the SSR for the site. The determination of each parameter was based on the assumptions that Peninsular Malaysia can be divided into five design regions having different uniform design storms, as shown in Figure 5.A1.

(a) Permissible Site Discharge (PSD)

The maximum allowable discharge leaving the site is given in litres/second/hectare (l/s/ha), or in litres/second (l/s) when applied to a specific site. The maximum PSD is about 65 l/s/ha. This needs to be adjusted in accordance with the design regions as shown in Figure 5.A1 and the results were summarised in



- approve preliminary drainage plan;
- development/subdivision consent conditions; and
- landscape and architectural/building plans.

- select discharge control device for each storage;
- establish level of outlet(s) and ensure free outfall if possible; and
- finalise required volume(s).

- distribute final storage volume(s) to minimize; nuisance ponding conditions to property owners;
- check underground storages for access and ease of maintenance; and
- ensure sufficient weir capacity for storage overflows.

- ensure storage design ARI flows are conveyed to storage for all areas designed to drain to storages; and
- check overland flowpaths have adequate capacity to ensure external flows bypass on-site storages.

- undertake structural design of system elements as necessary;
- prepare plans of sufficient standard an detail to allow builders/plumbers to construct system; and
- specify construction materials.

- prepare calculation sheets for each storage system; and
- prepare maintenance schedule outlining necessary maintenance practices.

- review other plans prepared for the development for any anomalies or conflicts with detailed drainage plan; and
- check all stormwater-related development consent conditions have been satisfied.

Figure 5.13: Detailed Design Procedure for OSD Storage Systems (DID, 2000)

Tabel 5.A1. Table 5.A3 shows the inlet sizes which represent the PSD capacity. The examples of how to use these tables and figures are given in Appendix 5.B.

(b) Site Storage Requirement (SSR)

The minimum volume (in m^3/hectare or in m^3 when applied to a specific site) is required for storage to ensure that spillage will not occur when the outflow is restricted to the PSD. The minimum SSR for an OSD storage that has a Discharge Control Pit (DCP) that achieves the full PSD early in the storm event is presented in Table 5.A1 which shows the values in accordance with the design region, in m^3/ha. Information about SSR for major towns in Peninsular Malaysia is provided in Table 5.A2.

Table 5.A3 can be used to determine the size of the OSD storage, and its inlet and outlet structures for different types of development projects in Malaysia when the project area, terrain slope and percentage of impervious area are known. Examples of how to use these tables are presented in Appendix 5.B.

(c) Minimum Outlet Size

To reduce the likelihood of the DCP outlet being blocked by debris, the outlet opening shall have a minimum internal diameter or width of at least 30 mm and shall be protected by an approved screen. Stainless steel well screen fabric is more preferable instead of mesh sreen in controlling trash entering the orifice due to the nature of easier cleaning after accumulating trash. Table 5.A3 provides the pipe diameter for the outlet in respect to the PSD requirements in different regions in Peninsular Malaysia.

(d) Overflow Pipe/Spillway

An overflow system must be provided to allow the storage compartment to surcharge in a controlled manner if the capacity of the system is exceeded due to a blockage of the outlet pipe or a storm larger than the design ARI. An overflow can be provided by installing a pipe/weir and the size of pipe is given in Table 5.A3. The weir size can be calculated based on pipe area, which can be determined for a given pipe diameter. Figure 5.A2 can also be used to determine the pipe sizes.

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APPENDIX 5.A DESIGN FIGURES AND TABLES

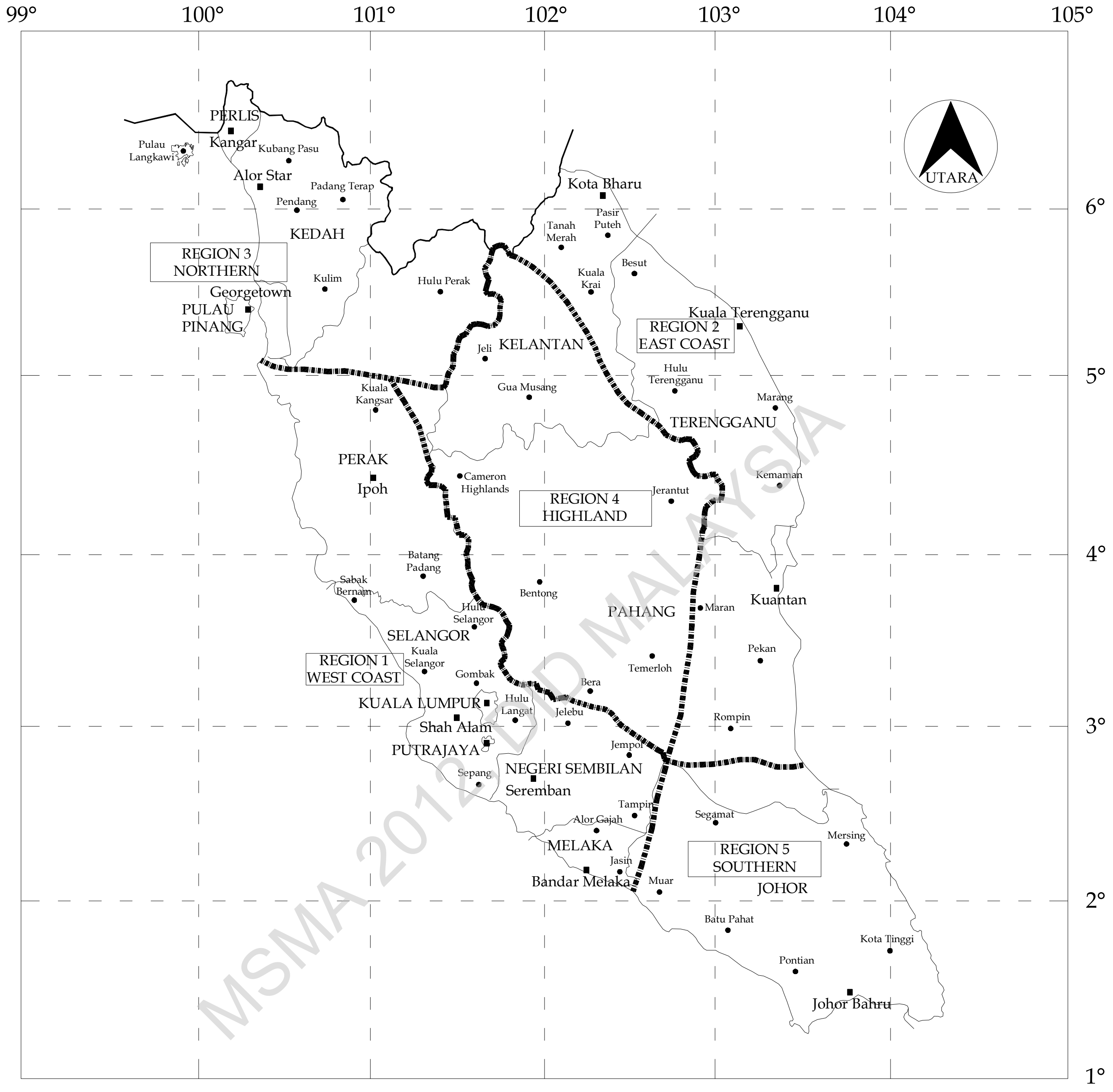


Figure 5.A1: Five (5) Design Regions

Table 5.A1: Maximum Permissible Site Discharge (PSD) and Minimum Site Storage Requirement (SSR) Values in Accordance with The Five Regions in Peninsular Malaysia

Terrain / Slope Condition	**PSD (l/s/ha)**					**SSR (m^3/ha)**				
	Impervious Area (as a Percentage of Project Area)									
	25%	**40%**	**50%**	**75%**	**90%**	**25%**	**40%**	**50%**	**75%**	**90%**
REGION 1 - WEST COAST										
Lowlying	63.4	64.2	64.5	65.2	65.5	322.2	363.0	394.2	478.3	540.4
Mild	76.7	77.5	77.9	78.7	79.1	306.6	340.0	367.2	448.5	504.7
Steep	87.7	88.6	89.1	90.1	90.5	294.0	327.0	350.5	426.7	478.8
REGION 2 - EAST COAST										
Lowlying	53.0	53.6	53.9	54.5	54.7	276.6	350.4	410.7	609.1	768.8
Mild	61.1	61.8	62.2	62.8	63.1	257.6	321.7	373.9	546.1	678.7
Steep	67.4	68.2	68.6	69.3	69.6	243.5	302.6	351.0	509.9	625.9
REGION 3 - NORTHERN										
Lowlying	54.8	55.4	55.7	56.3	56.5	311.1	353.3	389.7	493.3	564.4
Mild	68.0	68.8	69.2	69.9	70.2	295.5	328.3	360.3	454.0	521.6
Steep	77.3	78.2	78.6	79.5	79.8	284.8	316.2	341.8	430.3	492.6
REGION 4 - HIGHLAND										
Lowlying	42.6	43.1	43.4	43.8	44.0	227.8	285.7	331.4	460.5	546.6
Mild	49.6	50.2	50.5	51.0	51.2	212.3	266.0	307.3	428.2	509.2
Steep	55.0	55.6	56.0	56.5	56.8	202.1	252.3	291.0	405.5	484.1
REGION 5 - SOUTHERN										
Lowlying	61.1	61.9	62.2	62.8	63.1	315.0	362.0	398.4	501.0	572.7
Mild	74.8	75.7	76.1	76.9	77.2	298.5	340.9	372.6	465.9	532.3
Steep	83.4	84.3	84.8	85.7	86.1	288.5	323.3	352.5	442.8	505.0

Table 5.A2: Maximum Permissible Site Discharge (PSD), Minimum Site Storage Requirement (SSR) and Inlet Values in Accordance with The Major Towns in Peninsular Malaysia

Terrain / Slope Condition	PSD (l/s/ha)					SSR (m^3/ha)					Inlet (l/s/ha)				
	Impervious Area (as a Percentage of Project Area)														
	25%	40%	50%	75%	90%	25%	40%	50%	75%	90%	25%	40%	50%	75%	90%
BATU PAHAT															
Low-lying	45.1	45.6	45.9	46.3	46.6	249.2	293.7	331.1	437.3	502.6	107.0	125.0	137.0	166.0	184.0
Mild	55.6	56.2	56.6	57.1	57.4	234.7	273.8	304.7	403.7	468.4	127.0	146.0	158.0	189.0	208.0
Steep	62.3	63.0	63.4	64.0	64.3	225.6	261.8	291.3	383.5	447.1	140.0	159.0	172.0	204.0	224.0
JOHOR BAHRU															
Low-lying	61.1	61.9	62.2	62.8	63.1	315.0	362.0	398.4	501.0	572.7	138.0	157.0	170.0	200.0	221.0
Mild	74.8	75.7	76.1	76.9	77.2	298.5	340.9	372.6	465.9	532.3	163.0	184.0	200.0	231.0	252.0
Steep	83.4	84.3	84.8	85.7	86.1	288.5	323.3	352.5	442.8	505.0	181.0	200.0	215.0	250.0	271.0
SEGAMAT															
Low-lying	41.4	41.9	42.1	42.5	42.7	294.3	326.6	356.0	454.9	521.2	110.0	130.0	140.0	170.0	190.0
Mild	48.6	49.1	49.4	49.9	50.1	289.2	321.1	350.0	431.6	497.4	140.0	150.0	170.0	200.0	220.0
Steep	54.2	54.8	55.1	55.7	55.9	280.2	310.5	332.9	415.1	479.1	150.0	170.0	180.0	220.0	240.0
MELAKA															
Low-lying	57.9	58.5	58.9	59.5	59.7	296.1	349.2	388.4	487.4	547.3	130.0	150.0	170.0	200.0	220.0
Mild	71.4	72.2	72.6	73.4	73.7	272.3	318.6	355.5	453.1	512.4	130.0	150.0	170.0	200.0	220.0
Steep	78.3	79.2	79.6	80.4	80.8	262.9	306.5	339.7	436.4	505.4	170.0	190.0	200.0	240.0	260.0
KUALA PILAH															
Low-lying	35.7	36.1	36.3	36.7	36.8	225.2	262.3	289.8	373.1	429.6	90.0	110.0	120.0	140.0	160.0
Mild	44.6	45.2	45.4	45.9	46.1	214.5	247.5	273.5	349.2	402.1	110.0	130.0	140.0	170.0	180.0
Steep	50.6	51.2	51.5	52.0	52.2	207.6	238.0	263.7	335.2	385.6	120.0	140.0	150.0	180.0	200.0
SEREMBAN															
Low-lying	32.5	32.9	33.1	33.4	33.5	223.2	256.3	278.6	365.5	424.9	80.0	100.0	110.0	140.0	150.0
Mild	39.8	40.3	40.5	40.9	41.1	214.4	245.7	267.8	343.2	398.6	100.0	120.0	130.0	160.0	180.0
Steep	45.8	46.4	46.6	47.1	47.3	209.1	237.2	259.2	326.5	380.3	110.0	130.0	140.0	170.0	190.0
KUALA LUMPUR															
Low-lying	60.4	61.0	61.4	62.0	62.3	305.2	346.7	377.9	466.1	529.7	130.0	150.0	160.0	190.0	210.0
Mild	71.2	72.0	72.4	73.1	73.4	292.6	327.9	358.0	439.6	497.8	160.0	173.0	190.0	220.0	240.0
Steep	81.7	82.6	83.0	83.9	84.3	280.6	312.6	340.5	418.0	471.6	170.0	190.0	210.0	240.0	260.0
KUALA KUBU BAHRU															
Low-lying	42.6	43.1	43.4	43.8	44.0	227.8	285.7	331.4	460.5	546.6	100.0	120.0	130.0	160.0	170.0
Mild	49.6	50.2	50.5	51.0	51.2	212.3	266.0	307.3	428.2	509.2	120.0	140.0	150.0	180.0	200.0
Steep	55.0	55.6	56.0	56.5	56.8	202.1	252.3	291.0	405.5	484.1	130.0	150.0	160.0	190.0	210.0
RAUB															
Low-lying	39.7	40.2	40.4	40.8	41.0	252.4	318.6	371.3	523.0	624.0	110.0	120.0	140.0	160.0	180.0
Mild	46.6	47.2	47.4	47.9	48.1	232.3	295.6	342.6	484.8	579.5	120.0	140.0	150.0	180.0	200.0
Steep	52.0	52.6	52.9	53.4	53.6	217.7	279.7	323.4	457.8	549.1	130.0	150.0	160.0	190.0	210.0
CAMERON HIGHLAND															
Low-lying	34.1	34.5	34.7	35.1	35.2	192.8	237.7	269.9	358.0	415.0	83.0	100.0	110.0	136.0	150.0
Mild	42.4	42.9	43.1	43.6	43.7	180.1	219.2	250.2	334.8	388.1	100.0	116.0	130.0	155.0	172.0
Steep	47.4	47.9	48.2	48.7	48.9	173.4	208.9	239.1	321.1	374.1	108.0	126.0	140.0	167.0	180.0
KUANTAN															
Low-lying	61.6	62.3	62.6	63.3	63.6	322.5	383.7	430.0	547.2	615.7	148.0	170.0	180.0	212.0	230.0
Mild	72.9	73.7	74.1	74.9	75.2	301.8	356.3	397.9	513.7	584.0	170.0	193.0	210.0	234.0	259.0
Steep	79.8	80.7	81.2	82.0	82.4	289.9	340.1	380.4	493.7	563.6	188.0	208.0	220.0	255.0	280.0
TELUK INTAN															
Low-lying	46.5	47.1	47.4	47.8	48.0	331.5	375.8	412.9	526.1	597.6	130.0	150.0	160.0	200.0	210.0
Mild	57.0	57.7	58.0	58.6	58.9	319.0	355.6	388.7	492.0	562.8	160.0	180.0	190.0	220.0	240.0
Steep	64.9	65.6	66.0	66.7	67.0	309.7	342.3	373.1	468.3	537.4	170.0	190.0	210.0	240.0	260.0
IPOH															
Low-lying	63.4	64.2	64.5	65.2	65.5	322.2	363.0	394.2	478.3	540.4	130.0	150.0	170.0	200.0	220.0
Mild	76.7	77.5	77.9	78.7	79.1	306.6	340.0	367.2	448.5	504.7	160.0	180.0	200.0	230.0	250.0
Steep	87.7	88.6	89.1	90.1	90.5	294.0	327.0	350.5	426.7	478.8	160.0	200.0	220.0	250.0	270.0
KUALA KANGSAR															
Low-lying	47.2	47.7	48.0	48.5	48.7	283.3	335.0	365.0	465.0	540.0	120.0	130.0	150.0	180.0	190.0
Mild	60.8	61.5	61.8	62.5	62.8	267.0	302.8	340.0	424.0	486.0	140.0	160.0	170.0	200.0	220.0
Steep	74.4	75.2	75.7	76.5	76.8	251.3	290.0	318.0	413.0	488.0	150.0	170.0	190.0	220.0	240.0
KUALA TERENGGANU															
Low-lying	53.0	53.6	53.9	54.5	54.7	276.6	350.4	410.7	609.1	768.8	130.0	140.0	160.0	190.0	210.0
Mild	61.1	61.8	62.2	62.8	63.1	257.6	321.7	373.9	546.1	678.7	140.0	160.0	170.0	210.0	230.0
Steep	67.4	68.2	68.6	69.3	69.6	243.5	302.6	351.0	509.9	625.9	150.0	170.0	190.0	220.0	240.0
KOTA BAHRU															
Low-lying	60.8	61.5	61.8	62.5	62.8	335.1	399.8	448.3	571.1	645.5	150.0	170.0	180.0	210.0	230.0
Mild	72.1	72.9	73.3	74.0	74.4	311.3	367.1	414.0	535.3	608.9	170.0	190.0	210.0	240.0	260.0
Steep	79.2	80.1	80.5	81.3	81.7	298.9	351.6	393.1	513.2	586.4	190.0	210.0	220.0	260.0	280.0
ALOR SETAR															
Low-lying	54.8	55.4	55.7	56.3	56.5	311.1	353.3	389.7	493.3	564.4	130.0	150.0	160.0	190.0	210.0
Mild	68.0	68.8	69.2	69.9	70.2	295.5	328.3	360.3	454.0	521.6	150.0	180.0	190.0	220.0	240.0
Steep	77.3	78.2	78.6	79.5	79.8	284.8	316.2	341.8	430.3	492.6	170.0	190.0	210.0	240.0	260.0

Table 5.A3: OSD Volume, Inlet Size and Outlet Size for Five Different Regions in Peninsular Malaysia

Region 1

Project Area (ha)	Impervious Area (as Percentage of Project Area)														
	25%			40%			50%			75%			90%		
	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)
TERRAIN : LOWLYING, SLOPE 1 : 2000 TO 1 : 5000															
0.1	32	129	62	36	138	62	39	147	62	48	160	62	54	167	62
0.2	64	182	87	73	195	87	79	208	87	96	226	87	108	237	87
0.4	129	257	124	145	276	124	158	294	124	192	319	124	216	335	124
0.6	193	315	151	218	339	151	236	350	151	288	391	151	324	410	151
0.8	258	364	175	290	391	175	315	416	175	384	451	175	432	474	175
1	322	407	195	363	437	195	394	465	195	480	500.00	195	540	529	195
2	770	422	276	870	479	276	950	505	276	1190	553	276	1360	576	276
3	1155	517	339	1305	586	339	1425	618	339	1785	677	339	2040	705	339
4	1540	597	391	1740	677	391	1900	714	391	2380	782	391	2720	814	391
5	1925	668	437	2175	757	437	2375	798	437	2975	874	437	3400	910	437
TERRAIN : MILD, SLOPE 1 : 875 TO 1 : 1999															
0.1	31	143	69	34	151	69	37	160	69	45	171	70	51	178	70
0.2	61	202	97	68	214	97	74	226	97	90	242	98	101	252	98
0.4	122	286	137	136	303	137	148	319	137	180	342	139	202	357	139
0.6	183	350	168	204	371	168	222	391	168	270	419	170	303	437	170
0.8	244	404	194	272	428	194	296	451	194	360	484	197	404	505	197
1	305	451	217	340	479	217	370	505	217	450	541	220	505	564	220
2	740	451	307	840	505	307	910	529	307	1140	597	311	1300	618	311
3	1110	553	376	1260	618	376	1365	648	376	1710	731	381	1950	757	381
4	1480	638	434	1680	714	434	1820	749	434	2280	845	440	2600	874	440
5	1850	714	485	2100	798	485	2275	837	485	2850	944	492	3250	977	492
TERRAIN : STEEP, SLOPE 1 : 100 TO 1 : 874															
0.1	30	143	73	32.5	160	73	35	167	74	43	178	74	48	185	74
0.2	59	202	103	65	226	103	70	237	105	86	252	105	96	262	105
0.4	118	286	146	130	319	146	140	335	148	172	357	148	192	371	148
0.6	177	350	179	195	391	179	210	410	181	258	437	181	288	454	181
0.8	236	404	207	260	451	207	280	474	209	344	505	209	384	525	209
1	295	451	231	325	505	231	350	529	234	430	564	234	480	586	234
2	710	505	327	810	553	327	870	576	331	1080	638	331	1230	658	331
3	1065	618	401	1215	677	401	1305	705	405	1620	782	405	1845	806	405
4	1420	714	463	1620	782	463	1740	814	468	2160	903	468	2460	931	468
5	1775	798	517	2025	874	517	2175	910	523	2700	1010	523	3075	1041	523

Region 2

Project Area (ha)	Impervious Area (as Percentage of Project Area)														
	25%			40%			50%			75%			90%		
	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m^3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)
TERRAIN : LOWLYING, SLOPE 1 : 2000 TO 1 : 5000															
0.1	28	129	55	35	134	55	41	143	56	61	156	56	77	164	56
0.2	55	182	78	70	189	78	82	202	80	96	226	80	154	231	80
0.4	110	257	111	140	267	111	164	286	113	192	319	113	308	327	113
0.6	165	315	135	210	327	135	246	350	138	288	391	138	462	401	138
0.8	220	364	156	280	378	156	328	404	160	384	451	160	616	463	160
1	275	407	175	350	422	175	410	451	178	610	451	178	770	517	178
2	730	451	247	980	505	247	1190	529	252	1870	576	252	2290	618	252
3	1095	553	303	1470	618	303	1785	648	309	2805	705	309	3435	757	309
4	1460	638	350	1960	714	350	2380	749	357	3740	814	357	4580	874	357
5	1825	714	391	2450	798	391	2975	837	399	4675	910	399	5725	977	399
TERRAIN : MILD, SLOPE 1 : 875 TO 1 : 1999															
0.1	26	134	61	33	143	61	38	147	61	55	164	62	68	171	62
0.2	52	189	86	65	202	86	75	208	86	109	231	87	136	242	87
0.4	104	267	122	130	286	122	150	294	122	218	327	124	272	342	124
0.6	156	327	149	195	350	149	225	360	149	327	401	151	408	419	151
0.8	208	378	172	260	404	172	300	416	172	436	463	175	544	484	175
1	260	422	192	325	451	192	375	465	192	545	517	195	680	541	195
2	680	479	247	900	529	247	750	553	247	1710	618	276	2130	638	276
3	1020	586	303	1350	648	303	1125	677	303	2565	757	339	3195	782	339
4	1360	677	350	1800	749	350	1500	782	350	3420	874	391	4260	903	391
5	1700	757	391	2250	837	391	1875	874	391	4275	977	437	5325	1010	437
TERRAIN : STEEP, SLOPE 1 : 100 TO 1 : 874															
0.1	25	138	64	30.5	147	64	35	156	64	51	167	65	62.5	175	65
0.2	49	195	90	61	208	90	70	220	90	102	237	92	125	247	92
0.4	98	276	128	122	294	128	140	311	128	204	335	130	250	350	130
0.6	147	339	156	183	360	156	210	381	156	306	410	159	375	428	159
0.8	196	391	181	244	416	181	280	440	181	408	474	183	500	495	183
1	245	437	202	305	465	202	350	492	202	510	529	205	625	553	205
2	640	505	286	840	553	286	1010	576	286	1500	638	290	1810	677	290
3	960	618	350	1260	677	350	1515	705	350	2250	782	355	2715	829	355
4	1280	714	404	1680	782	404	2020	814	404	3000	903	410	3620	958	410
5	1600	798	451	2100	874	451	2525	910	451	3750	1010	458	4525	1071	458

Region 3

Project Area (ha)	**Impervious Area (as Percentage of Project Area)**														
	25%			**40%**			**50%**			**75%**			**90%**		
	Volume (m3)	**Inlet & Overflow Dia. (mm)**	**Outlet Dia. (mm)**	**Volume (m3)**	**Inlet & Overflow Dia. (mm)**	**Outlet Dia. (mm)**	**Volume (m3)**	**Inlet & Overflow Dia. (mm)**	**Outlet Dia. (mm)**	**Volume (m3)**	**Inlet & Overflow Dia. (mm)**	**Outlet Dia. (mm)**	**Volume (m3)**	**Inlet & Overflow Dia. (mm)**	**Outlet Dia. (mm)**
TERRAIN : LOWLYING, SLOPE 1 : 2000 TO 1 : 5000															
0.1	32	129	58	36	138	58	39	143	59	49.5	156	59	57	164	59
0.2	63	182	81	71	195	81	78	202	83	96	226	83	113	231	83
0.4	126	257	115	142	276	115	156	286	117	192	319	117	226	327	117
0.6	189	315	141	213	339	141	234	350	144	288	391	144	339	401	144
0.8	252	364	163	284	391	163	312	404	166	384	451	166	452	463	166
1	315	407	182	355	437	182	390	451	185	495	492	185	565	517	185
2	740	422	257	850	479	257	960	505	262	1250	553	262	1450	597	262
3	1110	517	315	1275	586	315	1440	618	321	1875	677	321	2175	731	321
4	1480	597	364	1700	677	364	1920	714	371	2500	782	371	2900	845	371
5	1850	668	407	2125	757	407	2400	798	415	3125	874	415	3625	944	415
TERRAIN : MILD, SLOPE 1 : 875 TO 1 : 1999															
0.1	30	138	65	33	151	65	36	156	65	46	167	65	53	175	66
0.2	60	195	92	66	214	92	72	220	92	91	237	92	105	247	93
0.4	120	276	130	132	303	130	144	311	130	182	335	130	210	350	132
0.6	180	339	159	198	371	159	216	381	159	273	410	159	315	428	161
0.8	240	391	183	264	428	183	288	440	183	364	474	183	420	495	186
1	260	437	205	325	479	205	375	492	205	545	529	205	680	553	208
2	710	451	290	820	505	290	910	529	290	1180	597	290	1370	618	294
3	1065	553	355	1230	618	355	1365	648	355	1770	731	355	2055	757	360
4	1420	638	410	1640	714	410	1820	749	410	2360	845	410	2740	874	416
5	1775	714	458	2050	798	458	2275	837	458	2950	944	458	3425	977	465
TERRAIN : STEEP, SLOPE 1 : 100 TO 1 : 874															
0.1	29	147	69	31.5	156	69	34.5	164	70	43	175	70	49.5	182	70
0.2	57	208	97	63	220	97	69	231	98	86	247	98	99	257	98
0.4	114	294	137	126	311	137	138	327	139	172	350	139	198	364	139
0.6	171	360	168	189	381	168	207	401	170	258	428	170	297	446	170
0.8	228	416	194	252	440	194	276	463	197	344	495	197	396	515	197
1	285	465	217	315	492	217	345	517	220	430	553	220	495	576	220
2	690	505	307	790	553	307	870	576	311	1130	638	311	1310	658	311
3	1035	618	376	1185	677	376	1305	705	381	1695	782	381	1965	806	381
4	1380	714	434	1580	782	434	1740	814	440	2260	903	440	2620	931	440
5	1725	798	485	1975	874	485	2175	910	492	2825	1010	492	3275	1041	492

Region 4

Project Area (ha)	Impervious Area (as Percentage of Project Area)														
	25%			40%			50%			75%			90%		
	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m3)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)
TERRAIN : LOWLYING, SLOPE 1 : 2000 TO 1 : 5000															
0.1	23	113	50	29	124	52	34	129	52	46	143	52	55	147	52
0.2	46	160	71	57	175	73	67	182	73	96	226	73	109	208	73
0.4	92	226	101	114	247	103	134	257	103	192	319	103	218	294	103
0.6	138	276	124	171	303	127	201	340	127	288	391	127	327	360	127
0.8	184	319	143	228	350	146	268	364	146	384	451	146	436	416	146
1	230	357	160	285	391	164	335	407	164	460	451	164	545	465	164
2	540	391	226	690	451	231	800	479	231	1100	529	231	1290	553	231
3	810	479	276	1035	553	283	1200	586	283	1650	648	283	1935	677	283
4	1080	553	319	1380	638	327	1600	677	327	2200	749	327	2580	782	327
5	1350	618	357	1725	714	366	2000	757	366	2750	837	366	3225	874	366
TERRAIN : MILD, SLOPE 1 : 875 TO 1 : 1999															
0.1	22	124	55	27	134	55	31	138	55	43	151	55	51	160	55
0.2	43	175	78	54	189	78	62	195	78	86	214	78	102	226	78
0.4	86	247	111	108	267	111	124	276	111	172	303	111	204	319	111
0.6	129	303	135	162	327	135	186	339	135	258	371	135	306	391	135
0.8	172	350	156	216	378	156	248	391	156	344	428	156	408	451	156
1	215	391	175	270	422	175	310	437	175	430	479	175	510	505	175
2	540	422	247	680	479	247	790	505	247	1090	553	247	1290	597	247
3	810	517	303	1020	586	303	1185	618	303	1635	677	303	1935	731	303
4	1080	597	350	1360	677	350	1580	714	350	2180	782	350	2580	845	350
5	1350	668	391	1700	757	391	1975	798	391	2725	874	391	3225	944	391
TERRAIN : STEEP, SLOPE 1 : 100 TO 1 : 874															
0.1	21	129	59	25.5	138	59	29	143	59	40.5	156	59	48.5	164	59
0.2	41	182	83	51	195	83	58	202	83	81	220	83	97	231	83
0.4	82	257	117	102	276	117	116	286	117	162	311	117	194	327	117
0.6	123	315	144	153	339	144	174	350	144	243	381	144	291	401	144
0.8	164	364	166	204	391	166	232	404	166	324	440	166	388	463	166
1	205	407	185	255	437	185	290	451	185	405	492	185	485	517	185
2	520	451	262	650	505	262	750	553	262	1040	597	262	1230	618	262
3	780	553	321	975	618	321	1125	677	321	1560	731	321	1845	757	321
4	1040	638	371	1300	714	371	1500	782	371	2080	845	371	2460	874	371
5	1300	714	415	1625	798	415	1875	874	415	2600	944	415	3075	977	415

Region 5

Project Area (ha)	Impervious Area (as Percentage of Project Area)														
	25%			40%			50%			75%			90%		
	Volume (m³)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m³)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m³)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m³)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)	Volume (m³)	Inlet & Overflow Dia. (mm)	Outlet Dia. (mm)
TERRAIN : LOWLYING, SLOPE 1 : 2000 TO 1 : 5000															
0.1	32	133	61	37	141	62	40	147	62	50	160	62	58	168	62
0.2	63	188	86	73	200	87	80	208	87	96	226	87	115	237	87
0.4	126	265	122	146	283	124	160	294	124	192	319	124	230	336	124
0.6	189	325	149	219	346	151	240	355	151	288	391	151	345	411	151
0.8	252	375	172	292	400	175	320	416	175	384	451	175	460	475	175
1	315	419	192	365	447	195	400	465	195	500	507	195	575	531	195
2	750	443	272	880	484	276	970	510	276	1260	569	276	1460	604	276
3	1125	542	333	1320	593	339	1455	624	339	1890	697	339	2190	739	339
4	1500	626	384	1760	685	391	1940	721	391	2520	804	391	2920	854	391
5	1875	700	430	2200	765	437	2425	806	437	3150	899	437	3650	954	437
TERRAIN : MILD, SLOPE 1 : 875 TO 1 : 1999															
0.1	30	144	68	35	153	68	38	158	68	47	172	69	54	179	69
0.2	60	204	96	69	217	96	75	224	96	93	243	97	107	253	97
0.4	120	288	135	138	306	135	150	317	135	186	343	137	214	358	137
0.6	180	353	166	207	375	166	225	388	166	279	420	168	321	439	168
0.8	240	408	192	276	433	192	300	448	192	372	485	194	428	507	194
1	300	456	214	345	484	214	375	501	214	465	542	217	535	567	217
2	740	482	303	850	522	303	940	546	303	1220	606	307	1420	638	307
3	1110	590	371	1275	639	371	1410	669	371	1830	742	376	2130	782	376
4	1480	681	428	1700	738	428	1880	772	428	2440	857	434	2840	903	434
5	1850	761	479	2125	826	479	2350	863	479	3050	958	485	3550	1010	485
TERRAIN : STEEP, SLOPE 1 : 100 TO 1 : 874															
0.1	30	152	71	32.5	160	71	35	165	72	44.5	178	72	50.5	186	72
0.2	59	215	101	65	227	101	70	234	102	89	252	102	101	263	102
0.4	118	304	143	130	321	143	140	331	145	178	357	145	202	372	145
0.6	177	372	175	195	393	175	210	405	177	267	437	177	303	455	177
0.8	236	429	202	260	454	202	280	468	204	356	505	204	404	526	204
1	300	480	226	320	507	226	290	523	229	405	564	229	485	588	229
2	700	512	319	800	555	319	880	580	323	1130	640	323	1300	673	323
3	1050	627	391	1200	680	391	1320	710	396	1695	784	396	1950	825	396
4	1400	724	451	1600	785	451	1760	820	457	2260	906	457	2600	952	457
5	1750	810	505	2000	878	505	2200	917	511	2825	1013	511	3250	1065	511

Table 5.A4: Discharge and Pipe Diameter Relationship

Terrain: Lowlying

Discharge (m^3/s)	Pipe Diameter (mm)
0.01	236
0.02	247
0.04	270
0.06	292
0.08	315
0.1	338
0.12	361
0.16	406
0.22	474
0.24	497
0.28	542
0.32	587
0.36	633
0.42	701
0.48	769
0.52	814
0.56	860
0.62	928
0.66	973
0.72	1041
0.78	1109
0.82	1154

Terrain: Steep

Discharge (m^3/s)	Pipe Diameter (mm)
0.01	259
0.02	269
0.04	288
0.06	307
0.08	326
0.1	345
0.12	364
0.16	402
0.22	459
0.24	478
0.28	516
0.32	554
0.36	592
0.42	649
0.48	707
0.52	745
0.56	783
0.62	840
0.66	878
0.72	935
0.78	992
0.82	1030

Terrain: Mild

Discharge (m^3/s)	Pipe Diameter (mm)
0.01	250
0.02	261
0.04	282
0.06	303
0.08	325
0.1	346
0.12	367
0.16	410
0.22	474
0.24	495
0.28	538
0.32	581
0.36	623
0.42	687
0.48	751
0.52	794
0.56	836
0.62	900
0.66	943
0.72	1007
0.78	1071
0.82	1114

APPENDIX 5.B EXAMPLE - SIZING OSD TANKS

5.B1 – Underground OSD Tank

Problem:

A multi-purpose hall is to be developed within UiTM Kuala Pilah campus area. The project area is 0.61ha. 75% of it shall be occupied by building, access road and pavement while 25% by garden and turfed areas. The catchment area of the project where it connects to the main drain is 0.61ha and has a terrain slope of about 1:2000. It is more economical to construct an OSD tank than to upgrade the existing drainage system for this new development. Based on the design procedure, calculate the Permissible Site Discharge (PSD), Site Storage Requirement (SSR) and the inlet and outlet pipe sizes.

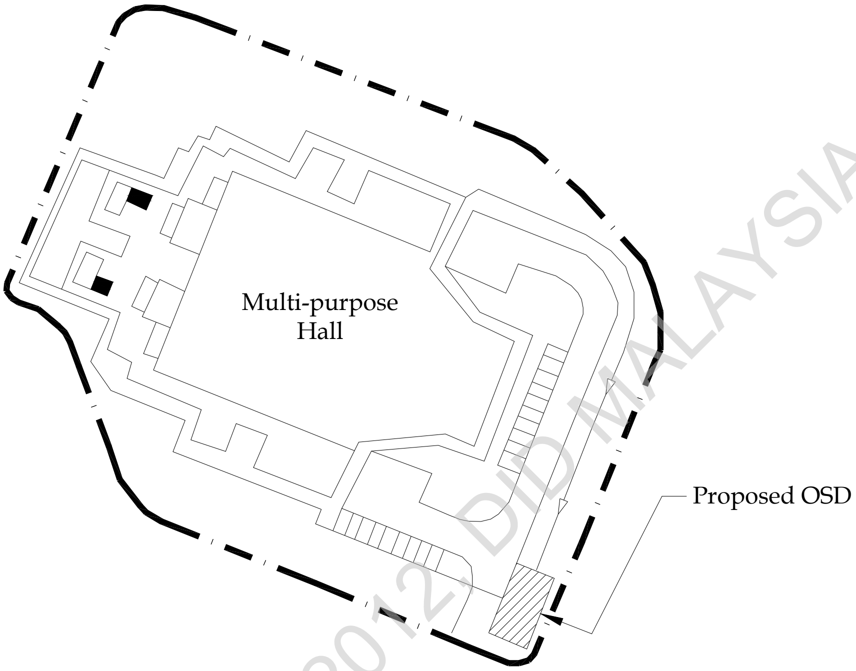


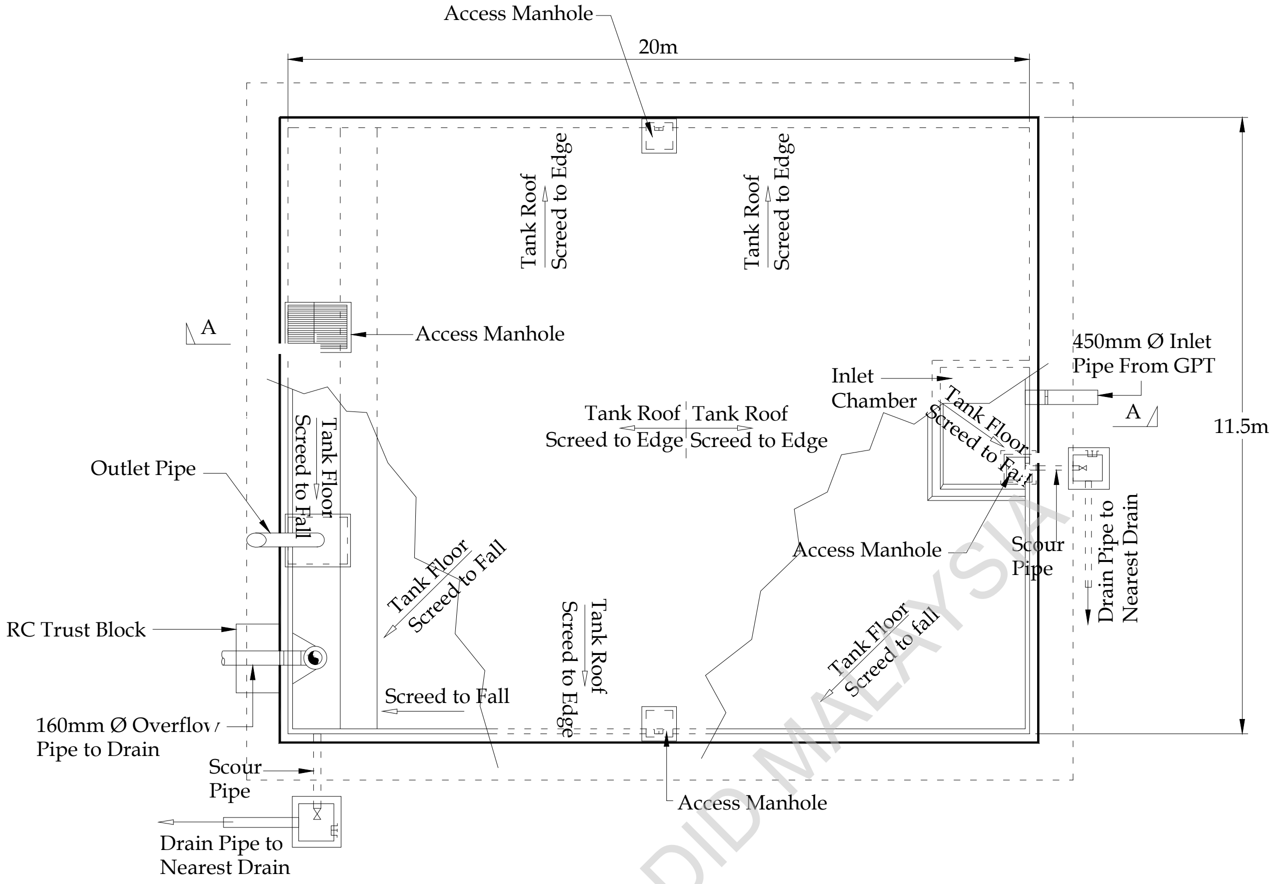
Figure 5.B1.1 Multi-Purpose Hall Layout at UiTM Kuala Pilah

Solution:

Reference	Calculation		Output
Figure 5.A1	Kuala Pilah falls under Region 1 — West Coast.		
	So, use OSD Characteristic for Region 1— West Coast.		
	Project Area = 0.61ha		
	Terrain = Mild		
	% of Impervious Area = 75%		
Table 5.A1	Permissible Site Discharge (PSD)/ha:	=	78.71 *l* /s/ha
	For area of 0.61 ha, PSD = 0.61 x 78.7	=	48.0 *l* /s
		=	0.048m^3/s

Reference	Calculation		Output
Table 5.A1	Site Storage Requirement (SSR)/ha:	=	$448.5m^3$/ha
	For area of 0.61ha, SSR = 0.61 x 448.5	=	$273.6m^3$
Table 5.A2	Inlet Flow/ha:	=	170.01/s/ha
	For area of 0.61 ha, Inlet Flow = 0.61 x 170.0	=	103.7 *l* /s
		=	$0.104m^3$/s
Table 5.A2	PSD/ha:	=	45.91/s/ha
	For area of 0.61 ha, PSD = 0.61 x 45.9	=	28.0 *l*/s
		=	$0.028m^3$/s
Table 5.A2	SSR/ha:	=	$349.2m^3$/ha
	For area of 0.61 ha, SSR = 0.61 x 349.2	=	$213.0m^3$
	Smaller PSD value is adopted for subsequent sizing of outlet pipe. Thus, PSD = $0.028m^3$/sec.	=	$0.028m^3$/s
Table 5.A3	Inlet Pipe: (adopt 450mm dia. as it is readily available in the market)	=	419mm dia. (Adopt 450mm dia.)
Table 5.A3	Outlet Pipe: (adopt 160mm dia. as it is readily available in the market)	=	170mm dia. (Adopt 160mm dia.)
Table 5.A4	Inlet Pipe: (with Inlet Flow of $0.104m^3$/sec)	=	350mm dia.
Table 5.A4	Outlet pipe: (with PSD of $.0028m^3$/sec)	=	270mm dia.
	Design Value Selected:		
	PSD: (whichever is smaller from Table 5.A1 and 5.A2)	=	28.0 *l*/s
	SSR: (whichever is larger from Table 5.A1 and 5.A2)	=	$273.6m^3$
	Sizing of OSD Tank: The required storage is $273.6m^3$ Adopt tank width of 20m, 12m length and a depth of 1.2m. Tank storage = 20m x 12m x 1.2m	=	$288m^3 > 273.6m^3$ ok
	Inlet Pipe: (whichever is smaller from Table 5.A3 and 5.A4)	=	350mm dia. Hence, adopt 350 mm dia. pipe size.
	Outlet Pipe: (whichever is smaller from Table 5.A3 and 5.A4)	=	170mm dia. Hence, adopt 160 mm dia. pipe size.

PLAN



SECTION A-A

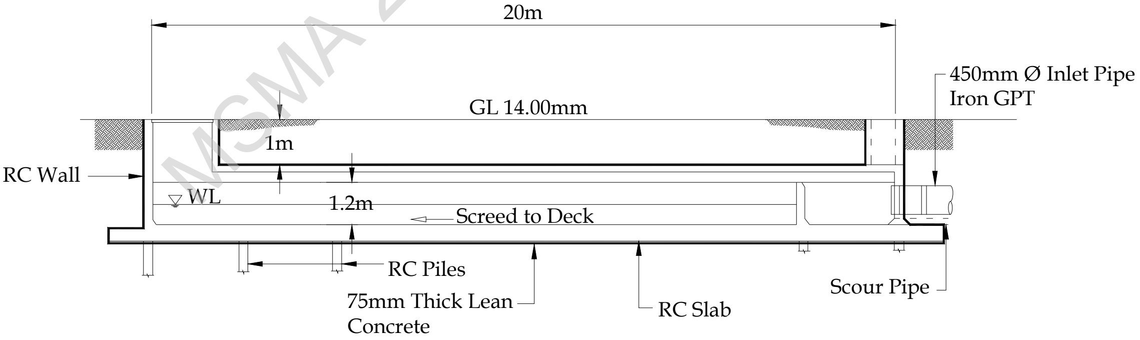


Figure 5.B1.2: Typical Detail Drawing for Below-Ground OSD Tank

5.B2 – Above Ground OSD Tank

Problem:

There is a tennis court located next to a multipurpose building of UiTM in Kluang, Johor. OSD tank needs to be built for the same reason as in Design Example 5.B1. The tennis court area is 0.7ha and 60% of it shall be paved. The catchment area of the project that connects to the main drain is about 0.7ha with a slope of about 1: 1800.

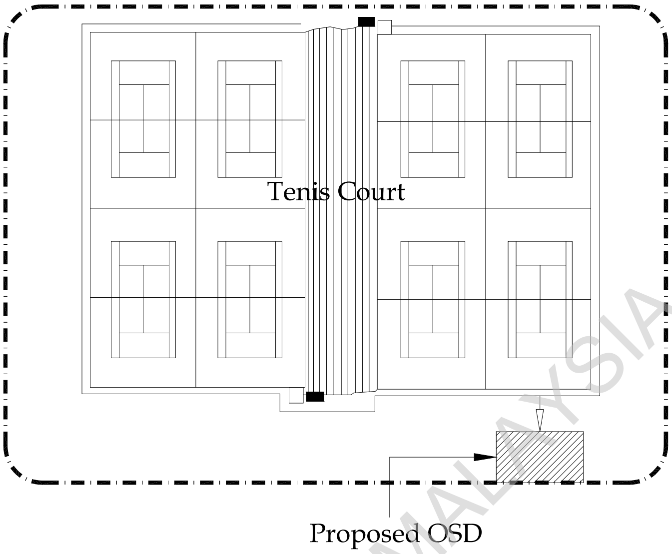


Figure 5.B2.1: Tennis Court at UiTM Kluang

Solution:

Reference	Calculation	Output
Figure 5.A1	Kluang falls under Region 5 - Southern.	
	So, use OSD Characteristic for Region 5— Southern.	
	Project Area = 0.70ha	
	Terrain = Mild	
	% of Impervious Area = 60%	
Table 5.A1	Permissible Site Discharge (PSD)/ha: =	76.4 *l*/s/ha
	For area of 0.7 ha, PSD = 0.7 x 76.4 =	53.5 *l*/s
	=	0.035m^3/s
Table 5A1	Site Storage Requirement (SSR)/ha : =	409.9m^3/ha
	For area of 0.7ha, SSR = 0.7 x 409.9 =	286.9m^3
	Sizing of OSD Tank: The required storage is 286.9m^3 Adopt tank width of 20m, 12m length and a depth of 1.2m. Tank storage = 20m x 12m x 1.2m =	288m^3 > 286.9m^3 ok

Reference	Calculation		Output
Table 5.A2	Inlet Flow: As Kluang is not in the list in Table 5.A2, refer to Table 5.A3 only.		
Table 5.A2	PSD: As Kluang is not in the list in Table 5.A2, refer to Table 5.A1 only.		
Table 5.A2	SSR: As Kluang is not in the list in Table 5.A2, refer to Table 5.A1 only.		
Table 5.A3	Inlet Pipe: (adopt 400mm Dia. as it is readily available in the market)	=	401mm dia. (Adopt 400mm dia.)
Table 5.A3	Outlet Pipe: (adopt 160mm Dia. as it is readily available in the market)	=	167mm dia. (Adopt 160mm dia.)
Table 5.A4	Inlet Pipe: As Kluang is not in the list in Table 5.A2, no checking for Table 5.A4 is required. Refer to Table 5.A3 only.		
Table 5.A4	Outlet pipe: As Kluang is not in the list in Table 5.A2, no checking for Table 5A4 is required. Refer to table 5.A3 only.		

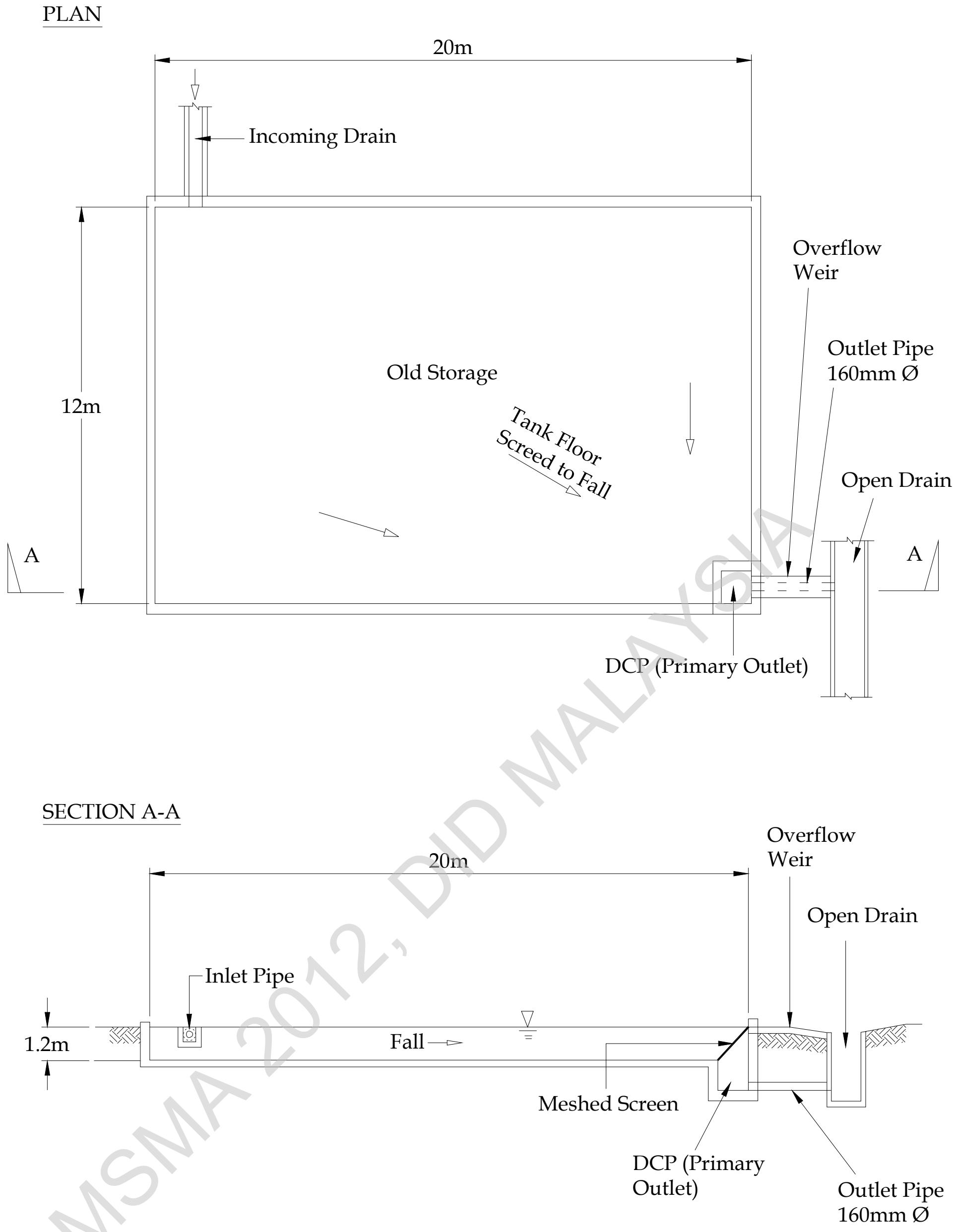
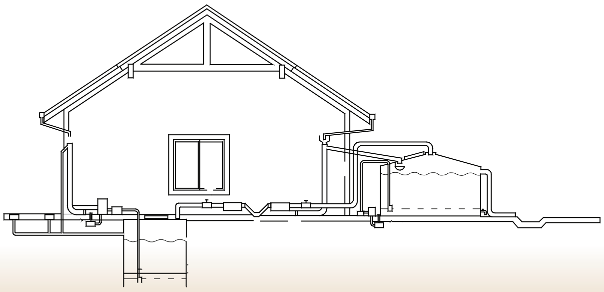


Figure 5.B2.2: Typical Detail Drawing for Above-Ground Tank



RAINWATER HARVESTING 6

CHAPTER 6 RAINWATER HARVESTING

6.1 INTRODUCTION

6.1.1 General

Rainwater harvesting is a technique of collecting rainfall as a supplementary source of water supply for households, commercial and industrial premises, landscape watering, livestock water, and irrigation of agriculture. The planning and development of rainwater harvesting systems shall be carried out adhering to the principles and guidelines described here. This is to ensure that the system complies with sustainability, water quality and design standard.

6.1.2 Principles

The value of rainwater as the primary source of clean water is always ignored. The aim of rainwater harvesting is to concentrate runoff and collect it in a basin for use. Rainwater harvesting using roof catchments is the easiest and most common method. Rainwater may also be collected from any impervious surface, such as stone, concrete, or asphaltic pavement. Landscape can also be contoured to maximize the catchment areas and runoff for rainwater collection.

6.1.3 Benefits

There are numerous benefits of rainwater harvesting:

- It provides an alternative water supply to supplement piped water;
- It is a green approach. It reduces the dependency of people on pipe water hence discourage dam construction and deforestation;
- It reduces water bills for consumer. Occasionally, there are economic advantages such as rebates from municipalities for a reduction in use and dependency on municipal water;
- On islands with limited fresh-water, rainwater harvesting is the major source of water for domestic use; and
- It educes stormwater flooding and soil erosion.

6.2 COMPONENTS OF A SYSTEM

6.2.1 Components

Whether it is large or small, a rainwater harvesting system (RWHS) has five basic components:

- Catchment area – the surface area which catches the rainfall. It may be a roof or impervious pavement and may include landscaped areas;
- Conveyance – channels or pipes that transport the water from catchment area to a storage;
- First flush – the systems that filter and remove contaminants and debris using separation devices;
- Storage tanks – where collected rainwater is stored; and
- Distribution – the system that delivers the rainwater to the point of use, either by gravity or pump.

In certain case where collected rainwater is for potable usage, purification involving filtering, distillation and disinfection are the optional components in rainwater harvesting system. The harvesting process from rainfall up to end user is conceptually shown in Figure 6.1. A typical detailing of the system either above-ground storage or below-ground storage is diagrammatically illustrated by Figure 6.2.

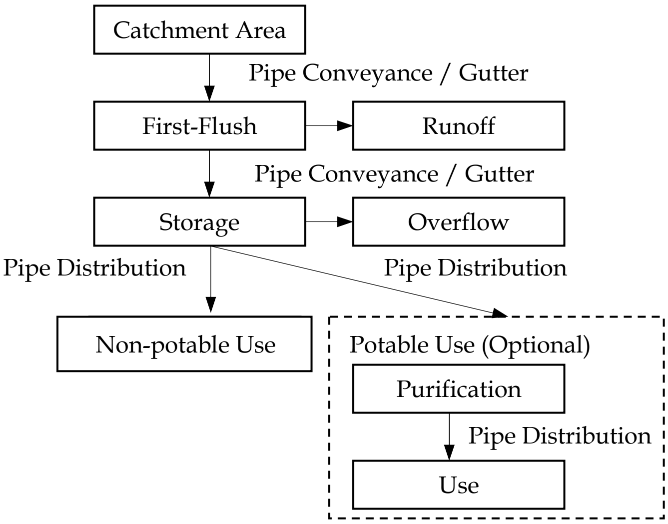
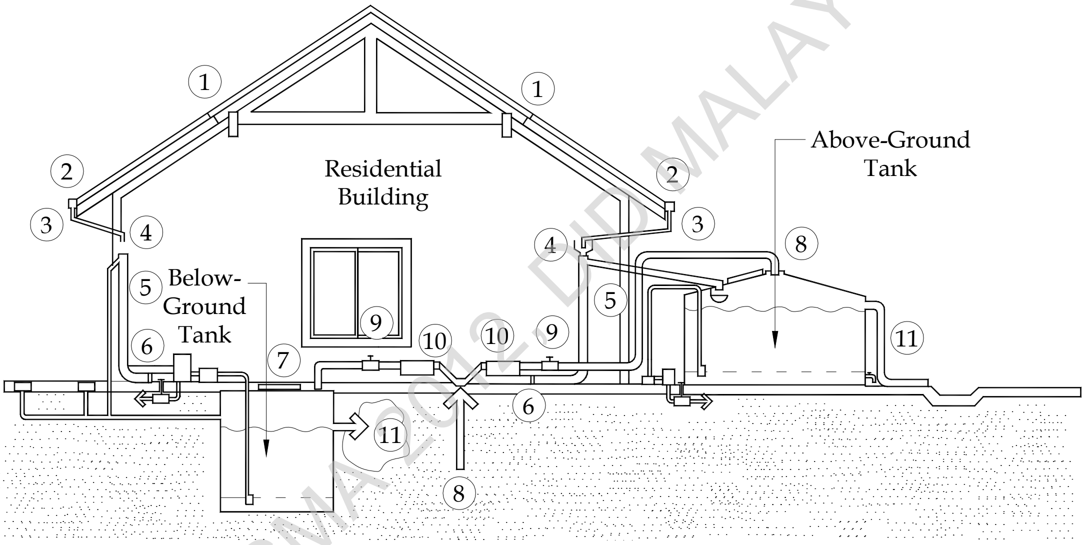


Figure 6.1: Rainwater Harvesting Process



Notes:
1. Roof collection
2. Gutter with leaf screen
3. Rain gutters
4. Downpipe
5. Debris and sediment removal, first flush device
6. Screw-off end cap for cleaning
7. Access for cleaning
8. Alternate water supply
9. Typical valve
10. Backflow prevention device
11. Overflow pipe

Figure 6.2: Typical Components of Rainwater Harvesting System for a Residential Building

6.2.2 Integration with OSD

The rainwater harvesting system can be integrated with OSD facilities (Chapter 5) that control a minor storm event. It is appropriate for large scale landscapes such as parks, school, commercial sites, parking lots, and apartment complexes and also small scale residential landscapes. Rainwater can be stored on flat roofs provided that adequate protection against leakage is catered for in the structural design of the building.

This type of storage has limited application in residential areas and is more suited to commercial and industrial buildings where large flat roof are available. OSD storage-cum-rainwater harvesting system can be provided as below-ground storages, above-ground storages or a combination of both.

6.2.3 Pumps

There are several types of pumps with different operating principles suitable for various conditions of use such as reciprocating pumps, centrifugal pumps, centripetal pumps, and centrifugal jet pumps. Each pump impeller has its own operating characteristics, which define its capability and efficiency. These can either be calculated for each individual pump and its impeller or the pump characteristic read from graphs and nomographs normally furnished by pump manufacturers for their products.

6.2.4 First Flush Device

Rainwater quality varies and is affected by environmental factors and commercial industrial activities in the area. The inclusion of the first flush device will improve the quality of the water. The device can be part of the rainwater downpipe, be separated from a tank or be attached to a tank. It can also be installed below ground.

Collection and disposal of the first flush of water from a roof, is of particular concern for any rainwater harvesting system. This is due to the fact that first flush picks up most of the dirt, debris, and contaminants, such as bird droppings that have been collected on the roof and in the gutters during dry periods. Multiple first flush devices may be required instead of a single first flush depending on slope of the catchments surface and time required for rainwater to reach the first flush device(s). Figure 6.3 shows the typical first flush systems.

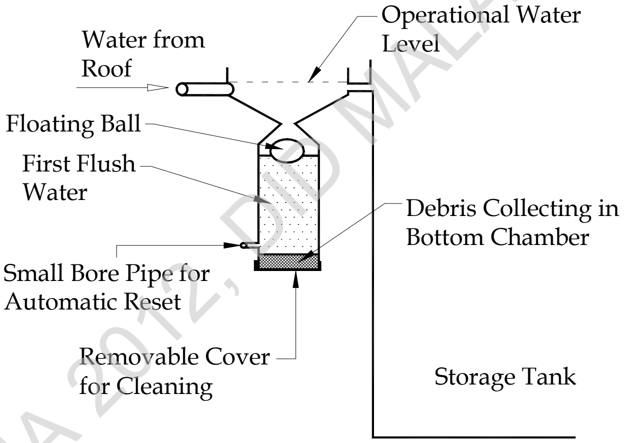


Figure 6.3: Typical First Flush Systems

6.3 CONFIGURATION OF TANK

6.3.1 Storage Type

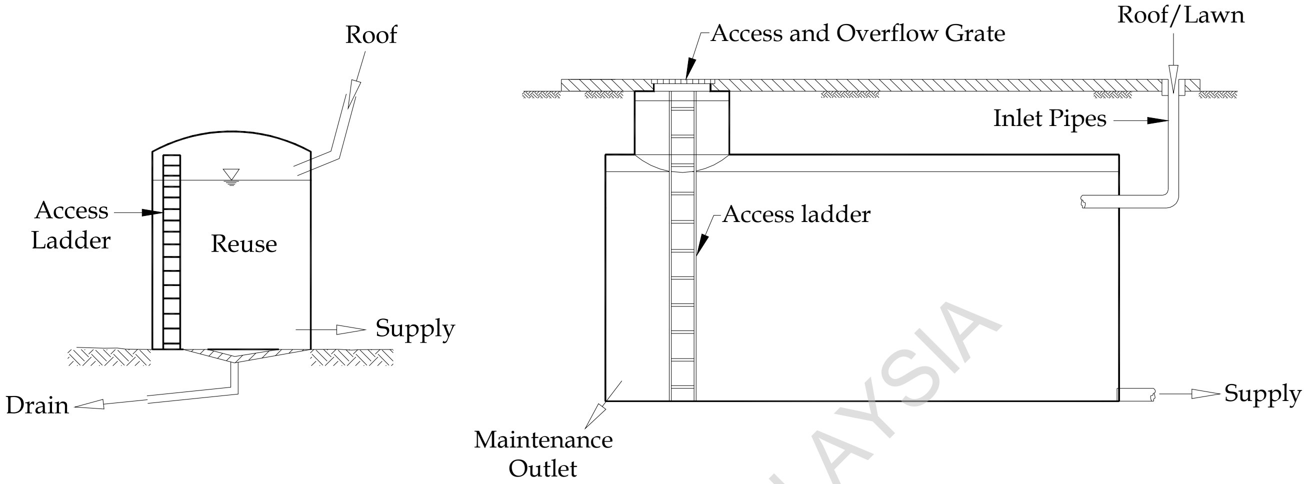
There are various types for rainwater harvesting tank. In general, the rainwater tank can be divided into either above-ground tank or below-ground tank as shown in Figure 6.4.

6.3.2 Shape of Storage Tank

The tank shapes can be circular, rectangular and others, such as trapeziums and pipes as provided in Table 5.2 of Chapter 5.0. There are a number of factors that govern what tank or tanks would be suitable in varying situation. These include:

- height of roof;
- roof catchment area identification;

- gutter and downpipe arrangements;
- space around the building;
- alignment of building in relation to boundaries; and
- local regulatory authority regulations with regard to water tanks.



a) Above-Ground

b) Below-Ground

Figure 6.4: Types of Storage Tank

Larger tanks often have to be cast in-situ. Fibreglass tanks, although more expensive, are more durable because they resist corrosion and are not generally affected by chemical or moisture. These tanks are manufactured with a good-grade coating on their interior surface. The tanks should also be manufactured to prevent the entry of light, which could encourage algae growth.

6.4 SIZING STORAGE TANK

6.4.1 Rainwater Demand

The rainwater demand depends on several factors. Table 6.1 shows the amount of water uses for different appliances and outdoor application which was adopted from Rainwater Harvesting Guidebook: Planning and Design (DID, 2009). Rainwater demand depends on:

- The number of people using the water;
- Average consumption per person; and
- The range of uses (drinking, bathroom, laundry, toilet, garden watering, etc.)

The rainwater demand can be reduced in certain water stress area and during dry period using water conservation devices.

6.4.2 Factors Affecting Rainwater Availability

Rainwater availability depends directly on several parameters such as rainfall characteristics, catchment area (roof area), and tank size. The other minor factors which affect the yield are first flush amount and losses on the roof, such as evaporation and splashing.

Table 6.1: Rainwater Demand for Domestic Application (DID, 2009)

Use (Appliance)	Type	Average Consumption	Average Total Rainwater Demand
A. Indoor			
Toilet	Single Flush	9 litres per flush	120 litres per day
	Dual Flush	6 or 3 litres per flush	40 litres per day
Washing Machine	Twin Tub (Semi- auto)		40 litres per wash
	Front Loading		80 litres per wash
	Top Loading		170 litres per wash
Dishwasher	-		20-50 litres per load
General Cleaning	-	10-20 litres per minute	150 litres per day
B. Outdoor			
Sprinkler or Handheld Hose		10-20 litres per minute	1000 litres per hour
Drip System			4 litres per hour
Hosing Paths/Driveways		20 litres per minute	200 litres per wash
Washing Car with a Running Hose		10-20 litres per minute	100-300 litres per wash

(a) *Rainfall Characteristics*

For a typical RWHS with a defined roof area, tank size and rainwater use pattern, the rainwater harvestable yield will depend mainly on the rainfall characteristics such as the sequence of rain-days and the daily rainfall amount. Generally, more rain-days yield higher rainwater amounts. Since the rainfall characteristics vary from place to place in Malaysia, the RWHS yield will therefore vary. A total of 17 major towns within Malaysia have been selected for the rainwater yield assessment. Table 6.2 shows the average annual rainfall and the number of rain-days for the selected towns.

Table 6.2: Mean Annual Rainfall and Number of Rain-days for Selected Towns

No.	Name of Town	Rainfall Station	Period of Record	Mean Annual Rainfall (mm)	Number of Annual Rain-day	Years of Record
1	Alor Star	6103047	1948-2007	2,365	147	42
2	Ipoh	4511111	1972-2008	2,288	181	38
3	Klang	3014084	1953-2008	2,197	132	55
4	Kuala Lumpur	3117070	1953-2008	2,527	177	56
5	Seremban	2719043	1959-2008	1,901	141	50
6	Melaka	2222010	1954-1998	1,989	179	45
7	Kluang	1833092	1948-2006	2,295	163	58
8	Johor Bahru	1537113	1948-2007	2,787	158	59
9	Kota Bharu	6121001	1981-2008	2,622	138	21
10	Kuala Terengganu	5331048	1954-2008	2,659	161	53
11	Kuantan	3833004	1948-2008	2,881	136	59
12	Kuching	1403001	1950-2008	4,043	242	59
13	Sibu	2219001	1999-2007	3,282	229	9
14	Bintulu	MMS 96441	1999-2008	4,136	225	10
15	Kota Kinabalu	5961002	1985-2009	2,629	177	25
16	Sandakan	5875001	1987-2008	3,070	190	18
17	Tawau	4278004	1989-2006	1,626	155	14

Note: When large gaps occur in a particular year, the entire data series of that year were discarded in analysis *(b) Rooftop Area*

The rooftop area corresponds to the rainwater catchment area. The rainwater harvestable yield is higher when rooftop area is larger.

(c) Rainwater Storage Tank Size

Rainwater tank is an important component in the RWHS from perspective of cost and space allocation. The rainwater harvestable yield is higher when the tank size is bigger. When tank size is increasing, the incremental increase in yield is decreasing because it is limited by roof area. Hence it is essential to determine the optimum tank size.

(d) First Flush Volume

The first flush is required to prevent contaminants collected at the roof surface from entering the storage tank. The first flush volume adopted is equivalent to 0.5mm of rainfall as shown in Table 6.3 (DID, 2009).

Table 6.3: First Flush Requirement According To Roof Area (DID, 2009)

Roof Area (m^2)	First Flush Volume (m^3)
Less than 100	0.025 - 0.05
100 - 4356	0.05 - 2.5
Greater than 4356	2.5

Note: Adopt first flush of 5 m^3 if surface contains excessive soil, dust or debris.

(e) Losses from Roofs

For houses in tropical region such as Malaysia, roof structures usually heat up due to prolonged exposure to sunshine. Other than in the Northeast Monsoon season, rainfall usually occurs in the late afternoon. Hence, 0.5 mm of rain falling on the roof is considered as evaporation and splashing losses.

6.4.3 Rainwater Availability Estimation

Two models are available in the rainwater availability estimation procedures suggested by Jenkins et al. (1987) which are:

- Yield Before Spillage (YBS) model; and
- Yield After Spillage (YAS) model.

The YBS model adopts an optimistic approach where the rainwater harvested will be supplied for daily consumption and the balance will be stored in the storage tank for next day use. On the other hand, the YAS model assumes a conservative approach where rainwater harvested will be channelled to the tank first, and the excess of rainwater will be overflowed. The daily consumption will be drawn from the tank.

With the RWHS described above and the adopted parameters, a daily water balance model has been configured to compute the daily rainwater yield based on YBS as shown in Figure 6.5. A spreadsheet has been developed to facilitate the daily water balance computation.

6.4.4 Average Annual Rainwater Yield Estimation

Estimation of the Average Annual Rainwater Yield (AARY) was carried out using daily water balance model for the selected towns in Malaysia adopting YBS method. The estimation was carried out to assess a typical case of household/residential house with 5 persons. Roof area of the building is 100m². The adopted first flush and losses from roof were estimated to be 1 mm as recommended in Section 6.4.2.

The analyses results shown that, e.g. in Alor Star, the AARY is 103m³ if tank size of 1m³ is used. When the tank size is 3 times bigger (3m³), the AARY is merely increased by 32% to 136m³ only. Therefore, tank size of 1m³ is the near optimum in term of highest AARY per unit tank size for Alor Star. Similarly for other towns, a tank size of 1m³ is the near optimum tank size. Table 6.4 below presents the AARY from the YBS model for all the selected towns.

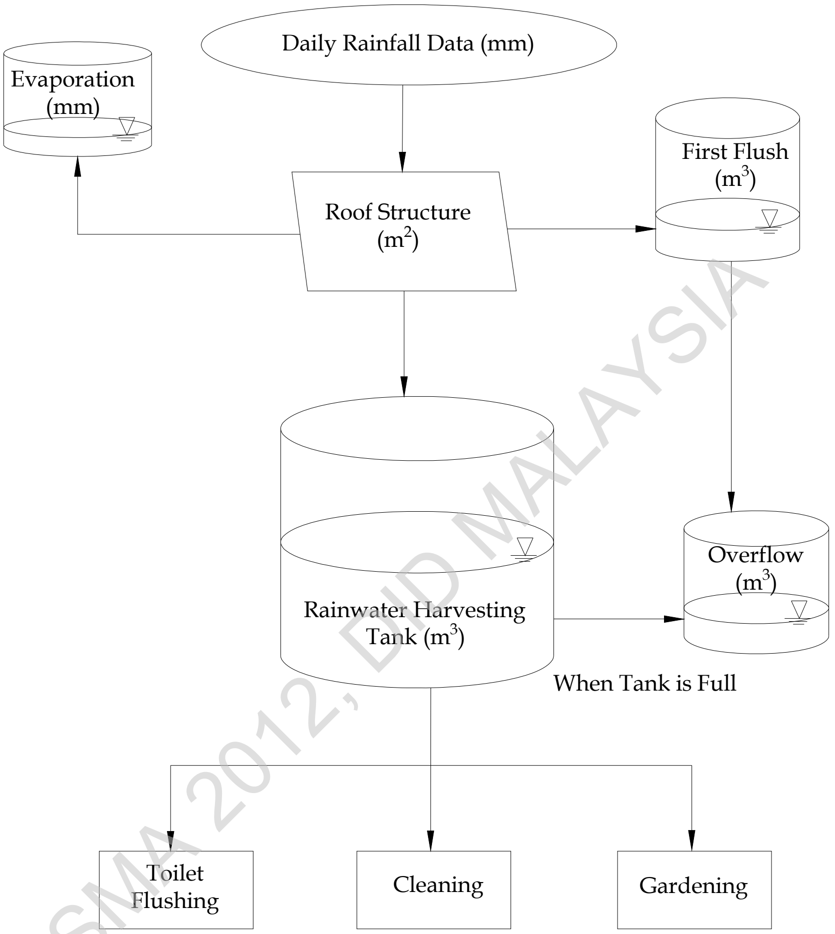


Figure 6.5: Schematic Diagram for the Daily Water Balance based on YBS Model

6.4.5 Tank Size Estimation

From Section 6.4.4, the tank size for Malaysia regardless of location is 1 m³ for roof area of 100m². It is equivalent to store 10mm of rainfall with 100m² of roof area. As such, tank size can be estimated with the following equation:

$$S_t = 0.01A_r \tag{6.1}$$

where,

S_t = Tank Size (m³); and
A_r = Rooftop Catchment Area (m²).

Equation 6.1 is established by assuming the demand is proportional to the roof area and tank size. The tank size with varying daily demand and roof area may have variation of ± 25% using the above simplified approach. A more methodical approach, i.e., a daily water balance model using the specific roof area and daily rainwater demand for a particular project can be carried out if a more accurate result is required.

Table 6.4: Average Annual Rainwater Yield for Selected Towns

No.	Name of Town	Average Annual Rainwater Yield (m^3)
1	Alor Star	103
2	Ipoh	99
3	Klang	107
4	Kuala Lumpur	116
5	Seremban	98
6	Melaka	100
7	Kluang	115
8	Johor Bahru	128
9	Kota Bharu	95
10	Kuala Terengganu	94
11	Kuantan	111
12	Kuching	156
13	Sibu	144
14	Bintulu	148
15	Kota Kinabalu	109
16	Sandakan	120
17	Tawau	89

Note: AARY was computed from tank size of $1m^3$ and roof area of $100m^2$.

6.5 SIZING CONVEYANCE AND DISTRIBUTION SYSTEMS

6.5.1 Rainwater Collection and Conveyance

This system component is designed based on the procedure presented in Chapter 4, Section 4.3. It comprises gutters, rainheads, sumps and downpipes.

6.5.2 Distribution Pipe

In designing for water supply installation, an assessment must first be made of the probable maximum water flow. In most buildings it seldom happens that the total numbers of appliances installed are ever in use at the same time, and therefore, for economic reasons, it is usual for a system to be designed for a peak usage which is less than the possible maximum usage. The probable maximum demand can be assessed based on the theory of probability. This method uses a loading unit rating which is devised for each type of appliance, based on its rate of water delivery, the time the taps are open during usage, and the simultaneous demand for the particular type of appliance. Table 6.5 gives the loading unit rating for various appliances.

In building where high peak demands occur, a loading unit rating for such appliances is not applicable and 100% of the flow rate for these appliances is required as shown in Table 6.6. The same applies to automatic flushing cisterns for urinals.

The pipe sizing can be determined using the Thomas-Box equation (DID, 2009):

$$q = \sqrt{\frac{d^5 \times H}{25 \times L \times 10^5}} \tag{6.2}$$

where,

q = Discharge through the pipe (L/s);

d = Diameter of pipe (mm);
H = Head of water (m); and
L = Total length of pipe (m).

The diameter of the pipe necessary to give a required flow rate will depend upon the head of water available, the smoothness of the internal bore of the pipe and the effective length of the pipe. An allowance for the frictional resistance set up by fittings such as elbows, tees, taps and valves must be added to the actual length of the pipe. Table 6.7 gives the allowance for fittings expressed in equivalent pipe lengths.

Table 6.5: Loading Unit Rating for Various Applications

Type of Appliance	Loading Unit Rating
Dwelling and Flats	
W.C. Flushing Cistern	2
Wash Basin	1.5
Bath	10
Sink	3 - 5
Offices	
W.C. Flushing Cistern	2
Wash Basin (Distributed Use)	1.5
Wash Basin (Concentrated Use)	3
School and Industrial Buildings	
W.C. Flushing Cistern	2
Wash Basin	3
Shower (with Nozzle)	3
Public Bath	22

Table 6.6: Recommended Minimum Flow Rate at Various Appliances

Type of Appliance	Rate of Flow (*l*/s)
W.C. Flushing Cistern	0.12
Wash Basin	0.15
Wash Basin with Spray Taps	0.04
Bath (Private)	0.30
Bath (Public)	0.60
Shower (with Nozzle)	0.12
Sink with 13 mm Taps	0.20
Sink with 19 mm Taps	0.30
Sink with 25 mm Taps	0.60

Table 6.7: Frictional Resistance of Fittings Expressed in Equivalent Pipe Length

Nominal Outside Diameter (mm)	Equivalent Pipe Length (m)		
	Elbow	Bend	Tee
15	0.5	0.4	1.2
20	0.6	0.5	1.4
25	0.7	0.6	1.8
32	1.0	0.7	2.3
40	1.2	1.0	2.7
50	1.4	1.2	3.4
65	1.7	1.3	4.2
80	2.0	1.6	5.3
100	2.7	2.0	6.8

In calculating the diameter of a pipe to supply individual fittings, the head loss through the draw-off tap should also be taken into account. Table 6.8 gives the allowances for draw-off taps expressed in equivalent pipe lengths.

Table 6.8: Frictional Resistance of Draw-off Taps Expressed as Equivalent Pipe Lengths

Fitting (BS 1010)	Discharge Rate with Tap Fully Open (litre/s)	Equivalent Pipe Length (m)	
		Copper	Galvanized Steel
15 mm Diameter Bib-tap or Pillar Tap	0.20	2.70	4.00
20 mm Diameter Bib-tap or Pillar Tap	0.30	8.50	5.75
25 mm Diameter Bib-tap or Pillar Tap	0.60	20.00	13.00

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1. Chong S.F., Lim C.K., Lee C.S. (2010). *Rainwater Harvesting Systems in Malaysia: How Much Can We Harvest?* IEM Colloquium.
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3. Jenkins, D., Pearson, F. Moore, E. Sun, K. Valentine, R. (1987). *Feasibility of Rainwater Collection Systems in California, Contribution no. 173*. University of California (UCLA).
4. Ministry of Housing and Local Government, Malaysia, (1999). *Guidelines for Installing a Rainwater Collection and Utilization System.*

APPENDIX 6.A SIZING CHARTS

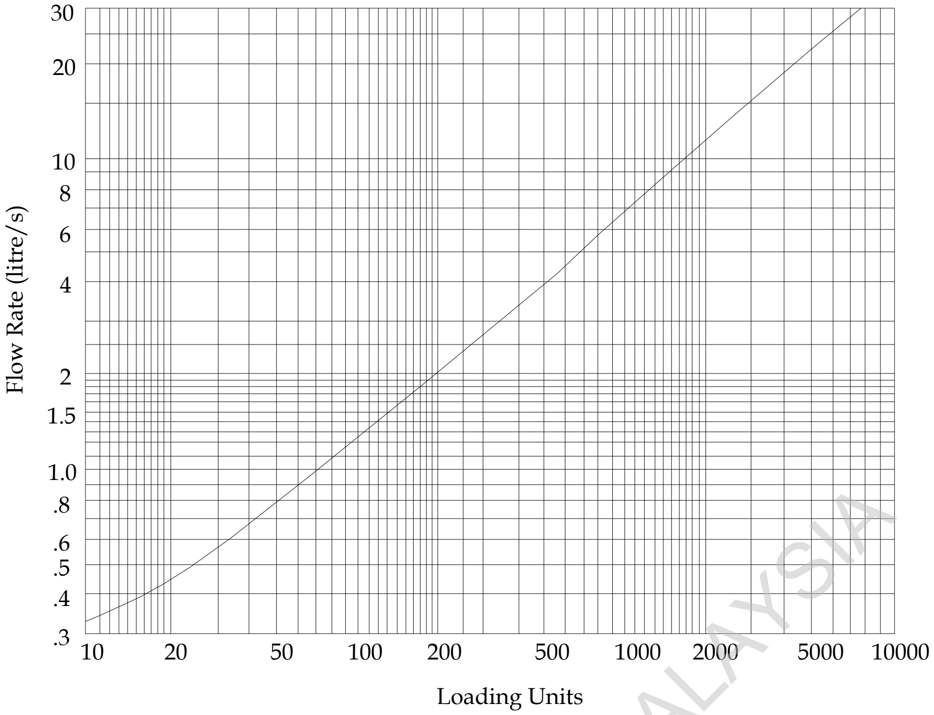


Figure 6.A1: Design Flow Rate for Various Loading Units

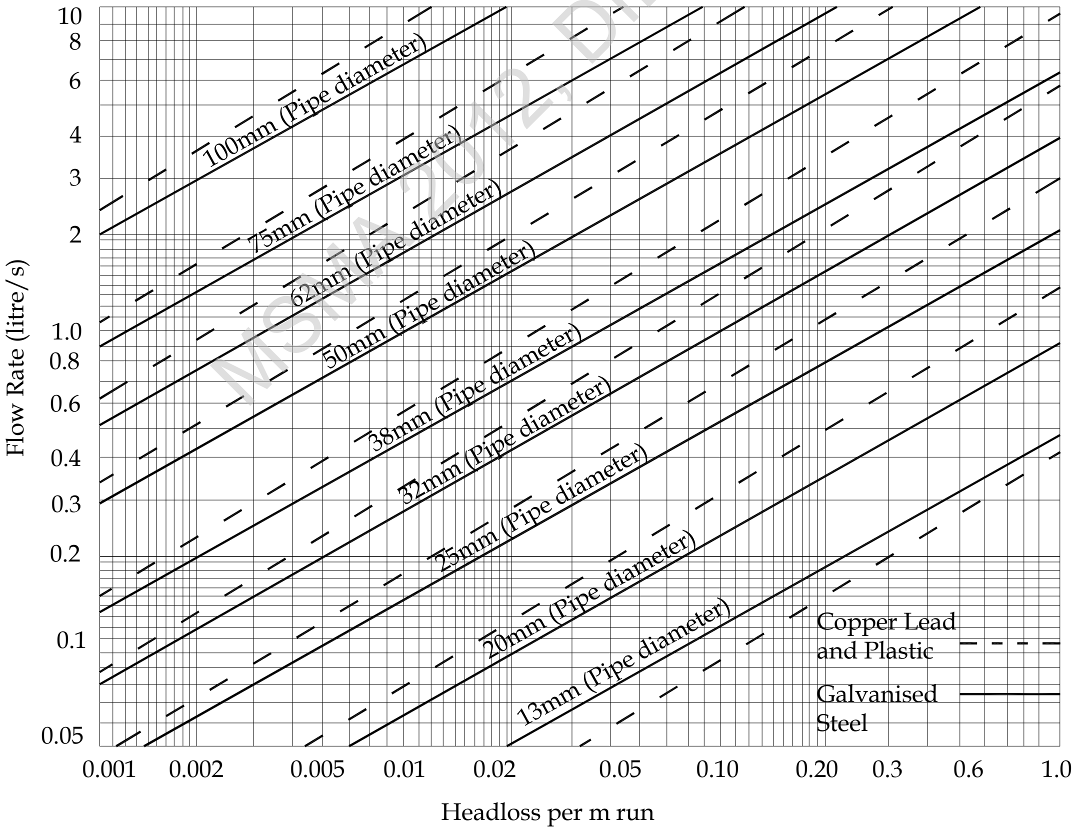


Figure 6.A2: Headloss for Various Pipe Size and Flow Rate

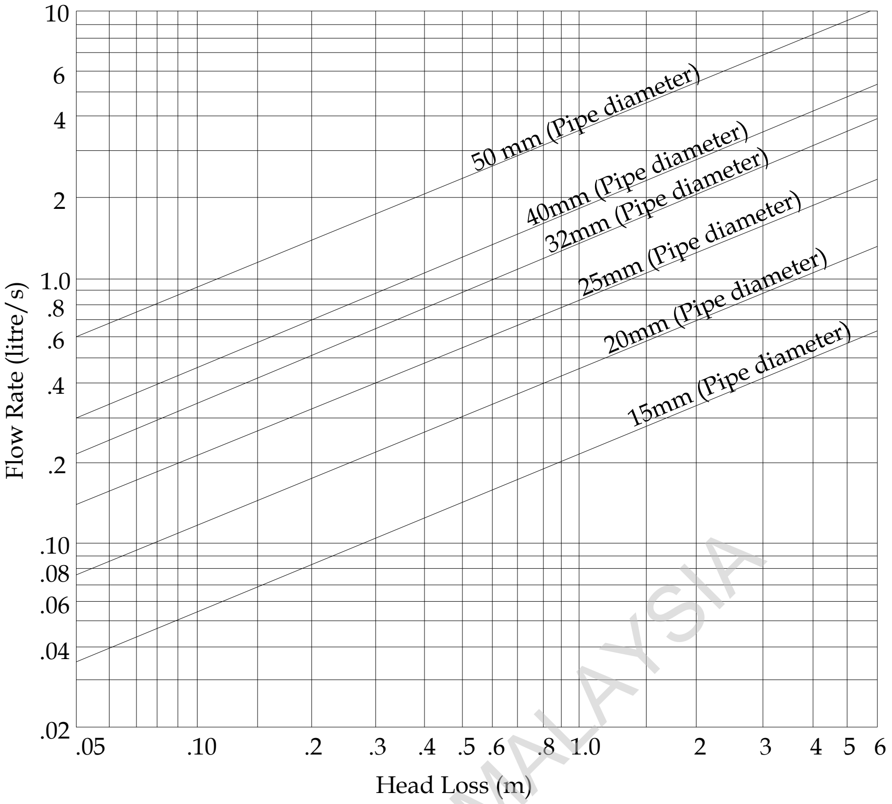


Figure 6.A3: Headloss through Stop Valve

APPENDIX 6.B EXAMPLE – SIZING TANK AND PIPE

6.B1 Tank Sizing for Residential Development

Problem:

A bungalow development is proposed in Kuala Lumpur with the inclusion of a rainwater harvesting system in the design. The roof area of each bungalow is 200m^2 with a car porch and garden. The bungalow is designed with five rooms with a twin sharing concept. Each room is equipped with one dual flush toilet. Given that domestic water demand is 250 litre/capita/day, compute:

- Annual rain water demand;
- Rainwater tank size;
- Percentage of rainwater yield over rainwater demand; and
- Percentage of rainwater yield over domestic water demand.

Solution:

<table>
<tr><th>Reference</th><th>Calculation</th><th colspan="2">Output</th></tr>
<tr><td></td><td>Compute annual rain water demand (m^3)</td><td></td><td></td></tr>
<tr><td>Table 6.1</td><td>
<table>
<tr><th>Use (Appliance)</th><th>Unit</th><th>Average Water Use</th><th>Total Water Use (litre/day)</th></tr>
<tr><td>Dual Flush Toilet</td><td>5 nos</td><td>40 litres/day</td><td>200</td></tr>
<tr><td>Washing Machine (Front Loading)</td><td>1 wash</td><td>80 litres/wash</td><td>80</td></tr>
<tr><td rowspan="2">Dishwasher/
General Cleaning</td><td rowspan="2">3 loads</td><td rowspan="2">50 litres per load</td><td>150</td></tr>
<tr><td>150</td></tr>
<tr><td>Gardening</td><td>20minutes</td><td>20 litres/minute</td><td>400</td></tr>
<tr><td>Washing 5 Cars with Running Hose for 20 Minutes/Car Once a Week</td><td>20minutes/ car</td><td>20 litres/minute</td><td>285.7
(5 x 20 x 20 /7)</td></tr>
<tr><td>Total</td><td></td><td></td><td>1266</td></tr>
</table>
</td><td></td><td></td></tr>
<tr><td></td><td>The annual rainwater demand = 365 days x 1266 l/day</td><td>=</td><td>462m^3</td></tr>
<tr><td></td><td>Tank size estimation (m^3)</td><td></td><td></td></tr>
<tr><td>Equation 6.1</td><td>With rooftop catchment, A_r = 200m^2, Tank Size , S_t = 0.01m x 200m^2</td><td>=</td><td>2m^3</td></tr>
<tr><td></td><td>Compute Average Annual Rainwater Yield (m^3)</td><td></td><td></td></tr>
<tr><td>Table 6.4</td><td>For Kuala Lumpur, the AARY for 2m^3 tank size = 2 x 116</td><td>=</td><td>232m^3</td></tr>
<tr><td></td><td>Compute percentage of water yield over rainwater demand</td><td></td><td></td></tr>
<tr><td></td><td>Percentage of rainwater yield over rainwater demand = 232/462 x 100</td><td>=</td><td>50.2%</td></tr>
<tr><td></td><td>Compute annual domestic water demand (m^3)</td><td></td><td></td></tr>
<tr><td></td><td>Annual domestic water demand = 365 days x 10 capita x 250 litres/capita/day</td><td>=</td><td>912m^3</td></tr>
<tr><td></td><td>Compute percentage of water yield over domestic water demand</td><td></td><td></td></tr>
<tr><td></td><td>Percentage of water yield over domestic water demand = 232/912 x 100</td><td>=</td><td>25.4%</td></tr>
</table>

6.B2 Tank Sizing for Industry/Factory

Problem:

A development project is proposed involving building a factory with roof area of 5000m^2 in Pasir Mas, Kelantan. The factory is designed to house 500 workers. The factory toilets are equipped with single flush system. This factory has a small landscaping area at the entrance of the factory. Compute:

- annual rain water demand;
- rainwater tank size; and
- percentage of rainwater yield over rainwater demand.

Solution:

<table>
<tr><th>Reference</th><th colspan="4">Calculation</th><th>Output</th></tr>
<tr><td></td><td colspan="4">Compute annual rain water demand (m^3)</td><td></td></tr>
<tr><td>Table 6.1</td><td>Use (Appliance)</td><td>Unit</td><td>Average Water Use</td><td>Total Water Use (l/day)</td><td></td></tr>
<tr><td></td><td>Single flush toilet with 9 l/flush</td><td>500 workers</td><td>5 flushes/day</td><td>22500</td><td></td></tr>
<tr><td></td><td>Gardening</td><td>40 minutes</td><td>20 l/minute</td><td>800</td><td></td></tr>
<tr><td></td><td>Total</td><td></td><td></td><td>23300</td><td></td></tr>
<tr><td></td><td colspan="4">The annual rainwater demand = 365 days x 23300 litres/day =</td><td>8504m^3</td></tr>
<tr><td></td><td colspan="4">Tank size estimation (m^3)</td><td></td></tr>
<tr><td>Equation 6.1</td><td colspan="4">With rooftop catchment, A_r = 5000m^2, Tank Size , S_t = 0.01m x 5000m^2 =</td><td>50m^3</td></tr>
<tr><td></td><td colspan="4">Compute Average Annual Rainwater Yield for town (m3)</td><td></td></tr>
<tr><td>Table 6.4</td><td colspan="4">Take the nearest Kota Bharu, the AARY for 50m^3 tank size = 50 X 95 =</td><td>4750m^3</td></tr>
<tr><td></td><td colspan="4">Compute percentage of water yield over rainwater demand</td><td></td></tr>
<tr><td></td><td colspan="4">Percentage of rainwater yield over rainwater demand = 4750/8504 X 100 =</td><td>55.9%</td></tr>
</table>

6.B3 Pipe Sizing

Problem:

This worked example shows how the main pipe can be sized up for rainwater tank serving a typical bathroom at the same factory in Worked Example 6.B2. The appliances in the bathroom consists of 5 W.C. flushing cisterns, 5 wash basins and 5 showers with nozzles. The actual length of the main pipe is 15 metres. The layout of the system is shown in Figure 6.B1. Assuming the system used 25mm (O.D) copper pipe with an available head of 5m:

- compute the design flow rate (l/s) of the loading units; and
- determine whether the pipe size used acceptable?

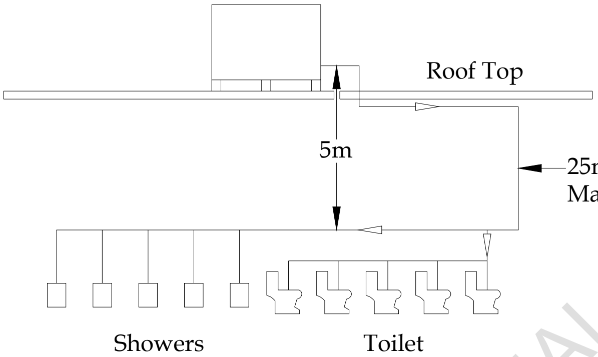
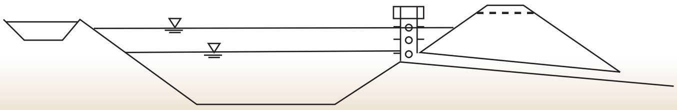


Figure 6.B1: Example layout of the Plumbing System

Solution:

<table>
<tr><th>Reference</th><th colspan="4">Calculation</th><th>Output</th></tr>
<tr><td></td><td colspan="4">Compute the loading rating per unit appliance</td><td></td></tr>
<tr><td>Table 6.5</td><td>Piping Component</td><td>Loading Rating per Unit Appliance (unit)</td><td>Number of Appliance (nos)</td><td>Total Loading (unit)</td><td></td></tr>
<tr><td></td><td>W.C. flushing system (WC)</td><td>2</td><td>5</td><td>10</td><td></td></tr>
<tr><td></td><td>Wash basin (WB)</td><td>3</td><td>5</td><td>15</td><td></td></tr>
<tr><td></td><td>Shower (SR)</td><td>3</td><td>5</td><td>15</td><td></td></tr>
<tr><td></td><td>Total loading</td><td></td><td></td><td>40</td><td></td></tr>
<tr><td></td><td colspan="4">Compute the flow rate</td><td></td></tr>
<tr><td>Figure 6A1</td><td colspan="4">The flow rate for 40 units loading =</td><td>0.70 l/s</td></tr>
<tr><td></td><td colspan="4">Compute the flow rate</td><td></td></tr>
<tr><td></td><td colspan="4">With 25 mm of OD copper pipe,</td><td></td></tr>
<tr><td>Table 6.7</td><td>Piping Component</td><td>Equivalent Length (m)</td><td>Number of Component (Nos.)</td><td>Total Headloss (m)</td><td></td></tr>
<tr><td></td><td>Elbow</td><td>0.7</td><td>4</td><td>2.8</td><td></td></tr>
<tr><td></td><td>Tee</td><td>1.8</td><td>1</td><td>1.8</td><td></td></tr>
<tr><td></td><td>Total Headloss due to friction resistance in fittings</td><td></td><td></td><td>4.6</td><td></td></tr>
</table>

Reference	Calculation		Output
	<u>Compute Effective Length of Pipe</u>		
	Effective length =15 + 4.6	=	19.6m
	<u>Compute headloss in pipe due to frictional resis6-7tance</u>		
Figure 6A2	The head loss in 25mm copper pipe due to frictional resistance	=	0.10m
	<u>Compute headloss due to fitting of stop valve</u>		
Figure 6A3	The head loss due to fitting of stop valve	=	0.6m
	<u>Compute total headloss</u>		
	The total headloss due to pipe and fittings = (19.6 × 0.1) + 0.6	=	2.56m
	<u>Compute residual head (>0)</u>		
	Residual head = available head – total headloss= 5 – 2.56	=	2.44m Since >0, OK
	<u>Compute required pipe size using Thomas-Box equation</u>		
Equation 6.2	$d = \sqrt[5]{\frac{0.70^2 \times 25 \times 19.6 \times 10^5}{2.44}}$	=	25.04mm 25mm is OK



DETENTION POND

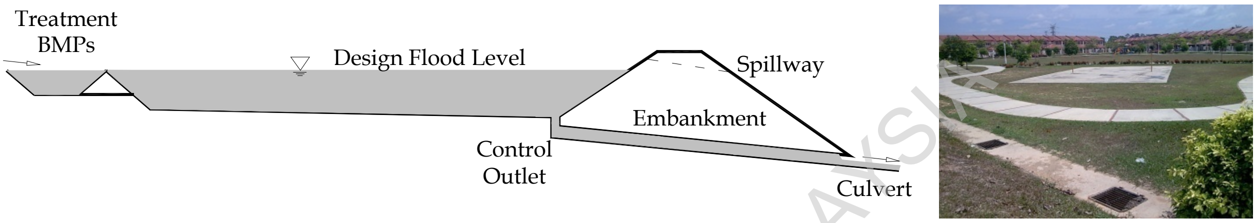
7

CHAPTER 7 DETENTION POND

7.1 INTRODUCTION

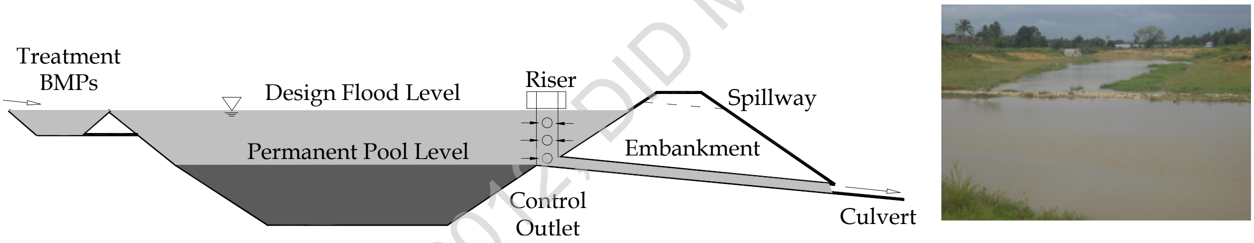
This Chapter provides guidelines and procedures for designing detention pond, a facility presently regarded as the most important measure in the stormwater management practice. Detention ponds are used for controlling stormwater quantity impacts resulting from larger urbanising catchment. The facilities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

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



Concept Completion

(a) Dry Type - Kemaman



Concept Completion

(b) Wet Type - Malacca

Figure 7.1: Typical Detention Ponds

7.1.1 System Components and Configurations

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

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

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

7.1.2 Water Quality Control Requirement

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

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

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

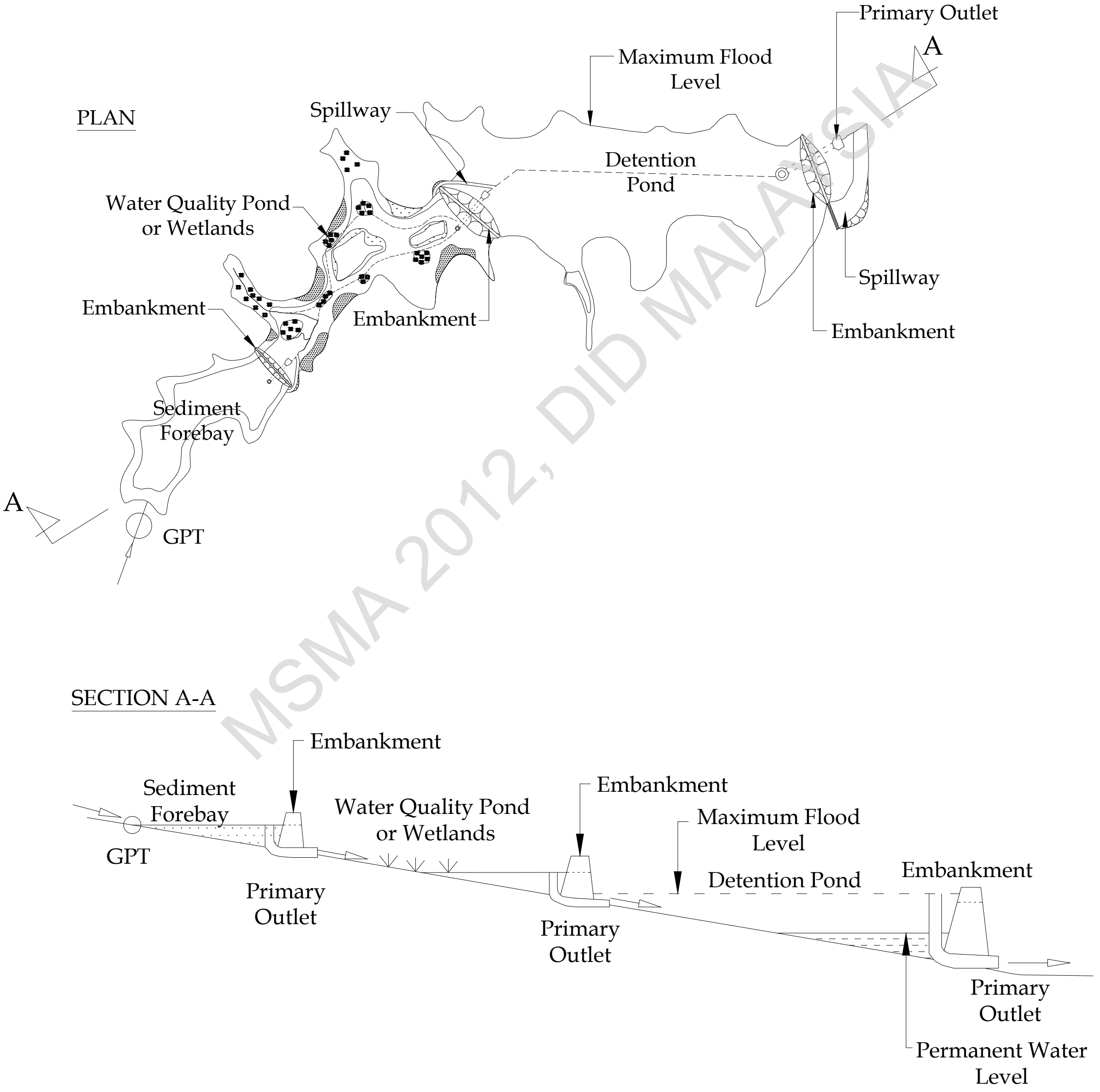


Figure 7.2: Typical Detention Pond Components

7.2 DESIGN CONSIDERATION

7.2.1 General

(a) Design and Analysis

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

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

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

(b) Release Timing

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

(c) Multiple Design Storms

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

(d) Extreme Floods

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

(e) Public Safety

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

Other requirements are set as follows:

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

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

7.2.2 Inlet

7.2.2.1 Inlet Structure

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

7.2.2.2 GPTs and Sediment Forebay

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

7.2.2.3 Water Quality Pond or Wetlands

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

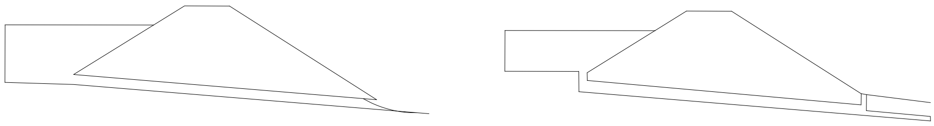
7.2.3 Primary Outlet

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

For a single stage, the facility is typically designed as a simple culvert. For multi-stage system, the outlet control structure is designed by considering a range of design flows. A stage-discharge curve is developed for the full range of flows that the structure would experience. The design flows are typically orifice flow through whatever shape chosen, while the higher flows are typically weir flow over the top of the control structure. The outlets are typically housed in a riser structure connected to a single outlet conduit that passes through the pond embankment and discharges to the downstream conveyance system. Orifices and weirs can be designed using the equations provided in Chapter 2.

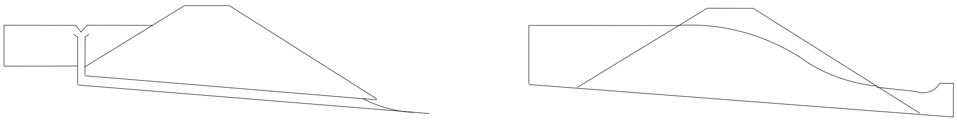
Primary outlets for detention ponds shall be designed to reduce post-development peak flows to pre-development peak flows for the minor and major system design storm ARI (Table 1.1). For this will require a two-staged outlet configuration, one to control the minor system design flow and the other to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

This requirement can readily be achieved for new development areas as sufficient land can be set aside to accommodate the necessary storage requirements. However, in existing developed areas, reducing the major system design ARI flow to the pre-development rate may not be practical in some cases due to limited availability of suitable detention sites. In such cases, detention facilities should be sized to attenuate the minor system design ARI only while addition of more OSD facilities are encouraged to lower peak flow for the major system instead.



a) Pipe or Box Culvert

c) Drop Inlet Pit
(surcharge pit or culvert outlet)



b) Riser Structure
(single and multi-level outlet)

d) Slotted Outlet

Figure 7.3: Typical Pond Primary Outlets

7.2.3.1 Culverts

The design of these outlets can be for either single or multi-stage discharges. A single stage discharge system typically consists of a single culvert entrance system, which is not designed to carry emergency flows. A multi-stage inlet typically involves the placement of a control structure at the inlet end of the culvert. The inlet structure is designed in such a way that the design discharge passes through a weir or orifice in the lower levels of the structure and the emergency flows pass over the top of the structure.

The culvert needs to be designed to carry the full range of design storm flows from the pond catchment area, without developing any internal pressure. Such surface flow condition within the culvert is necessary to avoid culvert failure that may lead to embankment failure. Rubber ring jointed pipes without lifting holes are recommended for pipe culverts. All culverts should be provided with suitable bedding and cutoff walls or seepage collars to prevent possible failure due to piping. Detail procedure for culvert design is given in Chapter 18.

7.2.3.2 Trash Racks

The susceptibility of inlets, such as small orifices, to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most case thrash racks is required to control the clogging. Track racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention pond outlets. For very small outlets, an even larger opening ratio is usually necessary to control the onrush of debris at the onset of a storm, and a high degree of maintenance is required. An example of a trash rack is shown in Figure 7.4.

The following points should be followed for the trash racks:

- The trash rack should have an area at least ten (10) times larger than the control outlet opening.
- The inclined bar racks are recommended.
- A maximum angle of 60^o to the horizontal can be allowed for the trash racks.
- The trash racks should be located at a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet.
- The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected.
- Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

- The racks should have hinged connections.
- The invert of the outlet structure for a dry pond should be depressed below the ground level to minimise clogging due to sedimentation.
- Depressing the outlet invert to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

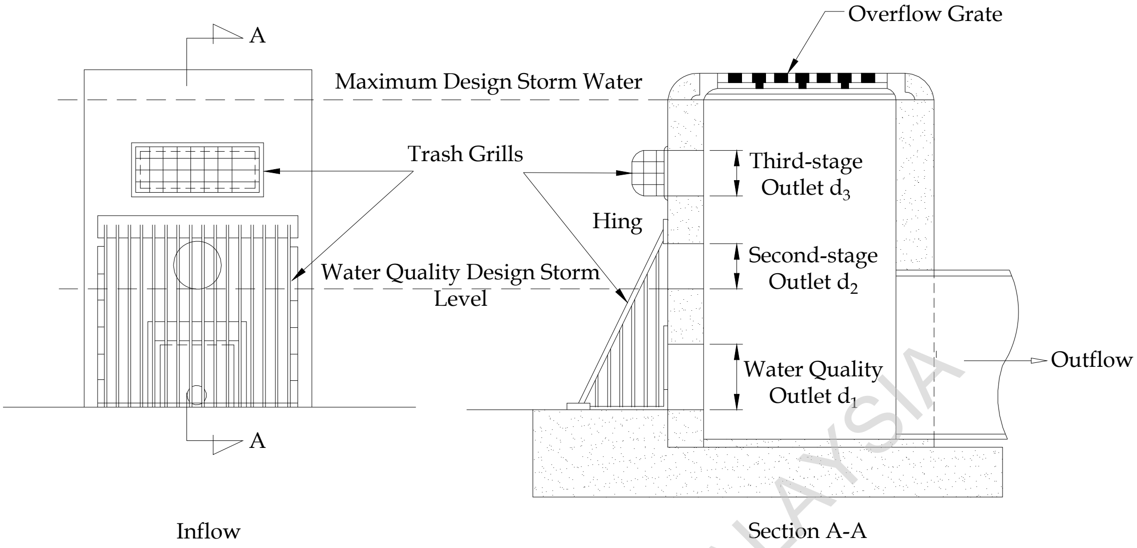


Figure 7.4: Typical Multistage Primary Outlet Trash Racks (DID, 2000)

7.2.4 Secondary Outlet (Emergency Spillway)

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm ARI for the storage facility. In many cases, stormwater detention structures do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. The catchment areas of many sites are small that only very short, sharp thunderstorms are apt to threaten overtopping or embankment failure, and such storms are localised. Also, capacities of the structures are usually too small to create a flood wave.

By contrast, sizable on-line facilities with settlement/valuable assets downstream may pose a significant hazard if failure was to occur, in which case emergency spillway considerations are a major design factor. The potential for loss of life or property damage must be categorised early in the design effort and a spillway design ARI up to and including the Probable Maximum Flood (PMF) may be warranted. A hazard rating for the pond should be determined and a secondary outlet (emergency spillway) design ARI selected in accordance with the Federal Government or relevant State Government Dam Safety Guidelines. The secondary outlets for all non-hazard small detention ponds shall be designed to safely pass a minimum design storm of 100 year ARI through the pond. For hazard ponds they must be designed as per the approved guidelines (ANCOLD, 1986; USBR, 1987; USBR, 1992; FEMA, 1987 and Golzé, 1977).

7.2.5 Storage Zone

7.2.5.1 Dry Time

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

7.2.5.2 Low Flow Channel

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

7.2.6 Embankment

7.2.6.1 Classification

Dry detention ponds are intermittent water-retaining structures and their embankments do not need to be designed rigorously as dams unless they are high, or special soil problems exist. An embankment that raises the water level a specified amount as defined by the appropriate dam safety group (generally 1.5 m to 3 m or more above the usual mean low water height, when measured along the downstream toe of the embankment to the emergency spillway crest), is classified as a dam. Such embankments must be designed, constructed, and maintained in accordance with the Federal Government or relevant State Government dam safety standards.

7.2.6.2 Design Criteria

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

a) Pond Water Depth

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

b) Embankment Top Widths

Minimum recommended embankment top widths are provided in Table 7.

Table 7.1: Minimum Recommended Top Width for Earthen Embankments (USDA, 1982)

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

c) Side Slopes

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

d) Bottom Grades

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

e) *Freeboard*

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

7.3 EROSION PROTECTION

7.3.1 Primary Outlet and Downstream

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

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

Below a pipe outlet, a suitable device is the 'impact energy dissipator'. An open stilling basin is generally more suitable downstream of weirs, gates and sluices, and other large control structures. The stilling basin must be of sufficient depth below the downstream tailwater level so as to drown the flow, thus enabling the hydraulic jump to be retained within the stilling basin at all flows.

The channel bed and banks immediately downstream of stilling basins should be protected by stone pitching or riprap. Where the outfall from the basin is a culvert, this should be provided for a distance of at least four times the diameter or height of the culvert. Information for stone pitching and riprap is provided in Chapter 20.

7.3.2 Secondary Outlet and Downstream

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

In places where immediate cover and protection of the completed earthworks against erosion is required, turf can be laid and held down by coarse-meshed wire netting and wire pins until it is firmly rooted. With all turfing methods, adequate subsoil cultivation is essential to encourage good root penetration. A large number of geotextiles are available which can be used for reinforcing turf enabling it to withstand velocities of up to 7 m/s. The use of concrete reinforcement in conjunction with grass should be considered where turf alone will not provide sufficient protection to a bank or spillway subject to high velocities for long periods. Details on erosion control practices can be referred to Chapter 12.

For reducing erosion an open stilling pond, as discussed in Chapter 20, may be required at the bottom of the spillway prior to discharge into the downstream waterway. It may be possible, and more cost effective, to provide a single stilling basin for both the emergency spillway and the primary outlet to the basin.

7.4 SAFETY AND AESTHETICS

7.4.1 Public Safety

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

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

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

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

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

7.4.2 Landscaping

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

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

7.5 SIZING FLOOD DETENTION SITE

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

7.5.1 Pond Volume Estimate

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

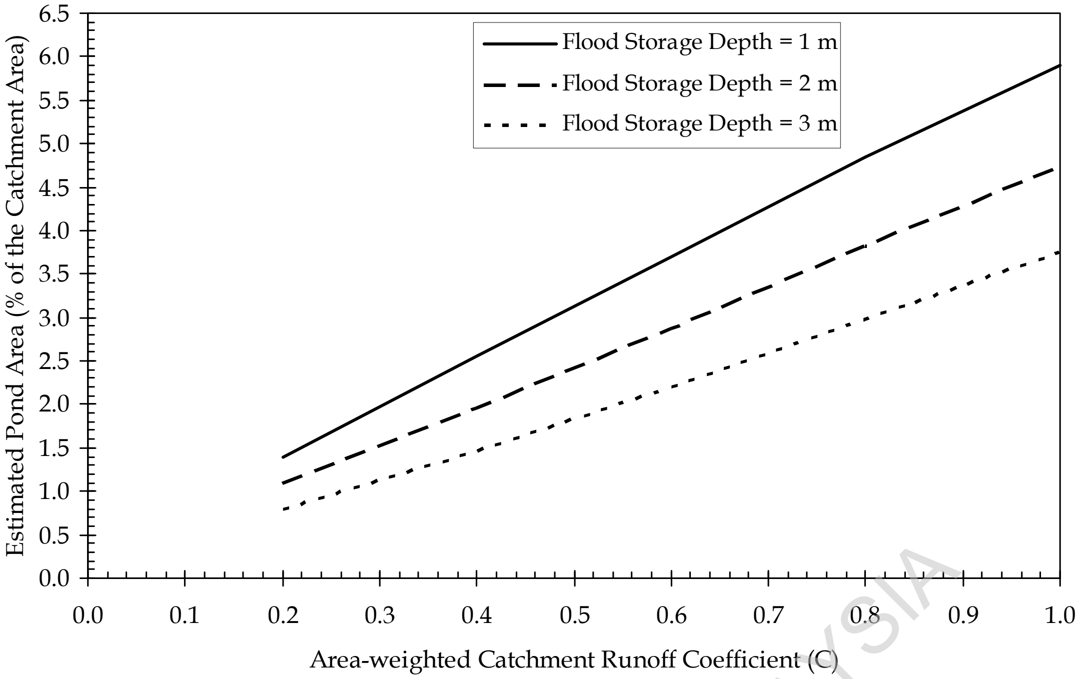
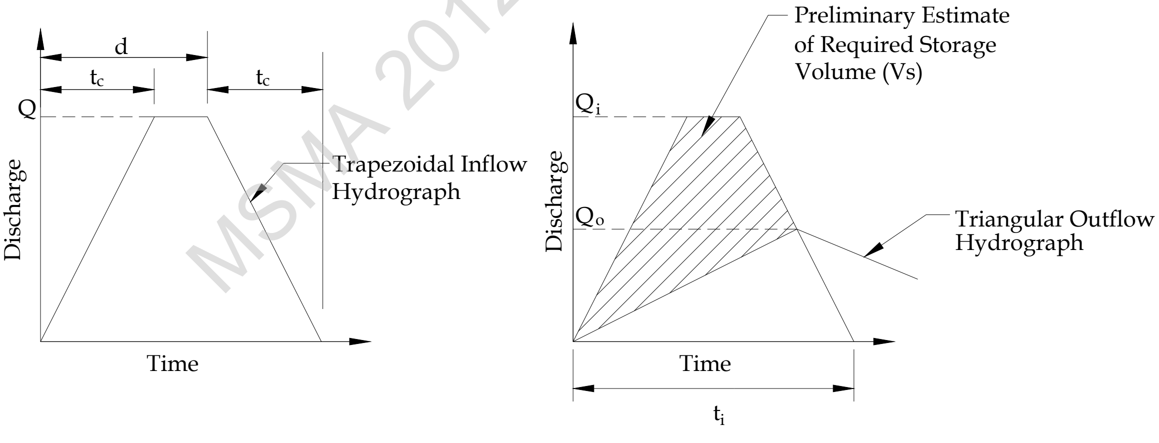


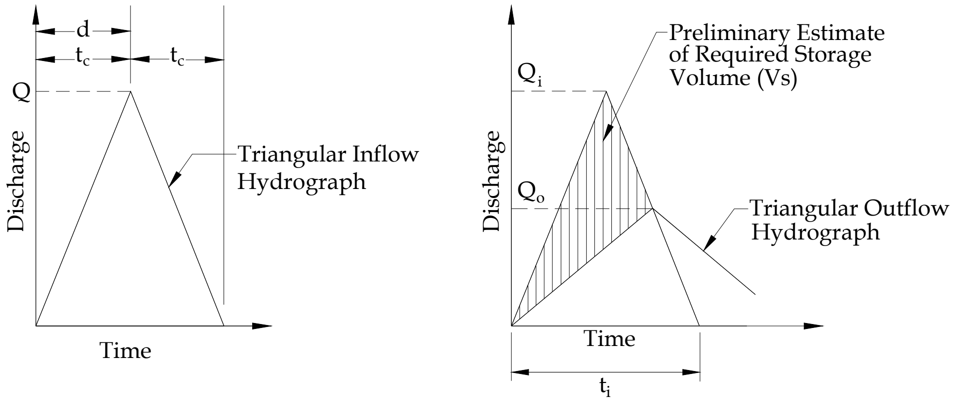
Figure 7.5: Estimate of Pond Area for Planning Purpose

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



a) Inflow Hydrograph b) Storage Volume Estimate

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



a) Inflow Hydrograph b) Storage Volume Estimate

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

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

$V_s = dQ_i - 0.5t_iQ_o$ for Type 1 (7.1a)

$V_s = 0.5t_i\,(Q_i - Q_o)$ for Type 2 (7.1b)

Where,

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

7.5.2 Stage – Storage Curves Development

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

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

where,

$V_{1,2}$ = Storage volume between elevations 1 and 2 (m^3);
A_1 = Surface area at elevation 1 (m^2);
A_2 = Surface area at elevation 2 (m^2); and
h = Change in elevation between points 1 and 2 (m).

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

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

where,

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

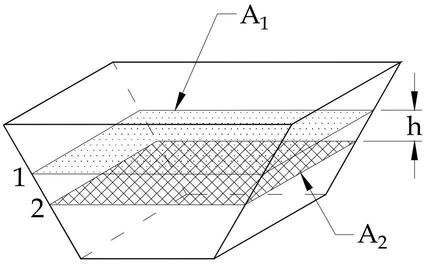


Figure 7.7: Frustum of a Pyramid

7.5.3 Stage – Discharge Curves

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

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

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

7.5.4 Sizing Steps

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

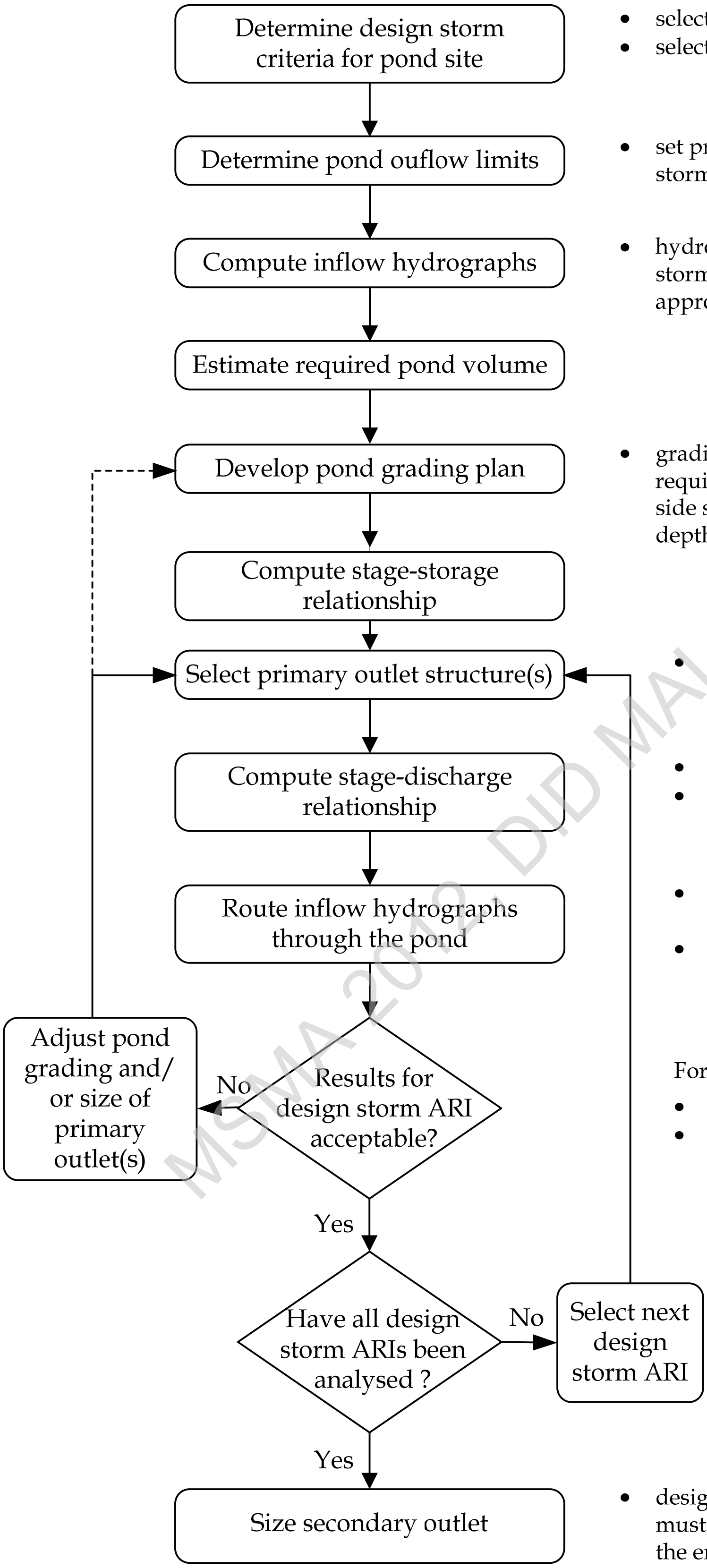
Step 1: Determine design storm criteria for the pond

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

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

Step 2: Determine the pond outflow limits

For each design storm ARI, the pond outflow limits are set as the maximum pre-development flow less any non-routed post-development bypass flow. Peak flows for the pre-development design storms and non-routed post-development bypass to be determined by the Time-Area hydrograph estimation technique.



- select primary outlet design storm ARIs
- select level of non-routed bypass flow

- set pre-development peak flow for each design storm ARI as pond outflow limit

- hydrographs will be required for each design storm ARI for durations ranging from approximately t_c to $3t_c$ for the basin catchment

- grading of pond must be in accordance with the requirements for pond floor slope, embankment side slopes, and maximum permissible ponding depths

- a multi-staged outlet configuration will be required

- determine outlet stages
- calculate and sum ratings for all outlets under consideration

- use a computer model or the routing procedure outlined in Section 2.5
- the critical duration hydrograph is that which results in the maximum depth and volume of ponded water in the pond

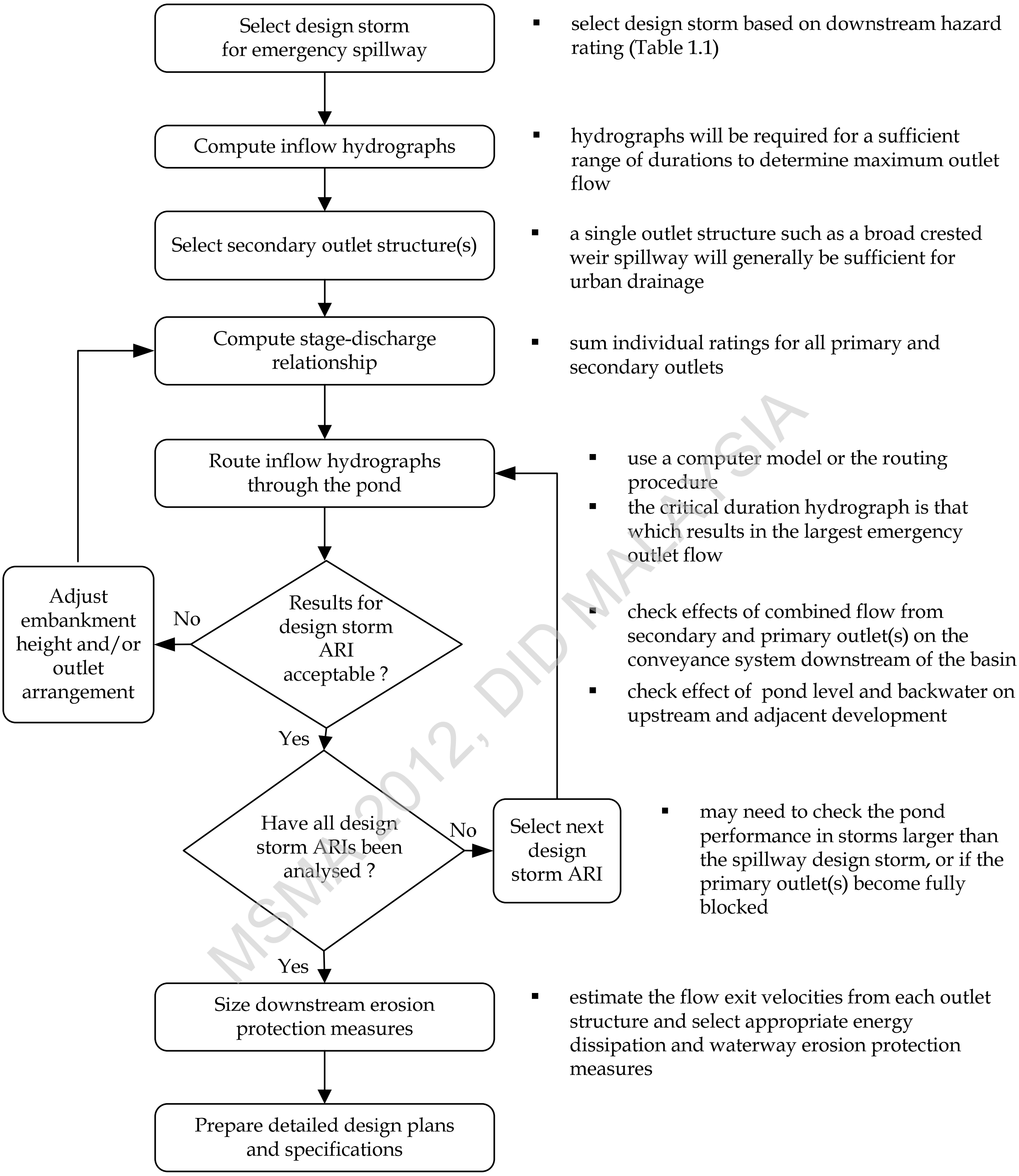
For the design storm under consideration:

- is the maximum pond outflow acceptable?
- is the maximum water depth acceptable?

- minor and major system design storm ARIs must both be analysed and result in satisfactory pond performance

- design storm for the emergency outlet must be routed through the pond to size the emergency spillway

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



- select design storm based on downstream hazard rating (Table 1.1)
- hydrographs will be required for a sufficient range of durations to determine maximum outlet flow
- a single outlet structure such as a broad crested weir spillway will generally be sufficient for urban drainage
- sum individual ratings for all primary and secondary outlets
- use a computer model or the routing procedure
- the critical duration hydrograph is that which results in the largest emergency outlet flow
- check effects of combined flow from secondary and primary outlet(s) on the conveyance system downstream of the basin
- check effect of pond level and backwater on upstream and adjacent development
- may need to check the pond performance in storms larger than the spillway design storm, or if the primary outlet(s) become fully blocked
- estimate the flow exit velocities from each outlet structure and select appropriate energy dissipation and waterway erosion protection measures

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

Step 3: Compute the pond inflow hydrograp*hs*

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

Step 4: Make a preliminary estimate of the required pond volume

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

A preliminary estimate may be obtained based on the post-development pond inflow hydrographs for the major system design ARI and the required outflow rate as shown in Figure 7.8. The outflow hydrograph can be approximated by drawing a straight line from the beginning of substantial runoff on the inflow hydrograph to the point on the receding limb corresponding to the maximum allowable peak outflow rate. The amount of storage required is equal to the representative volume (shaded area) between the inflow and outflow hydrographs. To determine the necessary storage, the shaded area can be planimetered or computed mathematically.

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

Step 5: Develop a pond grading plan

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

Step 6: Compute the stage-storage relationship

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

Step 7: Size the minor design storm primary outlet

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

(i) Select trial outlet arrangement

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

(ii) Compute the stage-discharge relationship

Compute the stage-discharge relationship by summing the individual discharge ratings for each outlet adopted. The maximum stage selected must be greater than the expected maximum water level in the pond so that it will not be exceeded in the routing calculations in the following step. Step 3: Compute the pond inflow hydrograp*hs*

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

for a particular pond grading and outlet configuration. The pond inflow hydrographs are obtained by subtracting the non-routed bypass flow from the total inflow hydrographs.

Step 4: Make a preliminary estimate of the required pond volume

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

A preliminary estimate may be obtained based on the post-development pond inflow hydrographs for the major system design ARI and the required outflow rate as shown in Figure 7.8. The outflow hydrograph can be approximated by drawing a straight line from the beginning of substantial runoff on the inflow hydrograph to the point on the receding limb corresponding to the maximum allowable peak outflow rate. The amount of storage required is equal to the representative volume (shaded area) between the inflow and outflow hydrographs. To determine the necessary storage, the shaded area can be planimetered or computed mathematically.

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

Step 5: Develop a pond grading plan

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

Step 6: Compute the stage-storage relationship

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

Step 7: Size the minor design storm primary outlet

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

(i) Select trial outlet arrangement

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

(ii) Compute the stage-discharge relationship

Compute the stage-discharge relationship by summing the individual discharge ratings for each outlet adopted. The maximum stage selected must be greater than the expected maximum water level in the pond so that it will not be exceeded in the routing calculations in the following step.

(iii) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond outflow and water level. The routing time step adopted should be a uniform integer value and should be small enough so that the change in inflow and outflow between time steps is relatively linear. A value of $2t_i$ / 300 may be used as a rough guide. However, for manual calculations, a minimum value of five (5) minute is recommended.

(iv) Route the inflow hydrographs through the pond

Route the inflow hydrographs through the pond, using a suitable computer model or the procedures presented in Chapter 2, to determine the maximum pond outflow and water level. The routing time step adopted should be a uniform integer value and should be small enough so that the change in inflow and outflow between time steps is relatively linear. A value of $2t_i$ / 300 may be used as a rough guide. However, for manual calculations, a minimum value of five (5) minute is recommended.

(iv) Check if results are acceptable

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

Step 8 Size the major design storm primary outlet

(vi) Select trial outlet arrangement

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

(vii) Compute the stage-discharge relationship

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

(viii) Route the inflow hydrographs through the pond

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

(ix) Check if results are acceptable

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

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

Step 9 Size the secondary outlet arrangement

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

(i) Select trial outlet arrangement

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

(ii) Compute the stage-discharge relationship

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

(iii) Route the inflow hydrographs through the pond

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

(iv) Check if results are acceptable

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

Step 10 Check behaviour under extreme conditions

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

Step 11 Prepare design drawings and specifications

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

REFERENCES

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

APPENDIX 7.A EXAMPLE – DETENTION POND DESIGN

Problem:

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

Pre-development catchment and stream properties

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

Post-development catchment and channel properties

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

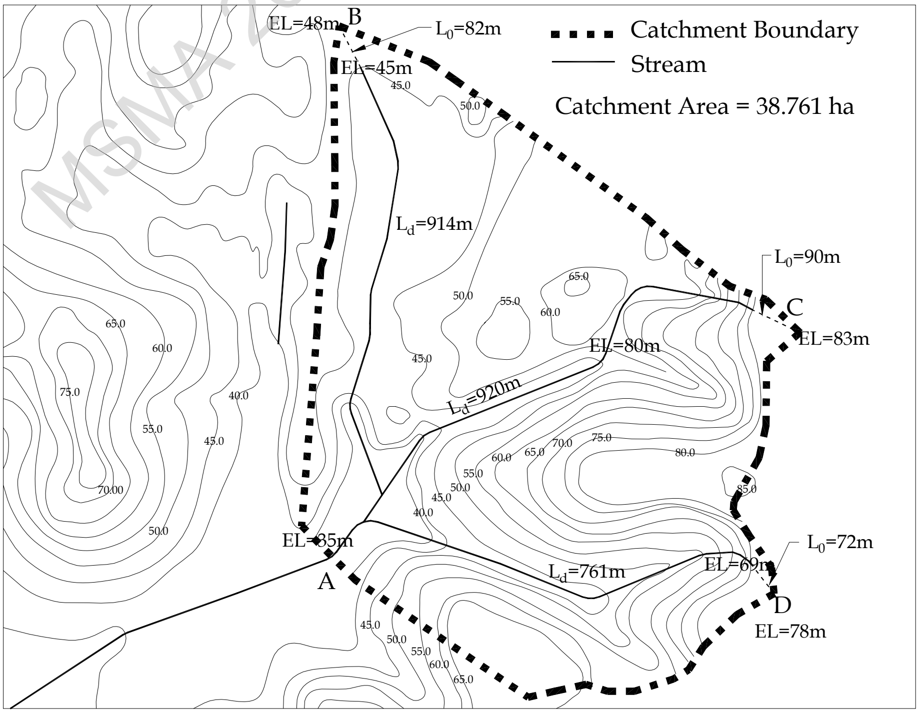


Figure 7.A1: Catchment Area

Solution:

Reference	Calculation	Output
	Step 1 : Setting the design storm criteria for the pond	
	As specified in Section 1.2.1, the runoff quantity control criterion for new development is to reduce post-development peak flows to less than or equal to the pre-development values. The majority of the proposed development upstream of the pond will be medium density housing lots. The minor and major design storms for primary outlet are 5 and 50 year ARI, while the secondary outlet is 100 year ARI (in accordance with Table 1.1, Section 7.2.3 and 7.2.4).	
	Step 2 : Compute the pond inflow hydrograph	
	Compute the Pre-development Time of Concentration, t_c	
	Reach AB:	
Table 2.1	$t_o = \frac{107.n^*.L_o^{1/3}}{S_o^{1/5}}$	
Table 2.2	L_o = 82.0 m, S_o = 3.66 %, n^* = 0.045, t_o =	16.1 min
Table 2.1	$t_d = \frac{n.L_d}{60R^{2/3}S_d^{1/2}}$	
Table 2.3	L_d = 914 m, S_d = 0.011 %, n = 0.050, $R = 0.11$ t_d =	32.5 min
	$t_c = t_o + t_d$ =	48.6 min
	Similarly for Reach AC:	
	L_o= 90 m, S_o = 3.33 %, n^* = 0.045, t_o =	17 min
	L_d = 920 m, S_d = 0.049 %, n = 0.050, $R = 0. 11$ t_d =	15.5 min
	$t_c = t_o + t_d$ =	32.4 min
	Similarly for Reach AD:	
	L_o= 72.0 m, S_o = 4.17 %, n^* = 0.045, t_o =	15.1 min
	L_d = 761 m, S_d = 0.053 %, n = 0.050, $R = 0.11$ t_d =	12.3 min
	$t_c = t_o + t_d$ =	27.4 min
	Thus, the pre-development t_c is 48.6 min (the longest).	
	Compute the Post-development Time of Concentration, t_c	
	Reach AB:	
	L_o= 82 m, S_o = 3.66 %, n^* = 0.035, t_o =	12.6 min

Reference | Calculation | Output

L_d = 914 m, S_d = 0.009 %, n = 0.035, $R = 0.15$ t_d = 20.2 min

$t_c = t_o + t_d$ = 32.7 min

Similarly for Reach AC:

L_o= 90 m, S_o = 3.33 %, n^* = 0.035, t_o = 13.2 min

L_d = 920 m, S_d = 0.024 %, n = 0.035, $R = 0.15$ t_d = 12.2 min

$t_c = t_o + t_d$ = 25.3 min

Similarly for Reach AD:

L_o= 72 m, S_o = 4.17 %, n^* = 0.035, t_o = 11.7 min

L_d = 761 m, S_d = 0.026 %, n = 0.035 $R = 0.15$ t_d = 9.7 min

$t_c = t_o + t_d$ = 21.4 min

Thus, the post-development t_c is 32.7 min, which is 15.9 minute shorter than the the pre-development t_c.

Compute the rainfall intensity, *i*

The nearest rainfall station is Station ID 3216004, SK Jenis Keb. Kepong. Referring to Table 2.B1, the fitting constants for the IDF Empirical Equation of this station are:

Table 2.B1.4

λ	κ	θ	η
73.602	0.164	0.330	0.874

The rainfall intensity for various durations and ARIs are given in Table 7.A2 and Table 7.A3 using Equation 2.2.

Equation 2.2

Step 3: Determine the pond outflow limits

Table 7.A1: Runoff Coefficient

Condition	C_{post}	C_{pre}
ARI < 10 yr	0.65	0.30
ARI > 10 yr	0.70	0.40

Compute the pre and post-development discharge

Table 2.5

Table 7.A2: Calculated Pre-development Discharge

ARI (year)	Storm duration, d	*d* (min)	*i* (mm/hr)	C_{pre}	*A* (ha)	Q_{pre} (m^3/s)
5	t_c	48.6	85.5	0.30	38.761	2.8
50	t_c	48.6	124.7	0.40	38.761	5.4
100	t_c	48.6	139.7	0.40	38.761	6.0

Reference | Calculation | Output

Table 7.A3: Calculated Post-development Discharge

Equaion 2.2
Equaion 2.3

ARI (year)	Storm duration, d (in terms of t_c)	d (min)	i (mm/hr)	C_{post}	A (ha)	Q_{post} (m^3/s)
5	t_c	32.7	107.7	0.65	38.761	7.5
	1.5 t_c	49.1	85.0	0.65	38.761	5.9
	2.0 t_c	65.4	70.5	0.65	38.761	4.9
	2.5 t_c	81.8	60.5	0.65	38.761	4.2
50	t_c	32.7	157.1	0.70	38.761	11.8
	1.5 t_c	49.1	124.0	0.70	38.761	9.3
	2.0 t_c	65.4	102.9	0.70	38.761	7.8
	2.5 t_c	81.8	88.3	0.70	38.761	6.7
100	t_c	32.7	176.0	0.70	38.761	13.3
	1.5 t_c	49.1	138.9	0.70	38.761	10.5
	2.0 t_c	65.4	115.3	0.70	38.761	8.7
	2.5 t_c	81.8	98.9	0.70	38.761	7.5

Equaion 2.2
Equaion 2.3

The pre-development flows for this duration will become the post-development flow limits in the channel immediately downstream of the pond. The pond outflow limits for 5, 50 and 100 year ARI are **2.8 m³/s, 5.4 m³/s** and **6.0 m³/s** respectively.

Step 4: Make a preliminary estimate of the required pond volume, Vs storm duration (based on 50 yr ARI design storm) using Rational Hydrograph Method (RHM)

For Type 1 hydrograph (storm duration *d* is greater than t_c),

$Vs = dQ_i – 0.5t_iQ_o$

Equation 7.1a

For Type 2 hydrograph (storm duration *d* is equal to t_c),

$Vs = 0.5t_i (Q_i – Q_o)$

Equation 7.1b

For 50 year ARI, the estimated pond volumes for various storm durations are:

For storm duration d=t_c = 32.7 min, t_i = 65.4 min, V_s = 12695 m^3

For storm duration d=1.5 t_c = 49.1 min, t_i = 81.8 min, V_s = 14325 m^3

For storm duration d=2.0 t_c = 65.4 min, t_i = 98.1 min, V_s = 14627 m^3

For storm duration d=2.5 t_c = 81.8 min, t_i = 114.5 min, V_s = 14190 m^3

Reference	Calculation	Output

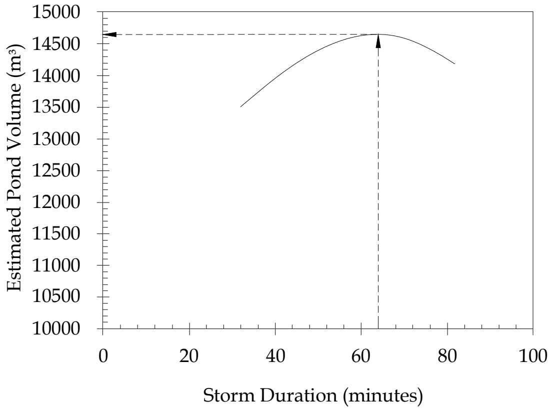


Figure 7.A2: Storm Duration vs Estimated Pond Volume Curve

From Figure 7.A2, the estimated pond volume required is 14630 m^3.

Step 5: Develop a pond grading plan

The location and grading of the pond embankment and storage area is selected by trial and error so that at least the preliminary estimated volume of 14630 m^3 is available in the pond to cater for the critical 50 year ARI design storm event.

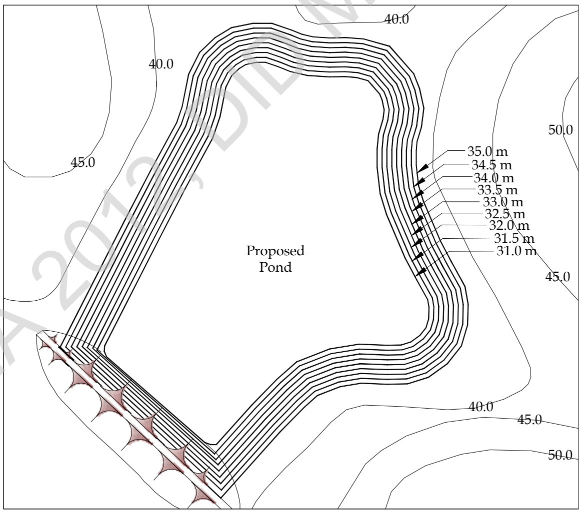


Figure 7.A3: Pond Grading Plan

Step 6: Compute stage-storage relationship

Table 7.A4: Stage-Storage Relationship

Stage (m)	Elevation (m)	Area (m^2)	Storage Volume (m^3)
0.00	31.00	6250	0
0.50	31.50	6435	3171
1.00	32.00	7275	6599
1.50	32.50	7360	10258
2.00	33.00	8245	14159
2.50	33.50	8365	18311
3.00	34.00	9278	22722

Reference	Calculation	Output

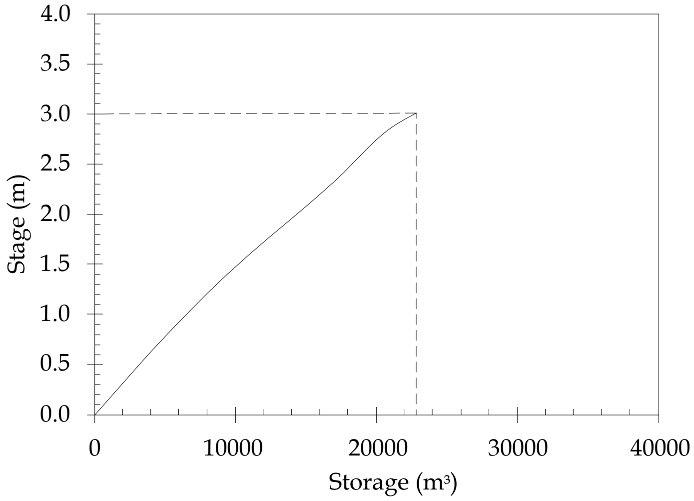


Figure 7.A4: Stage – Storage Curve

Step 7: Size the minor design storm primary outlet

The optimum size of the minor design storm primary outlet needs to be determined by trial and error to produce a maximum pond outflow that is as close as practicable to the required flow limit of **2.8m³/s**. This involves selecting an initial outlet arrangement and size, determining the stage-discharge relationship, and routing the pond inflow hydrographs through the pond to determine the maximum outflow and water level produced. The outlet arrangement and/or size is then refined, if necessary, and the process repeated until an acceptable maximum outflow and water depth is reached.

(1) Select trial outlet arrangement

To provide flow reduction for the 5 year ARI post-development design storm, a rectangular orifice measuring 2.0m (width) x 1.0m (depth) was initially selected. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond.

(2) Compute the stage-discharge relationship and storage indicator number

Calculate the stage-discharge relationship and storage indicator number from 5yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve.

Table 7.A5: Stage-Discharge Relation and Storage Indicator Numbers (5yr ARI 1st Trial)

H (m)	O (m³/s)	S (m³)	(S/Δt)+O/2 (m³/s)
0.00	0.00	0	0.00
0.50	1.20	3171	53.44
1.00	3.76	6599	111.85
1.50	5.32	10258	173.62
2.00	6.51	14159	239.23
2.50	7.52	18311	308.93
3.00	8.40	22722	349.57

C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), A= 2.0m², (Dimension=2.0m x 1.0m)

Reference	Calculation	Output

(3) Route the inflow hydrographs through the pond

Using a routing time step of 1.0 minutes, the 5yr ARI orifice produced a maximum discharge of **4.07 m³/s** which is NOT acceptable (Table 7.A6) as it is more than the 5 year ARI pond outflow limit of **2.8 m³/s**. The maximum water elevation in the basin is **32.1 m**, LSD. It is ok since it is less than the maximum allowable elevation (34.7m). However, the outlet needs to be resized since it exceeds the permissible discharge.

Table 7.A6: Routing Table – 5 yr ARI (1st Trial)

Time (min)	I (m^3/s)	$(I_1 + I_2)/2$ (m^3/s)	$S_1/\Delta t + O_1/2$ (m^3/s)	O_1 (m^3/s)	$S_2/\Delta t + O_2/2$ (m^3/s)	O_2 (m^3/s
0	0.00	0.00	0		0	0
1	0.15	0.07	0.00	0.00	0.07	0.00
2	0.30	0.22	0.07	0.00	0.30	0.00
3	0.45	0.37	0.30	0.00	0.67	0.00
4	0.60	0.52	0.67	0.00	1.20	0.00
5	0.75	0.67	1.20	0.00	1.87	0.00
6	0.90	0.82	1.87	0.00	2.69	0.06
7	1.05	0.97	2.69	0.06	3.61	0.06
8	1.20	1.12	3.61	0.06	4.67	0.06
9	1.35	1.27	4.67	0.06	5.88	0.12
10	1.50	1.42	5.88	0.12	7.18	0.12
Continued						
…	…	…	…	…	…	…
60	4.94	4.94	116.96	3.84	118.06	3.91
61	4.94	4.94	118.06	3.91	119.09	3.91
62	4.94	4.94	119.09	3.91	120.12	3.91
63	4.94	4.94	120.12	3.91	121.14	3.99
64	4.94	4.94	121.14	3.99	122.09	3.99
65	4.94	4.94	122.09	3.99	123.04	3.99
66	4.79	4.87	123.04	3.99	123.91	3.99
67	4.64	4.72	123.91	3.99	124.63	4.07
68	4.49	4.57	124.63	4.07	125.13	4.07
69	4.34	4.42	125.13	4.07	125.48	4.07
70	4.19	4.27	125.48	4.07	125.67	4.07
71	4.04	4.12	125.67	4.07	125.72	4.07
72	3.89	3.97	125.72	4.07	125.62	4.07
73	3.74	3.82	125.62	4.07	125.36	4.07
74	3.59	3.67	125.36	4.07	124.96	4.07
75	3.44	3.52	124.96	4.07	124.41	4.07
76	3.29	3.37	124.41	4.07	123.71	3.99
77	3.14	3.22	123.71	3.99	122.93	3.99
78	2.99	3.07	122.93	3.99	122.01	3.99
79	2.84	2.92	122.01	3.99	120.94	3.91
80	2.69	2.77	120.94	3.91	119.79	3.91
Continued						
…	…	…	…	…	…	…

Reference	Calculation	Output

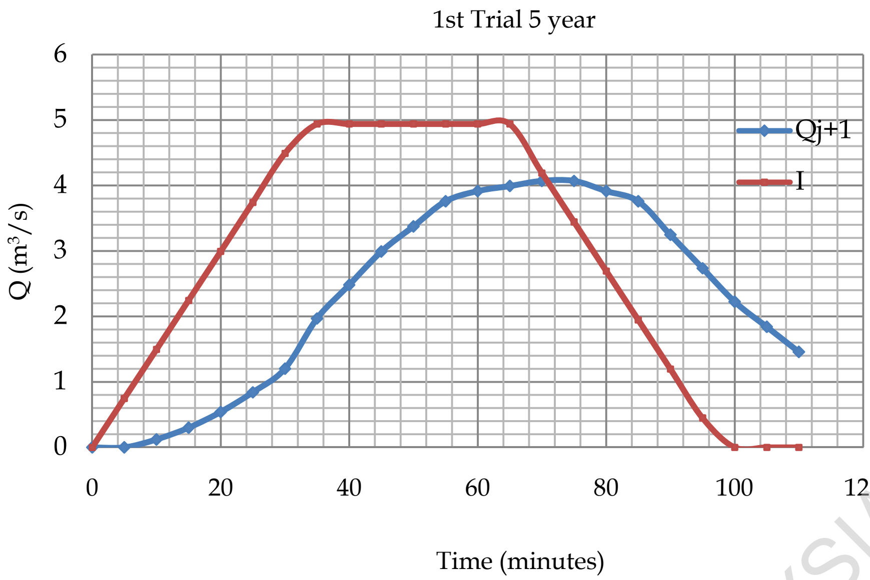


Figure 7.A5: Inflow and Outflow Hydrograph of 5yr ARI Storm Event (1st Trial)

(4) Re-Select trial outlet arrangement

To provide flow reduction for the 5 year ARI post-development design storm to a pre-development level, a smaller size outlet is selected. A 0.5 x 0.5 m rectangular orifice was selected. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond.

(5) Re-compute the stage-discharge relationship and storage indicator number

Calculate the stage-discharge relationship and storage indicator number from 5yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve.

Table 7.A7: Stage-Discharge Relation and Storage Indicator Numbers (5yr ARI 2nd Trial)

H (m)	O (m³/s)	S (m³)	(S/Δ*t*)+O/2 (m³/s)
0.00	0.00	0	0.00
0.50	0.87	3171	53.28
1.00	1.51	6599	110.73
1.50	1.94	10258	171.94
2.00	2.30	14159	237.12
2.50	2.61	18311	306.48
3.00	2.88	22722	380.14

C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), A= 0.25m², (Dimension=0.5m x 0.5m)

(6) Re-route the inflow hydrographs through the pond

Using a routing time step of 1.0 minutes, the 5yr ARI orifice produced a maximum

Reference	Calculation	Output

discharge of **2.34 m^3/s** which is acceptable (Table 7.A8) as it is less than the 5 year ARI pond outflow limit of **2.8 m^3/s**. The maximum water elevation in the pond is **32.63 m**, LSD. It is ok since it is less than the maximum allowable elevation (34.7m).

Table 7.A8: Re-routing Table – 5 yr ARI (2nd Trial)

Time (min)	I (m^3/	$(I_1 + I_2)/2$ (m^3/s)	$S_1/\Delta t + O_1/2$ (m^3/s)	O_1 (m^3/s	$S_2/\Delta t + O_2/2$ (m^3/s)	O_2 (m^3/s
0	0.00	0.00	0.00		0.00	0.00
1	0.15	0.07	0.00	0.00	0.07	0.00
2	0.30	0.22	0.07	0.00	0.30	0.00
3	0.45	0.37	0.30	0.00	0.67	0.00
4	0.60	0.52	0.67	0.00	1.20	0.00
5	0.75	0.67	1.20	0.00	1.87	0.00
6	0.90	0.82	1.87	0.00	2.69	0.04
7	1.05	0.97	2.69	0.04	3.62	0.04
8	1.20	1.12	3.62	0.04	4.70	0.04
9	1.35	1.27	4.70	0.04	5.93	0.09
10	1.50	1.42	5.93	0.09	7.27	0.09
Continued						
…	…	…	…	…	…	…
80	2.69	5.54	322.51	2.70	325.35	2.73
81	2.54	5.24	325.35	2.73	327.85	2.75
82	2.40	4.94	327.85	2.75	330.05	2.76
83	2.25	4.64	330.05	2.76	331.93	2.77
84	2.10	4.34	331.93	2.77	333.50	2.79
85	1.95	4.04	333.50	2.79	334.75	2.80
86	1.80	3.74	334.75	2.80	335.69	2.80
87	1.65	3.44	335.69	2.80	336.34	2.80
88	1.50	3.14	336.34	2.80	336.68	2.80
89	1.35	2.84	336.68	2.80	336.72	2.80
90	1.20	2.54	336.72	2.80	336.47	2.80
91	1.05	2.25	336.47	2.80	335.91	2.80
92	0.90	1.95	335.91	2.80	335.06	2.80
93	0.75	1.65	335.06	2.80	333.90	2.79
94	0.60	1.35	333.90	2.79	332.47	2.77
95	0.45	1.05	332.47	2.77	330.74	2.77
96	0.30	0.75	330.74	2.77	328.72	2.76
97	0.15	0.45	328.72	2.76	326.41	2.73
98	0.00	0.15	326.41	2.73	323.82	2.72
99	0.00	0.00	323.82	2.72	321.11	2.70
100	0.00	0.00	321.11	2.70	318.40	2.68
Continued						
…	…	…	…	…	…	…

Reference	Calculation	Output

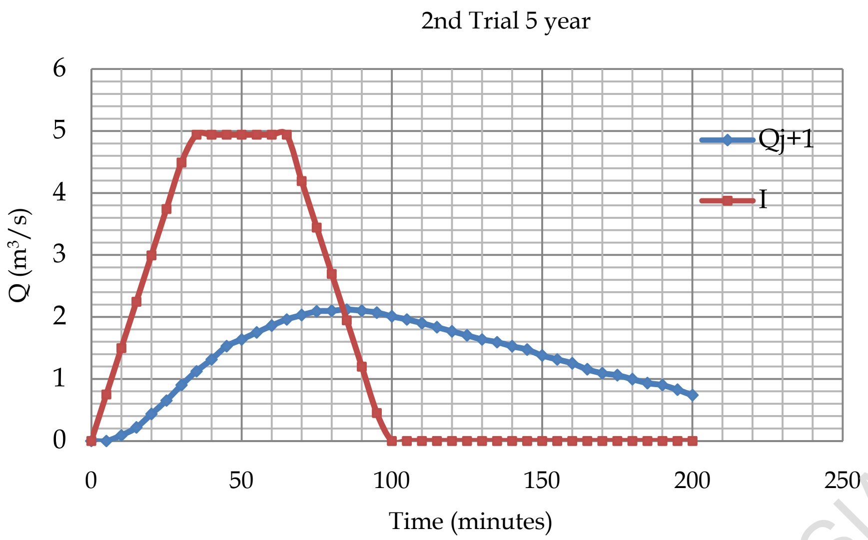


Figure 7.A6: Inflow and Outflow Hydrograph of 5yr ARI Storm Event (2nd Trial)

Step 8: Size the major design storm primary outlet

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

(iii) Select trial outlet arrangement

To provide flow reduction for the 50 year ARI post-development design storm, an additional 2.0 m x 2.0 m orifice was initially selected with an invert level of 33 m, LSD that corresponds to a stage of 2.0 m.

(iv) Compute the stage-discharge relationship

The stage-discharge relationship is the summation of the 5 year ARI and the 50 year ARI orifice capacities (Table 7.A9).

Table 7.A9: Combined Stage-Discharge – 5 yr and 50 yr ARI Orifice

H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge
(m)	(m³/s)	(m³/s)	(m³/s)
0	0.00	0	0
0.5	0.87	0	0.87
1	1.51	0	1.51
1.5	1.94	0	1.94
2	2.30	0	2.30
2.5	2.61	4.70	7.31
3	2.88	6.64	9.53

C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), A= 4 m² (Dimension=2.0m x 2.0m)

Reference	Calculation	Output

Table 7.A10: Combined Storage Indicator Numbers – 5 yr and 50yr ARI

H (m)	Q (m3/s)	S (m3)	(S/dt)+Q/2
0	0	0	0.00
1	1	3171	53.28
1	2	6599	110.73
2	2	10258	171.94
2	2	14159	237.12
3	7	18311	308.83
3	10	22722	383.46

(v) Route the inflow hydrographs through the pond

Using a routing time step of 2.5 minutes, the 5 year and 50 year ARI orifices produced a maximum discharge of **5.80 m³/s**. This is NOT acceptable (Table 7.A11) as it is more than the 50 year ARI pond outflow limit of **5.4 m³/s**. The maximum water elevation in the pond is **33.35m**, LSD that corresponds to a maximum water depth of 2.35m.

Table 7.A11: Routing Table – 50 yr ARI (1st Trial)

Time (min)	I (m³/s)	$(I_1 + I_2)/2$ (m³/s)	$S_1/\Delta t + O_1/2$ (m³/s)	O_1 (m³/s	$S_2/\Delta t + O_2/2$ (m³/s)	O_2 (m³/s
0	0.00	0.00	0.00	0.00	0.00	0.00
1	0.24	0.12	0.00	0.00	0.12	0.00
2	0.47	0.35	0.12	0.00	0.47	0.00
3	0.71	0.59	0.47	0.00	1.06	0.00
4	0.94	0.82	1.06	0.00	1.88	0.00
5	1.18	1.06	1.88	0.00	2.94	0.04
6	1.41	1.29	2.94	0.04	4.19	0.04
Continued						
…	…	…	…	…	…	…
65	7.76	7.76	276.09	4.86	278.99	5.1
66	7.52	7.64	278.99	5.11	281.52	5.4
67	7.29	7.41	281.52	5.37	283.55	5.4
68	7.05	7.17	283.55	5.37	285.36	5.6
69	6.82	6.94	285.36	5.63	286.67	5.6
70	6.58	6.70	286.67	5.63	287.74	5.8
71	6.35	6.47	287.74	5.88	288.33	5.8
72	6.11	6.23	288.33	5.88	288.68	5.8
73	5.88	6.00	288.68	5.88	288.80	5.8
74	5.64	5.76	288.80	5.88	288.68	5.8
75	5.41	5.53	288.68	5.88	288.32	5.8
76	5.17	5.29	288.32	5.88	287.73	5.8
77	4.94	5.06	287.73	5.88	286.90	5.6
78	4.70	4.82	286.90	5.63	286.10	5.6
79	4.47	4.59	286.10	5.63	285.06	5.6
80	4.23	4.35	285.06	5.63	283.78	5.6
Continued						
…	…	…	…	…	…	…

Reference	Calculation	Output

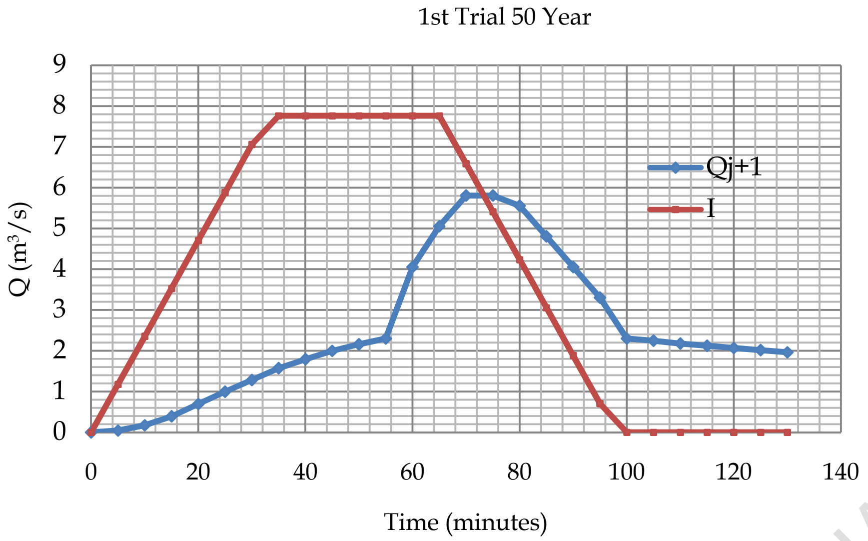


Figure 7.A7: Resulting Outflow Hydrograph –50 yr ARI (1st Trial)

(viv) Re-select trial outlet arrangement

To provide flow reduction for the 50 year ARI post-development design storm, a 1.2m x 1.2m square orifice was re-selected. The previous selection was a 2.0m x 2.0m square, which exceeded the 50yr ARI allowable discharge limit of 5.4 m^3/s. The invert level of the upstream end of the culvert was set at stage 31.00 m LSD in the pond.

(v) Re-compute the stage-discharge relationship

Re-calculate the stage-discharge relationship and storage indicator number from 50yr ARI orifice outlet then, plot stage-discharge curve and storage indicator curve.

Table 7.A12: Stage-Discharge – 5 yr and 50 yr ARI Orifice

H	5 yr ARI Orifice	50 yr ARI weir	Total Discharge
(m)	(m^3/s)	(m^3/s)	(m^3/s)
0	0.00	0	0
0.5	0.87	0	0.87
1	1.51	0	1.51
1.5	1.94	0	1.94
2	2.30	0.00	2.30
2.5	2.61	2.26	4.86
3	2.88	3.19	6.07

C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), A= 1.44 m^2 (Dimension =1.2m x 1.2m)

Reference	Calculation	Output

Table 7.A13: Combined Storage Indicator Numbers – 5 yr and 50yr ARI

H (m)	Q (m³/s)	S (m3)	(S/dt)+Q/2
0.00	0	0	0.00
0.50	0.87	3171	53.28
1.00	1.51	6599	110.73
1.50	1.94	10258	171.94
2.00	2.30	14159	237.12
2.50	4.86	18311	307.61
3.00	6.07	22722	381.74

(vi) Re-route the inflow hydrographs through the pond

Using a routing time step of 1.0 minutes, the 5yr and 50yr ARI orifices produced a maximum discharge of **4.9 m³/s**, which is acceptable as it is less than the 5 year ARI pond outflow limit of **5.4 m³/s**. The maximum water elevation in the pond is **33.48 m**, LSD. It is ok since it is less than the maximum allowable elevation (33.7m).

Table 7.A14: Routing Table – 50yr ARI (2nd Trial)

Time (min)	I (m³/s)	$(I_1 + I_2)/2$ (m³/s)	$S_1/\Delta t + O_1/2$ (m³/s)	O_1 (m³/s)	$S_2/\Delta t + O_2/2$ (m³/s)	O_2 (m³/s)
0	0.00	0.00	0.00	0.00	0.00	0.00
1	0.24	0.12	0.00	0.00	0.12	0.00
2	0.47	0.35	0.12	0.00	0.47	0.00
3	0.71	0.59	0.47	0.00	1.06	0.00
4	0.94	0.82	1.06	0.00	1.88	0.00
5	1.18	1.06	1.88	0.00	2.94	0.04
6	1.41	1.29	2.94	0.04	4.19	0.04
Continued						
…	…	…	…	…	…	…
65	7.76	7.76	280.13	4.22	283.67	4.38
66	7.52	7.64	283.67	4.38	286.94	4.53
67	7.29	7.41	286.94	4.53	289.81	4.53
68	7.05	7.17	289.81	4.53	292.45	4.69
69	6.82	6.94	292.45	4.69	294.69	4.61
70	6.58	6.70	294.69	4.85	296.54	4.61
71	6.35	6.47	296.54	4.85	298.15	4.73
72	6.11	6.23	298.15	5.01	299.37	4.73
73	5.88	6.00	299.37	5.01	300.35	4.73
74	5.64	5.76	300.35	5.01	301.10	4.86
75	5.41	5.53	301.10	5.17	301.45	4.86
76	5.17	5.29	301.45	5.17	301.57	4.86
77	4.94	5.06	301.57	5.17	301.45	4.86
78	4.70	4.82	301.45	5.17	301.10	4.86
79	4.47	4.59	301.10	5.17	300.51	4.73
80	4.23	4.35	300.51	5.01	299.85	4.73
Continued						
…	…	…	…	…	…	…

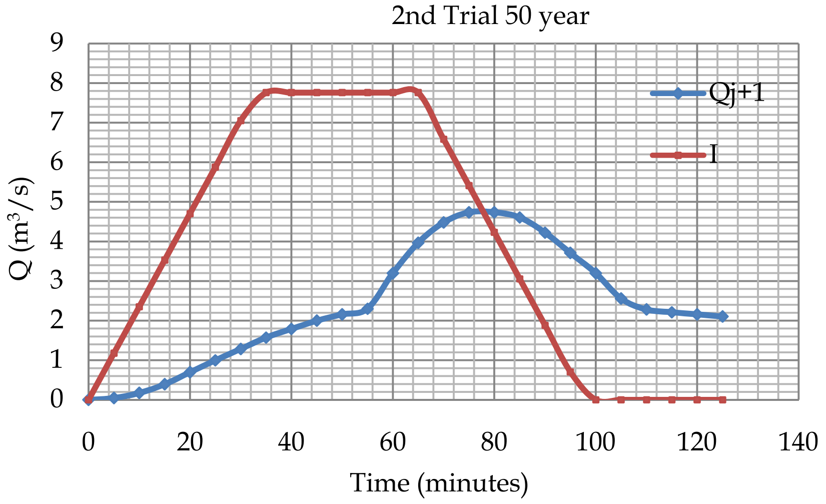


Figure 7.A8: Resulting Outflow Hydrograph –50 yr ARI (2nd Trial)

Step 9: Size the secondary outlet arrangement

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

(vii) Select trial outlet arrangement

A 10 m wide broad-crested weir spillway with 0(H):1(V) side slopes was initially selected as the pond secondary outlet. The spillway was set at the side of the embankment at an elevation of 33.5 m, LSD (50 year ARI maximum water level).

(viii) Compute the stage-discharge relationship

The stage-discharge relationship is the summation of the 5 yr and 50 yr ARI orifice and 100yr ARI spillway capacities.

Table 7.A15: Combined Stage-Discharge - 5 and 50 yr Orifice and 100 yr ARI Spillway

H	5 yr ARI Orifice	50 yr ARI Orifice	spillway	Total Discharge
(m)	(m³/s)	(m³/s)	(m³/s)	(m³/s)
0	0.00	0.00	0	0
0.5	0.87	0.00	0	0.87
1	1.51	0.00	0	1.51
1.5	1.94	0.00	0	1.94
2	2.30	0.00	0	2.30
2.5	2.61	2.26	0	4.86
3	2.88	3.19	6	12.08

C_o=1.7 (Weir Flow) and 0.6 (Orifice Flow), 5yr Orifice=0.5m x 0.5m, 50yr Orifice =1.2m x 1.2m and 100yr Spillway=10m x 0.5m

Reference	Calculation	Output

Table 7.A16: Combined Storage Indicator Numbers – 5yr, 50yr and 100yr ARI

H (m)	Q (m^3/s)	S (m^3)	(S/dt)+Q/2
0.00	0	0	0.00
0.50	0.87	3170.5	53.28
1.00	1.51	6598.5	110.73
1.50	1.94	10258	171.94
2.00	2.30	14158.5	237.12
2.50	4.86	18310.5	307.61
3.00	12.08	20722	351.41

(ix) Route the inflow hydrographs through the pond

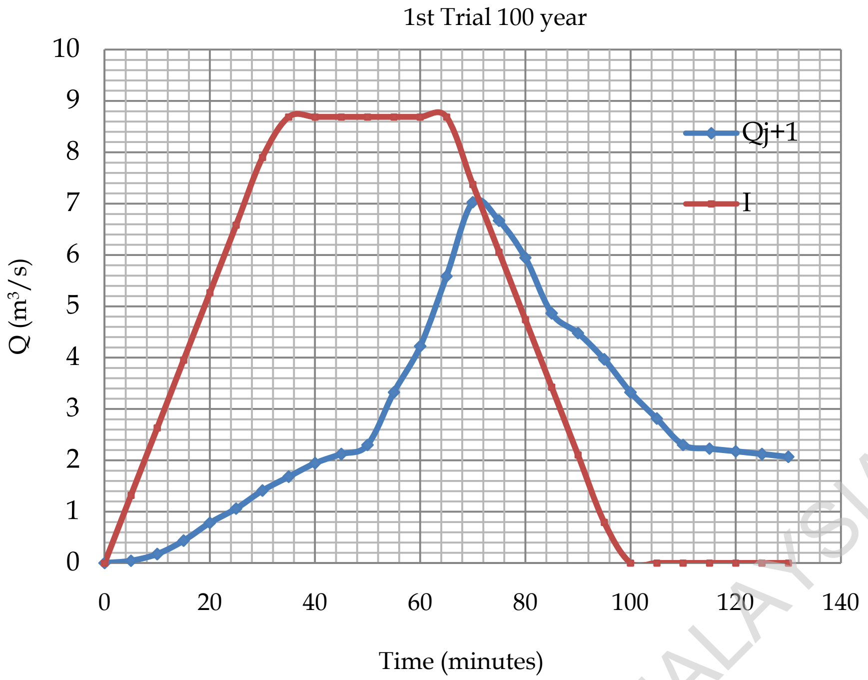
Using a routing time step of 1.0 minutes, the maximum water level in the pond is **33.65 m**, LSD that corresponds to a water depth of 2.60 m. Allowing a freeboard of 300 mm for wave action, the embankment crest elevation is set at **34.0 m**, LSD.

(iv) Check if the results are acceptable

The basin will provide a reduction in the 100 year ARI flow, being reduced from **8.69 m^3/s to 7.03 m^3/s** (more than the 100yr ARI pre-development discharge of **6.0 m^3/s**). Should be ok since the pond is to cater for 50 year ARI only. Since the maximum water level (33.60 m LSD) is within the allowable level, the pond sizing and routing is ok.

Table 7.A17: Routing Table – 5 yr, 50 yr and 100 yr ARI

Time (min)	I (m^3/s)	$(I_1 + I_2)/2$ (m^3/s)	$S_1/\Delta t + O_1/2$ (m^3/s)	O_1 (m^3/s	$S_2/\Delta t + O_2/2$ (m^3/s)	O_2 (m^3/s
0	0	0.00	0.00	0.00	0.00	0.00
1	0.26	0.13	0.00	0.00	0.13	0.00
2	0.53	0.40	0.13	0.00	0.53	0.00
3	0.79	0.66	0.53	0.00	1.19	0.00
4	1.05	0.92	1.19	0.00	2.11	0.00
5	1.32	1.19	2.11	0.00	3.29	0.04
Continued						
…	…	…	…	…	…	…
67	8.16	8.30	312.43	6.07	314.65	6.65
68	7.90	8.03	314.65	6.65	316.03	7.03
69	7.64	7.77	316.03	7.23	316.57	7.03
70	7.37	7.51	316.57	7.23	316.85	7.03
71	7.11	7.24	316.85	7.23	316.87	7.03
72	6.85	6.98	316.87	7.23	316.62	7.03
73	6.58	6.72	316.62	7.23	316.11	7.03
74	6.32	6.45	316.11	7.23	315.33	7.03
75	6.06	6.19	315.33	7.23	314.29	6.65
76	5.79	5.93	314.29	6.65	313.57	6.65
80	4.74	4.87	310.40	6.07	309.20	5.49
Continued						
…	…	…	…	…	…	…

Reference	Calculation	Output
	 Figure 7.A9: Resulting Outflow Hydrograph – 100yr ARI Step 10: Check behaviour under extreme conditions As the pond is small in volume and bund height it is not a hazardous structure. Therefore, no checking was required for the bund failure due to PMF. Step 11: Prepare design drawings and specifications Refer Figure 7.A11 and 7.A12.	

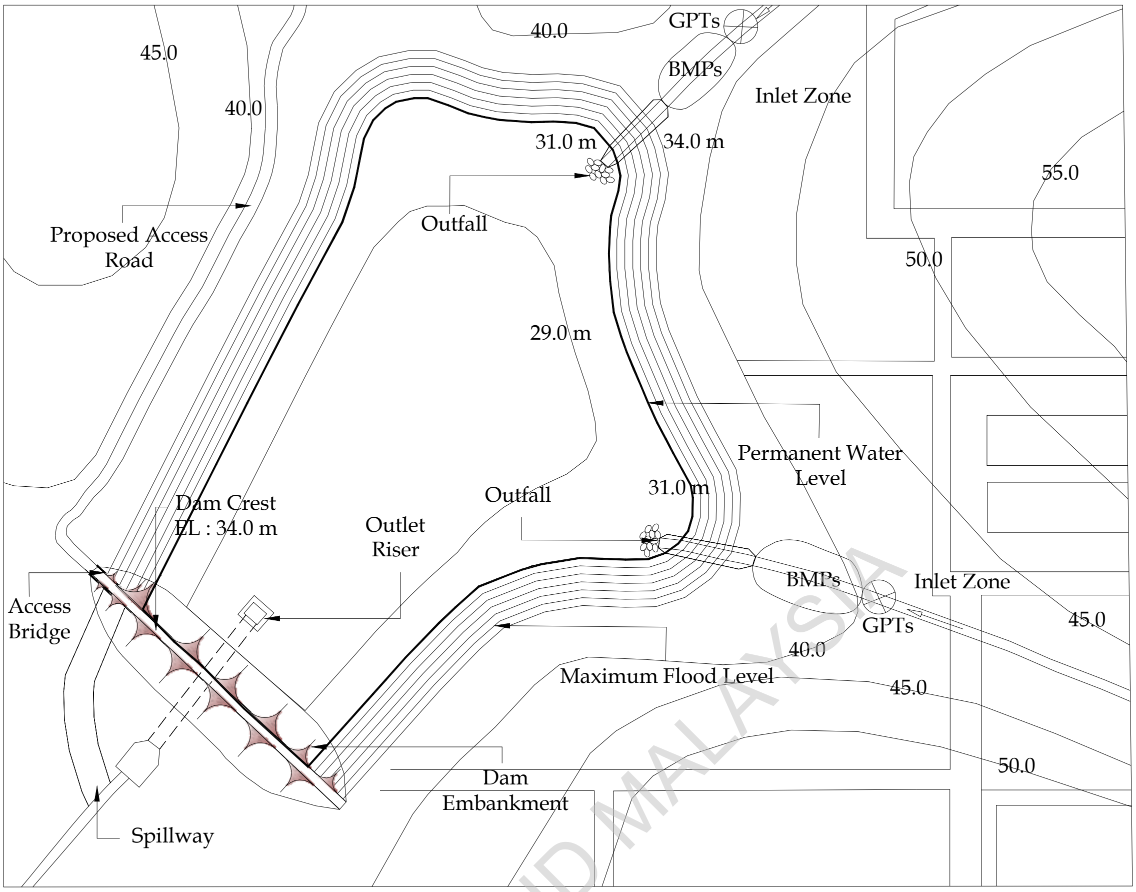


Figure 7.A10: Layout Plan of the Designed Detention Pond

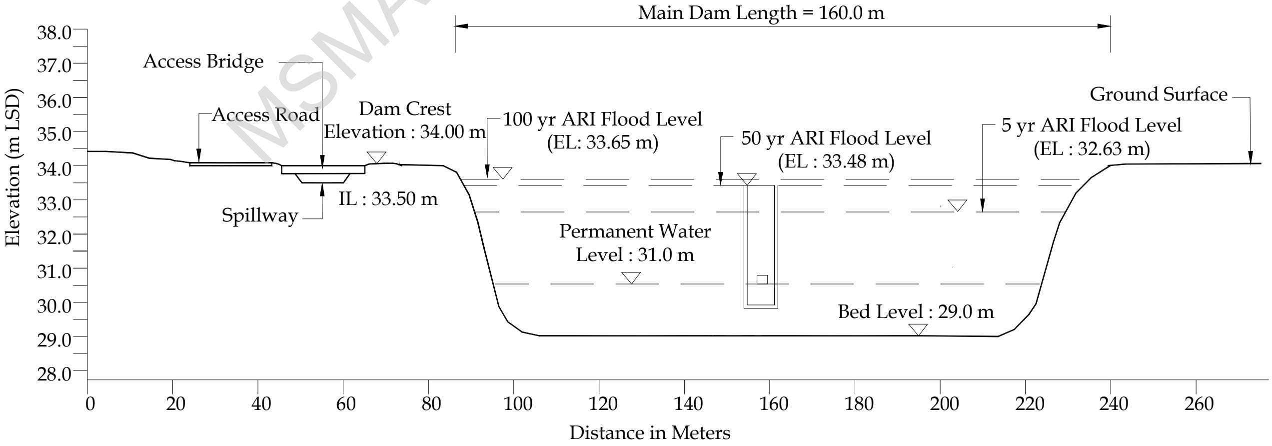


Figure 7.A11: Cross Section of the Designed Detention Pond

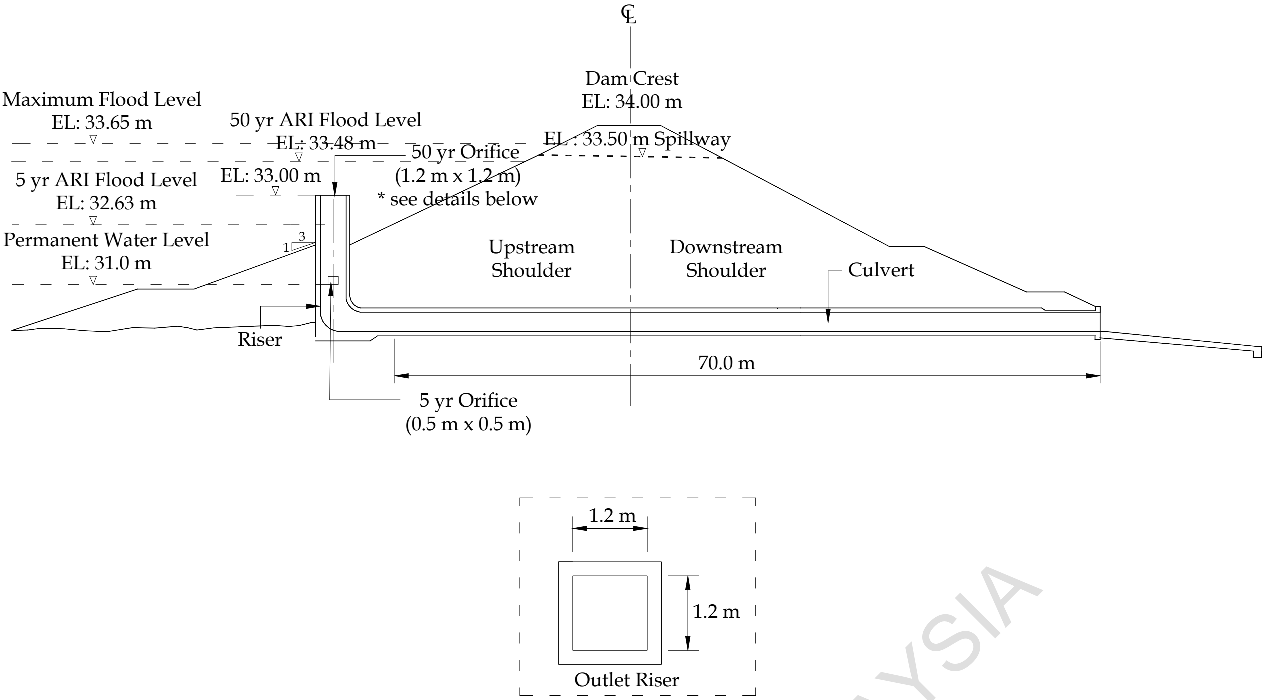
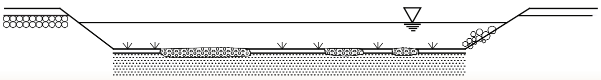


Figure 7.A12: Dam Cross-Section



INFILTRATION FACILITIES 8

CHAPTER 8 INFILTRATION FACILITIES

8.1 INTRODUCTION

This Chapter presents system, function and design of infiltration facilities. They serve primarily for removing the associated pollutants contained in the captured stormwater volume by filtration process through soil media above the groundwater table (GWT). The system can offer reduced pollutant loadings to downstream major runoff treatment BMPs, such as water quality ponds or wetlands. The main types of infiltration BMPs are sump, porous pavement, trench (Figure 8.1) and basin (Figure 8.2). The facilities are dry or empty when not in operation and are part of green park environment. The infiltration basin can provide additional functions of stormwater quantity controls and design of which shall be based on detention pond.

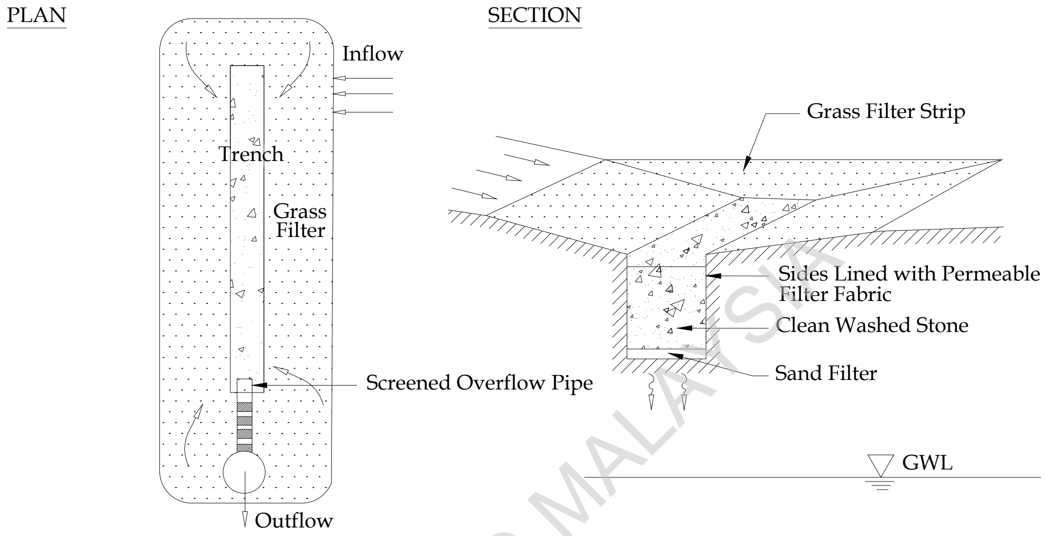


Figure 8.1: Typical Infiltration Trench Design

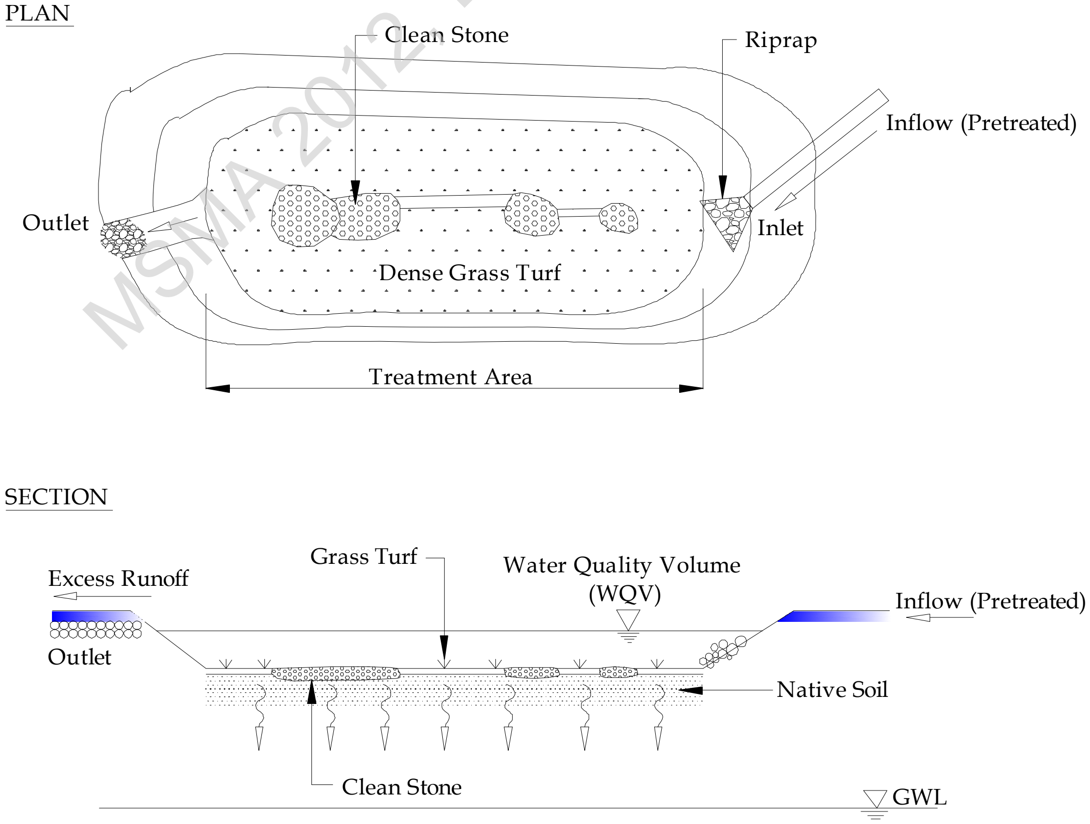


Figure 8.2: Typical Infiltration Basin Design

The facilities can be located on-site or along public drainage, depending on runoff contributing areas, pollution intensity and landuse practices being dealt with (Figure 8.3).

a) Infiltration Trench – Parking lot b) Infiltration Basin - Residential

Figure 8.3: Typical Infiltration Application

8.2 PLANNING AND FEASIBILITY ANALYSIS

The infiltration facilities must be carefully selected, located, designed, and maintained to achieve their design benefits as well as to protect areas where groundwater quality is of concern. Infiltration can be successfully utilised if adherence to proper construction and maintenance standards is followed in accordance with its design life.

8.2.1 Drainage Area

Infiltration BMPs are limited in their ability to accept and treat flows from larger drainage areas. The types of infiltration BMPs and their drainage area limitations are given in Table 8.1.

Table 8.1: Infiltration BMPs and Drainage Area Limit

Infiltration BMPs	Maximum Drainage Area
Dispersion trenches/Infiltration sumps	500 m^2
Infiltration trenches/Porous pavements	4 hectares
Infiltration basins	15 hectares

8.2.2 Pollutant Removal Capabilities

Pollutant removal mechanisms in an infiltration system include absorption, filtration and microbial decomposition. Pre-treatment BMPs are required to remove coarse particulate matter and to reduce/prevent excessive pollutant loads entering the facilities. The typical removal capacity of infiltration facilities are summarised in Table 8.2.

Table 8.2: Typical Pollutant Removal Efficiencies for Infiltration BMPs (Fletcher et al., 2003)

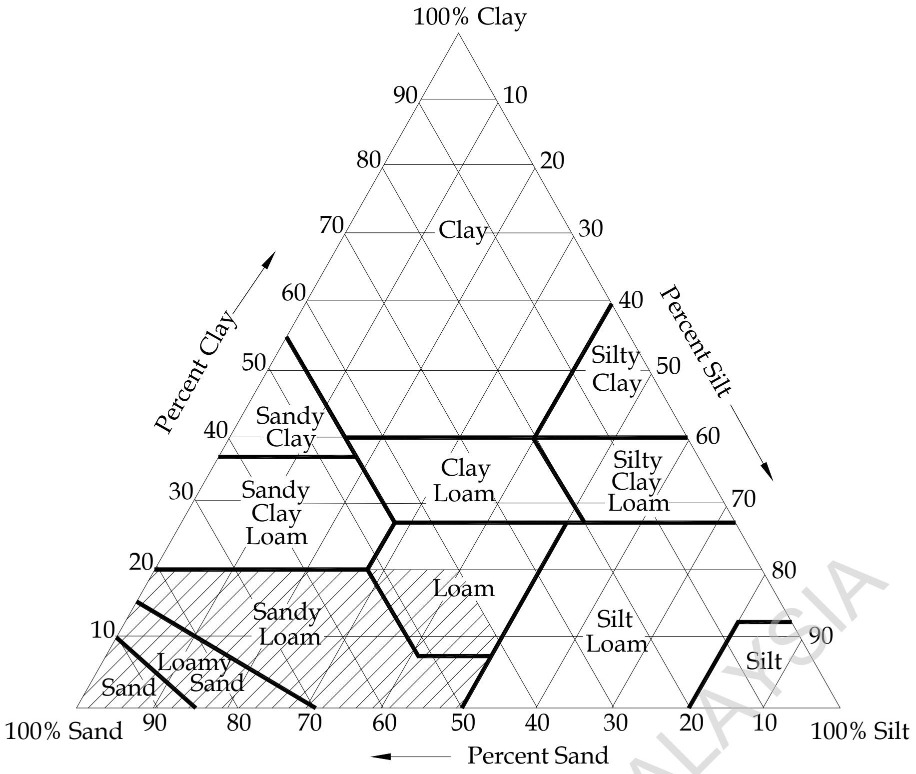
Pollutant	Expected Removal	Comments
Litter	>90%	Expected to trap all gross pollutants, except during high-flow bypasses
Total Suspended Solids	65 – 99%	Pre-treatment is required to reduce clogging risk
Total Nitrogen	50 – 70%	Depend on nitrogen speciation and state (soluble or particulate)
Total Phosphorus	40- 80%	Depend on nitrogen speciation and state (soluble or particulate)
Heavy Metals	50 – 95%	Depends on state (soluble or particulate)

8.2.3 Site Suitability Limitations

Soil types, surface geological conditions and groundwater levels determine the suitability of infiltration systems. For a site to be suitable, it must meet or exceed all of the criteria listed in Table 8.3. Should a site investigation reveal that any one of the Suitability Limitations cannot be met; the implementation of the infiltration practice should not be pursued.

Table 8.3: Site Suitability Limitations (DID, 2000).

Limitation	Comments
Soil	The suitability of soil for infiltration is to be based on evaluating the following: • Saturated soil infiltration rate should be a minimum rate of 13mm/hr • Soils with 30% or greater clay content or 40% greater silt/clay content should not be used (refer to the USDA Textural Triangle in Figure 8.4) • Aerobic conditions are to be maintained to the fullest extent possible for runoff treatment BMPs by designing them to drain the water quality design storm in 48 hours or less. • Infiltration systems should not utilise fill material or be placed over fill soils. (At locations where soils are unsuitable for infiltration or where water tables are high, it may be practical to provide infiltration through soil layers in fill, possibly collecting percolating water in pipes. This should not be ruled out, but suitable safeguards should be applied.)
Depth to Water Table, or Impermeable Layer (Bedrock)	The base of all facilities should be located at least 1.5m above the seasonal high ground water level and/or any impermeable layer (such as bedrock, clay lens, etc.).
Proximity to Drinking Water Wells, Building Foundations, Structures and Property Lines	The proximity of infiltration facilities to other structures and facilities must be taken into account where the potential exists to contaminate ground water, damage foundations and other property. The site investigation is required to determine the most appropriate locations of infiltration facilities; this is best done on a case-by-case basis but the following basic criteria are provided: • The facilities should not be allowed in well-field areas or near wells or springs used for drinking water supply. • Infiltration facilities should be situated at least 7m down-slope and 50m up-slope from building foundations. Infiltration sumps or trenches (on-site facilities) should be located at a minimum of 3m from any structure and 10m from a non-potable water supply well, septic tank or drain field. • Infiltration facilities on commercial and industrial sites, even though are not recommended, should be placed no closer than 35m from a non-potable water supply well, septic tank or drain field,
Land Slope	Infiltration facilities can be located on slopes up to 15% as long as the slope of the base of the facility is less than 3%. All basins should be a minimum of 20m from any slope greater than 15%.
Control of Siltation	During construction, it is critical not to excavate infiltration trenches or basins to final grade during this phase. After construction, it is vital to prevent as much sediment as possible from entering by first routing the water through a pre-treatment BMP. In the case of infiltration trenches, clogging occurs most frequently on the surface. Grass clippings, leaves and accumulated sediment should be removed routinely from the surface. If clogging appears to not be only at the surface, it may be necessary to remove and replace the first layer of filter media and the geotextile filter. In the case of infiltration basins, sediment should be removed when it is sufficiently dry so that the sedimentation layer can be readily separated from the basin floor



USDA Textural Triangle

Shaded area is applicable for design of infiltration BMPs

Figure 8.4: USDA Textural Triangle (USDA, 1971)

8.2.4 Design Criteria

8.2.4.1 Principles

The general principle in designing infiltration facilities is to capture the entire runoff water quality volume (WQV) of 40mm over the equivalent impervious contributing catchment area for water quality treatment purposes, whereas for water quantity regulation, the design acceptance criterion requires that the peak be reduced to the pre-developed value for the minor system design storm ARIs. Thus, the acceptance criteria for the design infiltration facilities require satisfying both standards.

8.2.4.2 Check List

The design criteria listed and discussed in this section are intended for designing infiltration facilities.

(i) Soil Investigation

Soil investigation is required for any type of infiltration facilities for each proposed development location in order to determine the soil type and groundwater level. Each soils log should extend a minimum of 3.0m below the bottom of the facility, describe the series of the soil, the textural class of the soil horizon(s) through the depth of the log and note any evidence of high ground water level. In addition, the location of impermeable soil layers or dissimilar soil layers should be determined.

(ii) Design Infiltration Rate

The design infiltration rate (f_d) should be one-half the infiltration rate (f_c) found from the soil textural analysis ($f_d = 0.5f_c$).

(iii) Runoff Pre-Treatment

Suspended solids and pollutants in runoff should be pre-treated prior to discharge to any infiltration facilities.

(iv) Fill Material

Aggregate materials (for trenches and upper layers of infiltration basins) should consist of a clean aggregate with void space (i.e., porosity) in the range of 30 to 40%. The aggregate should be poorly graded.

(v) Buildings

Dispersion trenches, infiltration sump, infiltration trenches or porous pavement should be located 3m from building foundations. Meanwhile, infiltration basins should be a minimum of 50m upslope and 7m downslope from any building. Further, these facilities should be a minimum of 20m from any slopes greater than 15%. A geotechnical report should address the potential impact of the basin infiltration upon the steep slope.

(vii) Overflow Route

An overflow route must be identified in the event that the facilities capacity is exceeded (greater than minor system design ARI). The overflows must be connected to a conveyance system.

(vii) Spillways

The spillway requirement is only applied for the infiltration basin. All aspects of the principal spillway design and the emergency spillway should follow the details provided for detention pond in Chapter 7.

8.3 DESIGN PROCEDURE

8.3.1 General

The design methodologies presented here concentrate on two types of infiltration facilities: infiltration trenches and infiltration basins. The design methodology of all infiltration facilities should be applied to all type of facilities that emphasise on their contributing drainage areas. There are two general types of situations where infiltration facilities may be used.

- One may be interested in the dimensions of an infiltration device that is required to provide storage of the water quality volume (*WQV*), and/or downstream protection volume of runoff.
- The site conditions may dictate the layout and capacity of infiltration measures and one might be interested in determining the level of control provided by such a layout. In this, control may not be sufficient to meet stormwater treatment requirements, and additional control, possibly using other acceptable control measures (such as conveyance systems) may be required.

8.3.2 Design of On-site Facilities

8.3.2.1 General Considerations

The design procedure outlined in this section should be used in designing on-site trench systems that include dispersion trenches, infiltration sumps and infiltration trenches that their drainage contribution area is less than 500m².

The design of a trench system is based on the textural class of the soils underlying. The design of a trench system is also based on the maximum allowable depth of the trench (d_{max}) which should meet the following criteria:

$$d_{max} = \frac{f_c T_s}{n} \tag{8.1}$$

where,

f_c = Final infiltration rate of the trench area (mm/hr);
T_s = Maximum allowable storage time (hr); and
n = Porosity of the stone reservoir.

Design criteria for on-site facilities are provided in Table 8.4. A trench system is sized to accept the design volume that enters the trench (V_w) plus the volume of rain that falls on the surface of the trench (PA_t) minus the exfiltration volume ($f_d TA_t$) out of the bottom of the trench (Figure 8.5). Based on the analysis, the effective filling time for most trenches (T) will generally be less than two hours. The volume of water that must be stored in the trench (V) is defined as:

$$V = V_w + PA_t - f_d TA_t \tag{8.2}$$

where,

P = Design rainfall event (mm);
A_t = Trench surface area (m^2);
V_w = Design runoff volume that enters the trench (m^3);
T = Effective filling time (hr), generally < 2; and
f_d = Design infiltration rate (mm/hr).

Table 8.4 Design Criteria of On-Site Infiltration Facilities

Design Parameters	Dispersion Trench/ Infiltration Sump	Infiltration Trench
Contributing drainage area	Up to 500m^2	Up to 500m^2
Soil investigation requirement	1 soil log test per site extending to 1.5m below of the trench bottom	1 soil log test every 15m of trench extending to 1.5m below of the trench bottom
Design infiltration rate (f_d)	f_d is equal to 0.5f_c with minimum f_c of 13mm/hr	f_d is equal to 0.5f_c with minimum f_c of 13mm/h
Maximum drawdown time	24hrs - Design ARI for Minor System	24hrs - Design ARI for Minor System
Design Runoff	Design ARI for Minor System (Quantity) Capture 40mm of rainfall over the contributing catchment area (Quality)	Design ARI for Minor System (Quantity) Capture 40mm of rainfall over the contributing catchment area (Quality)
Fill material	void space in the range of 30 to 40%	void space in the range of 30 to 40%
Maximum depth	1.5m above the seasonal groundwater or impermeable layer	1.5m above the seasonal groundwater or impermeable layer
Minimum clearance to special facilities	3m from building foundation 10m from non-potable water supply well	3m from building foundation 10m from non-potable water supply well
Overflow route	needs to be identified	needs to be identified
Observation well	1 every site (recommended)	1 every 15m of trench length

For most design storm events, the volume of water due to rainfall on the surface area of the trench (PA_t) is small when compared to the design volume (V_w) of the trench and may be ignored with little loss in accuracy to the final design.

The volume of rainfall and runoff entering the trench can be defined in terms of trench geometry. The gross volume of the trench (V_t) is equal to the ratio of the volume of water that must be stored (V) to the porosity (n) of the stone reservoir in the trench; V_t is also equal to the product of the depth (d_t) and the surface area (A_t):

$$V_t = \frac{V}{n} = d_t A_t n \qquad (8.3)$$

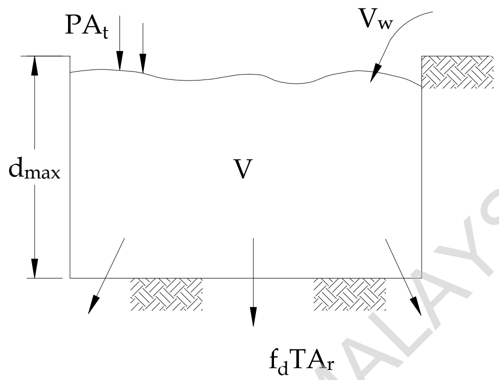


Figure 8.5: Model of Trench Hydrologic Balance

Combining Equations 8.2 and 8.3 yields the relationship: $d_tA_t n + f_dTA_t = V_w$. Because both dimensions of the trench are unknown, this equation may be rearranged to determine the area of the trench (A_t) if the value of d_t were set based on either the location of the water table or the maximum allowable depth of the trench (d_{max}):

$$A_t = \frac{V_w}{nd_t + f_dT} \qquad (8.4)$$

In the event that the side walls of the trench must be sloped for stability during construction, the surface dimensions of the trench should be based on the following equation:

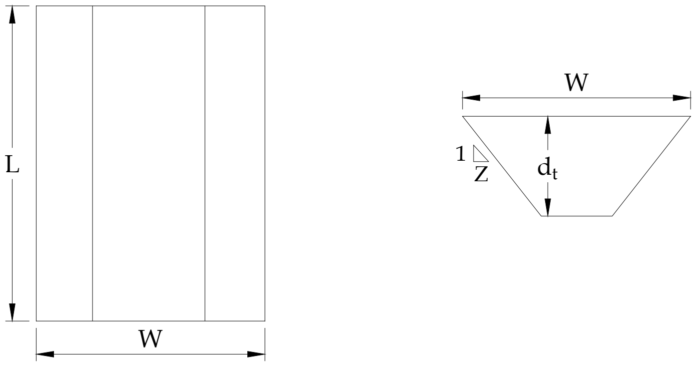
$$A_t = \left(L - Zd_t\right)\left(W - Zd_t\right) \qquad (8.5)$$

where,

L = The top length (m);
W = The top width (m); and
Z = The trench side slope ratio.

The design procedure would begin by selecting a top width (W) that is greater than 2 x Zd_t for a specified slope (Z). The side slope ratio value will depend on the soil type and the depth of the trench. The top length (L) is then determined as:

$$L = Zd_t + \frac{A}{W - Zd_t} \qquad (8.6)$$



a) Plan b) Side

Figure 8.6: Schematic of Trench

8.3.2.2 Design steps

This involves the following steps:

- Determine the contributed water quality volume (40 mm over the equivalent impervious contributing catchment area) from the development for storage to meet the runoff control requirement (acceptance criteria).
- Compute the maximum allowable depth (d_{max}) from the feasibility equation, $d_{max} = f_c T_s / n$.
- Compute volume of water that must be stored in the trench (V) as $V = V_w + PA_t - f_d TA_t$.
- Determine the surface area (A_t) dimensions of the trench as $A_t = (L - Zd_t)\,(W - Zd_t)$. This area should be
  greater or equal than the area considered in the preliminary sizing.

8.3.3 Design of Community Facilities

8.3.3.1 General Considerations

The same principles as those applied in the previous section can be used as general design consideration for infiltration trench and basin, serving for larger catchment areas. The design of an infiltration basin facility is based on same soil textural properties and maximum allowable depths as an infiltration trench. However, because the infiltration basin uses an open area or shallow depression for storage, the maximum allowable depth (d_{max}) should meet the following criteria:

$$d_{max} = f_c T_p \tag{8.7}$$

where,

f_c = Final infiltration rate of the basin area (mm/hr); and
T_p = Maximum allowable ponding time (hr).

Design criteria are set out in Table 8.5.

An infiltration basin is sized to accept the design volume that enters the basin (V_w) plus the volume of rain that falls on the surface of the basin (PA_b) minus the exfiltration volume (fTA_b) out of the bottom of the basin. Based on the analysis, the effective filling time for most infiltration basins (T) will generally be less than two hours. The volume of water that must be stored in the basin (V) is defined as:

$$V = V_w + PA_b - f_d TA_b \tag{8.8}$$

where,

P = Design rainfall event (mm);
A_b = Basin surface area (m^2);
V_w = Design volume that enters the basin (m^3);
T = Effective filling time (hr), generally < 2 ; and
f_d = Design infiltration rate (mm/hr).

For most design storm events, the volume of water due to rainfall on the surface area of the basin (PA_b) is small when compared to the design volume (V_w) of the basin and may be ignored with little loss in accuracy to the final design.

Table 8.5: Design Criteria of Community Infiltration Facilities

Design Parameters	Infiltration Trench	Infiltration Basin
Drainage area	Up to 4ha.	Up to 15ha
Soil investigation requirement	1 soil log test every 15m of trench 1.5m below of the trench bottom	1 soil log test every 500m² of basin area 1.5m below of the basin bottom
Design infiltration rate (f_d)	f_d is equal to $0.5f_c$ with a minimum f_c of 13mm/hr	f_d is equal to $0.5f_c$ with a minimum f_c of 13mm/hr
Maximum drawdown time	24 hours – 10 year ARI	24 hours – 10 year ARI
Runoff treatment	Design ARI for Minor System (Quantity) 40mm (Quality)	Design ARI for Minor System (Quantity) 40mm (Quality)
Backfill material	void space in the range of 30 to 40%	void space in the range of 30 to 40%
Maximum depth	3m with minimum 1.5m above the seasonal groundwater	3m with minimum 1.5m above the seasonal groundwater
Minimum proximity to special facilities or building foundation	7m (downslope) 50m (upslope)	7m (downslope) 50m (upslope)
Overflow route	needs to be identified	needs to be identified
Observation well	1 every 15m of trench length	1 every 50m² of basin area
Spillway and Embankment		Require and should be stabilised and planted with vegetation

The volume of rainfall and runoff entering the basin can be defined in terms of basin geometry. The geometry of a basin will generally be in the shape of an excavated trapezoid with specified side slopes. The volume of a trapezoidal shaped basin may be approximated by:

$$V = \frac{(A_b + A_t)d_b}{2} \tag{8. 9}$$

where,

A_t = Top surface area of the basin (m^2);
A_b = Bottom surface area of the basin (m^2); and
d_b = Basin depth (m).

The bottom length and width of the basin may be defined in terms of the top length and width as shown in Figure 8.6:

$$L_b = L_t - 2Z\,d_b \tag{8.10a}$$

$$W_b = W_t - 2Z\,d_b \tag{8.10b}$$

where,

L_b = Basin bottom length (m);
W_b = Basin bottom width (m);
L_t = Basin top length (m);
W_t = Basin top width (m); and
Z = Specified side slope ratio (H:V).

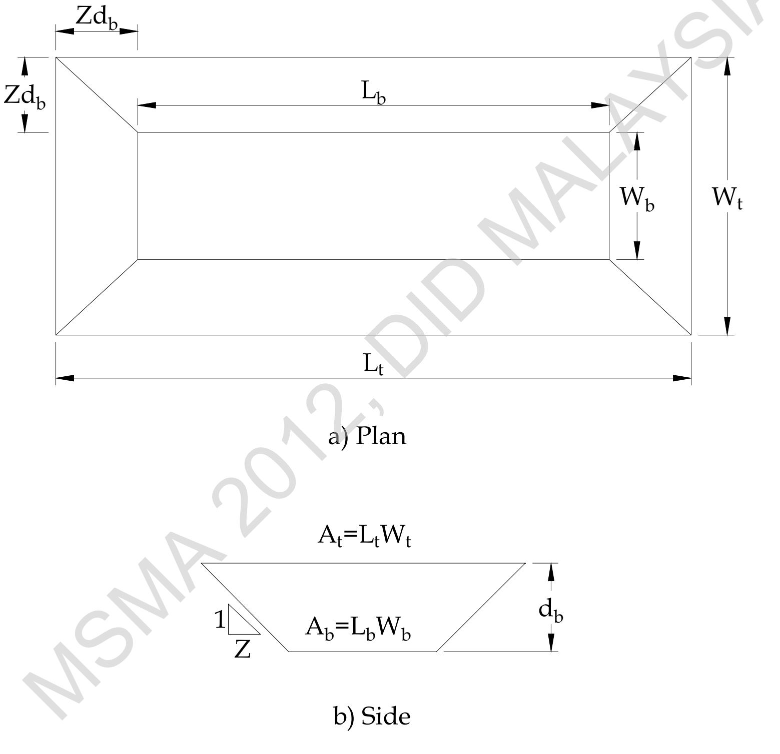


Figure 8.6: Schematic of Basin Nomenclature (Wisconsin Department of Natural Resources, 2004)

By setting Equations 8.8 and 8.9 equal and substituting the above relationships for L_b and W_b, the following equation is derived for the basin top length:

$$L_t = \frac{V_w + Zd_b\left(W_t - 2Zd_b\right)}{W_t d_b - Zd_b^2} \tag{8.11}$$

The infiltration basin usually adopts irregular shape in accordance with grading plan. Sizing is thus based on method described in Chapter on detention basin.

8.3.3.2 Design steps

This involves the following steps:

- Determine the contributed volume of water from the development for storage to meet the runoff control requirement (acceptance criteria).

- Compute the maximum allowable basin depth (d_{max}) from the feasibility equation, $d_{max} = f_c T_p$. Select the basin design depth (d_b) based on the depth that is the required depth above the seasonal groundwater table, or a depth less than or equal to d_{max} , whichever results in the smaller depth

- Compute the basin surface area dimensions for the particular soil type using Equation 8.11. The basin top length (L_t) and width (W_t) must be greater than $2Zd_b$ for a feasible solution. If L_t and W_t are not greater than $2Zd_b$ the bottom dimensions would less than or equal to zero. In this case, the basin depth (d_b) should be reduced for a feasible solution.

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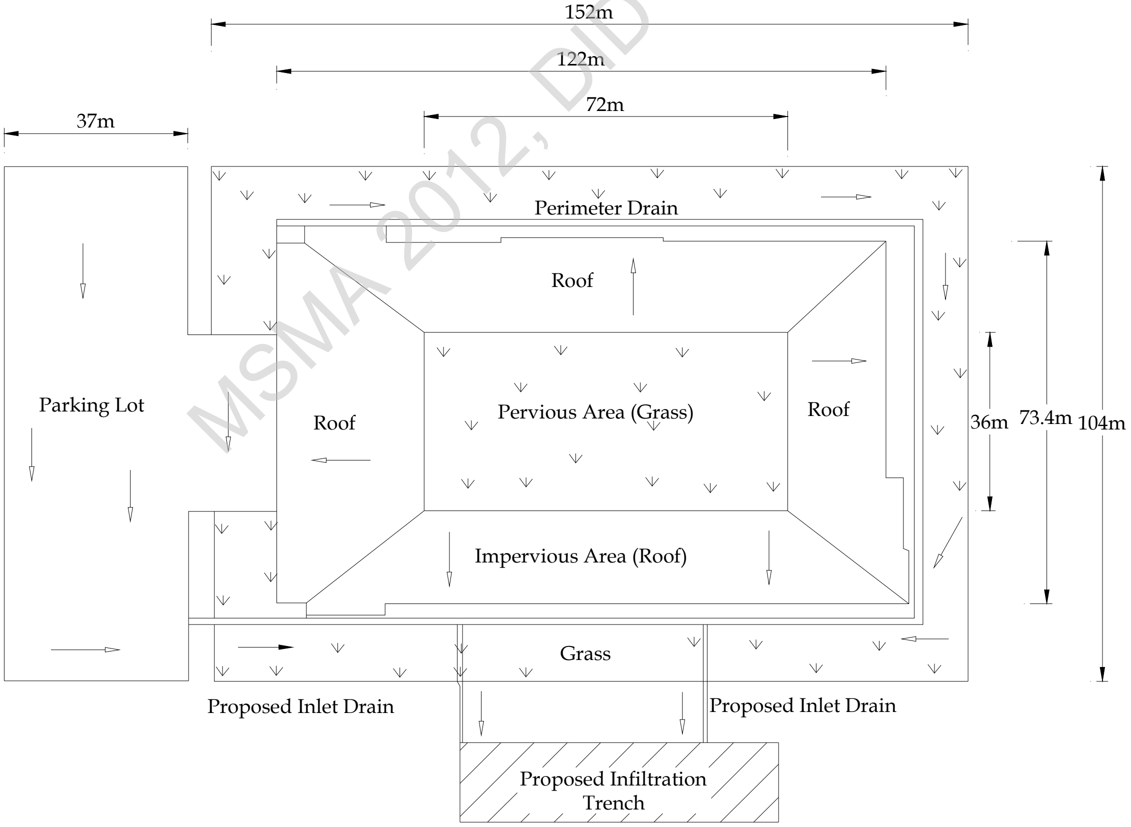
APPENDIX 8.A EXAMPLE – INFILTRATION TRENCH

Problem:

Estimate the preliminary size of an infiltration trench proposed for the USM School of Civil Engineering building in Nibong Tebal, Pulau Pinang with 1.59ha of institutional area. The impervious area is 65% of the catchment area. Given the value of infiltration rate, f = 35 mm/hr, maximum storage time, T_s = 24 hr, effective filling time, T_f = 2 hr and porosity of fill materials, n = 0.35.

a) Site Plan



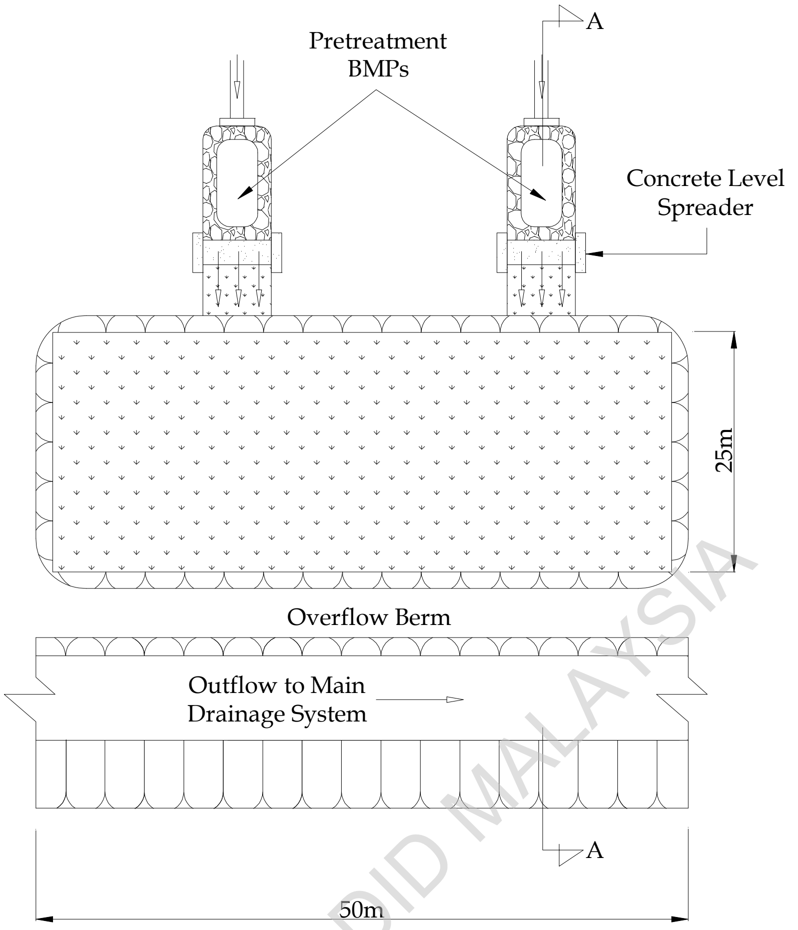
b) Building and Drainage Layout

Figure 8.A1: Proposed Trench Site and Design

Solution:

Reference	Calculation	Output
	Determine the contributed water quality volume WQV	
Table 2.5	Runoff coefficient for commercial and business centres = 0.95 Runoff coefficient for sport fields, park and agriculture = 0.40 Impervious area = 0.65 Pervious area = 0.35	
Equation 11.1	Required storage for WQV $= C_{ave} (P_d) A$ = (0.65*0.95 + 0.35*0.4)*0.04 x 15900 =	481.8m^3
	where: WQV = Water quality volume C_{ave} = Average rational runoff coefficient (Refer Table 2.5) P_d = Rainfall depth for water quality design storm (= 40 mm) A = Contributing drainage area (ha)	
	Determine of Trench Size	
	Porosity of fill materials, n =	0.35
	Maximum storage time, T_s (hours) =	24hr
	Effective filling time, T_f (hours) =	2hr
	Infiltration rate, f =	35mm/hr
Equation 8.1	Maximum allowable depth, $d_{max} = \frac{f_c T_s}{n}$	
	$= \frac{0.035 \times 24}{0.35}$ =	2.4m
	Thus, the proposed depth (d_t) of the trench =	1.0m
	Design infiltration rate (f_d) $= 0.5 f_c$ =	0.0175m/hr
	$A_t = \frac{V_w}{nd_t + f_d T_f}$	
	$A_t = \frac{481.8}{0.35 \times 1.0 + 0.0175 \times 2.0}$ =	1252m^2
	The dimension of the proposed infiltration trench is should be based on capture of the required water quality volume as 1252m^2 =	50m x 25m x 1m

PLAN



SECTION A-A

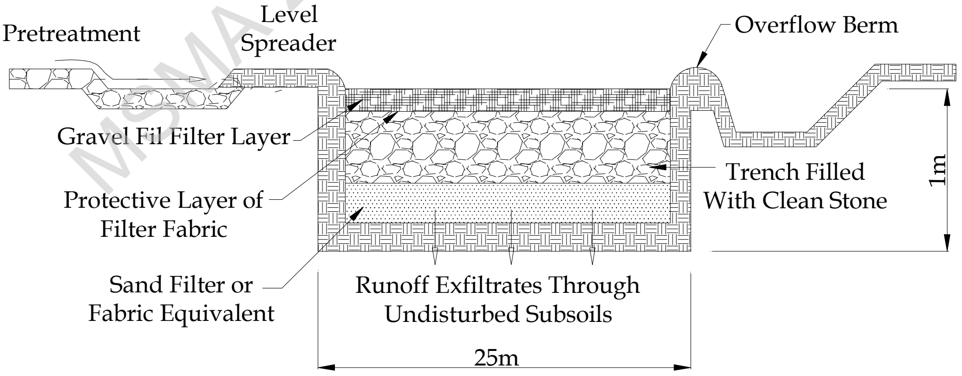


Figure 8.A2: Infiltration Trench Design

APPENDIX 8.B EXAMPLE – INFILTRATION BASIN

Problem:

Estimate the preliminary size of an infiltration basin proposed for the Bertam Perdana, Pulau Pinang. The catchment area is 12.70ha with 52% of impervious area. The site condition of pre-development is palm oil plantations. From initial site investigation, the characteristic of the catchment is as follows:

Soil type	: Sandy loam	
Infiltration capacity (f_c)	: 35mm/hr	
Ground water level	: 3m (below ground surface)	
The following assumptions are made	: Time of concentration pre-development, t_{cs}	= 45 minutes
	Time of concentrations, t_c	= 30 minutes
	Porosity of sandy loam, n	= 0.35
	Maximum storage time, T_s	= 24 hr
	Effective filling time, T_f	= 2 hr

Figure 8.B1: Site Plan

Reference	Calculation	Output
	Determine the contributed water quality volume WQV	
Table 2.5	Runoff coefficient for commercial and business centres = 0.95	
	Runoff coefficient for sport fields, park and agriculture = 0.40	
	Impervious area = 0.52	
	Pervious area = 0.48	
Equation 11.1	Required storage for WQV = C_{ave} (P_d) A = (0.52*0.95 + 0.48*0.4)x0.04 x 127000 =	3484.9m^2
	where: WQV = Water quality volume C_{ave} = Average rational runoff coefficient (Refer Table 2.5) P_d = Rainfall depth for water quality design storm (= 40 mm) A = Contributing drainage area (m^3)	

Reference	Calculation	Output
	<u>Determine of Basin Size</u>	
	Porosity of soil materials, n =	0.35
	Maximum storage time, T_s (hours) =	24hr
	Effective filling time, T_f (hours) =	2hr
	Infiltration rate, f =	35mm/hr
Equation 8.7	Maximum allowable depth, $(d_{max}) = f_c T_p$ = (35/1000) x 24 =	0.84m
	<u>Thus, the proposed depth (d_b) of the basin</u> =	0.5m
	The proposed side slope 1:4 (V:H)	
	Design infiltration rate (f_d) = 0.5 f_c = 0.5 x (35/1000) =	0.00875m/hr
Equation 8.10a	$L_b = L_t - 2Zd_b$	
Equation 8.10b	$W_b = W_t - 2Zd_b$	
Equation 8.11	$L_t = [\, V_w + Zd_b\,(W_t - 2Zd_b)] \,/\, [\, W_t\, d_b - Zd_b^2]$	
	V_w is equal to WQV and Say W_t is 100m	
	L_t = [3484.9+(4x0.5)(100 – 2x4x0.5)] / [(100 x 0.5)–(4x0.5^2)] =	75.04m
	The top dimension of the proposed infiltration basin is 100.0m x 72.0m x 0.5m	
	W_b = 75.04–(2x4x0.5) =	71.04m
	L_b = 100-(2x4x0.5) =	96m
	The bottom/floor dimension of the proposed infiltration basin =	96m x 71m

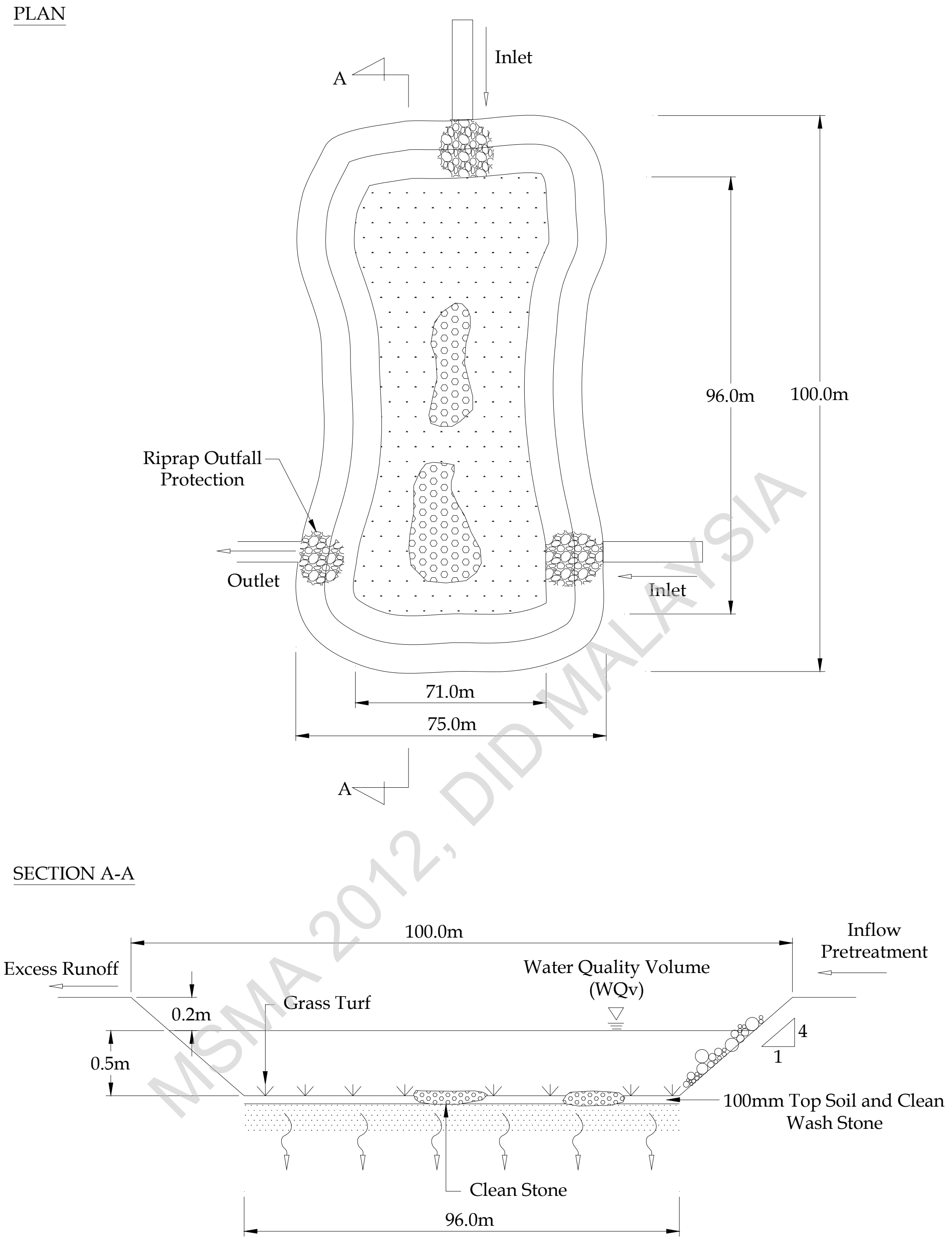
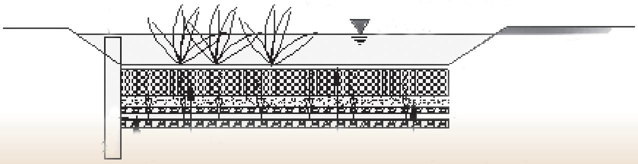


Figure 8.B3: Infiltration Basin Design



BIORETENTION SYSTEMS 9

CHAPTER 9 BIORETENTION SYSTEMS

9.1 INTRODUCTION

This Chapter provides bioretention systems in term of functions, components and design methods to best serve stormwater quality treatment objectives while promoting a pleasant and green living environment in the urban setting. Bioretention systems are a form of stormwater best management practices (BMPs) that use biological uptake and porous media filtration processes, combined, to treat stormwater runoff. The systems integrate vegetation, such as trees, shrubs and grasses, and layered media using soil, sand and mulches. These structural controls capture, temporarily detain and treat runoff from small rainstorms before release it back to the receiving waters. Runoff shall be pretreated and diverted into the systems that can be constructed from a shallow excavated site along a proposed drainage channel or swale.

9.1.1 System Components

Bioretention systems can be designed as permeable or impermeable systems (DOW, 2007). The permeable system (Figure 9.1) drains the water through the filtration media and sand bed layer before spreading to the surrounding native soil and finally recharging groundwater. The impermeable system (Figure 9.2) similarly drains the water from the filtration media through transition layers, however, intercepted by a subsoil pipe/underdrain located in the drainage layer. In an area where native soils have relatively low infiltration capacity or higher rainfall intensity is frequently experienced, such underdrain is required to carry excess water away from the site so that its storage capacity is available for the next storm. The components of a bioretention system consist of pre-treatment, inlet, an excavated basin area with plant and underlying mulch layer, soil bed, sand bed and drainage and outlet structure.

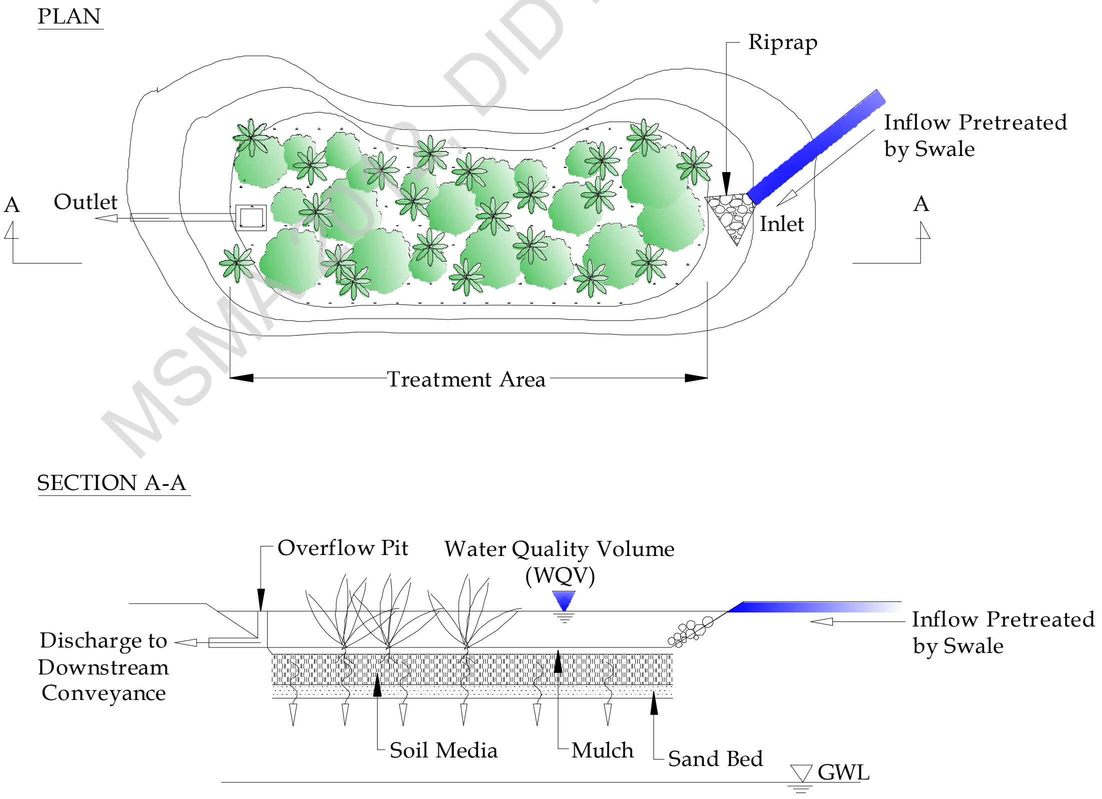


Figure 9.1: Permeable Bioretention Basin

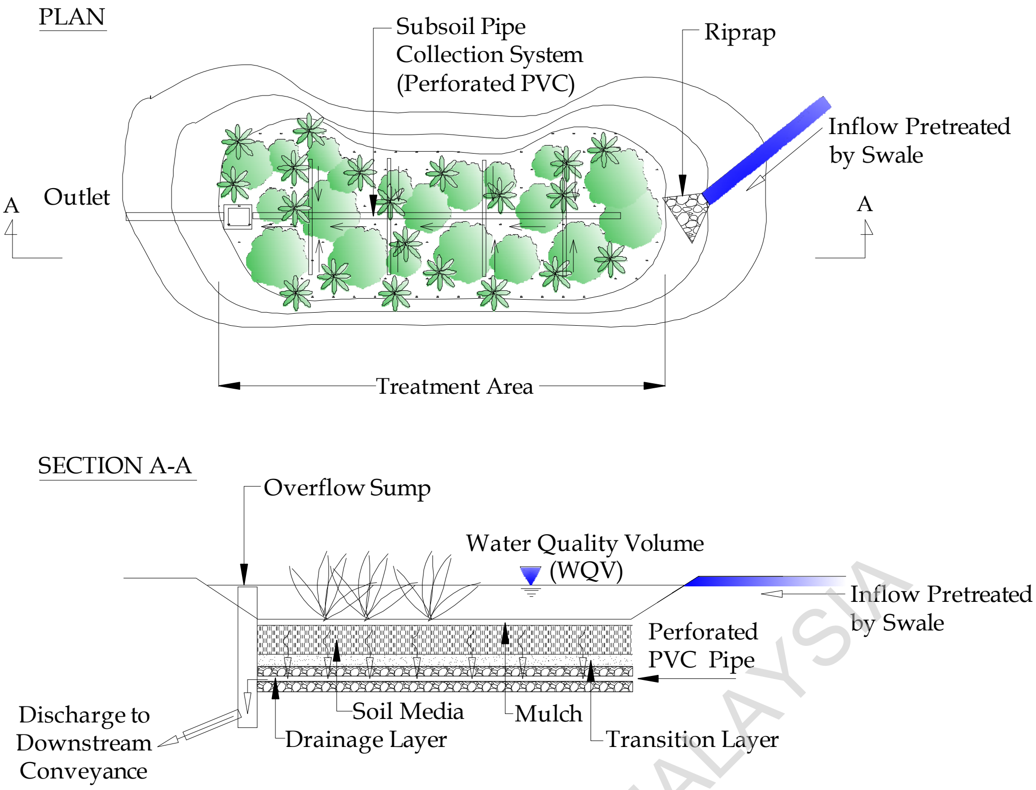


Figure 9.2: Impermeable Bioretention Basin

The selection of plant species can provide for a wide variety of landscape designs. The plants, soils, and organic matter such as compost and a mulch layer all play an important role in treating runoff by naturally breaking down pollutants. The underlying gravel beds (drainage layer) serve to temporarily store and infiltrate treated stormwater after percolating through the organic soil layer (filter media) or finally discharge treated water through under drains. They provide water quality treatment by removing fine sediment, trace metals, nutrients, bacteria and organics through a variety of pollutant removal mechanisms, including:

- Filtration, extended detention treatment;
- Adsorption to soil particles, denitrification; and
- Biological uptake by plants.

The systems provide a high degree of treatment as the increase in the organic content of soils used in bioretention cell promotes removal of pollutants in the water and also absorption of runoff. The organic soils act as a sponge to retain water, providing more storage capacity in the cell. The removal efficiency of a bioretention system for selected stormwater pollutants is given in Table 9.1 (ARC, 2001 & DOW, 2007).

Table 9.1: Removal Efficiency of Bioretention System

Pollutant	Removal Efficiency (%)
Total Suspended Solids (TSS)	80 (High)
Total Phosphorous (TP)	60 (Medium)
Total Nitrogen (TN)	50 (Medium)
Metals	80 (High)

The treated stormwater, filtered through the vegetation and soil media, is collected either in an underdrain system or allowed to infiltrate into the ground. Stormwater runoff higher than the design storm ARIs by passes the bioretention system.

9.1.2 Application

Bioretention systems are generally applied to small sites and ideally suited to many ultra urban areas, such as parking lots. They can fit into existing built up areas and provide visual enhancements to the urban landscape. The systems are typically placed close to runoff sources. They include use as off-line BMPs facilities located adjacent to parking lots, along highway and road drainage swales, around buildings and within landscaped islands in impervious or high-density environments. General form of bioretention systems can be very flexible, including as linear systems, basins and planter boxes (Figure 9.3).

Other application is at stormwater hotspot areas where landuse activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in stormwater. A typical example is a gas station or factory lot. Bioretention system can be used to treat stormwater hotspots as long as an impermeable liner is used at the bottom of the filter bed. The pollutant loading need to be considered for successful use in hotspot areas, as it is likely that the plants will not survive in some of the hotspots if the pollutant loads are high.

a) Residential - Basin

b) Road Side - Planter Box

c) Parking Lot - Linear

d) Factory (Hot Spot) - Basin

Figure 9.3: Typical Bioretention Application Sites

Bioretention can be used as a stormwater retrofit, by modifying existing landscaped areas, or if a parking lot is being resurfaced. In highly urbanised areas, they are one of the few retrofit options that can be employed. Bioretention systems have some limitations that they are used to treat runoff from small drainage areas. Although the systems itself do not consume a large amount of space, incorporating multiple bioretention systems into a high density or congested urban area may reduce the available space uses. Additionally, the construction and cumulative maintenance costs of bioretention systems are relatively high compared with other stormwater treatment BMPs.

9.2 DESIGN CONSIDERATIONS

9.2.1 General

Considerations for selecting the BMPs are the drainage area, landuse practices, slope, soil and subsurface conditions including the depth of the seasonally high groundwater table and the nearest impermeable layer below it.

Designers need to detail conditions and suitability at the site level and must incorporate design features to improve the longevity and performance of the device, while minimizing the maintenance cost.

9.2.1.1 Siting

Generally, the siting of a bioretention system should consider the followings:

- Closeness of placement of the facilities to the source of runoff generation, such as areas upstream from outfalls that receive sheet flow from graded areas.
- Site with land surface that permits the dispersion of flows relatively uniform.
- Space availability for easy installation considering setback requirements on residential subdivision lots and commercial lots.
- The systems should not be located near building areas (unless the design incorporates adequate waterproofing measures and are approved by a geotechnical engineer), well heads, and septic systems. Minimum setback distances from structures and property boundaries are shown in Table 9.2 for different soil type.
- Needs of stormwater management retrofit and redevelopment opportunities especially in areas where total stormwater management control is not feasible.
- The sites to be excavated or cut is suitable for the bioretention system construction.
- Existing wooded areas or other significant natural features should be avoided if possible.

Table 9.2: Minimum Setback Distances (Institute of Engineers Australia, 2006)

Soil Type	Saturated Hydraulic Conductivity (mm/hr)	Minimum Setback (m)
Sands	>180	3.0
Sandy Loam	36 to 180	3.0
Sandy Clay	3.6 to 36	4.0
Medium to Heavy Clay	0.0 to 36	5.0

9.2.1.2 Drainage Area

An individual bioretention system should usually be used to capture and treat runoff from small catchments, limited to less than 1.0 ha of impervious area. When used to treat larger areas, the systems tend to clog in shorter time. In addition, it is difficult to convey flow from a large area to a bioretention system. Generally, commercial or residential drainage areas, exceeding 0.5-1.0 ha in size, will discharge flows greater than the 5-year ARI storm event. When flows exceed this level, the designer should evaluate the potential for erosion to stabilized areas. Typically, flows greater than the 5-year ARI storm event will require channel/pipe enclosure across developed lots. However, by employing drainage runoff dispersion techniques and retaining existing contours, concentrated quantities of flow can be reduced below these thresholds, eliminating or reducing the need for a channel/pipe conveyance system. This may be accomplished by runoff dispersing larger drainage areas based on land grading design to multiple bioretention systems. This runoff dispersion of flow technique can reduce the cost of engineering design and site construction. In addition to reducing the need for drainage channel/pipe conveyance systems, runoff dispersion techniques can also eliminate the need for surface drainage easements or reserves.

9.2.1.3 Slope

Bioretention systems are best located on relatively small slopes (usually less than 5%). Sufficient grade is needed at the site to ensure that the runoff that enters a bioretention system can be connected to the storm drain system. It is important to note, however, that bioretention systems are most often located adjacent to parking lots or residential landscaped areas, which generally have gentle slopes.

9.2.1.4 In-Situ Soils

In an area where in situ soil is sandy and well drained it should be used for locating the permeable bioretention systems. For clayey and poorly drained soils impermeable system shall be used. Saturated hydraulic conductivity for typical soil types is shown in Table 9.3.

Table 9.3: Typical Soil Types and Associated Hydraulic Conductivity (Institute of Engineers Australia, 2006)

Typical Soil Types	Saturated Hydraulic Conductivity	
	(m/s)	(mm/hr)
Coarse Sand	$>1\times10^{-4}$	>360
Sand	$5\times10^{-5} - 1\times10^{-4}$	180 - 360
Sandy Loam	$1\times10^{-5} - 5\times10^{-5}$	36 - 180
Sandy Clay	$1\times10^{-6} - 1\times10^{-5}$	3.6 - 36
Medium Clay	$1\times10^{-7} - 1\times10^{-6}$	0.36 - 3.6
Heavy Clay	$< 1\times10^{-7}$	< 0.36

The permeable bioretention systems are recommended to be sited on in situ soils with saturated hydraulic conductivities of higher than 13mm/hr. The used of permeable bioretention system with lower saturated hydraulic conductivity of less than 13mm/hr results into prohibitively large bioretention area. In addition, soils with lower hydraulic conductivities will be more susceptible to clogging and will therefore require enhanced pretreatment.

9.2.1.5 Groundwater

Bioretention systems should be located above the groundwater table to ensure that groundwater never intersects with the bottom of the bioretention system, which prevents possible groundwater contamination and system failure. The minimum vertical distance between seasonal high water table and bottom of bioretention system should be 0.6m. Sites with shallow groundwater are not recommended as infiltration will not work effectively. Similarly, the organic absorption infiltration layer of a bioretention system is beneficial in pollutant removal where bedrock or impermeable layer is shallow.

9.2.2 System Design

9.2.2.1 Pre-treatment Area

Grass buffer strips or vegetated swales are commonly used as pretreatment devices. They are required where a significant amount of debris or suspended material is anticipated, such as from parking lots and commercial areas. Runoff enters the bioretention area as sheet flow after passing through grass buffer strips with reduced velocity and less particulate.

9.2.2.2 Inlet Controls

The flows may enter the system either through subsurface pipe, open channel/swale or as surface sheet flow contributed from an upstream catchment area. Scour protection such as light riprap with 6 to 12mm D50 is recommended to be used at the inlet into the system, to reduce localised flow velocities and to avoid erosion. The flow around the inlet should be non-erosive flow with velocity below 0.5m/s. Level spreader may be located to evenly distribute the incoming runoff onto the basin surface/cells for effective percolation.

9.2.2.3 Basin/Ponding Area

The ponding area provides surface storage of stormwater runoff before it filters through the soil bed. The ponding area also allows for evaporation and settling sediment. The ponding is typically limited to a depth of 150-300mm. A maximum 150mm additional freeboard depth should be provided for online systems to allow surcharge above the overflow outlet during larger storm events (> 3 month ARI). The ponding area is required to drain within 24 hours (MPCA, 2008 and CFWP & MDE, 2000). This is within the general practice of 72 hours maximum drain time of ponding to minimise mosquito breeding and other disease vectors. The 24 hours drain time is chosen based on the following consideration:

- The need to optimise the cost of the bioretention system as a shorter drain time of ponding area will lead to a bigger system.
- The pollutant removal based on 40mm of design storm is considered adequate for water quality control objective.

9.2.2.4 Mulch Layer

The organic mulch layer has several functions. It protects the soil bed from erosion, retains moisture in the plant root zone, provides a medium for biological growth and decomposition of organic matter, and provides some filtration of pollutants mainly larger sediment particles. The mulch layer should consist of 50-100mm depth of commercially-available fine shredded hardwood mulch or shredded hardwood chips.

9.2.2.5 Planting Soil Bed

The planting soil bed provides water and nutrients to support plant life in the bioretention system. Stormwater filters through the planting soil bed where pollutants are removed. The total depth of the planting soil bed should be between 450 to 1000mm.

Planting soils should be sandy loam, loamy sand, or loam texture with clay content ranging from 10 to 25%. The natural soil profile of silt loam with design hydraulic conductivity of 13mm/hr can be used for a planting bed. A much higher design hydraulic conductivity can be obtained with engineered soil mixture of sandy loam. The in-situ base soil is either mixed with loose non-angular sand (to increase saturated hydraulic conductivity) or conversely with loose non-dispersive soft clay (to reduce saturated hydraulic conductivity), to achieve the desired design saturated hydraulic conductivity of required engineered soil media. The engineered soil media should provide an organic matter contents that facilitates good plant root systems; and permeable substrata of around 25% porosity. Designer should strive for composition of organic matter content 15% and clay content of the mixture ≤25%. The recommended composition of engineered soil media for planting bed is given in Table 9.4 (CFWP & MDE, 2000). The pH of the planting soil bed should be 5.5 to 6.5.

Table 9.4: Engineered Soil Media Composition

Soil Mixture	Contents by Volume (%)
Top Soil (Sandy/Silt loam)	20-25
Medium Sand	50-60
Organic Leaf Compost	12-20

The maximum saturated hydraulic conductivity for engineered soil media should not be higher than 200 mm/hr. This is to ensure that the engineered soil media can retain sufficient soil moisture for sustaining vegetation growth. An appropriate saturated hydraulic conductivity is required to optimise the treatment performance of the bioretention system given site constraints and available engineered soil media surface area. The design infiltration rate (f_d) must be one-half the infiltration rate (f_c) found from soil textural analysis ($f_d = 0.5f_c$).

9.2.2.6 Sand Bed in Permeable System

In permeable systems where the underlying native soil has sufficient infiltration capacity to drain the water from the planting soil bed, the sand bed underlies the planting soil bed and allows water to drain into the underlying soil. The sand bed also provides additional filtration and allows for aeration of the planting soil bed. The sand bed should be 200-300mm thick. The sand should be clean and have less than 15% silt or clay content.

9.2.2.7 Drainage Layer in a Impermeable System

In impermeable systems, a drainage layer is used to convey treated flows into the subsoil/underdrain pipes. This layer is generally constructed using coarse sand or fine gravel (2mm to 5mm particle size). The layer should surround the subsoil pipe and is typically 200-400mm thick.

9.2.2.8 Transition Layer in a Impermeable System

A granular transition layer that is typically 100-150mm thick or a suitable geotextile fabric should be included between the planting soil bed and the drainage layer to prevent the filtration media from washing into the drainage layer and the subsoil pipes. This is desirable if the drainage layer is constructed using fine gravel. The material size differential should be approximately 10 between layers to avoid fine material being washed through the voids of a lower layer. The addition of a transition layer increases the overall depth of the bioretention area. This may be an important consideration for some sites and hence pipes with smaller perforations may be preferable. The material for transition layers must be sand/coarse sand material with typical specification given in Table 9.5.

Table 9.5: Typical Particle Size Distribution for Transition Layer
(Moreton Bay Waterways & Catchments Partnership, 2006)

Particle Size (mm)	% Passing
1.4	100
1.0	80
0.7	44
0.5	8.4

9.2.2.9 Plants

Plants are an important component of a bioretention system. They remove pollutants and nutrient through uptake. The plant species selected are designed to replicate a forested ecosystem and to survive stresses such as frequent periods of inundation during runoff events and drying during inter-event periods. The use of native plant species or plants harmonised to the area is recommended (Annex 1).

In addition to providing for treatment of stormwater, bioretention facilities, when properly maintained, can be aesthetically pleasing. More often local regulations frequently require site plans to incorporate a certain percentage of reserved open landscaped area, for incorporation of bioretention facilities. The layout of bioretention facilities can be very flexible, and the selection of plant species can provide for a wide variety of landscape designs. However, it is important that a landscape architect with adequate experience in designing bioretention areas be consulted prior to construction to ensure that the plants selected can tolerate the maturing conditions present in bioretention area.

9.2.2.10 Outlet Controls

The design flow of 3 month ARI from the catchment area will be retained in the ponding area of bioretention system. The ponding water will then infiltrate through the filtration media and for permeable system deep infiltrate will take place within the surrounding soil before it reached the ground water. In the case of impermeable system, the water will be discharged through the primary outlet consists of perforated pipe provided in the drainage layer and will be conveyed to the road reserve and/or by connection to an underground drainage system.

A secondary outlet should be incorporated into the design of a bioretention system to safely convey excess stormwater higher than 3 month ARI. This includes the flow diversion structure or overflow weir to bypass major storm discharge where applicable.

9.2.2.11 Design for Maintenance

The system design should incorporate features to reduce the long term maintenance. This include such as easy accessibility for maintenance. Like any other BMPs, bioretention systems will need regular maintenance and eventual rehabilitation as it degrades over a number of years (Annex 2).

9.2.2.12 Landscaping

Landscaping is critical to the function and appearance of bioretention system. It is preferred that native vegetation is used for landscaping, where possible. Plants should be selected that can withstand the hydrologic regime they will experience (i.e., plants must tolerate both wet and dry conditions). At the edges, which will remain primarily dry, upland species will be the most resilient. In general, it is best to select a combination of trees, shrubs, and herbaceous materials (Annex 1).

9.3 SIZING PROCEDURE

The sizing of a bioretention facility requires a consideration of various factors including:

- Purposes and function of the bioretention system;
- Site requirements for water quality controls;
- Design storm that is required to meet the stormwater management criteria;
- Capabilities of the bioretention system to be used for water quality controls; and
- Use of bioretention system independently of other BMP's, or to be installed along with other devices within a treatment train.

9.3.1 Filter Bed Area

The filtering treatment criteria should be as follows:

- The entire treatment system (including pretreatment) should temporarily hold the Water Quality Volume (WQ_v) derived as runoff from 40mm (i.e., 3 month ARI) design rainfall, prior to infiltration.
- The bioretention system must conform to the accepted specification shown in Figure 9.4 and Figure 9.5. Table 9.6 presents the physical specification and geometry of a bioretention system while Table 9.7 gives the coefficient of permeability (*k*) for various types of filter media.

Table 9.6: Physical Specification and Geometry

Parameter	Specification
Minimum Size	3m wide by 6m long
Length and Width ratio (optional)	2:1
Maximum Emptying Time	≤24 hours
Permeability of Planting Bed	≥13mm/hr
Ponding Depth	150mm – 300mm
Depth to Groundwater Table (below drainage layer)	0.60m (min)

Table 9.7: Coefficient of Permeability (*k*) for Various Types of Filter Media

Media Type	*k* (m/day)
Sand	1.00
Peat	0.60
Leaf Compost	2.65
Bioretention Soil	≥0.312

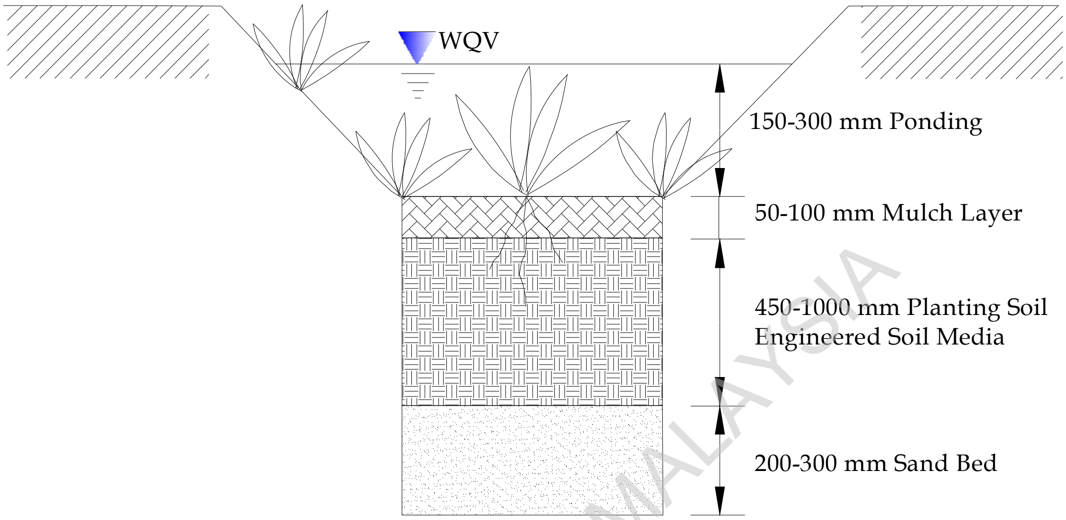


Figure 9.4: Specification for Permeable Bioretention System

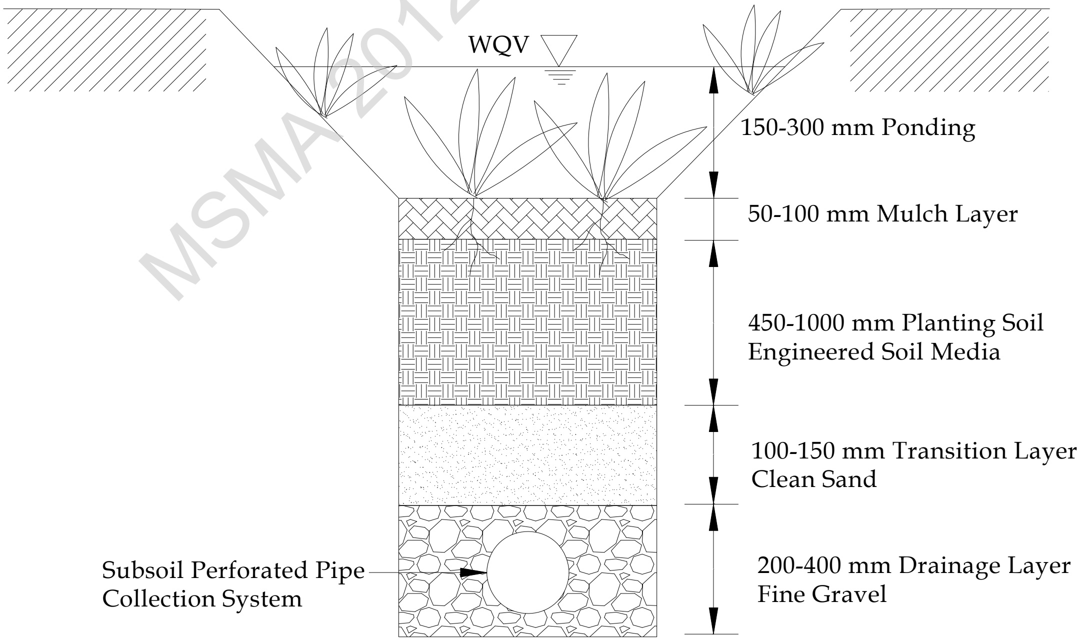


Figure 9.5: Specification for Impermeable Bioretention System

The required filter bed area (A_f) is computed based on the Darcy's Law. For those practices that are designed with an under drain (impermeable system), the following equation is used:

$$A_f = \frac{(WQ_v)(d_f)}{[(k)(h_f+d_f)(t_f)]} \tag{9.1}$$

where,

A_f = Surface area of filter bed (m^2);
WQ_v = Water quality volume (m^3);
d_f = Filter bed depth (m) (Figure 9.6);
k = Coefficient of permeability of filter media (m/day);
h_f = Average height of water above filter bed (m); and
t_f = Design filter bed drain time (day) – (1 day maximum).

For those systems that are designed without an under drain (permeable system), the area is estimated by:

$$A_f = \frac{(WQ_v)(d_f)}{[(i)(h_f+d_f)(t_f)]} \tag{9.2}$$

where,

A_f = Surface area of filter bed (m^2);
WQ_v = Water quality volume (m^3);
d_f = Filter bed depth (m);
i = Infiltration rate of underlying soils (m/day);
h_f = Average height of water above filter bed (m); and
t_f = Design filter bed drain time (day) – (1 day maximum).

9.3.2 Maximum Infiltration Rate

In the case of impermeable bioretention system, the maximum infiltration rate through the filtration media must be considered to allow for the subsoil drain to be sized. The capacity of the subsoil drain, when installed, must exceed the maximum infiltration rate to ensure free draining conditions for the filter media (Figure 9.6). The maximum infiltration rate reaching the perforated pipe at the base of the soil media is estimated applying the equation:

$$Q_{max}=kL_bW_b\frac{h_f+d_f}{d_f} \tag{9.3}$$

where,

Q_{max} = Maximum infiltration rate (m^3/s);
k = Hydraulic conductivity of the filter bed (m/s);
W_b = Base width of the ponded cross section above the filter bed (m);
L_b = Base length of the bioretention zone (m);
h_f = Height of water above the filter bed (m); and
d_f = Depth of filter media (m).

The suitability of the above formula for design purposes will need to be assessed for each individual site, considering the influence of both the annual maximum groundwater level and infiltration capacity of surrounding natural soils on bioretention system infiltration, particularly for permeable bioretention systems.

9.3.3 Subsoil/Underdrain Pipes

Subsoil pipes are perforated pipes placed at the base of impermeable bioretention systems to collect treated water for conveyance downstream. These collection pipes are sized to allow free draining of the filtration layer and prevent 'choking' of the system. Typically, subsoil pipes should be limited to approximately 150mm in diameter so that the thickness of the drainage layer does not become excessive. Where the maximum infiltration rate is greater than the capacity of the pipe consideration should be given to using multiple pipes. To ensure the subsoil pipes are of adequate size:

- Perforations must be adequate to pass the maximum infiltration rate into the pipe;
- The pipe itself must have adequate hydraulic capacity to convey the required design flow; and
- The material in the drainage layer must not be washed into the perforated pipes.

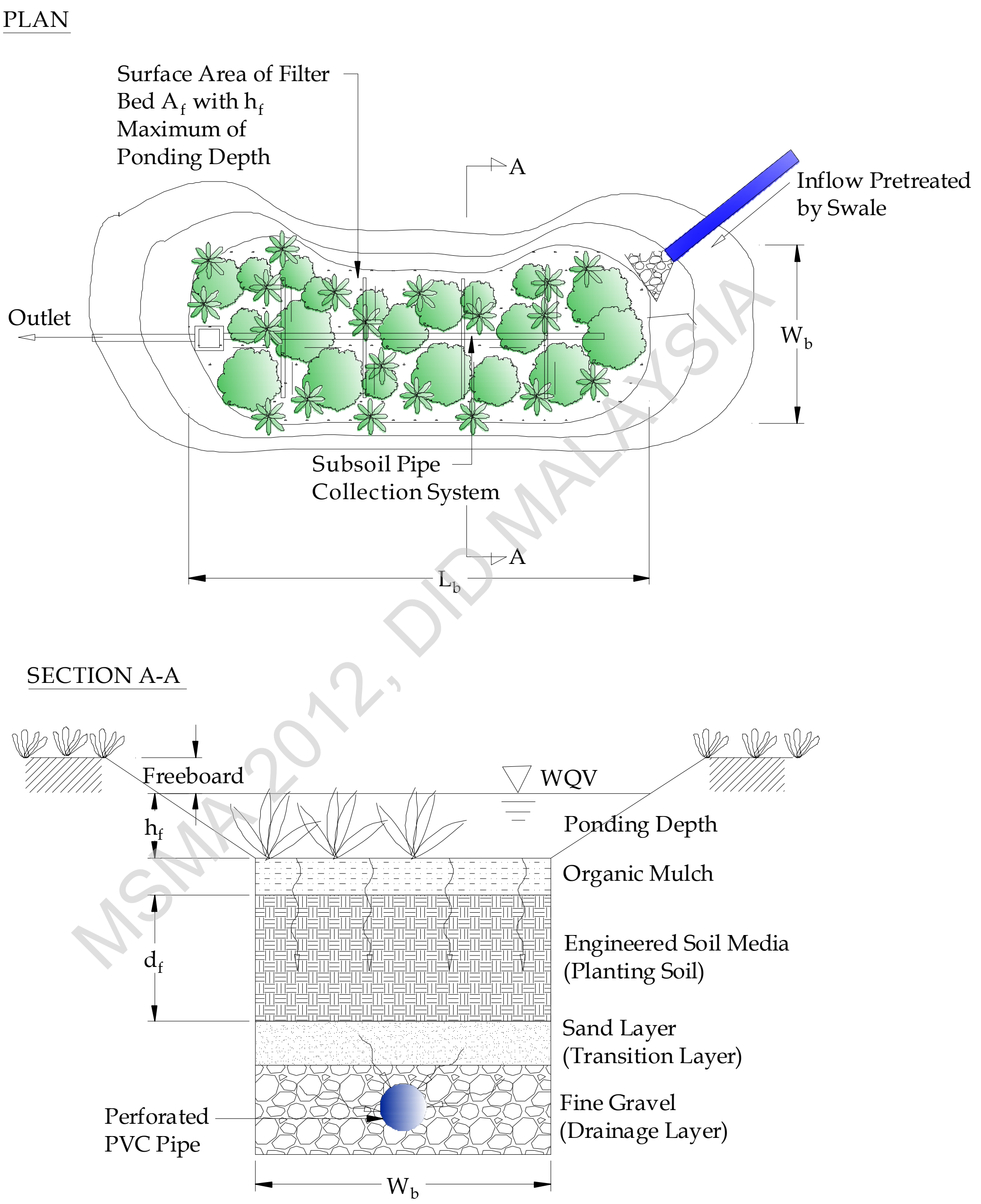


Figure 9.6 Hydraulic Variables for Subsoil Drainage

These requirements can be assessed using the Equation 9.4 and 9.5, or alternatively using manufacturers' design charts. To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp edged orifice equation can be used. The number and size of perforations need to be determined (typically from manufacturers' specifications) and used to estimate the total flow rate into the pipe. It is conservative but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. Flow per perforation is therefore estimated by:

$$Q_{perf}=\frac{C\,.A\sqrt{2gh}}{\Psi} \tag{9.4}$$

where,

Q_{perf} = Flow per perforation (m^3/s);
A = Total area of orifice (m^2);
h = Maximum head of water above the pipe (m) (filtration media depth plus ponding depth);
C = Orifice coefficient;
Ψ = Blockage factor (Ψ = 2 is recommended); and
g = Gravity constant (9.81 m^2/s).

For circular perforated pipes flowing full, using Manning Equation the flow in pipe (Q_{pipe}) is given by:

$$Q_{pipe}=\left(\frac{0.312}{n}\right)D^{2.67}S_o^{0.5} \tag{9.5}$$

where,

n = Manning Coefficient;
D = Pipe diameter (m); and
S_o = Slope of hydraulic grade line (m/m).

The composition of the drainage layer should be considered when selecting the perforated pipe system, as the slot sizes in the pipes may determine a minimum size of drainage layer particle size. Coarser material (e.g. fine gravel) should be used if the slot sizes are large enough that sand will be washed into the slots.

9.3.4 Outlet Structure

The excess flow of the ponding area of bioretention system, which is greater than design flow, will be conveyed to downstream drainage system via overflow pit and subsequently by the road reserve and/or by connection to an underground drainage system. To size a grated overflow pit, two checks should be made for either drowned or free flowing conditions:

- The broad crested weir equation to determine the length of weir required (assuming free conditions); and
- The orifice equation to estimate the area of opening required (assuming drowned outlet conditions).

The larger of the two pit configurations should then be adopted for design purposes. The weir flow equation for free overfall conditions is:

$$Q_{oflow}=C\,.L.H^{3/2} \tag{9.6}$$

where,

Q_{oflow} = Overflow (weir) discharge (m^3/s);
C = 1.7;
H = Head above weir crest (m); and
L = Length of weir crest (m).

Once the length of weir is calculated, a standard sized pit can be selected with a perimeter at least the same length of the required weir length. It is considered likely that standard pit sizes will accommodate flows for most situations. The orifice flow equation for drowned outlet conditions is:

$$Q_{oflow} = C \cdot A\sqrt{2gh} \tag{9.7}$$

where,

Q_{oflow} = Overflow (orifice) discharge (m^3/s);
C = 0.6;
h = Available head above weir crest (m);
A = Orifice area (m^2); and
g = Gravity constant ($9.81m^2/s$).

9.3.5 Scour Velocity of Inflows

It is recommended where possible, the overflow pit or bypass channel should be located near the inflow zone to prevent high flows from passing over the surface of the filter media. If this is not possible, then velocities during the minor storms (2-10 year ARI) and major storms (50-100 year ARI) should be maintained sufficiently low (preferably below values of 0.5m/s and not more than 1.5m/s for major storm) to avoid scouring of the filter media and vegetation.

Scour velocities over the vegetation in the bioretention basin are determined by assuming the system flows at a depth equal to the maximum ponding depth across the full width of the system. By dividing the minor and major storm design flow rates by this cross sectional flow area, an estimate of flow velocity can be made. It is a conservative approach to assume that all flows pass through the bioretention basin (particularly for a major storm), however, this will ensure the integrity of the vegetation.

If the inlet to the bioretention basin controls the maximum inflow to the basin then it is appropriate to use this maximum inflow to check velocities. In this case, velocities should be maintained below 0.5m/s.

9.3.6 Inlet Structure

Erosion protection should be provided for concentrated inflows to a bioretention system. Flows will enter the system from either surface flow system or a piped drainage system. Rock beaching is a simple method for dissipating the energy of concentrated inflow. The use of impact type energy dissipation may be required to prevent scour of the filter media.

This can be achieved with rock protection and by placing several large rocks in the in the flow path to reduce velocities and spread flow as shown in Figure 9.7 (Moreton Bay Waterways & Catchment Partnership, 2006). The detailing of the inlet structure is based on the hydraulic diameter, *DH* which is given as follows:

$$DH = \frac{4A}{P} \tag{9.8}$$

where,

A = Cross-section area; and
P = Wetted Perimeter.

For Trapezoidal channel the hydraulic diameter is

$$DH = \frac{4H(mH+BW)}{BW+2H\sqrt{1+m^2}} \tag{9.9}$$

where,

1:m (V:H) = Side slope;
BW = Bottom width (m) ; and
H = Depth (m).

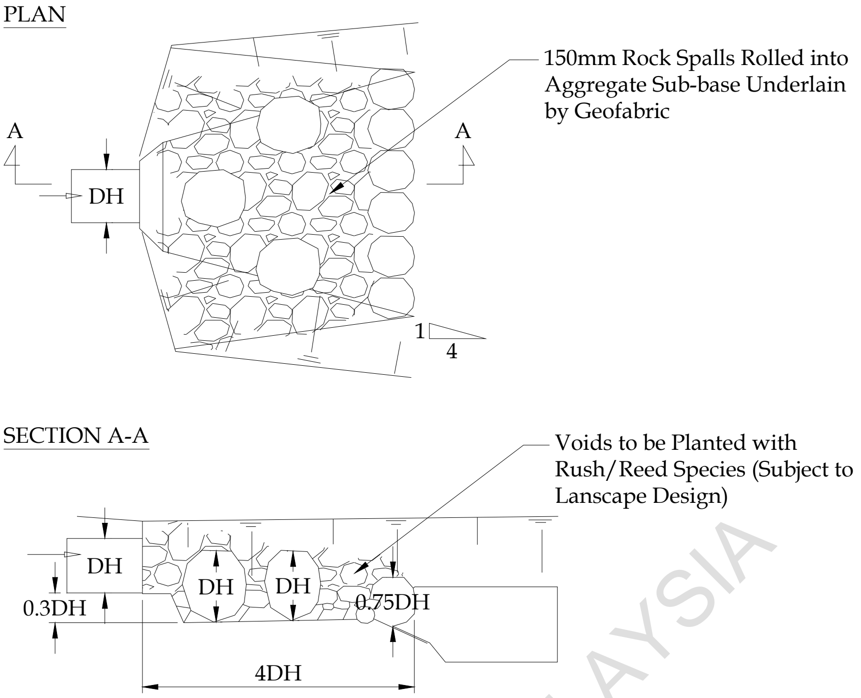


Figure 9.7: Typical Inlet Scour Protection Detail

9.4 DESIGN STEPS

Step 1: Site Evaluation

Make a preliminary judgment as to whether site conditions are appropriate for the use of a bioretention system, and identify the function of the practice in the overall BMPs treatment system. Consider these basic parameters in the site evaluation:

- Drainage area;
- Topography and slopes;
- Soil infiltration capacity (Chapter 8);
- Depth to ground water and bedrock;
- Location/minimum setbacks.

Determine how the bioretention system will fit into the overall stormwater treatment train system and decide whether bioretention is the only BMP to be employed, or if are there other BMPs addressing some of the treatment requirements. Decide on the site where the bioretention system is most likely to be located.

Step 2: Field Verification of Site Suitability

If the initial evaluation indicates that a bioretention system would be a good BMP choice for the site, it is recommended that soil borings or pits be carried out (in the same location as the proposed bioretention system) to verify soil types and infiltration capacity characteristics and to determine the depth to groundwater and bedrock. The number of soil borings should be selected as needed to determine local soil conditions. It is recommended that the minimum depth of the soil borings or pits be 1.5m below the bottom elevation of the proposed bioretention system.

It is recommended that soil profile descriptions be recorded and include the following information for each soil horizon or layer:

- Thickness;
- Feature colour; abundance; size and contrast;
- USDA soil textural class (Chapter 8);
- Soil structure, grade size and shape;
- Soil consistency; root abundance and size;
- Soil boundary; and
- Occurrence of saturated soil; impermeable layers/lenses; ground water; bedrock or disturbed soil.

Step 3: Estimating Design Flow

Calculate the design stormwater volume to be treated through bioretention system. If part of the overall design stormwater volume is to be treated by other BMPs, subtract that portion from the total design volume to determine the part of the design volume to be treated by the bioretention system. The design techniques adopted are meant to maximize the volume of stormwater being infiltrated. If the site layout and underlying soil conditions permit, a design water quality volume derived from 40mm of rainfall depth (i.e. 3 month ARI) may be adopted.

Step 4: Inlet Structure

The flows may enter the system either through subsurface pipe, open channel/swale or as surface sheet flow contributed from an upstream catchment area. Scour protection such as light riprap with 6 to 12mm D50 is recommended to be used at the inlet into the system, to reduce localised inflow velocities. The flow should be non-erosive to the basin with velocity below 0.5m/s.

Bioretention system which receives flows from larger catchment may require an impact type energy dissipator that can be achieved with rock protection and by placing several large rocks in the flow path to reduce the velocities and spread the flows.

Step 5: Select the Bioretention Type

The selection of design variant shall be based on the location of naturally occurring permeable soils, the depth to the water table, bedrock or other impermeable layers, and the contributing drainage area. The bottom of a bioretention system must have a minimum 0.6m height above the local water table. While the initial step in sizing a bioretention system is selecting the type of design variant for the site, the basic design procedures for each type of bioretention system are similar.

Information collected during the field verification of site suitability (Step 2) should be used to explore the potential for multiple bioretention systems application or in integration with other stormwater BMPs in a treatment train.

Step 6: Determine Site Infiltration Rates (Saturated Hydraulic Conductivity)

If the infiltration rate is not measured, select the design infiltration rate from Table 9.3 based on the least permeable soil horizon within the first 1.5m below the bottom elevation of the proposed bioretention system. The infiltration capacity of natural basins is inheritably different from constructed systems.

In the event that a natural depression is proposed to be used as a bioretention system, the designer must assess the followings: the infiltration capacity of the soil under existing conditions (mm/hr), the existing drawdown time for water at the maximum basin water level and the natural overflow elevation. The designer should also demonstrate that the operation of the natural depression under post-development conditions mimics the hydrology of the system under pre-development conditions.

If the infiltration rates are to be measured, the tests shall be conducted at the proposed bottom elevation of the bioretention system. If the infiltration rate is measured with a double-ring infiltrometer the requirements of standard practice should be used for the field test.

Step 7: Determine the Size of Bioretention Area

The size of bioretention area is determined using Darcy's equation (Equation 9.1 for impermeable system and Equation 9.2 for permeable system). The corresponding equation is applied depending on whether the system is impermeable or permeable. The composition of filter media is designed to achieve a minimum coefficient of permeability of 13mm/hr.

Step 8: Sizing of Perforated Collection Pipes

The inlet capacity of the underdrain system (perforated pipe) is estimated by assuming that 50% of the holes are blocked. To estimate the flow rate, an orifice equation is applied using the identified parameters for optimum performance. The Manning equation is applied to estimate the flow rate in the perforated pipe. The capacity of this pipe needs to exceed the maximum infiltration rate.

Step 9: Determine the Size of Outlet Structure

It is required that a secondary outlet or spillway be incorporated into the design of a bioretention system to safely convey excess stormwater. This includes the flow diversion structure to bypass major storm where applicable.

Step 10: Landscaping Design

The performance of the bioretention system is dependent on the selection of plant species. Native plant species should be considered for stormwater treatment and optimum maintenance (Annex 1).

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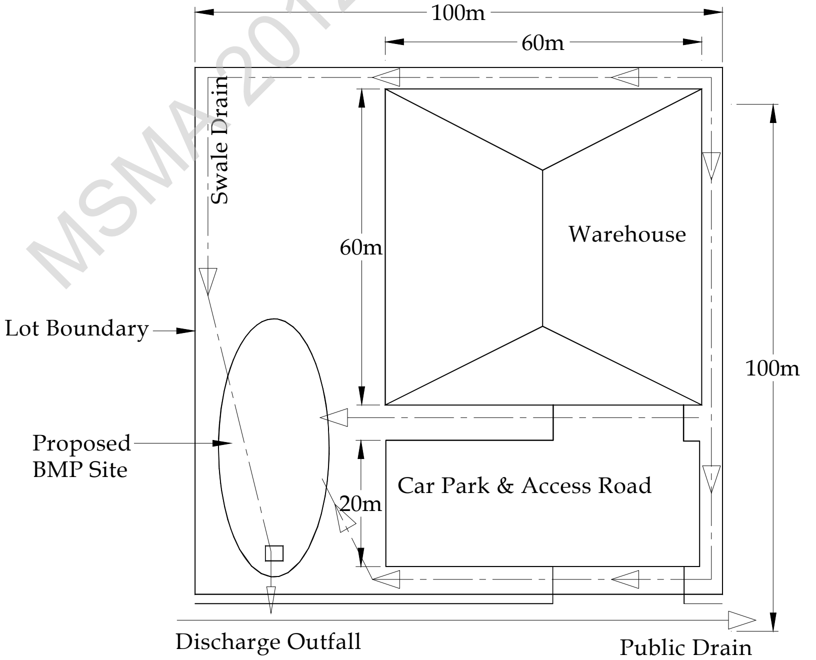
APPENDIX 9.A EXAMPLE - IMPERMEABLE BIORETENTION SYSTEM

Problem:

An industrial area located in Bukit Tengah, Seberang Perai Tengah, Pulau Pinang, has been adopted for the warehouse development with lot area of 1 ha (100m x 100m), floor area 60m x 60m and parking area 60m x 20m. The stormwater runoff from the impervious areas will be directed to the vegetated filter strip around the perimeter of the building and parking areas. The distributed stormwater runoff from vegetated filter strip will be collected in the grassed swale to be conveyed downstream of the lot area and treated by a bioretention facility of impermeable type. For such runoff quality treatment an appropriate sizing of the facility is required.

a) Site Plan



b) Warehouse and Drainage Layout

Figure 9.A1: Development Site

Solution:

Reference	Calculation	Output
	Design Rainfall Duration	
Figure 9.A1	Development Project Area	
	Catchment area, A =	1.0ha
	The time of flow in the channel (t_d) is derived based on the velocity (V) and length (L) of flow in the swale.	
	Given that:	
	Velocity, V =	0.25m/s
	Length, L =	175m
	Then, $t_d = L/V = 175/0.25$ =	11.7min
	=	12min
	Assume the overland flow time (t_o) is 5 minutes	
	Then, the time of concentration, $t_c = t_o + t_d = 5 + 12$ =	17min
	Design Rainfall Intensity	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d + \theta)^{\eta}}$	
	Where,	
	i = Rainfall Intensity (mm/hr) T = Average Recurrence Interval (year) d = Storm Duration (hr)	
	λ, K, θ, η = Coefficients	
	Given that,	
	Average Recurrence Interval, T =	5 yr
	Storm Duration, D =	17min
Table 2.B2	$\lambda = 52.771$, $\kappa = 0.203$, $\theta = 0.095$ & $\eta = 0.717$	
	Then,	
Equation 2.2	$i = \frac{52.771 \times 5^{0.203}}{(0.283+0.095)^{0.717}}$ =	146.88mm/hr
	Water Quality Volume	
	The water quality volume is estimated based on 40 mm of 3 month ARI rainfall.	
Table 2.6	Permeable area: Open spaces grass cover with runoff coefficient value, C =	0.40
Table 2.6	Impermeable area: Impervious roofs, concrete and roads with runoff coefficient value, C =	0.95

Landuse	C	P (mm)	A (m^2)	WQ_v (m^3)
Permeable	0.40	40	5200	83.2
Impermeable	0.95	40	4800	182.4

Reference	Calculation		Output
	The water quality volume, WQv	=	$265.6m^3$
	<u>Soil Mixture of Planting Bed</u>		
	Specification of Planting Bed:		
	Depth	=	750mm
	Porosity	=	25%
	Maximum Clay Content	=	5%
	Coefficient of Permeability, k	=	26mm/hr
		=	0.624m/day
	Recommended composition of planting bed is specified in Table 9.4.		
	<u>Surface Area of Filter Bed</u>		
	Given that:		
	Water Quality Volume, WQ_v	=	$265.6m^3$
	Planting Bed Depth, d_f	=	0.75m
	Saturated Hydraulic Conductivity, k = 0.5×0.624m/day	=	0.312m/day
	Average height of Water above Filter Bed, h_f	=	0.3m
	Design Filter Bed Drain Time, t	=	1.0day
	The surface area of filter bed for impermeable system is given as:		
Equation 9.1	$A_f = \frac{(265.6)(0.75)}{(0.312)(0.3+0.75)(1.0)}$	=	$608.1m^2$
	<u>Maximum Infiltration Rate</u>		
	Given that,		
	Saturated Hydraulic Conductivity, k	=	$7.222\times10^{-6}m/s$
	Base Width of the Ponded Cross Section above the Filter Bed, W_b	=	15.00m
	Base Length of the Bioretention Zone, L_b	=	41.00m
	Average height of water above the filter bed, h_f	=	0.30m
	Depth of Planting Bed, d_f	=	0.75m
	Therefore, the maximum infiltration rate reaching the perforated pipe at the base of the soil media is given as:		
Equation 9.3	$Q_{max} = (7.222 \times 10^{-6})41 \times 15\left(\frac{0.30 + 0.75}{0.75}\right)$	=	$6.22 \times 10^{-3}m^3/s$

Reference	Calculation	Output
	Sub-Soil Pipe	
	The impermeable bioretention system employs a UPVC perforated sub-soil pipe to convey the stormwater runoff to the downstream conveyance.	
	Given that,	
	Diameter of Perforated Pipe =	100mm
	Diameter of Perforations =	5mm
	Spacing of Perforations =	25mm c/c
	Circumference of Perforated Pipe =	314.2mm
	Number of Rows (of Perforations) =314.2/25 =	12.57
	Number of Column (of Perforation/m) =1000/25 =	40.00
	Number of Perforations = 12.57 × 40.00 =	502.72
	Number of Perforations (assume 50% blockage) = 502.72/2 =	251.36
	Area of each Perforation =	$19.63 mm^2$
	And so,	
	Total Area of the Orifices, *A* = $19.63 \times 251.36 \times 10^{-6}$ =	$0.0049 m^2$
	Maximum Head of Water above the Pipe (m) (filtration media depth plus ponding depth), *h* =	1.4m
	Orifice Coefficient, *C* =	0.60
	Blockage Factor (Ψ = 2 is recommended), Ψ =	2.00
	Gravity Constant, *g* =	$9.81 m^2/s$
	Note that, the maximum head of water above the pipe (*h*) is given as: 300 mm (ponding depth) + 100 mm (mulch layer) + 750 mm (planting bed) + 100 mm (transition layer) + 100 mm (part of drainage layer above the pipe) + 50 mm (radius of perforated pipe) = 1400 mm	
	The flow per perforation is given as:	
Equation 9.4	$Q_{perf}=\frac{0.60 \times 19.63 \times 251.36 \times 10^{-6} \sqrt{2 \times 9.81 \times 1.4}}{2.0}$ =	$0.0078 m^3/s/m$
	Inlet capacity per meter × total length: 0.0078 × 41 = 0.318 m^3/s > 0.0062 m^3/s (maximum infiltration). Hence, a single pipe of 100 mm diameter is sufficient to pass the flow into the perforation.	
	The Manning equation can then be applied to estimate the flow rate in the perforated pipe.	
	Given that,	
Table 2.3	Manning Roughness, *n* =	0.01
	Diameter of the Perforated Pipe, *D* =	0.1m
	Slope HGL, S_o =	0.01m/m
	The flow rate in the perforated pipe is given as:	
Equation 9.5	$Q_{pipe}=\left(\frac{0.312}{0.01}\right)0.1^{2.67}0.01^{0.5}$ =	$0.0067 m^3/s$

Reference	Calculation	Output

This is higher than the maximum infiltration capacity; hence the single 100 mm diameter perforated pipe is adequate for the bioretention system.

Estimation of Peak Discharge

The computation of peak flow for 5 year ARI is tabulated below.

Landuse	C	I (mm/hr)	A (m^2)	Q (l/s)
Permeable	0.40	146.88	5200	84.9
Impermeable	0.95	146.88	4800	186.1

The peak flow for 5 year ARI contributed from the permeable and impermeable areas is estimated to be 271 l/s.

Size of Overflow Sump

Sizing an overflow structure to convey stormwater in the basin in excess of the 300 mm ponding depth from the bioretention system to the downstream conveyance.

The broad crested weir equation (Equation 9.6) is initially used to determine the length of weir required by assuming free overfall conditions.

Given that,

Overflow (weir) discharge, Q_5 = $0.271 m^3/s$
Flow Coefficient, C = 1.7
Head above weir crest, H = 0.15m

The length of weir crest is computed as,

Equation 9.6

$$L = \frac{0.271}{1.7 \times 0.15^{1.5}} = 2.74m$$

The required length of weir crest, L is equal to 2.74 m, which can be provided by 785 mm × 785 mm square overflow sump.

Secondly, checking for drowned outlet conditions using orifice equation (Equation 9.7):

Given that,

Overflow (Orifice) Discharge, Q_5 = $0.271 m^3/s$
Flow Coefficient, C = 0.6
Head above Weir Crest, h = 0.15m
Gravity Constant, g = $9.81 m^2/s$

Reference	Calculation	Output
	The area of orifice is given as,	
Equation 9.7	$A = \dfrac{0.271}{0.6 \times \sqrt{2 \times 9.81 \times 0.15}}$ =	$0.263m^2$
	The discharge area required is *A* = 0.263 m², which can be provided by 515 mm × 515 mm overflow sump. Hence, the free overfall conditions dominate the overflow design. A pit size of 785 × 785 mm is adopted. The size of the overflow sump needs also to consider an allowance for the grate bars, which will reduce the available perimeter and area.	
	Details of the Inlet Structure	
	Used the typical trapezoidal swale for design flow 5 year ARI as follows:	
	Bottom Width, *BW* =	500mm
	Depth of the Swale, *H* =	300mm
	Side Slope of 1:3 (*V*:*H*), *m* =	3
	Then,	
	Cross-section Area, *A* =	$0.42m^2$
	Wetted Perimeter, *P* =	2.40m
	Hence,	
Equation 9.9	Hydraulic Diameter, *DH* =	700mm

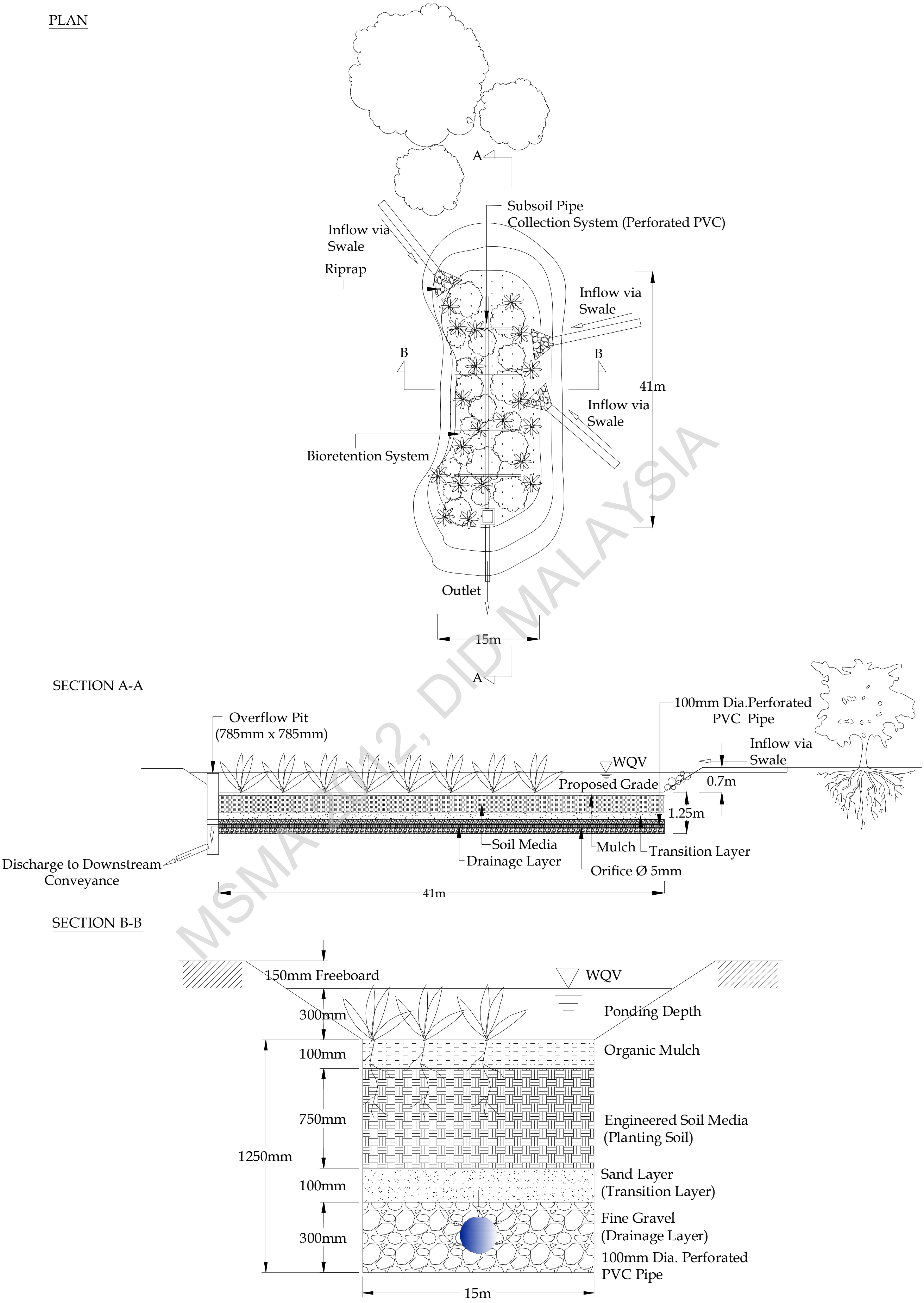
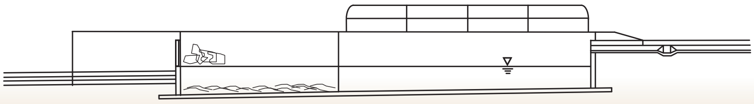


Figure 9.A2: Design Details of the Bioretention System



GROSS POLLUTANT TRAPS

10

CHAPTER 10 GROSS POLLUTANT TRAPS

Comment [G1]: GENERAL COMMENTS: This chapter is based on old methods, the most recent reference being the 2000 MSMA. Design equations are the same as those in the 2000 MSMA. Generally the materials are based on MSMA2000. The materials are proven from Australian experience.

I cannot be enthusiastic about this chapter. It seems to be a rehash of the 2000 version, with some new material on oil and grease separators, which I find to be very questionable.
O&G is the suggestion and requirement to be included by DID

GPTs of the type defined here are no longer built in Australia, although older installations are still operating.
It is still being practice locally.

Is there any experience of the operation of these devices in Malaysia? If so, conclusions and design requirements can be drawn from this. I see no evidence of this here.
The application of GPT in Malaysia is very new hence there is very limited evidence.

10.1 INTRODUCTION

This Chapter presents design procedures for gross pollutant traps (GPTs) which are installed to remove litter, debris, coarse sediment and hydrocarbon from stormwater. They may be used as pre-treatment BMPs for flow into a pond or wetlands or to remove coarse sediment before the flow enters infiltration devices. Most GPTs will provide some reduction in other associated pollutants that are attached to the trapped sediments for subsequent removal.

Comment [G2]: A large part of the material collected may be leaves and vegetative material.

10.2 TYPES AND APPLICATION

There are a wide range of devices used for the trapping of gross solids. Selection of suitable devices depends on many factors including catchment size, incoming pollutant load, type of drainage system and whole life cost. Table 10.1 provides three main types of GPTs for practice, each with its function, suitable catchment areas and installation option. Figure 10.1 presents typical configuration of built GPTs Type 1, 2 and 3 assets.

Table 10.1: GPTs Types and Application

Class		Function	Catchment Area	Installation
Type 1	- Floating Debris Traps	Litter capture on permanent waterbodies	> 200 ha	Proprietary and purposely built
	- Trash Racks & Litter Control Devices	Litter capture on drainage conveyance	2 – 40 ha	Purposely built from modular components
Type 2	- Sediment Basin and Trash Rack (SBTR) Traps	Sediment and litter capture on drainage conveyance	5 – 200 ha	Purposely built
Type 3	- Oil and Grease Interceptor	Oil, grease and sediment capture on drainage conveyance	< 40 ha	Purposely built and Proprietary

a) Floating Debris Trap - Type 1 b) Trash Rack - Type 1

c) SBTR Traps - Type 2 d) Oil and Grease Interceptor – Type 3

Figure 10.1: Typical GPTs Types

10.2.1 Floating Debris Traps

Floating debris traps or booms should be a main component of GPTs Type 1. They are installed primarily on streams, slow moving main drainage and pond inlets where there is permanent water body. Booms are only effective as a control measure for floatable pollutants under certain conditions. The requirements for a suitable site include the followings (Willing & Partners, 1989):

- Favourable currents;
- Location relative to major sources, such as tributary stormwater drains;
- Availability of access for maintenance;
- Ability to handle the effects of water level changes; and
- Suitable locations for attachment and anchorage.

Booms are generally not effective unless there is a steady current to force trapped material into the boom. Tidal flow reversals or strong adverse winds may disperse the trapped material, rendering the boom ineffective. They are also not effective when the current velocities are high and turbulent.

Installation of the boom will mainly be governed by site conditions. Sufficient slack must be provided to allow the boom level to rise and fall with water level variations, such in tidal and flooding conditions.

The pollutant materials collected in urban areas are potentially offensive, hazardous or infectious wastes including discarded syringes which necessitate arrangements for mechanical cleaning. Decision to install a boom or a trash rack is governed by a number of factors including (Nielsen and Carleton, 1989):

- Trash types - Booms were found to be effective in retaining floating and partially submerged objects; and
- Hydraulic - The trash retaining performance of booms decreases at higher flows because trash is forced under and over them. The minimum flow velocity at which trash escapes by being forced underneath a boom depends largely on the shape and weight of the boom and has been observed to be as low as 1m/s.

10.2.2 Trash Racks and Litter Control Devices

Trash racks is another component of GPTs Type 1. Trash racks range from relatively small screens installed at the outlets of stormwater pipes to large steel trash racks on main drains and open channels and more recently "soft" trash racks (litter control devices) that are installed in open channels and at the outlets of piped drains. The consideration of implementing either a boom or trash rack depends on the characteristic of the floating debris and the site suitability.

Comment [G3]: Is this right?

The preferred trash rack arrangement, suggested by a number of researchers, is that a trash rack with horizontal bars set at an angle to the flow should be self-cleansing, since the flow would push debris towards the sides of the rack. The effectiveness of such an approach would appear to depend on the shape and surface finish of the bars and their angle relative to the flow.

The litter control devices collect litter, as do trash racks. This "soft" trash rack are a series of nylon mesh "socks" which are attached to a rectangular metal frame that is mounted vertically and perpendicular to the flow. The socks are cleaned by removing each sock in turn, undoing the tie at the base of the sock and dumping the collected material into a truck. The base of the sock is then re-tied and it is slotted back into place. Due to the effectiveness of the socks it has been found that during periods of rainfall the soft trash racks may need to be cleaned every two to three days. These types of devices require a high maintenance and on-going cost. If not maintained, the upstream storm flow capacity is greatly compromised.

Comment [G4]: These devices are out of favour in Australia.

10.2.3 Sedimentation Basin and Trash Rack

Sedimentation basin functions by providing an enlarged waterway area and/or reduced hydraulic gradient to reduce flow velocities and allow bedload sediment to be trapped and suspended sediments to settle out of suspension. They do not provide litter removal. In urban areas, the presence of litter makes it preferable to build a Sedimentation Basin and Trash Rack (SBTR) type GPTs.

Comment [G5]: They do capture some litter, but it has to be removed by hand. The same comment applies to bioretention basins.

SBTR traps, GPTs Type 2, combine the functions of a sedimentation basin and a fixed trash rack. The trap is a major basin designed to intercept litter, debris and coarse sediment during storm flows and to act as an efficient retarding basin. This trap draws on the experience of sedimentation basins with incorporation of additional features to intercept trash and debris.

Major SBTR traps are typically located in trunk drainage channels and engineered waterways to intercept medium to high stormwater flows from large urban catchments. They are visually unattractive and generally should be placed away from residential areas. Indicative arrangement of major SBTR traps is shown in Figure 10.2.

Minor SBTR traps, covered in-ground, are used at the downstream end of pipe or open drains. They are less visually intrusive and hence are more suitable for residential areas. Due to the cost of the structure they are usually smaller in size and are only suitable for treating small catchment areas. Typical arrangement of minor SBTR traps is illustrated in Figure 10.3.

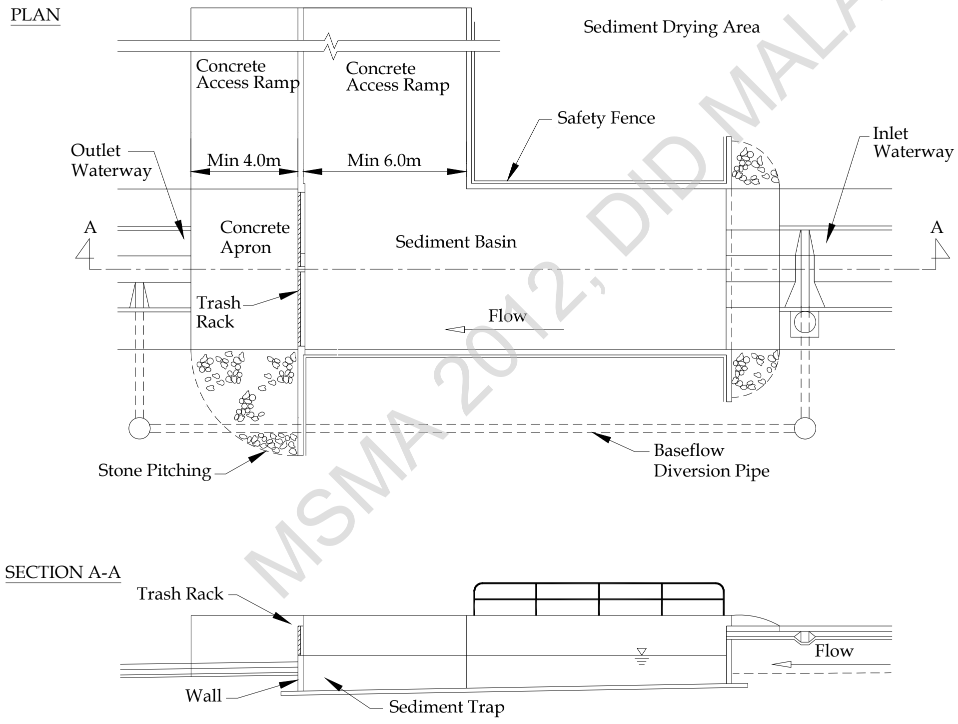


Figure 10.2: A Typical Major SBTR Trap Arrangement

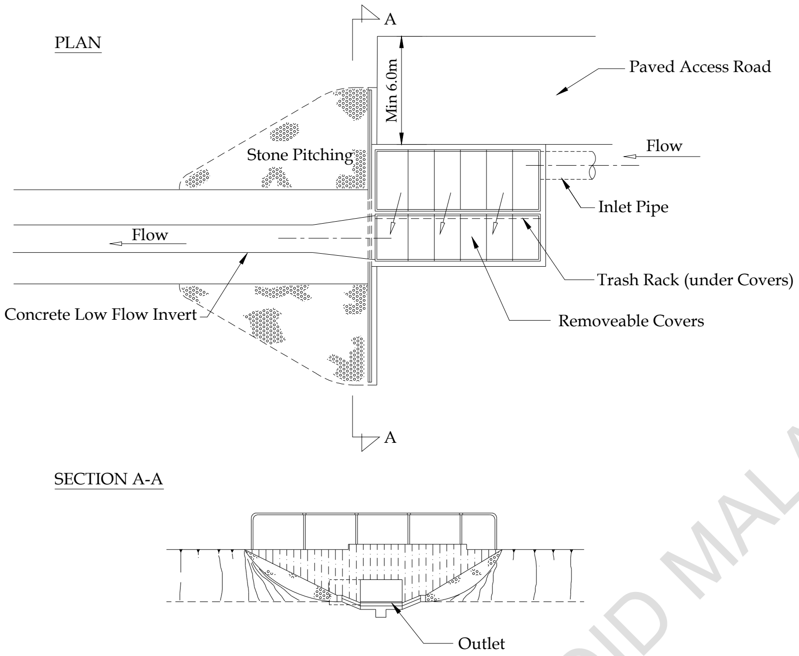


Figure 10.3 : A Typical Minor SBTR Trap Arrangement

10.2.4 Oil and Grease Interceptor

Oil and grease interceptor (OGI), GPTs Type 3, is intended to separate out oil and other hydrocarbons, grit, and coarse sediments from runoff. They are often used to pretreat runoff from parking lots and other impervious surfaces before it enters stormwater system. Because of the short runoff retention time, they are effective only in reducing the amounts of coarse-grained sediments, debris, oil and grease present in runoff. They must be cleaned out frequently, as needed, to remove collected grit and pollutants for disposal.

Schematic diagrams of typical three-chamber OGI are shown in Figure 10.4. Runoff enters the first chamber through a storm drain pipe or curb inlet, passes through screened orifice opening/baffle to the second chamber, then passes through another baffle or an inverted pipe elbow to the third chamber. Flow exits the third chamber through a storm drain outlet pipe.

The first chamber is used to trap floatable debris and settle coarse particles such as sediment. The second chamber is used to separate out oil, grease and other hydrocarbons that float to the water surface while the runoff is withdrawn from the opening of the baffle or elbow. The floating hydrocarbons remain in this chamber until removed at clean-out or adsorbed by sediment and settled out. The third chamber will have a permanent pool if the outlet pipe is elevated above the floor. Additional sediment may settle out in this chamber. An overflow opening is included in each chamber for large stormwater flows to bypass the orifice openings between chambers. Clean-out access is available through manhole openings.

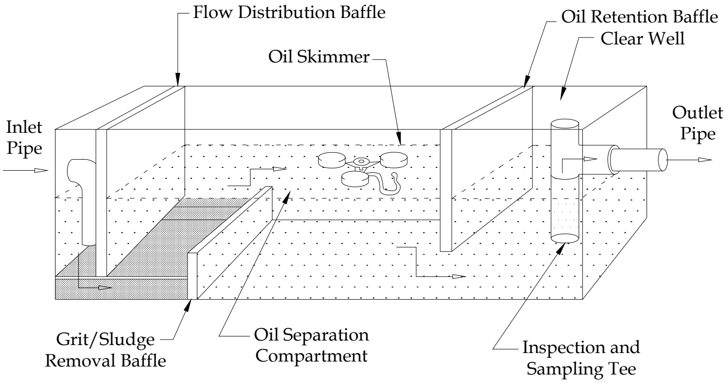
Comment [G6]: This use of separators in stormwater drainage systems is very strange to me. I am unaware of any large oil and grease separators being used as part of public stormwater systems in Australia, or anywhere else.

Separators are used in commercial and industrial properties such as fast food shops, petrol stations and manufacturing plants. They are relatively small and are usually placed underground. The property owner is responsible for their operation and maintenance.

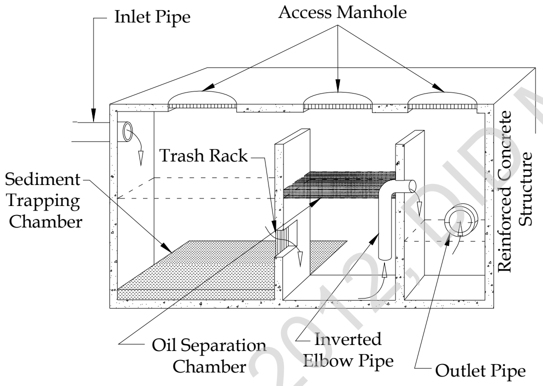
Here, you are saying that they can be installed in catchments up to 40 ha.

If oil and grease is a problem in a catchment of this size, the solutions are prosecution and fining of emitters of these pollutants, and requirements for source controls such as separators in individual properties.

Oil and grease can be handled in GPTs by use of baffles, screens and scum boards, but effective removal is unlikely.



a) Conventional Gravity Separator (CDM, 1993)



b) Water Quality Inlet (Dewberry & Davis, 1966)

Figure 10.4: Typical Oil and Grease Interceptors Arrangement

OGIs are applicable to situations where the concentration of oil and grease related compounds will be abnormally high and source control cannot provide effective removal. The general types of activity where this situation is likely are truck, car and equipment maintenance and washing business, as well as a business that performs maintenance on its own equipment and vehicles. Public facilities where OGIs may be required include marine ports, airfields, fleet vehicle maintenance and washing facilities and mass transit park-and ride lots. Conventional OGIs are capable of removing oil droplets with diameters equal to or greater than 150 microns.

10.3 GENERAL DESIGN CONSIDERATIONS

Design of GPTs requires information on the hydrology and hydraulics of the drainage system. For each GPT, considered as part of a "treatment train", a primary treatment objective or performance criteria related to a specific pollutant shall be defined. This is the target pollutant that is to be reduced to a recommended level.

10.3.1 Hydrology and Hydraulics

Peak inflows shall be computed using the Rational Method or Rational Hydrograph Method. Normally these calculations will be done as part of the hydraulic design of the drainage system. More important for GPT design is the magnitude of sediment and other pollutant loads that will determine the frequency of cleaning.

The pollutant reduction performance must be maintained up to the design discharge. If design flows are exceeded, the GPT should not allow any significant re-mobilisation of trapped material. The GPT must be designed so as to prevent any additional surcharge in the stormwater system in the event of partial or complete blockage. Tidal influence and backwater effects must be considered.

10.3.2 Ease of Maintenance

Problems with maintenance can be partly overcome by appropriate design. Considerations should be addressed during the preliminary design stage of a GPT. This include adequate provision of road access to the site for maintenance vehicles and equipments. Suitable walkways, ladders, manholes and plinths shall be provided within the structure for access.

10.3.4 Health and Safety

Open GPTs can present a safety hazard from the followings:

- Raised structures that children can fall off and sudden drops into deep water; and
- Sudden changes in flow velocities or water levels.

Therefore GPTs should be fully fenced off, if possible. Such fencing should be designed so that it does not interfere with the hydraulics of the flow structure.

Provision shall be made to minimise mosquito hazard as follows:

- Keeping the sediment trap wet with a low or trickle flow; and or
- Using biodegradable slow release larvicides (note: full environmental impact assessment of the larvicide would be needed prior to the adoption of this alternative).

10.4 DESIGN OF SEDIMENT BASIN AND TRASH RACK

The major SBTR traps are designed as open traps on large open drains or engineered waterways where they are installed at or below ground level and serve the catchment area in the range of 5 to 200 ha. Whereas, the minor SBTR trap is enclosed and installed below ground with contributing catchment area of less than 5 ha. SBTR traps permit coarse sediment to settle to the bottom by decreasing the stormwater flow velocity through increased width and/or depth of the drain.

The trash rack is intended to collect floating and submerged debris. Experience has shown that it should be located at the downstream end of the sediment basin compartment.

10.4.1 Design Standard

The SBTR type GPTs should be designed to retain all sediments, litter and debris based on the water quality design storm of 3 month ARI that has been described in Chapter 3 and to comply with the size requirements of Design Chart 10.A1 in Appendix 10.A.

Comment [G7]: How is this to be calculated? Chapter 2 does not provide any guidance on this. I think that you mean to use 40 mm depth.

Traps designed according to these criteria are expected to remove, on an annual average basis, 70% of the sediment with a grain size ≥ 0.04mm. This sizing criterion may not be attainable in the case of very fine-grained soils (silts and clays).

10.4.2 Design Parameters

The SBTR trap relies on reducing the flow velocity sufficiently to allow settling by gravity. These principles apply to both major SBTR and minor SBTR traps. The design parameters are given below:

- The ratio of length: width of the sediment basin should be between 2 and 3;
- Velocity through the sediment basin should not exceed 1.0m/s, to minimise re-suspension;
- For a sediment trap volume greater than 5m^3, a sediment drying area with a minimum area equal to 1.5m^2 for each m^3 of trap volume shall be provided, where sediment may be dried prior to transportation. The drying area shall be surfaced with 300mm of compacted gravel or other approved surfacing;
- Trash rack shall have maximum bar spacing 50mm to retain a small plastic bottle or an aluminium drink can;
- Trash racks shall be sized to operate effectively whilst passing the design flow without overtopping and with 50% blockage;
- Trash racks shall be structurally stable when overtopped by flood events up to the major design storm when fully blocked. Trash racks and their supporting structures shall be designed to withstand impact of a large floating object together with its drag loads or debris loads (100% blocked);
- The design must allow water to flow past or over the trash rack when the trash rack is blocked; and
- Vehicular access must be provided for maintenance.

10.4.3 Sizing Steps

The efficiency of the trap will vary with soil type and adjustment factors for different soils are given in Design Chart 10.A2 in Appendix 10.A The chart shows typical soil gradations and the relevant adjustment factors F_1 and F_2.

Sizing for SBTR trap involves the steps set out below:

Step 1: Determine the required removal efficiency of coarse sediment ≥ 0.04mm diameter, $P_{0.04*}$.

Step 2: Determine the catchment area A_c (m^2) served by the sediment trap and the applicable degree of urbanisation [U] within that catchment. Allow for future catchment development, if appropriate.

Step 3: Select an initial trial trap area ratio R:

$$R = \frac{A_t}{A_c} \tag{10.1}$$

where,

A_t = Area of trap (m^2); and
A_c = Area of catchment (m^2).

Step 4: Find $P_{0.04}$ for the reference soil and degree of urbanisation [U] from the appropriate Design Chart 10.A1 and Factor F_1 from Design Chart 10.A2. Calculate the actual trap removal efficiency for the site soil:

$$P_{0.04*} = P_{0.04} \times F_1 \tag{10.2}$$

where,

F_1= Factor from Design Chart 10.A2.

Adjust R if necessary by trial and error to obtain the required performance.

Step 5: Select the length L_t (m) and width W_t (m) of the sediment trap to give the required area A_t such that the length to width ratio is between 2 and 3, and the width is not less than 2m.

Step 6: Determine the average annual TSS load L (tonnes) using Equation 3.2 then estimate sediment load, M (tonnes) with grain size ≥ 0.01mm.

Step 7: Determine the average annual percentage retention $P_{0.01}$ of sediment ≥ 0.01mm for the reference soil from the applicable Curve B in Design Chart 10.A1 for the selected trap area ratio (A_t /A_c). Then determine the adjusted average annual percentage retention $P_{0.01^*}$ of sediment ≥ 0.01mm from the equation:

$$P_{0.01^*} = P_{0.01} \times F_2 \qquad (10.3)$$

where,

F_2 = Factor from Design Chart 10.A2.

Step 8: The required sediment trap volume is a function of the average frequency of cleaning. Assuming that the trap is cleaned two times per year and that it is half full when cleaned, the required depth D_w is given by:

$$D_w = 0.0065 \times P_{0.01^*} \times M / A_t \qquad (10.4)$$

where,

D_w = Depth of the sediment trap below trash rack (m); and

M = Annual sediment load (tonnes).

This relationship is based on a sediment density of 2.65 tonnes/m^3 and a sediment porosity of 0.42.

Step 9: Determine the design flow in the water quality design storm of 3 months ARI, $Q_{0.25}$.

Step 10: Determine the trash rack height, based on the rack not being overtopped in the water quality design storm when the rack is 50% blocked.

The presence of a downstream hydraulic control can lead to the downstream submergence of the trash rack and an increase in the pool level upstream of the trash rack. Under these conditions the trash rack height should be sized by a hydraulic analysis of the site and the trash rack. The sizing method for a standard vertical-bar trash rack is presented herein (Willing & Partners, 1992).

Under unsubmerged conditions, the required height of the trash rack [H_r] is twice the depth at critical flow [y_c] through the unblocked trash rack.

$$H_r = 2y_c$$

$$= 2\left(\frac{Q_{0.25}{}^2}{g \cdot L_e{}^2}\right)^{1/3} \qquad (10.5)$$

where,

H_r = Required height of trash rack (m);

$Q_{0.25}$= The design flow (m^3/s), of 3 month ARI;

g = Gravitational acceleration, 9.8m/s^2; and

L_e = The effective length of flow through an unblocked trash rack (m).

Using a standard design of vertical 10mm galvanised flat steel bars at 60 mm centres and a coefficient [C_c] of 0.8 to account for contraction of flow through the trash rack, gives:

$$H_r = 1.22\left(\frac{Q_{0.25}}{L_r}\right)^{2/3} \tag{10.6}$$

where,

L_r = Actual length of the trash rack (m)

Adjust the sediment trap dimensions to ensure that the velocity through the sediment trap when it is full does not exceed 1.0m/s in the water quality design storm, to minimise the re-entrainment of deposited sediment.

Determine the nominal design flow velocity $V_{0.25}$ in the water quality design storm using,

$$V_{0.25} = \frac{Q_{0.25}}{(D_w + H_r)W_t} \tag{10.7}$$

in which W_t is the width of the sediment trap, normal to the direction of flow. Increase the dimensions of the sediment trap pool or increase the track rack height if the resulting velocity is greater than 1.0m/s.

Step 11: An additional step is necessary for covered (minor SBTR traps) to minimise the potential for upstream surcharge. Provide a minimum overflow clearance above the trash rack that is sufficient to discharge the flow of the inlet pipe even if the trash rack is fully blocked. The required clearance, B, is given by Equation 10.8 must be a minimum of 0.35m.

$$B = \left(\frac{Q_p}{1.7\,L_r}\right)^{2/3} \tag{10.8}$$

where,

L_r = Length of trash rack (m); and

Q_p = Inlet pipe capacity (m^3/s).

Where possible a step shall be incorporated at the outlet of the SBTR trap to minimise submergence effects at any trash rack provided. The step should be determined using hydraulic principles but should desirably be 80 mm or greater.

An energy dissipation device shall be provided at the inlet to the SBTR trap where the velocity of the inflow stream under design flow conditions exceeds 2m/s. Excessive inlet velocities and turbulence will inhibit sedimentation action in the trap.

10.4.4 Special Design Considerations

(a) Major SBTR Traps

The longitudinal axis of the trap should be as close as possible to the centreline of the incoming drain or engineered waterway. Eliminate unnecessary angles in the flow, thus, it is proposed to have a long, straight basin.

For major SBTR traps as in Figure 10.2, a base flow bypass shall be provided around the sediment trap to divert low flows during cleaning. The bypass shall operate under gravity and shall have a minimum diameter of 300 mm to prevent blockage. The design parameters are given below:

- The floor of the sediment trap shall be graded to a dewatering sump located at the side of the sediment trap but clear of vehicle or equipment paths;
- Side walls shall be provided to reduce scour of the surrounding banks when the trash rack is overtopped. The minimum level of the top of the side walls shall be the greater of: (i) the level of the 3 month ARI flow when the trash rack is fully blocked, or (ii) 300mm higher than the top of the trash rack;

Comment [G8]: Calculated how?

- Provision shall be made for a plinth or access walkway 800mm wide immediately upstream of the trash rack to allow access for cleaning or raking of collected material from the trash rack;
- Reduce the effect of wind-induced turbulence. Large open water surfaces are affected by wind, which produces cross-and counter currents that hinder settling and may resuspend bottom deposits; and
- Suitable landscaped screening should be considered.

(b) Minor SBTR Traps

For minor SBTR traps in Figure 10.3, the design parameters are given below:

- Pipe entries shall, where possible, be either parallel (preferred) or perpendicular to the major axis of the sediment trap;
- Low-flow bypasses are not normally required;
- The maximum allowable depth from the top of the surround to the lowest level of the sediment trap is limited by the reach of the equipment that will be used for cleaning. For an extended-arm backhoe, this is approximately 4.5m;
- The top of the structure should be at least 150mm above the surrounding ground level and/or protected by barriers to prevent vehicles from being driven over the trap;
- Lockable, removable covers shall be provided for access and maintenance; and
- Step irons shall be provided for access, in a position, which will not interfere with the operation of the cleaning equipment.

10.5 DESIGN OF OIL AND GREASE INTERCEPTOR

10.5.1 Design Criteria

OGI is typically an engineered precast tank designed to separate oil and grease from water through the use of baffled compartments and corrugated plates. Sizing determinations must be based on relevant information regarding the facility site, characteristics of the oil and grease loading in the stormwater flow, and upstream/downstream drainage network. Sizing must allow for adequate retention time and workable maintenance schedule.

OGI shall be constructed of impervious materials capable of withstanding abrupt and extreme changes in temperature. They shall be of substantial construction, watertight and equipped with easily removable covers which, when bolted in place, shall be gastight and watertight.

It must be located where maintenance can be easily performed. The installation should allow the cover to be visible and easily removable for cleaning, and clearances should be such that the internal baffling can be serviced. With the cover removed, all wetted surfaces should be visible. This is necessary not only for access to clean the interceptor, but also to have the capability to easily inspect the interior for potential problems such as damaged baffles and blocked air relief bypasses.

Sizing is related to anticipated influent oil concentration, water temperature and velocity, and the effluent goal. To maintain reasonable interceptor size, it should be designed to bypass flows in excess of 3 month ARI storm.

The sizing of OGIs is based upon the rise rate velocity of oil droplet and rate of runoff. However, with the exception of stormwater from oil refineries there are no data describing the characteristics of petroleum products in urban stormwater that are relevant to design; either oil density and droplet size to calculate rise rate or direct measurement of rise rates. Further, it is known that a significant percentages of the petroleum products are attached to the fine suspended solids and therefore are removed by settling.

10.5.2 Sizing for Conventional Gravity Interceptor

With small installations, a conventional gravity OGI has the general appearance of a septic tank, but is much longer in relationship to its width. Larger facilities have the appearance of a municipal wastewater primary sedimentation tank.

The sizing of a conventional gravity interceptor is based upon the calculation of the rise rate of the oil droplets using the following equation (API, 1990):

$$V_r = (g\ d_o^2(\rho_o - \rho_w))/18\ \mu \qquad (10.9)$$

where,

V_r = Oil droplet rise rate (cm/s);
µ = Absolute viscosity of the water (poise);
ρ_o = Density of the oil (g/cm^3);
ρ_w = Density of the water (g/cm^3);
d_o = Diameter of the droplet to be removed (cm); and
g = Gravitational constant, 980 cm/s^2.

A water temperature must be assumed to select the appropriate values for water density and viscosity from Table 10.3. There are no data on the density of petroleum products in urban stormwater but it can be expected to lie between 0.85 and 0.95. To select the droplet diameter the designer must identify an efficiency goal based on an understanding of the distribution of droplet sizes in stormwater. However, there is no information on the size distribution of oil droplets in urban stormwater. The designer must also select a design influent concentration, which carries considerable uncertainty because it will vary widely within and between storms.

Table 10.2: Water Viscosities and Densities (Daugherty & Franzini, 1977)

Temperature °C	Density g/cm³	Viscosity Poises
25	0.997	0.0008972
26	0.997	0.0008756
27	0.997	0.0008540
28	0.997	0.0008324
29	0.996	0.0008107
30	0.996	0.0007891
31	0.996	0.0007675
32	0.996	0.0007458
33	0.996	0.0007242
34	0.996	0.0007026
35	0.995	0.0006810

Sizing conventional gravity OGI (API, 1990), is based on following equation:

$$D = \sqrt{\frac{1}{2}\frac{Q}{V}} \qquad (10.10)$$

where,

D = Depth (m), which should be between 1.0 - 2.5;
Q = Design flow rate (m^3/s); and
V = Allowable horizontal velocity (m/s) which is equal to 15 times the design oil rise rate (V_r) but not greater than 0.015m/s.

If the depth exceeds 2.5m, design parallel units dividing the design flow rate by the number of units needed to reach the maximum recommended depth of 2.5m. Equation 10.10 is simplified from equations in API (1990) based on a recommended width to depth ratio of 2. The constant in Equation 10.10 can be changed accordingly if a different ratio is assumed. Some engineers may wish to increase the facility size to account for flow turbulence.

Then, estimate length (L) and width (W) using the following equations:

$$L = VD/V_r \tag{10.11}$$

$$W = Q/(VD) \tag{10.12}$$

Note that the width should be 2 to 3 times the depth, but not to exceed 6m and baffle height to depth ratio of 0.85 for top baffles and 0.15 for bottom baffles. Locate the distribution baffle at 0.10L from the entrance, add 300mm for freeboard and install a bypass for flows in excess of the design flow.

Determining the design flow, Q, requires identification of the design storm. The OGI is expected to operate effectively at all flow rates equal to or less than the peak runoff rate of the design storm. If sized to handle a storm frequency between the 3-month to 1-year ARI, the facility will effectively treat the vast majority of stormwater that occurs overtime. For the design storm selected, calculate the peak runoff rate using the Rational Method.

10.6 PROPRIETARY DEVICES

A number of proprietary designs for gross pollutant traps have been developed. Most of the proprietary devices developed to date are intended for use on piped drainage systems, rather than open channels.

This Manual seeks to encourage the development and application of suitable proprietary devices in Malaysia. Manufacturers seeking to market GPTs in Malaysia should provide full details, together with design guidelines and testing to DID or the local authority.

This may require the supplier to provide information on the catchment area, conduit size and its depth, estimated ARI flow capacity of the system, pollutant loading, and the required performance (% removal). Most of the devices include an internal bypass arrangement designed by the manufacturer.

REFERENCES

Comment [G9]: This chapter is based on old methods, the most recent being the 2000 MSMA. Elaborate design procedures are defined, based on these older references, that are no longer used in their country of origin.

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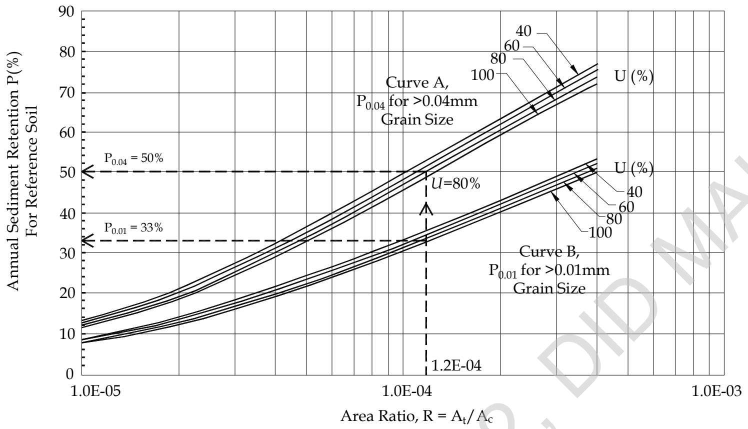
APPENDIX 10.A DESIGN CHARTS FOR SBTR TRAPS

10.A1 Average Sediment Retention against Area Ratio R

Design Chart 10.A1 shows the average annual sediment retention percentage as a function of the trap area ratio *R*, and the degree of urbanisation (*U*) in the catchment. Curve group (A), for particles ≥ 0.04mm is used to select the trap area A_t in order to achieve the specified design criteria. Curve group (B), for particles ≥ 0.01mm is used in calculating the trap volume for the sediment storage. In each case use the curve appropriate to the catchment urbanisation factor, *U*.



Design Chart 10.A1: Average Annual Sediment Retention against Area Ratio for Reference Soil (DID, 2000)

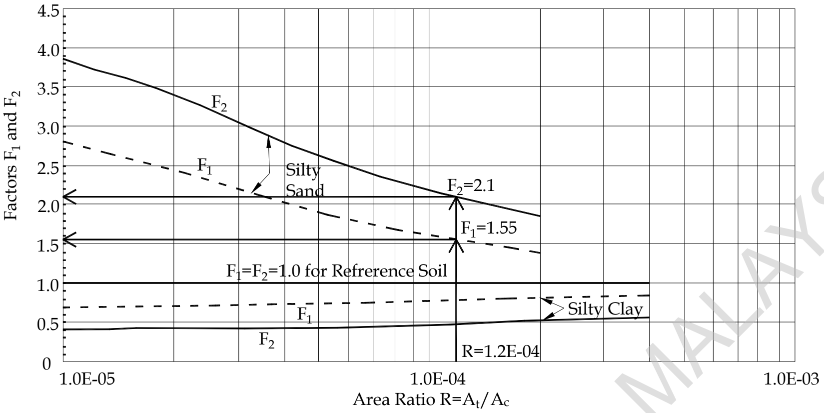
<u>How to Use the Chart</u>

An example is shown where *U* = 80% and the trap area ratio *R* = 1.2 E-04, the predicted removal of sediment for particle size ≥ 0.01mm is $P_{0.01}$ = 33% and ≥ 0.04mm is $P_{0.04}$ = 50%.

10.A2 Soil Type Adjustment Factors F_1 and F_2

Design Chart 10.A2 gives recommended values for the soil type adjustment factors F_1 and F_2 *as* a function of the soil type in the catchment.

These factors have been derived by repeating the calculations for Design Chart 10.A1, for other typical soil gradings.



Design Chart 10.A2: Soil Type Adjustment Factors for Trap Area and Sediment Volume (DID, 2000)

<u>How to Use the Chart</u>

Read factors F_1=1.55 and F_2=2.1 from the curves for the chosen trap area ratio R=1.2E-04.

APPENDIX 10.B EXAMPLE – SIZING OF SBTR (MAJOR)

Problem:

Determine the size required for a major SBTR GPT to be constructed in the Sg. Rokam, Ipoh, Perak with catchment area 113.8ha, 80% urbanisation and silty sand soil. It is an example of a community-level stormwater system as shown in Figure 10.B1.

Figure 10.B1: Site Plan

Solution:

Reference	Calculation	Output
	Step 1 : Determine the required removal efficiency. =	70%
Table 10.2	Step 2 : Detemine the catchment area, % urban area and soil type in the catchment.	
	Catchment Area, A_c =	113.8ha
	% Urban Area, U =	80%
	Soil Type =	Silty Sand
	Step 3 : Select trial trap area ratio R. First use trial area ratio R = 2.0 E-4	
	Step 4 : Calculate the required trap area by trial and error :	
Design Chart 10.A1	From Design Chart 10.A1, Curve A, $P_{0.04}$ value obtained =	61%
Design Chart 10.A2	From Design Chart 10.A2,F1 value obtained =	1.40
Equation 10.2	$P_{0.04*}$ = 61% x 1.40 = = (This is more than required so the trap size can be reduced)	85.4%

Reference	Calculation	Output
	Try R = 1.5 E-4. For this value of R, Design Chart 10.A1, Curve A gives $P_{0.04}$ = 54% for the reference soil and Design Chart 10.A2 gives F1 = 1.50; so the calculated removal efficiency $P_{0.04*}$ for the site soil is :	
	$P_{0.04*}$ = 54% x 1.50 =	81.0%
	(This is more than required so the trap size can be reduced)	
	Try R = 1.0 E-4. For this value of R, Design Chart 10.A1, Curve A gives $P_{0.04}$ = 47% for the reference soil and Design Chart 10.A2 gives F1 = 1.65; so the calculated removal efficiency $P_{0.04*}$ for the site soil is :	
	$P_{0.04*}$ = 47% x 1.65 =	77.6%
	(This is more than required so the trap size can be reduced)	
	Try R = 0.7 E-4. For this value of R, Design Chart 10.A1, Curve A gives $P_{0.04}$ = 44% for the reference soil and Design Chart 10.A2 gives F1 = 1.60; so the calculated removal efficiency $P_{0.04*}$ for the site soil is :	
	$P_{0.04*}$ = 44% x 1.60 =	70.4% (acceptable)
	Therefore the required minimum trap size is: $A_t = R \times A_c$ $=0.7 \times 10^{-4} \times 113.8 \times 10^4 m^2$ =	$79.7m^2$
	Step 5 : Determine the trap length and width to give a ratio L_t/W_t of between 2 and 3	
	The following trial dimensions are selected: L_t = 13.0m, W_t = 6.5m. Then the trap length and width ratio is equal to 2.0, and actual trap area $A_t = 84.5m^2$. =	$84.5m^2$
	Step 6 : Determine the average annual sediment export with grain size ≥ 0.01mm: (assumed 75% of TSS)	
Equation 3.2 Table 3.2	The TSS load estimated by EMC method is $L = R.EMC.A.C_v / 100$ = 3000 x 128 x 113.8 x 0.80 / 100 =	349594 kg @ 350 tonne
	Sediment export M = 0.75 x 350tonne =	262.5tonne
Design Chart 10.A1B & Design Chart 10.A2	Step 7 : Determine $P_{0.01}$, the average annual pollutant retention ≥ 0.01mm diameter for the reference soil from the relevant Curve B in the lower part of Design Chart 10A.1, and Volume Factor F_2 from Design Chart 10A.2:	
	Pollutant retention for reference soil $P_{0.01}$ = 29%, and F_2 = 2.2.	
Equation 10.4	Pollutant retention for site soil $P_{0.01*}$ = 29% x 2.2 =	63.8%
	Step 8 : Determine the required minimum sediment trap depth	
	$D_w = 0.0065 \times P_{0.01*} \times M / A_t$ = 0.0065 x 63.8 x 262.5/84.5 =	1.30m

Reference	Calculation	Output
	Step 9 : Determine the rainfall in the *water quality design storm* (usually 3 month ARI, then find $Q_{0.25}$ using the data of Politeknik Ungku Omar, Ipoh in Table 2.B2, rainfall intensity can be calculated :	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$ $= \frac{62.9315 \text{ x } 0.25^{0.3439}}{(1 + 0.1703)^{0.8229}}$ =	34.33mm/hr
Equation 2.3	$Q = \frac{C.i.A}{360}$ $= \frac{0.8 \text{ x } 34.33 \text{ x } 113.8}{360}$ =	8.68m^3/s
	Step 10 : Determine the trash rack height from Equation 10.6. Try a trash rack length L_r = 7.5m to match the height of the sediment trap:	
Equation 10.6	$H_r = 1.22\left(\frac{Q_{0.25}}{L_r}\right)^{2/3}$ $= 1.22 \text{ x } \left(\frac{8.68}{6.5}\right)^{2/3}$ =	1480mm
	Trash rack length L_r of 6.5m gives H_r = 1.48m which is reasonable Determine the nominal flow velocity $V_{0.25}$ in the water quality design storm. Increase the dimensions of the sediment trap pool or increase the track rack height if the flow velocity V is greater than 1.0m/s, to minimise the re-entrainment of deposited sediment	
Equation 10.7	$V_{0.25} = \frac{Q_{0.25}}{(D_w + H_r)W_t}$ $= \frac{8.68}{(1.30 + 1.48) \ 6.5}$ =	0.48m/s (which is acceptable)
	For a major SBTR, determine required clearance above the trash track from Equation 10.8. The open trash rack will be overtopped in floods greater than the 3 month ARI flood (if 50% blocked) and the open channel must be designed accordingly. The resulting 'theoretical' concept design for the SBTR trap is as shown below. In reality, the concept and dimensions may have to be adjusted if required to suit site conditions.	

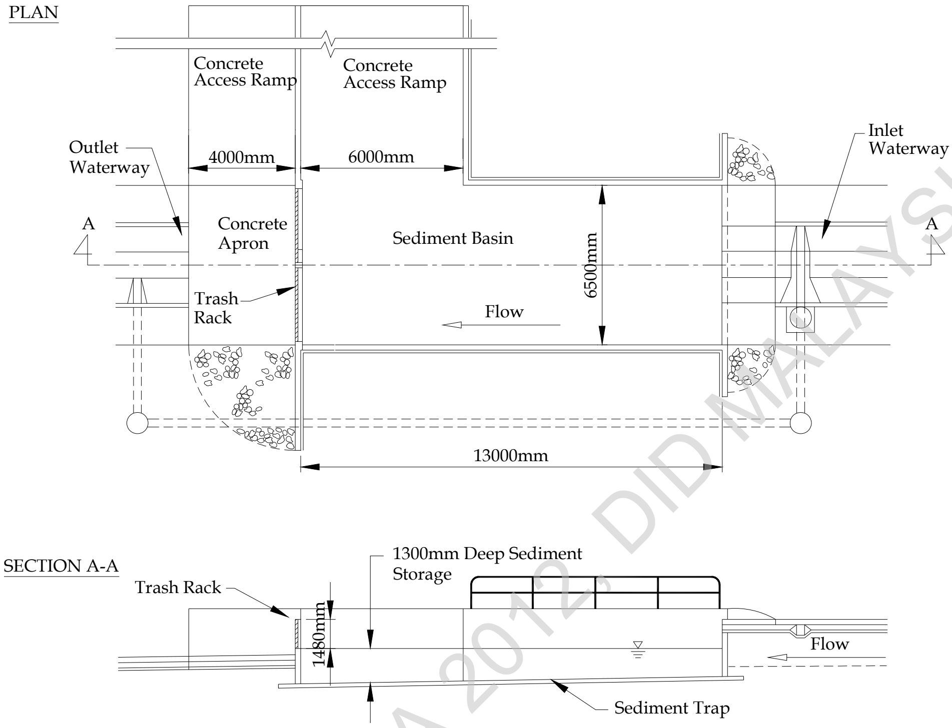
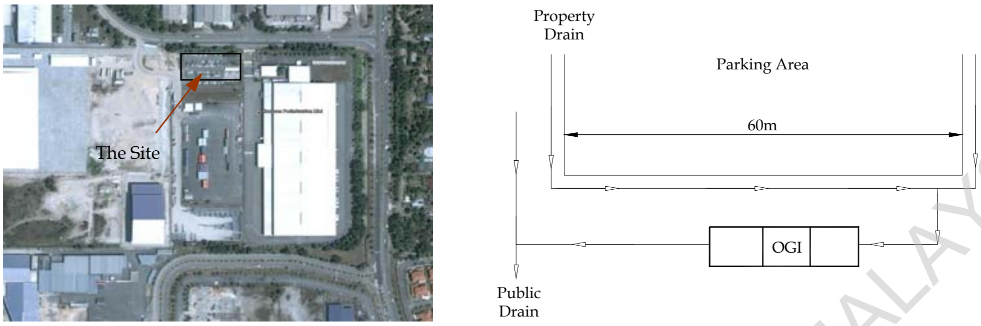


Figure 10.B2: Layout of Proposed Major SBTR

APPENDIX 10.C EXAMPLE – SIZING OF CONVENTIONAL GRAVITY OGI

Problem :

A conventional gravity OGI is to be used to trap oil and grease in runoff from a 0.2 ha parking lot of a factory in Bandar Baru Bangi, Selangor. Assume it is to be sized to treat runoff with flow rate of 0.0702 m³/s per hectare when the area is 100 % impervious. Given: $\rho_o = 0.898$ g/cm³, $d_o = 90 \times 10^{-4}$ cm and temperature = 30 °C.



a) Site Plan

b) Layout

Figure 10.C1: Proposed OGI Site and Layout

Solution :

Reference	Calculation	Output
Table 10.3	With the temperature = 30 °C, the water density = 0.996g/cm³ and μ = 0.0007891 poises	
Eq. 10.9	$V_r = (g\, d_o^2(\rho_o - \rho_w))/18\, \mu$ = (980 x (90 x 10⁻⁴)²(0.898-0.996))/18(0.0007891) the negative symbol of V_r only represent the direction of the particle. Thus the value of V_r =	-0.559cm/s 5.59 x 10⁻³m/s
	Allowable horizontal velocity is equal to 15 times the design oil rise rate, V_r	
Eq. 10.10	V = 15 x 5.59 x 10⁻³	0.0838m/s
Eq. 10.10	D = (Q/2V)^0.5 =(0.2 x 0.0702/(2 x 0.0838))^0.5	0.3m
Eq. 10.11	L = VD/V_r = 0.0838 x 0.3/5.59 x 10⁻³	4.5m
Eq. 10.12	W = Q/(VD) = (0.2 x 0.0702)/(0.0838 x 0.3) = 0.6m W = 0.6m, since W is equal to 2 x D, so W	0.6m
	Thus, a conventional OGI sized to capture runoff from the parking lot D = 0.3m W = 0.6m L = 4.5m	

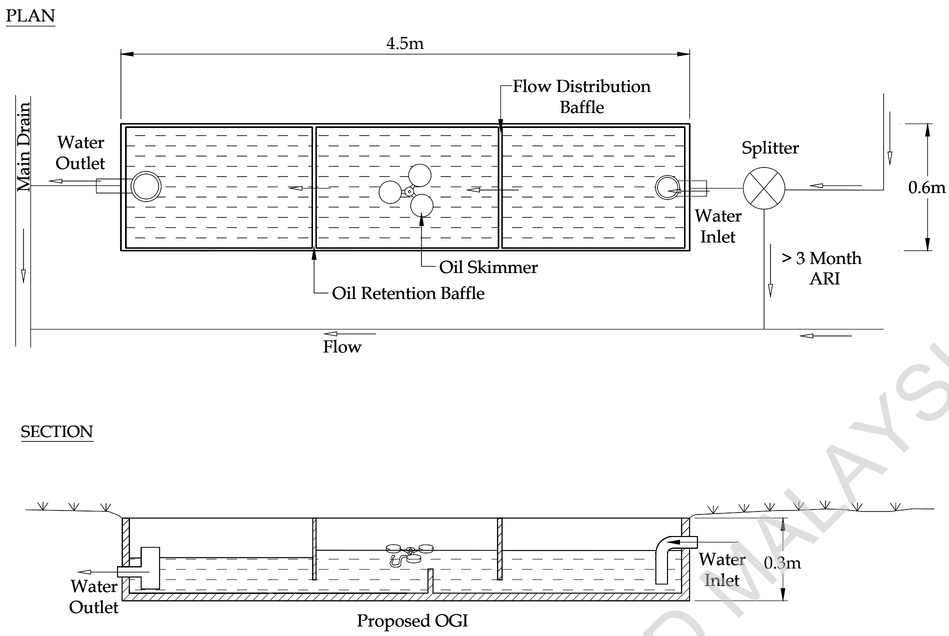
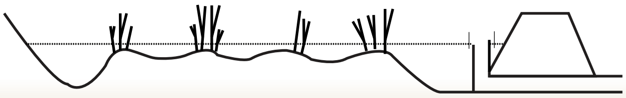


Figure 10.C2: Proposed Size of Conventional Gravity OGI



WATER QUALITY POND AND WETLANDS

11

CHAPTER 11 WATER QUALITY POND AND WETLANDS

11.1 INTRODUCTION

This Chapter contains guidance for the selection and design of water quality ponds and or/wetlands for new or existing developments areas. Guidances provided include descriptions, design criteria and sizing procedures.

If there is a need to design a storage facility for both water quality and quantity control purposes, the water quality pond should be considered and used in conjunction with detention pond design.

11.2 WATER QUALITY POND

11.2.1 Description and Types

Water quality ponds, with extended detention (ED), are effective for removing suspended constituents. Removal rates of solids and many dissolved constituents by wet water quality pond outperform dry type (Iowa Natural Resources Conservation Service, 2008). Dry type water quality ponds provide moderate pollutant removal, particularly soluble pollutants because of the absence of a permanent pool.

A practice has developed in some overseas countries with temperate climates, of designing a pond so that the runoff from a storm is retained over an extended period of several days or longer. This water quality pond allows for greater removal of certain pollutants, such as suspended constituents and nutrients, by biological action and settlement. The water quality pond uses a much smaller outlet than a flood control detention basin, which extends the emptying time for the more frequently occurring runoff events to facilitate pollutant removal (UDFCD, 2008). Runoff from small storms is attenuated.

When sized and designed appropriately with an appropriate drawdown time, water quality pond can provide the required sediment capture. In dry type ponds there is little biological uptake due to the lack of vegetation and relatively shorter detention times. Sediment re-entrainment may occur if the pond is not designed properly when subjected to high flows. Wet type ponds that are permanently full (wet extended detention ponds) are preferred if very high quality effluent is desired. Both dry and wet type water quality ponds can provide extended detention, the latter by having a surcharge water quality capture volume that is released slowly to mitigate some of the erosion effects in downstream waterways.

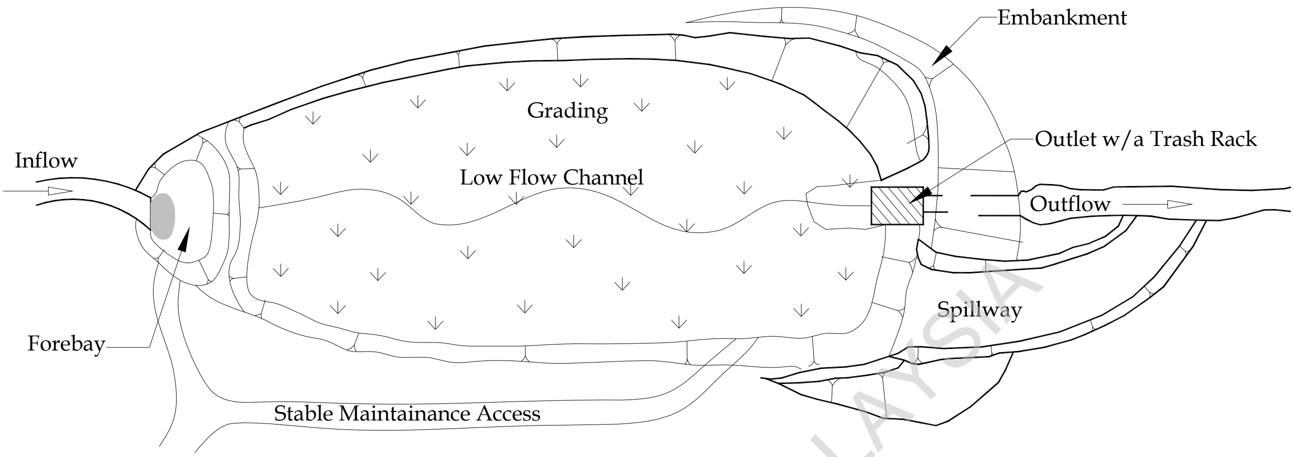
Other issues to be considered are:

- Water quality ponds will attenuate small storms (such as the water quality design storm), but due to their relatively small size and lower release rates, they will not provide significant peak flow attenuation for large storms. However, the water quality pond can be incorporated as the bottom stage of a larger flood control basin that provides flow attenuation for large storms.
- Due to the low amount of vegetation and the absence of permanent water, the habitat value may not be high. Water quality ponds may have less visual appeal than ponds that are permanent pool.
- The high probability of repeated storms within the detention period will make it difficult to achieve full emptying.
- Periodically inundated areas are often wet and boggy, so they may be difficult to mow and walk on and create favourable conditions for mosquito breeding. It may be difficult to maintain wetlands vegetation because the water level changes may be large and frequent.
- High sediment and debris loadings in many areas of Malaysia will create a serious risk of blockage of the small outlets that are typically used for extended detention.

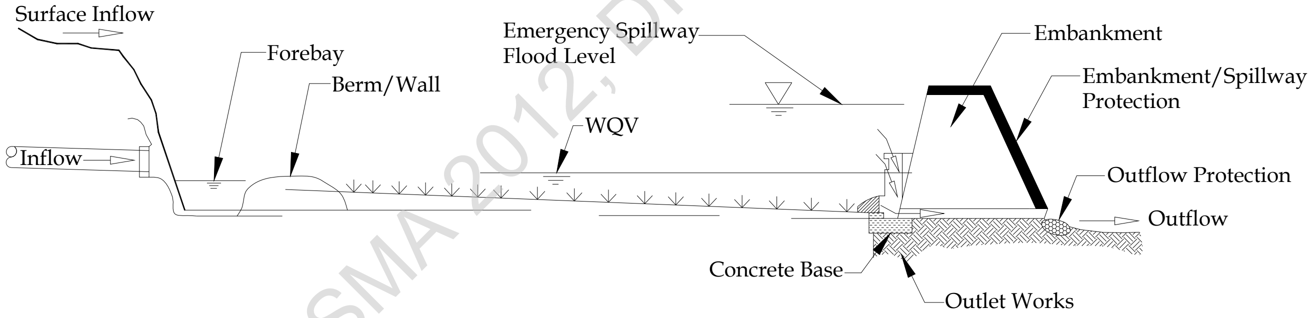
These issues require careful consideration by the designer under Malaysian conditions. There may be particular situations where water quality ponds would be appropriate. For example, they may be used in areas where pervious soils, lack of impermeable liner, lack of adequate inflow during dry weather periods, or low groundwater table would preclude the use of a permanent pool pond. On the other hand, when these conditions can be satisfied, wet detention basins would do a very good job in reducing pollutants leaving the site.

11.2.1.1 Dry Type

A dry type water quality pond is a sedimentation basin designed to totally drain dry after runoff from a storm ends (Figure 11.1). It is an adaption of the detention ponds used for flood control. The primary difference is in the outlet design. The dry type water quality pond uses a much smaller outlet that extends the emptying time of the more frequently occurring runoff events to facilitate pollutant removal. A drain time (time to fully evacuate the design volume) of 24 hours to 48 hours is recommended to remove a significant portion of the fine particulate pollutants found in urban stormwater runoff (UDFCD, 2008).



(a) Plan



(b) Profile

(c) Examples (UDFCD, 2008)

Figure 11.1: Dry Type Water Quality Pond (Modified from UDFCD, 2008)

Dry type water quality pond can be used to enhance stormwater runoff quality and reduce peak stormwater runoff rates. If these basins are constructed early in the development cycle, they can also be used to trap sediment from construction activities within the tributary drainage area. The accumulated sediment, however, will need to be removed after upstream land disturbances cease and before the pond is placed into permanent use. Also, a dry type water quality pond can sometimes be retrofitted into existing flood control detention ponds. Dry type water quality pond can be used to improve the quality of urban runoff coming from roads, parking lots, residential neighbourhoods, commercial areas, and industrial sites and are generally used for regional or development-wide treatment (UDFCD, 2008).

Since the dry type water quality pond is designed to drain very slowly, its bottom and lower portions will be inundated frequently for extended periods of time. Grasses in this frequently inundated zone will tend to be stressed, with only the species that can survive the specific environment at each site eventually prevailing. In addition, the bottom will be the depository of all the sediment that settles out in the pond. As a result, unless the inflow, concrete trickle channel and properly designed outlet are provided, the bottom can be muddy and may have an undesirable appearance to some. To reduce this problem and to improve the pond's availability for other uses (such as open space habitat and passive recreation), it is suggested that wet type water quality pond to be provided, in which the settling occurs primarily within the permanent pool.

11.2.1.2 Wet Type

Wet type water quality ponds (Figure 11.2) are very similar to wet detention ponds with the difference that their design is more focussed on attenuating water quality and peak runoff flows from smaller runoff event. Soluble pollutant removals are provided by a permanent ponding area in the bottom of the pond to promote biological uptake.

Since a water quality pond is designed to drain very slowly, its bottom and lower portions will be inundated frequently for extended periods of time. Grasses in this frequently inundated zone will tend to be stressed, with only the species that can survive the specific environment at each site eventually prevail. In addition, the bottom will be the depository of all the sediment that settles out in the pond. As a result, unless an inflow, concrete trickle channel and properly designed outlet are provided, the bottom can be muddy and may have an undesirable appearance to some. To reduce this problem and to improve the pond's availability for other uses (such as open space habitat and passive recreation), it is suggested that a wet type water quality pond be provided, in which the settling occurs primarily within the permanent pool. The outfall structure for wet type water quality ponds should be sized to allow for complete drawdown of the surcharge water quality volume stored above the permanent pool in no less than 12 hours (UDFCD, 2008).

11.2.2 Design Considerations and Requirements

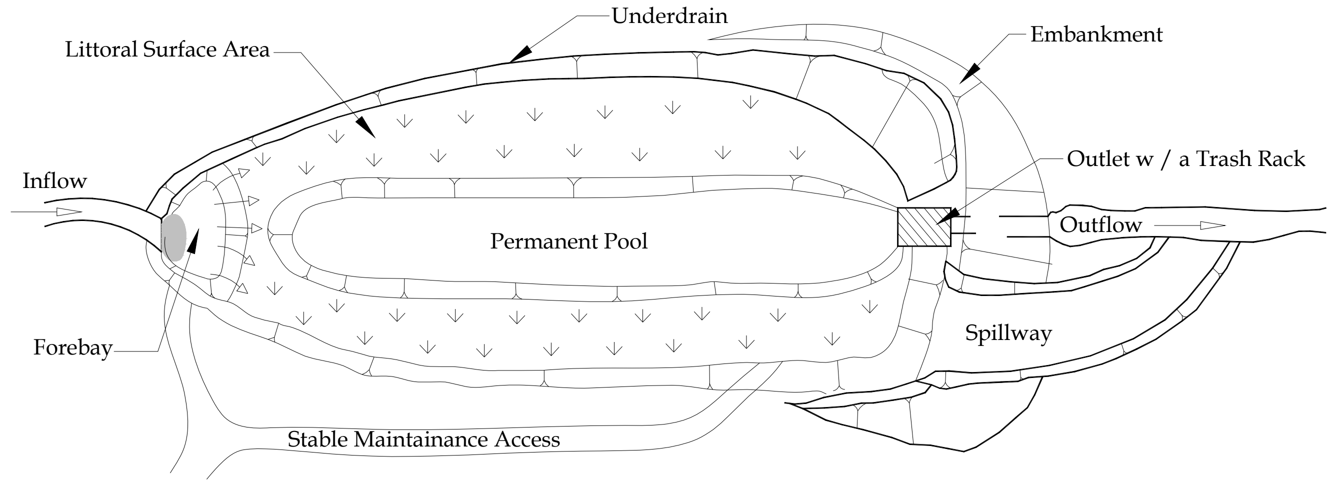
Specific designs may vary considerably, depending on site constraints or preferences of the designer or community.

11.2.2.1 Drainage Area

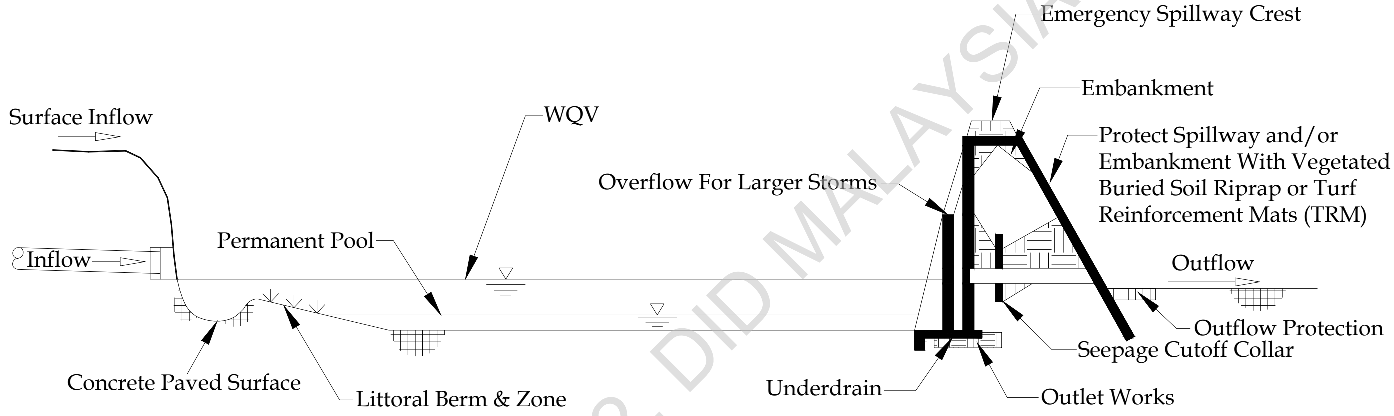
The minimum recommended catchment area is 2.5ha for commercial or industrial landuses and 3.5ha for residential landuses. A single water quality pond shall not have a contributing drainage area less than minimum recommended catchment area unless specifically approved by the local regulatory authority because it may require a very small orifice that would be prone to clogging when sized to allow for complete drawdown of the water quality volume for a specific period.

11.2.2.2 Location and Site Suitability

It is recommended that water quality ponds be located where the topography allows for maximum runoff storage at minimum excavation or embankment construction costs (Iowa Natural Resources Conservation Service, 2008). When locating a water quality pond, planners and designers should also consider the location and use of other land use features, such as planned open spaces and recreational areas, and should attempt to achieve multi-use objectives where these can be safely achieved. Water quality ponds shall not be located on unstable slopes or slopes greater than 15% (Iowa Natural Resources Conservation Service, 2008).



(a) Plan



(b) Profile

(c) Example (Iowa NRCS, 2008)

(d) Example (UDFCD, 2008)

Figure 11.2: Wet Type Water Quality Pond (Modified from UDFCD, 2008)

11.2.3 Design Criteria

Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. There are some features, however, that should be incorporated into most water quality pond designs.

11.2.3.1 Geometry

Design flow paths shall be designed to minimise potential short-circuiting by locating the outlet as far away from the inlet structures as possible. The recommended length-to-width ratio of a basin should be at least 2:1 (and preferably 3:1).

11.2.3.2 Water Quality Volume

The water quality volume (WQV) shall be determined for a water quality design storm (runoff from a 40mm rainfall depth over the contributing catchment). An effective stormwater enhancement facility has to balance the pond brim-full volume and its drawdown time.

In practice, the drain time is determined by the required pollutant settling time. If sediment characteristics are not known, a 12-hour drawdown time for a wet type water quality pond and a 48-hour drawdown time for dry type water quality pond are recommended for stormwater quality control designs. It was reported that about 80 to 90% sediment removal can be achieved using the above drain times (EPA, 1983). Therefore, it is expected that a large detention volume draining in a short time will not sufficiently remove sediment and the water quality volume shall be determined with the water quality pond design.

The required storage volume for WQV can be obtained using the Equation 11.1 as follows:

$$\text{WQV} = C.(P_d).A \tag{11.1}$$

where,

WQV = Water quality volume (m^3);
C = Volumetric Rational Method runoff coefficient (Refer Table 2.5);
P_d = Rainfall depth for water quality design storm (m); and
A = Contributing drainage area (m^2).

11.2.3.3 Outlet Works

The outlet works are to be designed to release the water quality volume over a proposed drain time period (Refer to Chapters 2 and 20 pertaining to outlet type: orifice plate or perforated riser pipe; hydraulic structures geometries; grates, trash racks, and screens; and all other necessary components). The number of columns of perforations used should be minimised and the diameter of the perforation hole(s) should be maximised when designing outlets to reduce chances of clogging by accepting the orifice size that will empty the water quality volume in a specified time (UDFCD, 2008).

The outlet works are designed to provide a drawdown of the water quality pond over a specific time. The target average discharge from the outlet works is computed as the ratio of the required storage volume for the WQV to the drawdown time as follows:

$$Q_{ave} = \frac{\text{Required Storage for WQV } (m^3)}{\text{Drawdown Time } (s)} \tag{11.2}$$

11.2.3.4 Safety

Public safety should be considered, particularly in residential areas. Designers should avoid steep slopes and drop-offs and depths over 1.0m whenever possible. If it is not possible, safety features such as fences shall be provided unless buffered by shallow marshlands (littoral zone) on all sides accessible by the public.

11.2.3.5 Water Quality Volume for Detention Ponds

Wherever possible, it is recommended that water quality pond be incorporated into stormwater quantity detention facilities. This is relatively straight forward for water quality pond, constructed wetlands, wetpond and detention pond. When combined, the 2, 5, 10, and/or 100-year ARI detention levels are provided above the water quality volume and the outlet structure is designed to control two or three different releases. When a local policy is lacking, the following approach is suggested as a minimum:

- Water quality: The full water quality volume is to be provided according to the design procedures;
- Minor storm: The full water quality volume plus the full minor storm quantity detention volume is to be provided; and
- Major storm (100-year storm or other event): One-half the water quality volume plus the full 100-year (or other major storm event) detention volume is to be provided.

The designer should be aware that the two functions (stormwater quantity and quality controls) can be provided by a same pond, and that incorporating the two functions in one pond will be more cost-effective than creating two separate ponds.

11.2.4 Design Procedure for Water Quality pond

The following design steps are recommended when designing water quality pond:

Step 1: Confirm Treatment Performance of Concept Design.

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the required area from the concept design is adequate to deliver the required level of stormwater quality improvement. This design process assumes a conceptual design has been undertaken. The pollutant reduction curve for water quality pond as a function of catchment impervious area shown in Figure 3.1 can be used to undertake this verification check. The curve is intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough design process.

Step 2: Determine Water Quality Volume.

Designer should determine the runoff coefficient for the catchment area to calculate the WQV and set the permanent pool elevation and water level based on the WQV calculated.

Step 3: Outlet Works.

The outlet works are to be designed to release the WQV (above the permanent pool elevation) over a proposed drain time period (Refer to Chapters 2 and 20 pertaining to outlet type) based on the types of water quality pond.

Step 4: Design Embankment(s) and Spillway.

Designers must calculate the peak of storm water surface elevation and size an emergency spillway to provide a controlled overflow for flows in excess of the design water quality storm. They must then set the top of the embankment elevation.

11.3 CONSTRUCTED WETLANDS

Constructed wetlands, like ponds, are used to improve water quality. The wetlands constructed at Putrajaya and Engineering Campus, Universiti Sains Malaysia (Figure 11.3) were designed for this purpose. Ponds and wetlands are complementary. The pollutant removal efficiency of a pond can be increased by including an area of wetlands.

11.3.1 Description and Components

Wetlands is an area of land where soil is saturated with water either permanently or seasonally. Such areas also need to be covered, at least partially, by shallow pools of water. Wetlands include swamps, marshes and bogs, among others. The water found in wetlands can be saltwater, freshwater or brackish.

(a) Putrajaya

(b) Engineering Campus,
Universiti Sains Malaysia

Figure 11.3: Examples of Constructed Wetlands in Malaysia

Wetlands are considered the most biologically diverse of all ecosystems. Plant life found in wetlands includes mangrove, water lilies, cattails, sedges, tamarack, black spruce, cypress, and many others. Animal life includes many different amphibians, reptiles, birds and furbearers.

Wetland systems can be explicitly designed to aid in pollutant removal from stormwater. Wetlands also provide for quantity control of stormwater by providing a significant volume of temporary water storage above the permanent pool elevation. Water levels rise during rainfall events and outlets are configured to slowly release flows, typically based on the design WQV, back to dry weather water levels. As stormwater runoff flows through the wetlands, pollutant removal is achieved via settlement and biological uptake within the BMPs. Wetlands are among the most effective stormwater BMPs in terms of pollutant removal, and also offer aesthetic value.

Constructed surface flow wetlands systems remove pollutants in stormwater through sedimentation, filtration of fines and biological uptake. The main components of a wetlands are the inlet zone, macrophyte zone and then open water zone (Figure 11.4).

11.3.1.1 Inlet Zone

The function of the inlet zone (sediment basin or forebay) is to encourage settling of coarse to medium sediments from the water column, reduce flow velocities and distribute flow across the wetlands. The installation of sediment traps and trash skimmers helps in the function of this zone.

The inlet zone of a wetlands is designed as a sediment basin and serves two functions:

- Pre-treatment of inflow to remove coarse to medium sized sediment; and
- The hydrologic control of inflows into the macrophyte zone and bypass of floods during 'above design' operating conditions.

The inlet zone consists of the following elements:

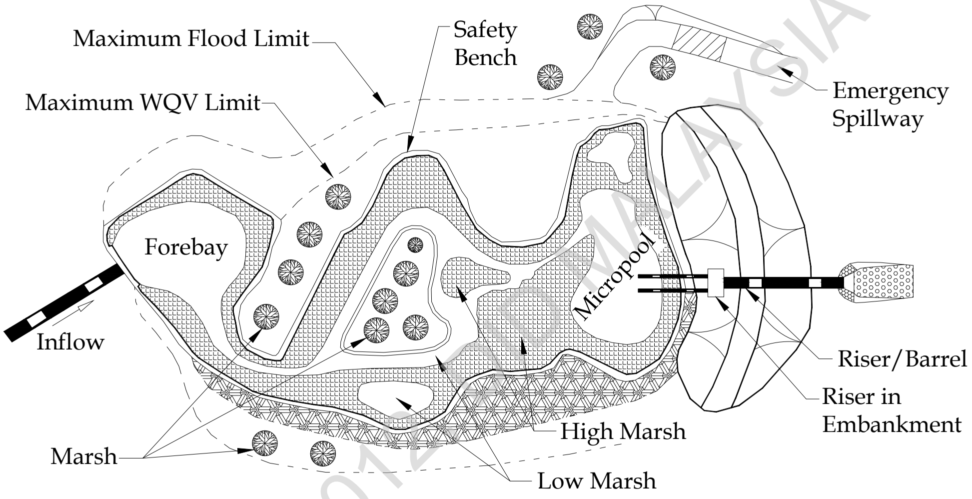
- Sedimentation basin 'pool' (or sediment forebay) to capture coarse to medium sediment;
- Inlet zone connection to the macrophyte zone (or control structure) normally consisting of an overflow pit within the inlet zone connected to one or more pipes through the embankment separating the inlet and macrophyte zones; and
- High flow bypass weir (or 'spillway' outlet structure) to deliver 'above design' flood flows to the high flow bypass channel.

11.3.1.2 Macrophyte Zone

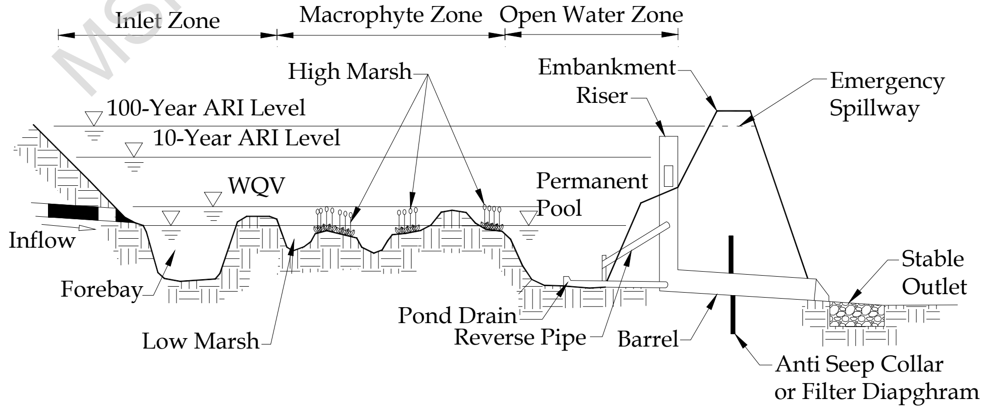
Macrophytes are large, emergent aquatic plants. For macrophyte zones to function efficiently, flows that pass through the vegetation must be evenly distributed. Beds of macrophytes filter out fine particles and directly take up contaminants. They enhance sedimentation and the absorption of pollutants onto sediments.

Macrophyte species should be selected from plants provided in Annex 1 and they shall meet the following requirements:

- Native to the area;
- Known to be likely to establish and grow under conditions applying at the site;
- Unlikely to colonise outside the designated area;
- With a maximum height of the plants must be consistent with maintaining desirable visual characteristics of the pond; and
- Not growing to a density that would provide suitable habitats for mosquito breeding.



(a) Plan



(b) Profile

Figure 11.4: Wetlands Component (Modified from MDE, 2000)

11.3.1.3 Open Water Zone

An open water zone is a deeper area that allows time for fine particles to flocculate to the bed and allows sunlight to kill bacteria. Decomposition and grazing of organic matter will occur in this zone. Periodic algal growth may occur here and this will also trap dissolved nutrients and allow them to enter the food chain or to settle to the bed of the pond.

11.3.2 Design Considerations and Requirements

Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. There are some features, however, that should be incorporated into most wetlands designs.

11.3.2.1 Drainage Area

Wetlands require a positive water balance, through continuous base flow or groundwater seepage, so that permanently wet conditions can be sustained. They are therefore limited to areas where springs or land drainage provides base flow during dry weather (CIRIA, 2007).

11.3.2.2 Space Requirement

At a minimum, the surface area of a wetlands should be no less than 1% of the contributing drainage area. The surface area can be determined from the annual pollutant curve as a function of pond area ratio in Chapter 3.

11.3.2.3 Location and Site Suitability

Wetlands are normally located at a position that can receive all the site runoff (i.e. at the lowest point in the site), generally at minimum excavation (or construction) cost. Where the urban design of an area permits, the location of a wetlands should take account of natural site features that might be used as additional temporary storage areas when the wetlands capacity is exceeded during extreme events. If the wetlands design criteria include protection of the watercourse during floods, then the system should not be located in the floodplain, where there is a risk that the detention storage will be lost through inundation at the critical time. River flood waters also tend to carry high sediment and debris loads, which will necessitate additional wetlands maintenance. Wherever possible, wetlands should be located in, or adjacent to, non-intensively managed landscapes where natural sources of native species are likely to be plentiful (CIRIA, 2007).

11.3.2.4 Site Slope and Stability

Wetlands basin require a near-zero (almost horizontal) longitudinal slope, which can be provided using embankments, if required. Wetlands should not be sited on unstable ground and ground stability should be verified by assessing site soil and groundwater conditions. Wetlands should not be considered within or over waste fill materials, uncontrolled or non-engineered fill (CIRIA, 2007).

11.3.2.5 Subsurface Soil and Groundwater

The soil below a wetlands must be sufficiently impermeable to maintain wet conditions, unless the wetlands intersects the water table. In permeable strata, a liner (or other impermeable material) will be required to prevent drying out. As the wetlands evolves, loss of water should become negligible as the soils on the floor of the basin become more organic, reducing the potential for exfiltration. Loams soils are needed in a wetlands bottom to permit plants to take root. Assessment of soils should be based upon an actual subsurface analysis and permeability tests (CIRIA, 2007).

In areas with contaminated soils or contaminated groundwater, wetlands can be used providing that the system is fully sealed, preventing exchange of water between the facility and groundwater. Any excavation or earthmoving processes required must be managed to ensure that mobilisation of contamination does not occur. Any permeability of the subsurface soils will require the use of a liner (or equivalent). Where the groundwater table is close to the base of the wetlands, the operation of the outfall should be confirmed for the annual maximum water table level. The maximum expected groundwater level should be beneath the storage zone (CIRIA, 2007).

11.3.3 Design Criteria

The primary objectives of the design are to meet the statutory requirements with regard to water quality and also to be seen as sensitive to the environment and proactive in meeting community expectations. The constructed wetlands is designed for stormwater pollution control in the project area before discharging into the nearest receiving waterway.

11.3.3.1 Design Storm

The water quality design storm with 40mm rainfall depth over the contributing catchment should be determined at the pre-treatment inlet. If the wetlands is to be constructed offline, then an upstream bypass structure can be used to divert the water quality storm volume into the wetlands.

The principal function of the storage in a wetlands is to provide a variable wetting-drying cycle, which encourages growth and diversity of macrophytes. A depth range of 0.3m to 1.2m and a hydraulic residence time of at least 12 hours for a design storm may be suitable. However, peak flow control and extended detention are achieved by temporarily ponding above the wetlands permanent water level during rainfall events, and then releasing it in no less than 12 hours to 24 hours is recommended for its biological treatment function.

11.3.3.2 Allocation of Surface Area

In general, wetlands designs are unique for each site and application. However, the allocation of surface area and limiting depths for the design of constructed wetlands shall be considered for adequate pollutant removal, ease of maintenance and improve safety. Note that if the surface area criteria is in conflict with the volume allocations, the surface area allocations are more critical for an effective design.

a) Inlet Zone

A sediment forebay is designed at the inlet zone to remove incoming sediment from the stormwater flow before runoff enters the wetlands marsh. This ensures the vegetation in the macrophyte zone is not smothered by coarse sediment and allows the macrophyte zone to target finer particulates, nutrients and other pollutants. The forebay shall consist of a separate cell, formed by an acceptable barrier and provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetlands facility.

Typically the forebay should be about 15 to 30% of the total wetlands area and around 1.5 to 2.0m deep. A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

b) Macrophyte Zone

The wetlands arrangement in a recreation reserve should be chosen to maximise its effectiveness while minimising the impact on open space. The wetlands replicates a natural river system with a long, narrow plan form with deeper ponds. Macrophytes would be planted in the shallow sections to assist with water treatment. This and relatively dense plantings of macrophytes also form a barrier around the edges of the wetlands to discourage people from entering the water.

The macrophyte zone should be provided around the wetlands edges downstream of the main inlets to filter out sediment, nutrients and toxicants, to disperse the inflowing waters and to reduce their velocity. Plantings should be on the perimeter, arranged such that there is opportunity for water in the open pond zone to circulate through the macrophyte zone. An allocation of 50 to 80% of the total wetlands area and a depth of 0.3 to 1.2m is recommended for the macrophyte zone.

c) Open Water Zone

The open water zone attracts plant and wildlife diversity and has the potential to be used for aesthetic and passive recreational activity. Typically, at least 5% of the total wetlands area, a minimum depth of 1.0m and a maximum depth of 2.0m are recommended for the open water zone.

11.3.3.3 Geometry

a) Length to Width Ratio

The length of time that water is retained in a pond is the main variable in determining how effective the wetlands will be in trapping pollutants. Increasing the volume and hence, retention time allows for more sedimentation of particles, and more assimilation of pollutants. Wetlands should be long relative to their width in order to provide optimum flow circulation, with minimum length to width ratio (L:W) in the range of 2:1 to 5:1.

b) Macrophyte Zone Bathymetry

It is a good design practice to provide a range of habitat areas within the macrophyte zone to support a variety of plant species, ecological niches and perform a range of treatment processes. The macrophyte zone therefore typically comprises of four marsh zones (defined by water depth). The four marsh zones are:

- Ephemeral zone (from 0.20m above the pool or water surface elevation): These areas are located above the permanent pool that are inundated during larger storm events. This zone supports a number of species that can survive flooding.
- High marsh zone (from 0.30m below the pool to the normal pool elevation): This zone will support a greater density and diversity of wetlands species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.
- Low marsh zone (from 0.30 to 0.60m below the normal permanent pool or water surface elevation): This zone is suitable for the growth of several emergent wetlands plant species.
- Deep mash zone (from 0.60 to 1.2m deep; includes the outlet micropool and deepwater channels through the wetlands facility): This zone supports little emergent wetlands vegetation, but may support submerged or floating vegetation.

The recommended vegetation planting density for the macrophyte zone is summarised in Table 11.1.

Table 11.1: Vegetation Planting Density

Zone	Depth Range (m)	Planting Density (plant/m^2)
Ephemeral zone	0 to 0.20m above permanent pool	8
High marsh zone	0 to 0.30m below permanent pool	10
Low marsh zone	0.30 to 0.60m below permanent pool	6
Deep marsh zone	0.60 to 1.20m below permanent pool	4

c) Water Depth

Wetlands should contain a mixture of water depths. Water levels in excess of 1m should not occur over more than 20% of the wetlands pond surface area (preferably limited to the inlet zone and outlet open water zones) as deep water supports relatively less of plant species compared to shallow zones.

Where wetlands are designed for flow attenuation, the depth of temporary storage above the permanent water level should not exceed 1.5m to allow for colonisation for submerged macrophytes. The water depth in the wetlands shall range between 0.1m and 1.2m with an average of 0.6m. Changes in water level shall be limited to about 0.6m. Wetlands that are associated with ponds used for flood control shall be designed to accommodate submergence to depths between 1m and 2m with the maximum velocity not exceeding 0.1m/s.

d) Slopes

For safety, stability and to promoting the growth of macrophytes, slopes within the wetlands shoreline area should be in the range of 6(H): 1(V) to 8(H): 1(V). After reaching a depth of 1m, the slope can be increased. The maximum slope is set by the angle of repose of the saturated soil.

Side slopes above water level should also be gentle, both for safety reasons and to limit the potential for erosion. However, the slope should not be so flat that it creates ponding areas. A minimum side slope of 10(H): 1(V) to 20(H): 1(V) is recommended for a distance of 5m from the wetlands edge, to allow maintenance access.

Wetlands that are also intended for flood control will be subject to variable water levels. This creates problems around the water edge due to alternate wetting and drying of soil, making it difficult to establish or maintain grass. In this situation grass is not suitable and a hard edge, lined with rock gabions or a low concrete wall, is preferable where visibility and public access are provided. For those parts of the pond that are inaccessible, emergent plants such as reeds, which will tolerate water level changes, can be used.

e) Water Balance

The designer should check that the permanent pool will not dry out during extended dry periods. This can be done by means of a continuous water balance calculation, allowing for runoff inputs, evaporation and infiltration over a period of least 12 months. If excessive exfiltration is likely it may be necessary to specify an impermeable lining, either with clay or a synthetic liner.

In the humid tropical climate of Malaysia, the risk of a permanent pool drying out is less likely than in drier climates.

11.3.3.4 Water Quality Treatment

a) Pollutant Removal Capabilities

Wetlands design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed wetlands facilities can reduce pollutant removal performance. The pollutant reduction curve in Figure 3.1(c) can be used to check the expected performance of the wetlands system.

b) Water Quality Volume

Pollutants are removed from stormwater runoff in a wetlands through uptake by wetlands vegetation and algae, vegetative filtering, and through gravitational settling in the slow moving marsh flow. Other pollutant removal mechanisms are also take place in a stormwater wetlands, including chemical and biological decomposition, and volatilization. The required storage for WQV of a wetland can be determined from Equation 11.1.

c) Hydraulic Residence Time

The hydraulic residence time is the permanent pool volume, divided by the average outflow discharge rate. The longer the residence time, the higher the pollutant removal.

Using 2 x water quality volume to size the permanent pool means that smaller storms (1 x water quality volume) will displace only half of the pool volume of the wetlands, thus providing for extended residence times. Larger treatment volumes with respect to the watershed size (3 x water quality volume) will provide longer residence times and, therefore, greater efficiencies. In certain situations, using these larger volumes and efficiencies may be desirable, especially where downstream waters may be sensitive to pollutants from urban runoff. However, the challenge is to provide the recommended minimum surface area allocations for the different depth zones as previously discussed.

11.3.3.5 Inlet and Outlet Configuration

a) Inlet Design

When applying the design procedure, the following should be used as a guide (Brisbane City Council, 2005):

- The inlet zone typically must comprise of a deep open water body (1.5m to 2.0m) that operates essentially as a sedimentation basin designed to capture coarse to medium sized sediment
- It may be necessary for a GPT to be installed at the end of incoming waterway (or pipe), so that litter and large debris can be captured before entering the open water of the inlet zone. A floating skimmer installed at the downstream end of the forebay can be used instead to capture the floatable debris.
- The crest of the overflow pit must be set at the permanent pool level of the inlet zone (which is typically set 0.3m above the permanent water level of the macrophyte zone).
- The overflow pit and pipe that transfer flows from an inlet zone, or pond to the macrophyte zone controls the water level in the inlet pond and the maximum level flow rate that can reach the macrophyte zone. This structure needs to have sufficient capacity to convey a 1-year ARI flow, assuming the macrophyte zone is at the permanent pool level and without resulting in any flow over the high flow bypass weir.
- An energy dissipater is usually required at the end of the pipes to reduce velocity and distribute flow into the macrophyte zone.
- The inlet zone is to have a structural base (e.g. coarse rock or cobble) to define the base during removal of silt deposits in its bottom and provide support for maintenance plant/ machinery when entering the basin for maintenance.
- The high flow bypass weir (spillway outlet) is to be set at the same level as the top of extended detention in the macrophyte zone to enable stormwater to safely discharge from the inlet zone around the macrophyte zone in periods of high flow.
- The high flow bypass channel may be used to protect the wetlands from the large, infrequent flows, bypassing excess flows around the wetlands, as well as reducing the risk of damage from erosion, scour and re-mobilisation of pollutants. The design of the high flow bypass channel needs to be carefully considered to provide recreational and landscape opportunities during times outside of the above design flow events.

b) Outlet Design

The use of a multiple stage riser-type outlet is generally more suitable for controlling the water level regime in a wetlands than a weir because it gives more control over the stage-discharge relationship. However there is scope for the design of innovative outlet arrangements such as proportional weirs to suit Malaysian conditions. However, flow control from a stormwater wetlands is typically accomplished with the use of a concrete or corrugated metal riser and barrel.

The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (Figure 11.5). The riser should be located within the embankment for maintenance access, safety and aesthetics. A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of the pond design (Iowa Natural Resources Conservation Service, 2008).

In the case of an extended detention (ED) wetlands, there is generally a need for an additional outlet (usually an orifice). This additional outlet that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which shall be sized to release the water quality volume no less than 12 hours to 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 0.3m below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the water quality volume and is sized to release the channel protection storage volume. However, in most instances trash racks will be needed to avoid the orifice to be clogged by debris. The trash rack design should conform to the design criteria as shown in Chapter 7.

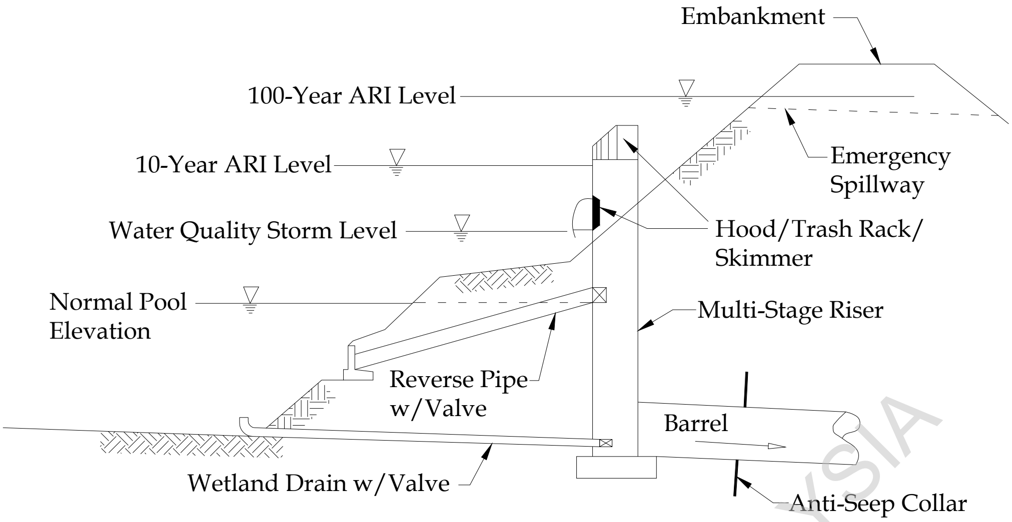


Figure 11.5 Typical Wetlands Facility Outlet Structure (Modified from MDE, 2000)

Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch or proportional weirs, or an outlet pipe protected by a hood that extends at least 0.3m below the normal pool level. After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars shall be installed on the outlet barrel to reduce the potential for pipe failure. Riprap, plunge pools or pads, or other energy dissipaters are to be placed at the outlet of the barrel to prevent scouring and erosion (Iowa Natural Resources Conservation Service, 2008).

If a wetlands facility discharges to a channel with dry weather flow, care should be taken to minimise tree clearing along the downstream channel, and to re-establish a forested riparian zone in the shortest possible distance. If a low flow discharge pipe is used, it should be constructed on a reverse slope and extended into the wetlands below the pool surface elevation but above the bottom elevation. This helps to prevent clogging, since a typical wetlands environment consists of floating plant debris and possible sediment and organic accumulation at the bottom. The wetlands facility must have a bottom drain pipe located in the open water zone with an adjustable valve that can completely or partially dewater the wetlands within 12 to 24 hours. The wetlands drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a hand wheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they will not normally be inundated and can be operated in a safe manner (Iowa Natural Resources Conservation Service, 2008).

c) *Flow Distribution*

Wetlands must be designed to provide an even flow distribution and avoid short-circuiting (direct flow from inlet to outlet). A long, narrow shape is recommended for this reason. Urbonas and Stahre (1993) proposed that the ideal shape is an extended oval, with inlet and outlet at opposite ends. Inlet structures should be designed to spread the flow as much as possible. This may involve providing flow baffles or a weir.

11.3.3.6 Spillway

Spillways are required to accommodate the design flood, whether or not the wetlands is intended to provide flood control. Spillways provide a controlled discharge, protecting the earth embankment from being overtopped and washed away by the flood.

For wetlands with large pool area, spillway safety is a major consideration and designs need to be checked for rare floods. In Malaysia, DID (1975) requires that all embankments be checked for stability during the inflow flood resulting from the Probable Maximum Precipitation (PMP) storm events.

An emergency spillway is to be included in the stormwater wetlands design to safely pass flows that exceed the largest design storm flows for that facility, whether this be the water quality design storm or the 100-year ARI if the wetlands basin is a multi-purpose stormwater facility. The spillway prevents the flood flows from overtopping the embankment and causing structural damage or catastrophic failure. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges. All wetlands should be provided with a spillway able to safely discharge at least the 100-year ARI flood. For on-line ponds, this will be the flood in the river after taking into account storage routing caused by the flood storage in the pond. For off-line ponds, the emergency spillway only need to cater all the diverted flows, including any that may be larger than the design ARI due to the capacity of the diversion system and actual diversions entering the diversion system, plus any local catchment inflows. One of the advantages of off-line facilities is that the emergency spillway requirements are much smaller and simpler than for in-line facilities.

If the wetlands is also intended to have a flood storage component, the form of the emergency spillway becomes important as it will affect the stage-discharge relationship. In this case the spillway should be designed as part of the normal outlet. A fixed weir set at the permanent pond water level provides a suitable outlet. The weir length should be computed so that it can discharge the design flood discharge allowing for storage routing, at the design flood storage level. Because the weir characteristic affects the storage routing, the design calculations must be checked using a storage-routing computer model.

In spillway design, it is also necessary to ensure that receiving channel downstream of the structure is sufficiently protected against the erosive flow from the spillway. Riprap lining or energy dissipaters may be necessary.

11.3.3.7 Embankment and Maintenance Access

The water-retaining embankments for the wetlands basin may have the characteristics of small dams, and should be designed as such. There are many different types of dams, including concrete, rockfill and earthfill. Earthfill dams are by far the most commonly used for detention facilities in urban development projects. Regardless of the size or location of a proposed facility, or the similarity to other projects nearby, the design of the pond or wetlands, and especially the embankment, should always be based on site-specific information.

11.3.3.8 Safety

Constructed wetlands need to conform to public safety requirements for new developments. Generally, the owner of the wetlands is responsible for ensuring that it does not pose risk to public health or safety. These include reasonable batter profiles for edges to facilitate public egress from areas with standing water and fencing or railings where water depths and edge profile requires physical barriers to deny public access.

Mosquito-borne diseases are a serious concern in tropical areas. The wetlands design should minimise the risk of mosquito breeding there. Mosquito control strategies include:

- Interception of water-borne rubbish which creates a mosquito breeding environment;
- Selection of plants which provide a breeding ground for predator insects, such as dragonflies that feed on mosquitoes;
- Varied depths as described in this chapter, to encourage breeding of fish and other predator species;
- Shaping of wetlands to avoid shallow stagnant areas with poor circulation;
- Shaping of wetlands edges to avoid the trapping of water in depressions as the pond water level changes;
- Providing a mechanism to regulate water levels in order to disturb any breeding larvae; and
- Selection and control of aquatic plants to avoid the creation of habitats favoured for mosquito breeding.

The pond itself can present a hazard to small children. The designer should concentrate on avoiding serious safety hazards such as:

- Sudden drops into deep water;
- Sudden changes in flow velocities or water levels; and
- Raised structures that children can fall off.

Inlet and outlet structures can be particularly dangerous because of the high flow velocity. It may be desirable to fence off the inlet and outlet structures. Such fencing should be designed so that it does not interfere with the hydraulics of the flow structure.

11.3.4 Design Procedure for Constructed Wetlands

The following design steps are recommended when designing wetlands:

Step 1: Confirm Treatment Performance of Concept Design.

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the required wetlands area (i.e. the macrophyte zone surface area) from the concept design is adequate to deliver the required level of stormwater quality improvement. This design process assumes a conceptual design has been undertaken. The pollutant reduction curve for wetlands as a function of catchment impervious area shown in Figure 3.1 can be used to undertake this verification check. These curves are intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough design process.

Step 2: Determine Design Flows.

A design operation flow (based on water quality storm) is required for sizing the inlet zone (i.e. sedimentation basin) and the control outlet structure (i.e. overflow pit and pipe connection) discharging to macrophyte zone.

<u>Water Quality Volume</u>

Next, the designer should calculate the WQV and set the WQV permanent pool elevation based on the volume calculated and the WQV water level.

Step 3: Designing the Inlet Zone.

The design of inlet zone applies the same processes as the procedure for design forebay or sediment basins. Inlet zone connection to the macrophyte zone normally consists of an overflow pit within the inlet zone connected to one or more pipes through the embankment separating the inlet zone and the macrophyte zone. The inlet zone shall be provided with a gross pollutant trap (GPT) to remove larger particles and debris including sediment.

Step 4: Designing the Macrophyte Zone.

<u>Length to Width Ratio and Hydraulic Efficiency</u>

To optimise wetlands performance, it is important to avoid short circuiting of flow paths and poorly mixed regions within the macrophyte zone. One way to minimise this is to adopt a high length to width ratio not less than 2 to 1 for the macrophyte zone. Length to width ratios less than this can lead to poor hydrodynamic conditions and reduced water quality treatment performance.

Designing the Macrophyte Zone Bathymetry

It is good design practice to provide a range of habitat areas within the macrophyte zone to support a variety of plant species (Refer Annex 1) and ecological niches to perform a range of treatment processes. The macrophyte zone therefore typically comprises four marsh zones (defined by water depth) and an open water zone. The bathymetry across the four marsh zones is to vary gradually ranging from 0.2m above the permanent pool level (i.e. ephemeral marsh) to a maximum of 1.2m below the permanent pool level (i.e. deep marsh).

Macrophyte Zone Edge Design for Safety

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety. It is recommended that a gentle slope to the water edge and extending below the water line be adopted before the batter slope steepens into deeper areas.

Macrophyte Zone Soil Testing

Constructed wetlands are permanent water bodies and therefore the soils in the base must be capable of retaining water. Geotechnical investigations of the suitability of the in-situ soils are required to establish the water holding capacity of the soils. Where the infiltration rates are too high for permanent water retention, tilling and compaction of in-situ soils may be sufficient to create a suitable base for the wetlands. Where in-situ soils are unsuitable for water retention, a compacted clay liner may be required (typically 300mm thick). Specialist geotechnical testing and advice must be sought.

Step 5: Design Open Water Zone Outlet.

An open water zone outlet has two purposes: (1) hydrologic control of the water level and flows in the macrophyte zone to achieve the design detention time; and (2) to allow the wetlands permanent pool to be drained for maintenance.

Outlet Works – Size and Location of Orifices

The riser outlet is designed to provide a uniform notional detention time in the open water zone over the full range of the extended detention depths. In the case of an extended detention wetlands, there is generally a need for an additional outlet that is sized to pass the water quality volume that is superimposed on top of the permanent pool no less than 12 hours to 24 hours.

Maintenance Drains

To allow access for maintenance, the wetlands should have appropriate allowance for draining. A maintenance drainage pipe should be provided that connects the low points in the macrophyte zone bathymetry to the macrophyte zone outlet. A valve is provided on the maintenance drainage pipe), which can be operated manually. The maintenance drainage pipe should be sized to draw down the permanent pool within 12 hours. If a weir plate is used as a riser outlet, provision should be made to remove the weir plate and allow drainage for maintenance.

Discharge Pipe

The discharge pipe of the wetlands conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). The conveyance capacity of the discharge pipe is to be sized to match the higher of the two discharges (i.e. maximum discharge from the riser or the maximum discharge from the maintenance drain).

Step 6: Design Embankment(s), Spillway and High Flow Bypass Channel.

Designers must next size an emergency spillway, and calculate the peak major storm water surface elevation. They must then set the top of the embankment elevation.

The bypass channel accepts 'above design flow' from the inlet zone of the wetlands via the bypass weir and conveys these flows downstream around the macrophyte zone of the wetlands. The bypass channel should be designed using standard methods (i.e. Manning's Equation) to convey the 'above design flow' and to avoid bed and bank erosion.

Step 7: Specify Vegetation.

Refer to Annex 1 for advice on selecting suitable plant species for wetlands.

Step 8: Design Calculation Summary.

Prepare a design calculation summary sheet and checklist for the key design elements.

REFERENCES

1. Brisbane City Council (2005). *Water Sensitive Urban Design Engineering Guidelines: Stormwater.* Draft, Brisbane City Council, Brisbane, Australia.
2. Construction Industry Research and Information Association, CIRIA. (2007). *The SUDS Manual*. CIRIA Report C697. London, UK.
3. Drainage and Irrigation Division, DID (1975). *Urban Drainage Design Standards and Procedures for Peninsular Malaysia: Planning and Design Procedure No. 1.* Ministry of Agriculture, Malaysia.
4. Environmental Protection Agency, EPA (1983). *Results of the Nationwide Urban Runoff Program*. Final Report, U.S. Environmental Protection Agency, NTIS No. PB84-185545, Washington DC, USA.
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6. Maryland Department of the Environment, MDE. (2000). *Maryland Stormwater Design Manual*. Baltimore, MD, USA.
7. Iowa Natural Resources Conservation Service. (2008). *Iowa Stormwater Management Manual*. Iowa State University, Center for Transportation Research and Education. Ames, USA.
8. Urbonas, B.R. and Stahre, P. (1993). *Stormwater - Best Management Practices Including Detention*. Prentice Hall, Englewood Cliffs, N.J., USA.
9. Urban Drainage and Flood Control District, UDFCD (2008). *Urban Storm Drainage Criteria Manual (USDCM)*. Denver, Colorado, USA.

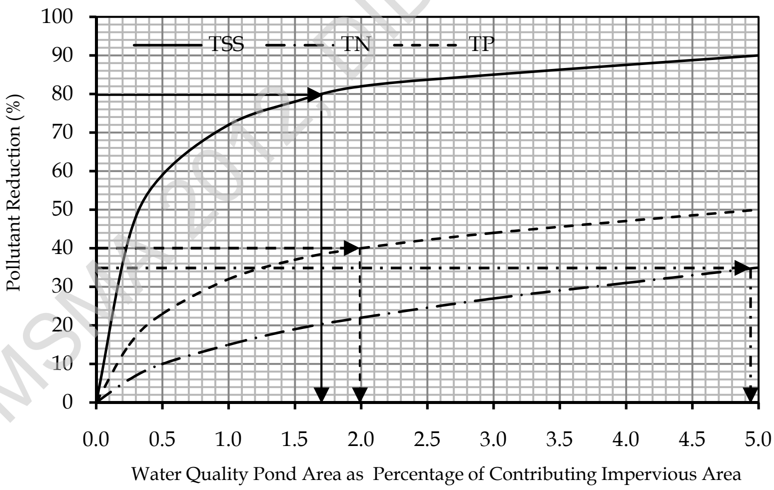
APPENDIX 11.A EXAMPLE – WATER QUALITY POND

Problem:

A water quality pond is proposed for the new development of 2.5 hectare college in Nibong Tebal, Pulau Pinang based on the following assumptions:

- average runoff coefficient, C_{ave} was estimated to be 0.45 based on the imperviousness of the mixed development area
- fraction of impervious area, f_{imp} is estimated to be 0.65

Solution:

Reference	Calculation	Output
Relevant Layout Plan	Development Project Area	
	Catchment area, *A* =	2.5ha
	Confirm treatment performance of design concept	
Section 11.3.3.4 (a)	Reduction of total suspended solids, TSS =	80%
Table 1.4	Reduction of total nitrogen, TN =	35%
	Reduction of total phosphorus, TP =	40%
Figure 3.1(b)	Determine required water quality pond area ratio for the targeted pollutant removal efficiencies	
		
	TSS: 80% removal corresponds to an area ratio of 1.70%. TN: 35% removal corresponds to an area ratio of 4.95%. TP: 40% removal corresponds to an area ratio of 2.00%.	
	The largest ratio must be selected therefore adopt an area ratio =	4.95%
	Calculate required water quality pond size knowing the catchment area	
	The catchment area =	2.50ha
	Contributing impervious area = f_{imp} x *A* = 0.65 x 2.5 =	1.625ha

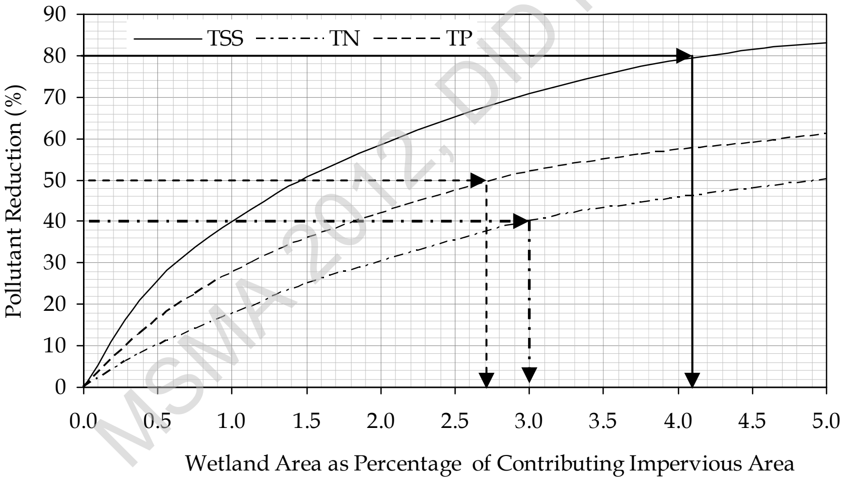
Reference	Calculation	Output
	where: f_{imp} = Fraction of impervious area A = Catchment area	
	The required water quality pond area = $1.625 \times \left(\frac{4.95}{100}\right)$	= 0.080438ha = 804.38m^2
	Determine WQV	
Equation 11.1	Required storage for WQV = C_{ave} (P_d) A = 0.45 x 0.040 x 25000	= 450m^3
Section 2.3.1.1 11.2.3.2	where: WQV = Water quality volume (m^3); C_{ave} = Average rational runoff coefficient; P_d = Rainfall depth for water quality design storm (= 40 mm); A = Contributing drainage area (m^2).	
	Designing Water Quality Pond	
	Width, W	= 20m
	Length, L	= 60m
	Surface area	= 1200m^2 (>804.38m^2)
Section 11.2.3.1	Length to width ratio	3:1
	Extended Detention Depth:	
	Depth (Above the permanent pool water level)	= 0.4m
	Side slope	= 1:3
	Water quality pond volume = 0.5 x (20+22.4) x 0.5 x (60+62.4) x 0.4	= 518.98m^3 (>450m^3)
	Option 1: Wet Type Water Quality Pond	
Section 11.2.1.4(b)	Drawdown time	= 12-Hour
Equation 11.2	$Q_{ave} = \frac{\text{Required Storage for WQV } (m^3)}{\text{Drawdown Time (s)}} = \frac{450}{12\times60\times60}$	= 0.0104m^3/s
	Outlet works	
	Outlet sized to discharge the WQV	= orifice
Equation 2.6	$Q = C_o A_o \sqrt{2gH_o}$	
	where, Q = The orifice flow rate (m^3/s); C_o = Orifice discharge coefficient (= 0.60);	

Reference	Calculation	Output
	A_o = Area of orifice (m^2), $\pi D_o^2/4$; D_o = Orifice diameter (m); H_o = Effective head on the orifice measured from the centre of the opening (m); and g = Acceleration due to gravity ($9.81 m/s^2$)	
	Say, Orifice diameter, D_o = =	0.09m 90mm
	Effective head on the orifice measured from the centre of the opening, H_o =	0.355m
	$Q = C_o A_o \sqrt{2gH_o} = 0.60 \times \frac{\pi(0.09)^2}{4} \sqrt{2g(0.355)}$ =	$0.010 m^3/s$ ($<0.0104 m^3/s$)
	Option 2: Dry Type Water Quality Pond	
Section 11.2.1.4(a)	Drawdown time =	24-Hour
Equation 11.2	$Q_{ave} = \frac{\text{Required Storage for WQV } (m^3)}{\text{Drawdown Time (s)}} = \frac{450}{24\times60\times60}$ =	$0.00521 m^3/s$
	Outlet works	
	Say, Orifice diameter, D_o = =	0.06m 60mm
	Effective head on the orifice measured from the centre of the opening, H_o =	0.37m
	$Q = C_o A_o \sqrt{2gH_o} = 0.60 \times \frac{\pi(0.06)^2}{4} \sqrt{2g(0.37)}$ =	$0.00457 m^3/s$ ($<0.00521 m^3/s$)
Section 7.4.3	Trash Racks	
	Provide trash racks of sufficient size that do not interfere with the hydraulic capacity of the outlet. Refer Chapter 7 for minimum trash rack sizes.	
	Design for Safety	
	Embankment height =	0.6m
	Side Slope =	1 : 4
	An emergency spillway is to be included in the water quality pond design to safely pass the flow larger than water quality design storm. The spillway prevents pond water levels from overtopping the embankment and causing structural damage.	

APPENDIX 11.B EXAMPLE – WETLANDS

Problem:

A constructed wetlands is proposed with the Bio-Ecological Drainage System (BIOECODS) for the new development of engineering campus, Universiti Sains Malaysia in Nibong Tebal, Pulau Pinang. The campus area is 320 acre. However, the total contributing drainage area is 250 acre.

Reference	Calculation	Output
Relevant Layout Plan	Development Project Area	
	Total contributing drainage area, A =	250 acre
	=	1,011,714.11m^2
	Confirm treatment performance of design concept	
11.3.3.4 (a)	Reduction of total suspended solids, TSS =	80%
Table 1.4	Reduction of total nitrogen, TN =	40%
	Reduction of total phosphorus, TP =	50%
Figure 3.1(e)	Determine required wetland area ratio for the targeted pollutant removal efficiencies	
		
	TSS: 80% removal corresponds to an area ratio of 4.10%	
	TN: 40% removal corresponds to an area ratio of 3.00%	
	TP: 50% removal corresponds to an area ratio of 2.70%	
	The largest ratio must be selected therefore adopt an area ratio =	4.10%
	Calculate required wetland size knowing the catchment area	
	Total contributing drainage catchment area, *A* =	1,011,714.11m^2
	Contributing impervious area = $f_{imp} \times A$ = 0.30 x 1,011,714.11 =	303,514.23m^2
	Where, f_{imp} = Fraction of impervious area *A* = Catchment area	

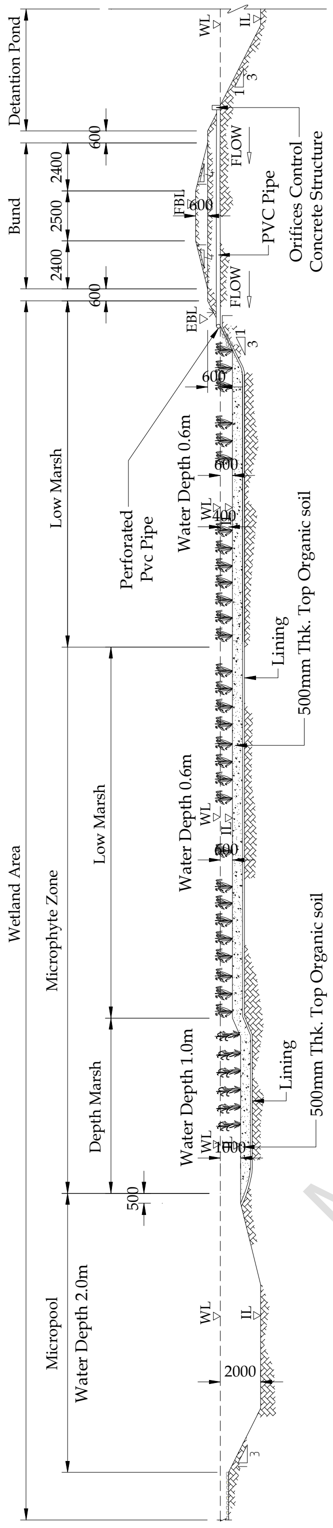
<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td><table>
<tr><th>Catchment Characteristic</th><th>Area (m²)</th><th>Area (%)</th><th>f_{imp}</th></tr>
<tr><td>Buildings/Lots, Road/Footpath</td><td>202,342.82</td><td>20.00</td><td>0.70</td></tr>
<tr><td>Green Area</td><td>809,371.29</td><td>80.00</td><td>0.20</td></tr>
<tr><td>Total</td><td>1,011,714.11</td><td>100.00</td><td>-</td></tr>
</table></td><td></td></tr>
<tr><td></td><td>The required wetland area = $303514.23\times\left(\frac{4.10}{100}\right)$ =</td><td>12,444.08m²</td></tr>
<tr><td>Section 11.3.3.2</td><td>Allocation of Surface Area
Inlet zone – Not available since design as off-line wetland. Detention pond can design as sediment forebay to reduce incoming sediment stormwater flow before runoff enter wetland.</td><td></td></tr>
<tr><td></td><td>Macrophyte Zone = 80% x 12,444.08m² =</td><td>9,955.26m²</td></tr>
<tr><td></td><td>Open Water Zone (Micropool) = 20% x 12,444.08m² =</td><td>2,488.82m²</td></tr>
<tr><td></td><td>Determine Design Flows</td><td></td></tr>
<tr><td></td><td>Design inflow rate (water quality storm), Q_{peak} =
(Routing result from detention pond)</td><td>0.275m³/s</td></tr>
<tr><td></td><td>Determine WQV</td><td></td></tr>
<tr><td>Equation 11.1</td><td>Required storage for $WQV = C_{ave}\ (P_d)\ A$
$= 0.35 \times 0.040 \times 1,011,714.11$ =</td><td>14,164.00m³</td></tr>
<tr><td>Section
2.3.1.1
11.2.3.2</td><td>where:
WQV = Water quality volume (m³);
C_{ave} = Average rational runoff coefficient;
P_d = Rainfall depth for water quality design storm (= 40mm);
A = Contributing drainage area (m²).</td><td></td></tr>
<tr><td></td><td>Designing wetland and bathymetry</td><td></td></tr>
<tr><td></td><td>Length to Width Ratio:</td><td></td></tr>
<tr><td></td><td>Width, W =</td><td>50m</td></tr>
<tr><td></td><td>Length, L =</td><td>250m</td></tr>
<tr><td></td><td>Surface area =</td><td>12,500m²
(>12,444.08m²)</td></tr>
<tr><td>11.3.3.3</td><td>Length to width ratio =</td><td>5:1</td></tr>
<tr><td></td><td>Extended Detention Depth:</td><td></td></tr>
<tr><td></td><td>Depth (Above the permanent pool water level) =</td><td>1.2m</td></tr>
<tr><td></td><td>Side slope =</td><td>1:3</td></tr>
<tr><td></td><td>Wetland volume = 0.5 x (50+57.20) x 0.5 x (250+257.20) x 1.2 =</td><td>16,311.55m³
(>14,164.00m³)</td></tr>
</table>

Reference	Calculation	Output
	<u>Designing macrophyte zone</u>	
	Length to width ratio and hydraulic efficiency:	
	Width, W =	50m
	Length, L =	200m
	Surface area =	10,000m^2 (>9,955.26m^2)
	Length to width ratio =	4:1
	Designing the macrophyte zone bathymetry:	
	(i) Area: =	10, 000m^2
	Deep marsh =	900m^2 (9.0%)
	Low marsh =	6,700m^2 (67.0%)
	High marsh =	2,400m^2 (24.0%)
	(ii) Depth:	
	Deep marsh =	1.0m
	Low marsh =	0.6m
	High marsh =	0.3m
	Macrophyte zone edge design for safety requirements:	
	Embankment height =	0.6m
	Side slope =	1 : 3
	Macrophyte zone soil testing:	
	Provide compacted clay liner (pea gravel and soil mixture) as specified by engineer	
	Vegetation specifications:	
Annex 1 Table 11.2		

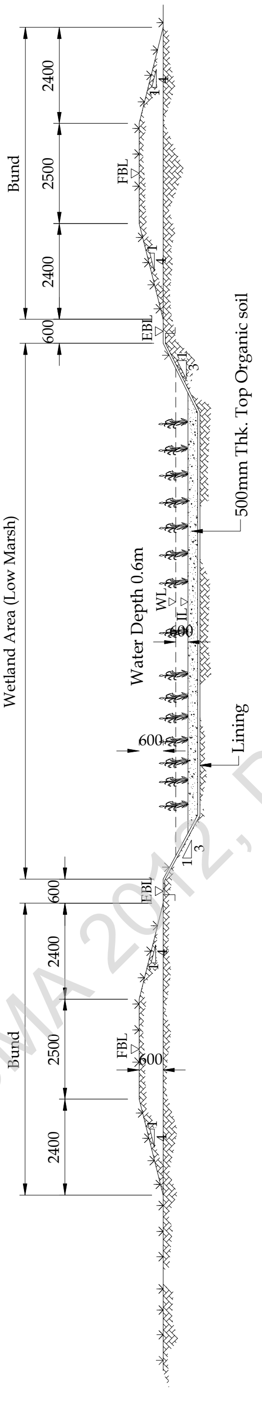
Zone	Water Depth (m)	Plant Species	Planting Density (plant/m^2)
High marsh zone	0.3	Eleocharis Variegata	10
High marsh zone	0.3	Eleocharis Dulchis	10
High marsh zone	0.3	Hanguana Malayana	10
Low marsh zone	0.6	Lepironia Articulata	6
Low marsh zone	0.6	Typha Augustifolia	6
Deep mash zone	1.0	Phragmites Karka	4

Reference	Calculation	Output
	<u>Designing open water zone</u>	
	(i) Micro pool area =	2,500m^2 (>2,488.82m^2)
	(ii) Micro pool depth =	2.0m
	Outlet works	
	After considering design concept and requirements, several factors and constraints of the application Bio-Ecological Drainage System (BIOECODS) in the Enginnering Campus, USM, to allow for tidal effects, one unit of check valve is incorporated into a concrete structure proposed at the recreational pond located downstream of the wetland. The check valve is approximately 0.6 m in diameter.	

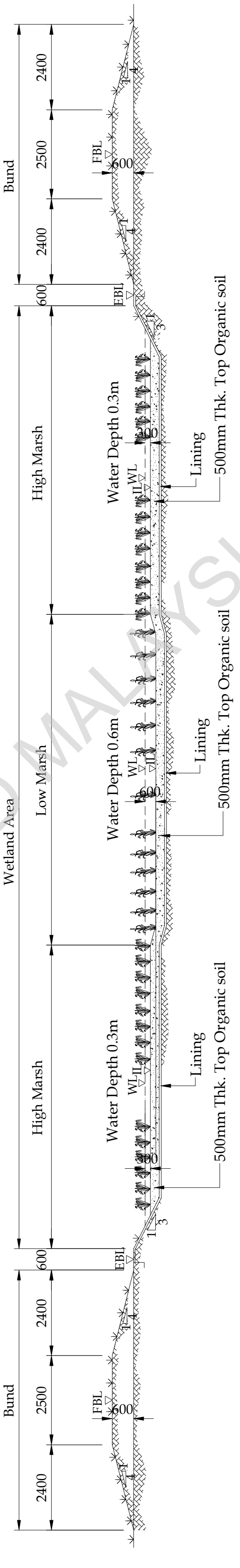
Layout



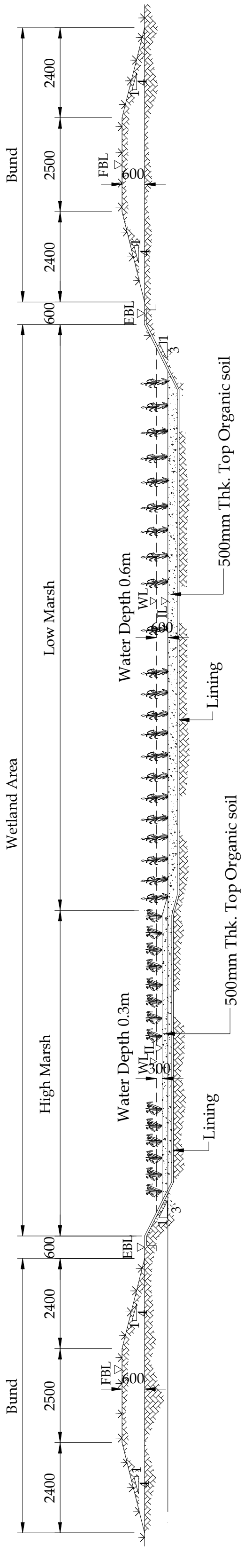
Section A - A



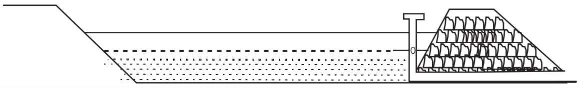
Section B - B



Section C - C



Section View



EROSION AND SEDIMENT CONTROL 12

CHAPTER 12 EROSION AND SEDIMENT CONTROL

12.1 INTRODUCTION

Uncontrolled land disturbance, deforestation and construction activities, exposed to rainfall and runoff, cause excessive erosion and sedimentation particularly in tropical environment including Malaysia. The effect has been in obvious water quality deterioration of a number of watercourses and receiving waters due to severe siltation. Therefore, comprehensive control is required to regulate and manage the processes and to mitigate the impacts of erosion and sedimentation.

This Chapter provides relevant guidance and procedures for reduction and control of erosion and sedimentation so that developmental/construction activities can be planned and executed in a judicious manner with minimum land, water quality and environmental degradation. The contents include preventive methods, micro model-based erosion estimates, BMPs design procedures and control plan preparation.

The amount of soil eroded from land development and sediment delivered to waterways is very large, majority transported to the sea, while the rest deposited in floodplains, rivers, lakes and reservoirs. Sediment is always the number one pollutant in waterways, where siltation and nutrients impair more kilometres of rivers and streams than any other pollutant. The economic costs of erosion and sedimentation are substantial, from resource damages on land and receiving waters as well as money spent for removal and/or dredging works (Gray & Sotir, 1996).

12.2 EROSION AND SEDIMENTATION PROCESSES

Soil erosion or surficial erosion is the detachment, entrainment, and transport of soil particles from ongoing land development and construction areas by the rainfall and runoff activities. Erosion occurs starting with raindrop splash. At the onset of runoff sheet, water collects into small rivulets, which may erode very small channels called rills. These rills may eventually coalesce into larger and deeper channels called gullies (Figure 12.1).

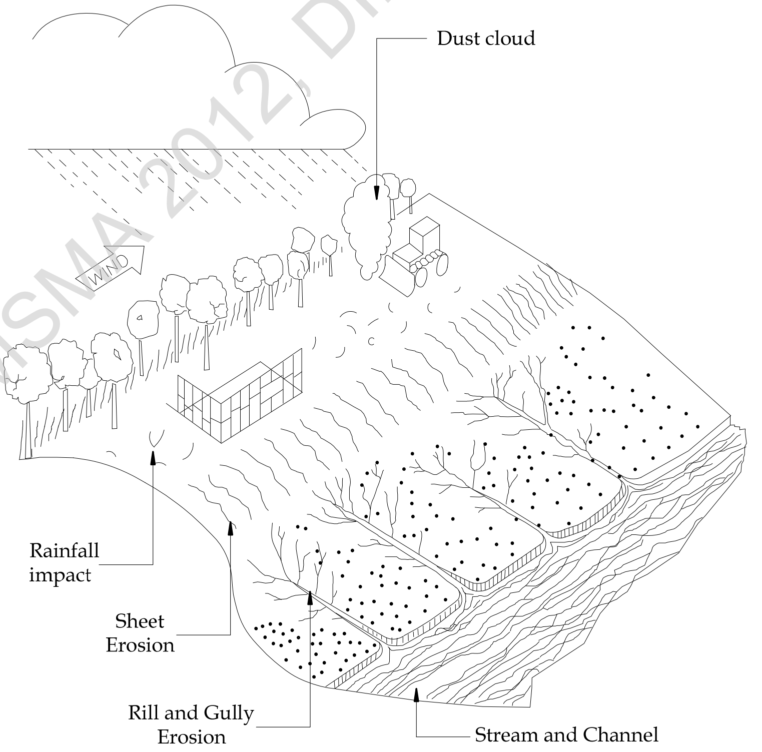


Figure 12.1: Types of Erosion Occurring at a Construction Site (CDM, 1993)

The mechanics of the erosion process is shown Figure 12-2 (Gray & Sotir, 1996). Drag or tractive forces exerted by the flowing stormwater runoff are resisted by inertia or cohesive forces between particles. The forces are measured by water velocity, discharge and soil particle shape and roughness. Erosion is initiated by drag, impact, or tractive forces acting on soil particles.

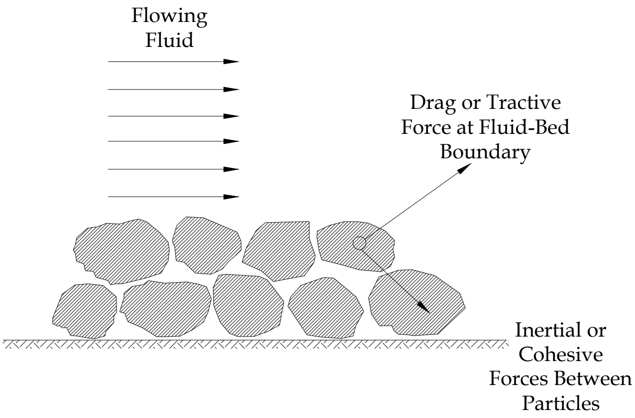


Figure 12.2: Mechanic of Erosion

Based on very limited database, it shows that the erosion rates from forest and agricultural land in a typical Malaysian tropic catchment are roughly 0.5 and 25 tons/ha/yr, while the erosion rates from construction sites have wider variations, ranging from approximately 1,000 to 10,000 tons/ha/yr. It is interesting to note that soil erosion rates from the tropical catchment in Malaysia is in the order of 10 to 20 times greater than those from catchments in the temperate climate (metropolitan areas of Washington D.C. and Baltimore, Maryland) (Chen, 1990). By comparing typical distribution patterns of rainfall intensities for tropical and temperate rainfall, it shows that the erosive power of the tropical rain is as much as 16 times greater than that of the temperate rain (Hudson, 1971).

Sedimentation is the build-up (aggradation) of sediment on the land surface or the bed of receiving waters. . Sedimentation leads to the rising of bed levels contributing to increased floods levels and escalates the destruction of aquatic habitats and fisheries (Figure 12.3). It is a dynamic process and is dependent upon the geomorphic and hydraulic characteristics of the drainage system and the nature of the receiving water body. The deposited sediment tends to remain in place sometimes for short periods of time, where subsequent rainstorms flush the sediment downstream and sometimes for very long times, the later being the case in estuaries and lakes. Sediment tends to be transported in pulses depending on the flow characteristics of the drainage systems.

a) On Flood Plain

b) In Receiving Waters

Figure 12.3: Impacts of Sedimentation

12.3 EROSION AND SEDIMENT CONTROL PRINCIPLES

There are primarily eight (8) principles of erosion and sediment control recommended (Figure 12.4). The following sub-sections provide detailed descriptions of these principles.

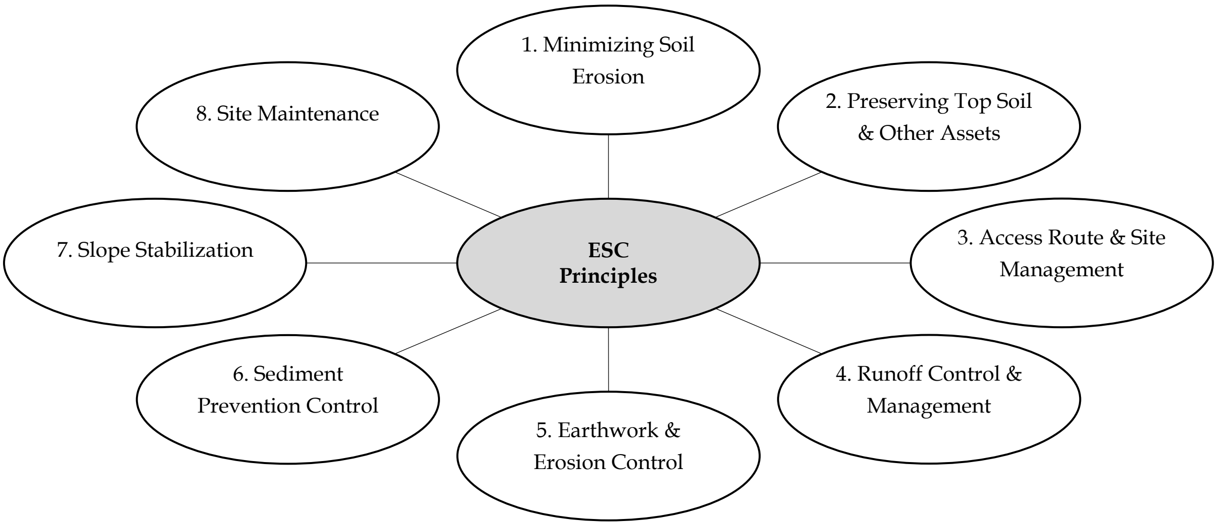


Figure 12.4: Principles of Erosion and Sediment Control

12.3.1 Minimising Soil Erosion

- Before development begins, preventative measures to minimise erosion shall be planned through the preparation of an ESCP.
- The working area for various facilities within a development site shall be reduced to less than twice the plan area of all buildings.
- The contractor shall regulate phases of earthwork and construction through proper development scheduling. All clearing, grading, and stabilisation operations shall be completed before moving onto the next phase. Major earthwork shall be scheduled to avoid wet seasons (monsoon).
- Existing vegetation shall be maintained to the maximum extent possible to filter runoff and provide erosion protection.
- Stream buffers shall be established, and natural waterway reserves should be delineated as recommended by Department of Irrigation and Drainage (DID) Malaysia.

12.3.2 Preserving Topsoil and Other Assets

- Sensitive ecological areas within a development site such as wetlands, natural springs, etc. shall be demarcated and preserved.
- All excavated topsoil shall be stockpiled, protected from erosion, and later used for revegetation.
- Vegetation of high ecological or social value should be identified, protected, and if required, transplanted.

12.3.3 Access Routes and Site Management

- All right-of-ways or access routes shall be shown on the ESCP and it shall be the responsibility of the developer to ensure that all vehicular traffic stays within the designated rights-of-way.
- Vehicle movements on access roads should be kept to a minimum with other areas off-limits to traffic.
- All movements of vehicles over unpaved areas should be kept to a minimum.
- Unpaved roads shall be sprayed with water to reduce dust pollution during dry periods.
- Access roads to the site shall be paved for at least 10 m into the site from any existing paved roads.
- Washing bays shall be provided to remove excessive sediment from out bound vehicles at all site access points.

12.3.4 Drainage Control and Runoff Management

Concentrated runoff often causes more erosion than sheet flows due to higher shear stress. Erosion control measures in locations of concentrated flow can have a major effect in reducing the risk of downstream sedimentation. Hence, proper runoff management during earthworks and construction can contribute greatly to erosion and sediment control. The following practices shall be followed:

- The main principle for establishing a good temporary drainage system in development sites is to direct runoff water so that it does not run across disturbed and unstable areas.
- Runoff from undisturbed areas and natural watercourses shall be diverted away from disturbed land using runoff management best management practices (BMPs) such as earth banks and diversion drains.
- Runoff from disturbed area shall be collected by a temporary (or permanent) drainage system and treated (using available sediment control BMPs), before being released (complying with Department of Environment (DOE) Malaysia environmental regulations) into natural watercourses.
- Temporary drainage system should be designed such that the system does not contribute to the sedimentation problems (stable channel design).
- No watercourse or reserve along a watercourse shall be disturbed until full plan details of the proposed works have been submitted to and approved by DID.
- Ineffective drainage controls shall be noted (especially during wet weather) and promptly corrected.

12.3.5 Earthworks and Erosion Control

- Generally, slopes should not be steeper than 2(H):1(V), or as designed by an experienced geotechnical engineer through slope stability analysis.
- All earthworks shall be stabilised as early as possible to minimize the rates of soil erosion using structural stabilisation or erosion control BMPs.
- Inactive working areas (areas not anticipating any construction activity in upcoming 30 days) shall be stabilised within 7 days with proper stabilization techniques.

12.3.6 Sediment Prevention and Control

- Sediment control BMPs shall be designed and provided at all construction/earthwork sites. These BMPs function by creating impoundments, resulting in sediment settling out from runoff.
- Permanent water quality control measures such as ponds and gross pollutant traps can be constructed and temporarily used as sediment basins, provided they are satisfactorily maintained and cleaned out after development to ensure efficient operation as designed.
- Sediment traps and other temporary control measures should only be removed and dismantled when the permanent vegetative cover and control measures are satisfactorily established.

12.3.7 Slope Stabilisation

- All critical areas along streams must be marked on the ESCP and the recommended methods of stabilisation indicated.
- There shall be no obstruction or interference with natural waterways. Where a road is to be cut across a river or stream, bridges and culverts as prescribed by the enforcement authority shall be constructed and maintained according to specifications.
- For hilly land (greater than 12° or 20%) terracing shall be built and maintained. Cover plants shall be established on the slopes of the platforms and walls of the terrace immediately after commencement of earthworks.
- Slope steeper than 35° or 70% shall not be worked and should instead be identified, stabilised and maintained.

12.3.8 Site Maintenance

- A maintenance programme shall be prepared to include plans for the removal and disposal of unwanted sediments, the repair of structural damages, and improvement or modification of BMPs (based on engineer's recommendation).
- Regular inspections should also be planned for on a fixed interval as well as before and after each storm event.
- All erosion and sediment control measures shall be constructed and maintained by the developer.
- Final discharge(s) from the development site shall comply with ambient standards for TSS (150 mg/l and below (DOE, 1996)) and turbidity for the designated beneficial use of the receiving water.

12.4 DESIGN GUIDELINES FOR EROSION AND SEDIMENT CONTROL BMPs

This section describes specific erosion and sediment best management practices (BMPs) for common construction activities. These BMPs can be further categorized into three types, i.e. erosion control BMPs, runoff management BMPs and sediment control BMPs, based on the natural functions and design objectives of each facility.

Erosion control BMPs emphasise the provision of cover protection to soil, while runoff management BMPs are temporary facilities provided to minimise channel erosion at construction sites. Over time, excessive sediments will be produced downstream of construction sites and they will have to be trapped by providing sediment control BMPs. The design of these BMPs as part of a functioning ESCP will help to ensure that runoff discharges from construction sites have minimal effects on natural watercourses.

While erosion control BMPs can be applied without detailed design, runoff management BMPs and sediment control BMPs shall be properly designed to ensure the facilities provided are able to cope with the on-site demand. This section introduces the Universal Soil Loss Equation (USLE) to be used to assess site erosion risk for preparation of an ESCP, and the Modified Universal Soil Loss Equation (MUSLE) to be used to estimate load for BMPs design.

12.4.1 Soil Loss and Sediment Yield Estimation

12.4.1.1 Soil Loss

The Universal Soil Loss Equation (USLE) is a semi-empirical equation used to assess soil losses (sheet & rill erosion) under different cropping systems and land management practices (Wischmeier and Smith, 1978). The USLE is given as,

$$A=R.K.LS.C.P \qquad (12.1)$$

where,

A = Annual soil loss, in tonnes/ha/year;

R = Rainfall erosivity factor, an erosion index for the given storm period in MJ.mm/ha/h;

K = Soil erodibility factor, the erosion rate for a specific soil in continuous fallow condition on a 9% slope having a length of 22.1 m in tonnes/ha/(MJ.mm/ha/h);

LS = Topographic factor, which represents the slope length and slope steepness. It is the ratio of soil loss from a specific site to that from a unit site having the 9% slope with a slope length of 22.1 m when other parameters are held constant;

C = Cover Management factor, which represents the protective coverage of canopy and organic material in direct contact with the ground. It is measured as the ratio of soil loss from land cropped under specific conditions to the corresponding loss from tilled land under clean-tilled continuous fallow (bare soil) conditions. This is the factor that indicates the effect of erosion control facilities in an ESCP; and

P = Support practice factor, which represents the soil conservation operations or other measures that control erosion, such as contour farming, terraces, and strip cropping. It is expressed as the ratio of soil loss with a specific support practice to the corresponding loss with up and-down slope culture. This is also the factor that indicates the effect of sedimentation control facilities in an ESCP.

This equation produces the annual soil loss (erosion rate) of a site, and hence is very useful in assessing erosion risk of a development site. By assessing different site conditions (pre-, during, and post- construction), Engineer can quantify how erosion risk changes throughout the course of development, and consequently design proper BMPs to reduce soil loss. Using the same equation, an engineer is able to predict the effect of BMPs implementation as well. The assessment of soil loss is required and should be documented in a written report, submitted as part of an ESCP for approval. An example of calculations using this equation is given in Appendix 12.C.

(a) *Rainfall Erosivity Factor (R)*

The rainfall erosivity factor relates soil loss to rainfall parameters. When other functions are held constant, soil losses are directly proportional to a rainstorm parameter: The total storm energy (E) times the maximum 30-minute intensity (I_{30}). The average annual total of the storm (EI_{30}) in a particular locality is represented by the rainfall erosivity factor, R, for that locality. The relation is expressed in Equation (12.2). Detailed calculations of R factors can be found in *Guideline for Erosion and Sediment Control in Malaysia* (DID, 2010).

$$R = \frac{1}{n}\sum_{j=1}^{n}\left[\sum_{k=1}^{m}(E)(I_{30})_k\right] \qquad (12.2)$$

where;

E = Total storm kinetic energy (MJ/ha);

I_{30} = Maximum 30 minute rainfall intensity;

j = Index for the number of years used to compute the average;

k = Index of the number of storms in each year;

n = Number of years to obtain average R; and

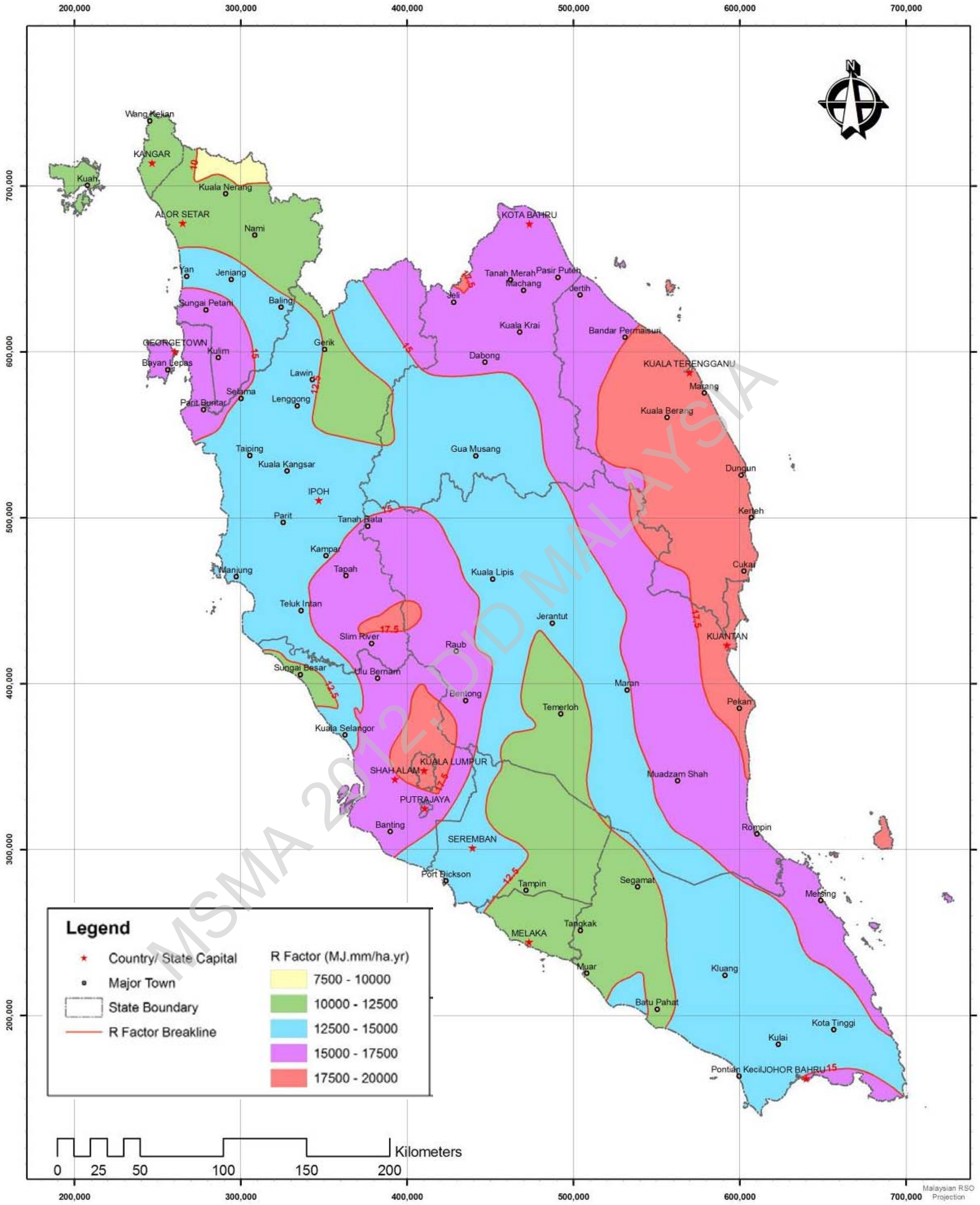
m = Number of storms in each year.

Hourly rainfall data from 241 rain gauge stations for the past 10 years were used to produce R factors for major locations in Peninsular Malaysia. The isoerodent map is given in Figure 12.5. For East Malaysia, the maximum R Factor, 20,000 MJ.mm/ha/yr shall be adopted. Soil loss prediction for a fraction of a year can be estimated using a Monthly Modification Factor (M) given in Table 12.1 below.

Table 12.1: Modification Factor (M) for Regions in Peninsular Malaysia

Region*	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Northern	0.030	0.033	0.073	0.100	0.070	0.077	0.083	0.087	0.123	0.163	0.097	0.063
Central	0.064	0.054	0.088	0.102	0.070	0.066	0.074	0.064	0.084	0.114	0.130	0.090
Eastern	0.085	0.045	0.055	0.050	0.070	0.055	0.055	0.070	0.085	0.100	0.140	0.190
Southern	0.120	0.045	0.090	0.075	0.065	0.060	0.060	0.060	0.075	0.095	0.110	0.145

*Note: Northern Region - Perlis, Kedah, Pulau Pinang
Central Region - Perak, Selangor, Wilayah Persekutuan Kuala Lumpur, Negeri Sembilan, Melaka
Eastern Region - Terengganu, Kelantan
Southern Region - Johor, Pahang



* Note: The highest value found in Peninsular Malaysia (20,000) shall be applicable for the entire East Malaysia.

Figure 12.5: Rainfall Erosivity Map for Peninsular Malaysia

(b) Soil Erodibility Factor (K)

Soil erodibility defines the resistance of the soil to both detachment and transport. It is an important index to measure soil susceptibility to water erosion, and an essential parameter for soil erosion prediction. The soil-erodibility factor (*K*) represents the effect of soil properties and soil profile characteristics such as soil texture, aggregate stability, shear strength, infiltration capacity, and organic and chemical contents on soil loss.

Equation 12.3 (Tew, 1999) has been found to give the most satisfactorily estimation of *K* factors for Malaysian soil series, and is therefore recommended for the calculation of *K* factors in Malaysia. Percentage of soil particle class can be obtained through standard laboratory analysis of site samples, while soil structure code and permeability class can be obtained from Table 12.2.

$$.K=\left[10\times10^{-4}\left(12\text{-}OM\right)M^{1.14}+4.5\left(S\text{-}3\right)+8.0\left(P\text{-}2\right)\right]/100 \qquad (12.3)$$

where,

M = (% silt + % very fine sand) x (100 - % clay);
OM = % of organic matter;
S = soil structure code; and
P = permeability class.

Table 12.2: Soil Structure Code and Permeability Class for Various Soil Textures

Soil Texture	Permeability Code[1]	Hydrologic Soil Group[2]	Soil Structure Code[3]
Heavy Clay	6	D	4
Clay	6	D	4
Silty Clay Loam	5	C	4
Sandy Clay	5	C	4
Sandy Clay Loam	4	C	4
Clay Loam	4	C	4
Loam	3	B	2
Silty Loam	3	B	3
Loamy Sand	2	A	1
Sandy Loam	2	A	2
Sand	1	A	1

Note: 1 – National Soil Survey Handbook (NRCS, 2005)
2 – National Engineering Handbook (NRCS, 2007)
3 – Field Manual for Describing Soils in Ontario (Ontario Centre for Soil Resource Evaluation, 1993)

(c) Slope Length and Steepness Factor (LS)

The rate of soil erosion by water is very much affected by both slope length (*L*) and slope steepness (*S*) in terms of gradient/ percent slope. By definition, the factor *L* is a ratio of field soil loss to that from a 72.6-foot slope, the value of L may be expressed as $(\lambda/\psi)^m$, where λ is the field slope length in feet (or m), Ψ is 72.6-foot (or 22.13 m) and the exponent *m* in this expression is not the same for all location. The slope-steepness factor (*S*) reflects the influence of slope gradient on erosion. It is the ratio of soil loss from the field slope gradient to that from a 9% slope under otherwise identical conditions. *LS* factor can be obtained directly from Table 12.3. Linear interpolation is allowed for values obtained from the table.

(d) Cover Management (C) and Practice Support (P) Factors

Crop Management, *C* factor and Practice Support, *P* factor are two management factors that can be used to control soil loss at a specific site. The *C* factors, which represent various types of covers introduced to protect bare ground from rain splash and sheet erosion, are important to reduce soil erosion at a construction site or disturbed land. If erosion has already taken place then the *P* factor is needed to stop the silt and sediment in flowing water from running off the site. Combining both the techniques for *C* and *P* factors, it is possible to minimize erosion at a construction site and reduce sediment loading to downstream receiving water bodies.

Recommended *C* and *P* values of commonly found erosion (normally associated with *C* factor) and sediment (normally associated with *P* factor) control BMPs are given in Tables 12.4 and 12.5, respectively.

Table 12.3: *LS* Factor for Various Slopes and Slope Lengths

Slope Steepness, *S* (%)	Slope Length, λ (m)											
	2	5	10	15	25	50	75	100	150	200	250	300
0.1	0.043	0.052	0.059	0.064	0.071	0.082	0.089	0.094	0.102	0.108	0.113	0.117
0.5	0.055	0.067	0.076	0.083	0.092	0.106	0.114	0.121	0.131	0.139	0.146	0.151
1.0	0.057	0.075	0.093	0.105	0.122	0.150	0.170	0.185	0.209	0.228	0.243	0.257
2.0	0.089	0.117	0.144	0.163	0.190	0.234	0.264	0.288	0.325	0.354	0.379	0.400
3.0	0.100	0.144	0.190	0.224	0.275	0.362	0.426	0.478	0.563	0.631	0.690	0.742
4.0	0.135	0.195	0.257	0.302	0.371	0.489	0.575	0.646	0.759	0.852	0.932	1.002
5.0	0.138	0.218	0.308	0.377	0.487	0.688	0.843	0.973	1.192	1.376	1.539	1.686
6.0	0.173	0.273	0.387	0.474	0.612	0.865	1.059	1.223	1.498	1.730	1.934	2.119
8.0	0.255	0.404	0.571	0.699	0.903	1.277	1.564	1.806	2.212	2.554	2.855	3.128
10.0	0.353	0.559	0.790	0.968	1.250	1.767	2.165	2.499	3.061	3.535	3.952	4.329
15.0	0.525	0.909	1.378	1.757	2.388	3.619	4.616	5.486	6.997	8.315	9.506	10.605
20.0	0.848	1.470	2.228	2.841	3.860	5.851	7.463	8.869	11.311	13.442	15.368	17.145
25.0	1.249	2.164	3.279	4.183	5.683	8.613	10.986	13.055	16.651	19.788	22.623	25.239
30.0	1.726	2.991	4.533	5.782	7.855	11.906	15.185	18.046	23.017	27.353	31.272	34.887
40.0	2.911	5.045	7.646	9.752	13.250	20.083	25.614	30.440	38.824	46.139	52.749	58.846
50.0	4.404	7.631	11.567	14.753	20.044	30.382	38.749	46.050	58.733	69.798	79.798	89.023
60.0	6.204	10.751	16.296	20.784	28.239	42.802	54.590	64.875	82.744	98.333	112.420	125.416
70.0	8.312	14.404	21.833	27.846	37.833	57.344	73.138	86.917	110.856	131.741	150.615	168.026
80.0	10.728	18.590	28.177	35.938	48.827	74.008	94.391	112.174	143.070	170.025	194.383	216.854
90.0	13.451	23.309	35.329	45.060	61.221	92.793	118.350	140.648	179.386	213.182	243.723	271.898
100.0	16.482	28.560	43.289	55.212	75.014	113.700	145.016	172.337	219.803	261.214	298637	333.159

Table 12.4a: Cover Management, *C* Factor for Forested and Undisturbed Lands

Erosion Control Treatment	C Factor
Rangeland	0.23
Forest/Tree	
25% cover	0.42
50% cover	0.39
75% cover	0.36
100% cover	0.03
Bushes/ Scrub	
25% cover	0.40
50% cover	0.35
75% cover	0.30
100% cover	0.03
Grassland (100% coverage)	0.03
Swamps/ mangrove	0.01
Water body	0.01

Note: The values are compiled from Layfield (2009), Troeh et al. (1999), Mitchell and Bubenzer (1980), ECTC (2006) and Ayad (2003).

Table 12.4b: Cover Management, *C* Factor for Agricultural and Urbanized Areas

Erosion Control Treatment	C Factor
Mining areas	1.00
Agricultural areas	
Agricultural crop	0.38
Horticulture	0.25
Cocoa	0.20
Coconut	0.20
Oil palm	0.20
Rubber	0.20
Paddy (with water)	0.01
Urbanized areas	
Residential	
Low density (50% green area)	0.25
Medium density (25% green area)	0.15
High density (5% green area)	0.05
Commercial, Educational and Industrial	
Low density (50% green area)	0.25
Medium density (25% green area)	0.15
High density (5 green area)	0.05
Impervious (Parking lot, road, etc.)	0.01

Note: The values are modified from Layfield (2009) and Troeh et al. (1999).

Table 12.4c: Cover Management, *C* Factor for BMPs at Construction Sites

Erosion Control Treatment	C Factor
Bare soil / Newly cleared land	1.00
Cut and fill at construction site	
Fill Packed, smooth	1.00
Freshly disked	0.95
Rough (offset disk)	0.85
Cut Below root zone	0.80
Mulch	
plant fibers, stockpiled native materials/chipped	
50% cover	0.25
75% cover	0.13
100% cover	0.02
Grass-seeding and sod	
40% cover	0.10
60% cover	0.05
≥90% cover	0.02
Turfing	
40% cover	0.10
60% cover	0.05
≥90% cover	0.02
Compacted gravel layer	0.05
Geo-cell	0.05
Rolled Erosion Control Product:	
Erosion control blankets / Turf reinforcement mats	0.02
Plastic sheeting	0.02
Turf reinforcement mats	0.02

Note: The values are compiled from Layfield (2009), Troeh et al. (1999), Mitchell and Bubenzer (1980), ECTC (2006), Israelsen et al (1980), Weischmeier and Smith (1978), and Kuenstler (2009).

Table 12.5: Support Practice, *P* Factor for BMPs at Construction and Development Sites

Support/ Sediment Control Practice	P Factor
Bare soil	1.00
Disked bare soil (rough or irregular surface)	0.90
Wired log / Sand bag barriers	0.85
Check Dam	0.80
Grass buffer strips (to filter sediment laden sheet flow)	
Basin slope (%)	
0 to 10	0.60
11 to 24	0.80
Contour furrowed surface (maximum length refers to downslope length)	
Slope (%) Maximum Length (m)	
1 to 2 120	0.60
3 to 5 90	0.50
6 to 8 60	0.50
9 to 12 40	0.60
13 to 16 25	0.70
17 to 20 20	0.80
> 20 15	0.80
Silt fence	0.55
Sediment containment systems (Sediment basin/Trap)	0.50
Berm drain and Cascade	0.50
Terracing	
Slope (%)	
1 to 2	0.12
3 to 8	0.10
9 to 12	0.12
13 to 16	0.14
17 to 20	0.16
> 20	0.18

Note: The values are compiled from Layfield (2009), Troeh et al. (1999), Mitchell and Bubenzer (1980), ECTC (2006), Israelsen et al (1980), Weischmeier and Smith (1978) and Kuenstler (2009)

12.4.1.2 Sediment Yield

The Modified Universal Soil Loss Equation (MUSLE) is recommended for sediment yield estimation of a catchment as a result of a specific storm event (Williams, 1975). The estimated amount of sediment storage volume is used in sediment basin/ trap design. This empirical relationship is expressed by the following equation for individual storm event:

$$.Y = 89.6\left(VQ_p\right)^{0.56}(K.LS.C.P) \tag{12.4}$$

where,

Y = Sediment yield per storm event (tonnes);

V = Runoff volume (m^3);

Q_p = Peak discharge (m^3/s); and

K, LS, C, P = USLE factors.

Runoff volume and peak discharge can be estimated using procedures described in Chapter 2 of this manual. The most common application would be the Rational Method. Soil Loss factors namely, *K*, *LS*, *C* and *P*, can be determined using equations and recommendations specified in Section 12.4.1.1.

Equation 12.4 is used to produce an event-based sediment yield, i.e. the amount of sediment expected at the end drainage point from the designated site. Hence, it is useful to predict (in mass) the amount of sediment for sizing and maintenance of sediment control BMPs. Many of these BMPs have a designated sediment storage zone. Using the MUSLE, the amount of sediment from a design water quality storm can be estimated. An example of calculations using this equation can be found in Appendix 12.D. Its application in BMPs design is shown in Appendix 12.E and Appendix 12.F.

12.4.2 Erosion Control BMPs

12.4.2.1 Seeding and Planting

(a) Description

Seeding of grasses and planting of trees, shrubs, and ground covers provide long-term stabilisation of soil (Figure 12.6). For example, vegetation may be established along landscape corridors and buffer zones where they may act as filter strips. Additionally, vegetated swales, mild slopes and stream banks can also serve as appropriate areas for seeding and plantings. In general, vegetation is the most suitable and most cost-effective cover for any disturbed or bare soil. However, it should be noted that this BMP has limitations. Seeding and planting on steep slopes can be difficult and may require the help of geo-mats. Vegetation will require proper irrigation during drier days and is slow to stabilise. Excessive use of fertilisers can also worsen stormwater pollution issues.

Maintenance wise, seeding and planting should not require costly or too much of regular maintenance, apart from regular watering during drier months to help plant establish. Fertilizers and pesticide may be required depending on plant selection, although the use of native species normally minimise such needs. Pruning or mowing may be required from time to time, as is weed removal. Eroded areas would require re-vegetation or reinforcement with other BMPs.

a) Turfing in Progress b) Completed Hydroseeding

Figure 12.6: Examples of Turfing and Stabilised Hydroseeding

(b) Design and Application Criteria

- Suppliers must be consulted to confirm appropriate method of seeding and seed species to ensure successful germination and provide an effective measure;
- Seeding is effective on mild slopes typically (H) 2:(V) 1 or flatter;
- For interim erosion control measures, the contractor must ensure no sediment is entrained off the area and must provide at minimum temporary seeding of native or non-invasive species;
- A minimum 150 mm of top soil should be applied to all areas subjected to permanent landscaping;
- All containerized nursery stock should be kept alive and healthy prior to planting;
- Botanist advice shall be acquired for specific site and plant requirements; and
- Personnel for maintenance should be adequately trained in gardening skills.

12.4.2.2 Mulching

(a) Description

Mulching is a temporary ground cover that protects the soil from rainfall impacts, increases infiltration, conserves moisture around vegetation, prevents compaction and cracking of soil, and aids the growth of seedings and plantings by holding the seeds, fertilisers, and topsoil in place until growth occurs. Figure 12.7 shows the examples of mulching works.

Mulching can be used either to temporarily or permanently stabilise cleared or freshly seeded areas. Types of mulches include organic materials, straw, wood chips, bark or other wood fibres. A variety of mats of organic or inorganic materials and chemical stabilisation may be used with mulches. Mulching requires periodical and after storm inspections. Effort should be made to replace or reinforce eroded mulch.

Figure 12.7: Examples of Mulch Application

(b) Design and Application Criteria

The choice of mulch should be based on the size of the area, site slopes, surface conditions (such as hardness and moisture), weed growth, and availability of mulch materials.

- Mulch may be used with netting to supplement soil stabilisation;
- Binders may be required for steep areas, or if wind or runoff is a problem;
- Types of mulch, binders, and application rates should be recommended by manufacturer; and
- Mulch should not be applied in areas subjected to concentrated flows, as it is highly erodible.

12.4.2.3 Geotextiles and Mats

(a) Description

Mattings made of natural or synthetic material, are often used to temporarily or permanently stabilise soil (Figure 12.8), reduce erosion from rainfall impact, hold soil in place, and absorb and hold moisture near the soil surface. Additionally, mattings may be used alone or with mulch during the establishment of protective cover on critical slopes. This technique is suitable for protection of steep slopes or stream channels with high shear force, as the mats provide structural support to hold soil (in many cases containing seeds or seedlings) together.

Geotextiles and mats require inspection for loose matting, which should be re-anchored or rejoined. Once stabilised, this BMP will require minimal maintenance. It should be noted that matting is relatively more costly than other erosion control BMPs. Its effectiveness heavily relies on the skill of installation workers. The use of synthetic matting such as plastic sheet can increase runoff rates in downslope areas and shall be avoided or only used as emergency or very temporary protection.

Figure 12.8: Example of Geotextile Applications for Erosion Control

(b) Design and Installation Criteria

The application of geotextile does not require professional design for most uses. If hydrostatic pressure is a concern for stability of a retaining wall, contractor should consult a professional experienced in the selection of geotextile fabric. Matting for conveyance systems such as swales or channel banks shall fulfil hydraulic requirements of the conveyance system as described in Chapter 1.

Geotextiles should be installed according to manufacturer's specifications. Among some aspects of installation to be emphasised are: toes and anchors should be sufficiently secure against uplift, overlap span should be sufficient, jointing methods used are to be those recommended by the manufacturer and performed to specification by skilled workers, and backfilling materials are chosen so as to not to damage the mats.

Due to differences in manufacturing materials, techniques, product properties, site constraints, and working environments, methods of installation may vary significantly.

12.4.3 Runoff Management BMPs

Runoff management is a process to control the direction, volume and velocity of the transport medium and safely convey stormwater so that its potential for erosion is reduced. Runoff management BMPs help to direct stormwater away from exposed soils. Transport control should direct the flow to areas where the sediment can be trapped and removed. The primary mechanisms used to control runoff include: reducing the amount of runoff, detaining runoff (reducing velocity), diverting runoff, and dissipating the energy of runoff. The runoff management BMPs introduced here are temporary structures specifically designed to support runoff management during construction. Whenever appropriate, permanent structures such as open channels, engineered waterways, and permanent culverts can be constructed at early stages of construction to manage runoff during construction.

12.4.3.1 Earth Bank

(a) Description

A temporary earth bank is a temporary berm or ridge of compacted soil used to divert runoff or channel water to a desired location, thereby reducing the potential for erosion and off-site sedimentation. Earth banks may also be used to divert runoff from off-site and from undisturbed areas away from disturbed areas, sheet flows away from unprotected or unstable slopes, and polluted runoff into sediment control or permanent stormwater BMPs. Earth banks require minimal maintenance, but should be checked after major storms to repair failed, overtopped or eroded areas. Figure 12.9 provides an engineering drawing for an earth bank.

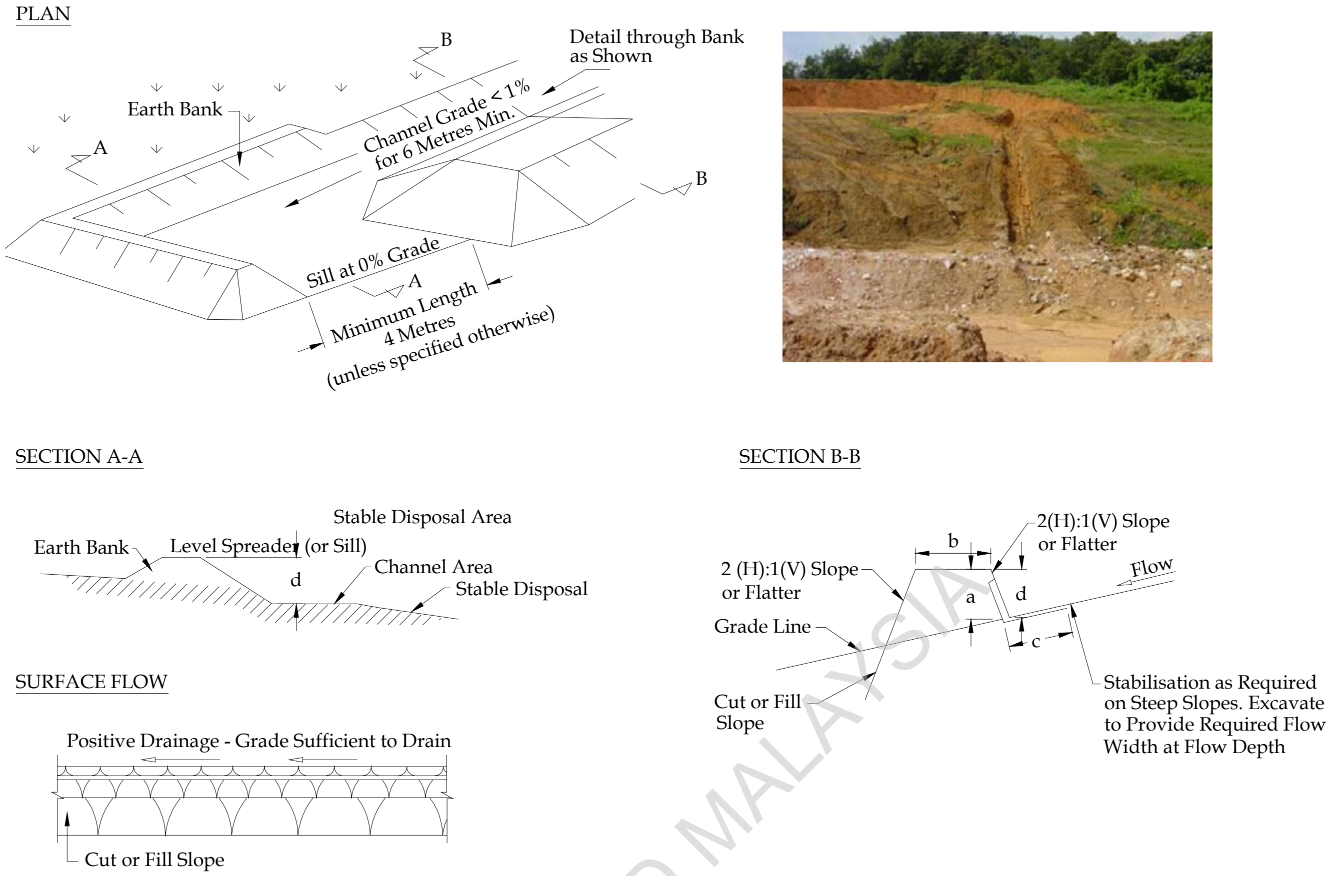


Figure 12.9: Example of an Earth Bank

(b) *Design Criteria*

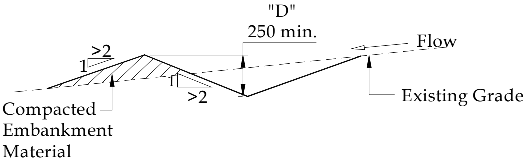
Table 12.10: Design Criteria for Temporary Earth Bank

Parameter	Requirement
Design Storm	2-year ARI
Catchment Area	Not more than 4 ha
Dimension	Side Slope: 2(H):1(V) or flatter Height: Minimum of 450 mm Top Width: Minimum of 600 mm
Flood Protection	Ensure upstream/ downstream flooding condition not aggravated
Scour Protection	Proper scour protection to be provided on the face contacted by flowing water
Embankment Material	95% compaction by earth moving machinery

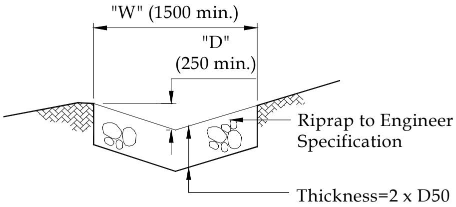
12.4.3.2 Diversion Channel

(a) *Description*

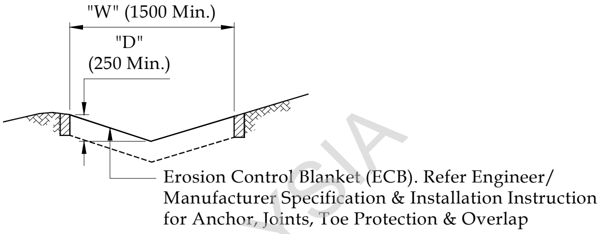
Temporary diversion channels (Figure 12.10) may be used to divert off-site runoff around the construction site, divert runoff from stabilised areas around disturbed areas, and to direct runoff into sediment control BMPs. Diversion channels should be installed when the site is initially graded and remain in place until permanent BMPs are installed and/or slopes are stabilised. Diversion channels should be regularly checked to prevent clogging, unexpected sediment built-up or channel erosion, overtopping, and cover failure, preferably after every storm. Any defects shall be repaired immediately.



(a) Unlined



(b) Ripraplined



(c) Lined - Erosion Control Blanket (ECB)

Figure 12.10: Diversion Channels

(b) Design Criteria

Table 12.11: Design Criteria for Temporary Earth Bank

Parameter	Requirement
Design Storm	2-year ARI
Dimension	Side Slope (if applicable): 2(H):1(V) or flatter
Flood Protection	Ensure upstream/ downstream flooding condition not aggravated.
Hydraulic Criteria	Shall be designed as described in Chapter 14 (Drains and Swales) of this manual.
Scour Protection	• Inlet and outlet protection shall be provided • Channel bed and banks can be stabilised using various erosion control methods such as turf, riprap or geo-mats. • Perform checking of maximum shear and velocity
Embankment Material	95% compaction by earth moving machinery

(c) Installation and Application Criteria

Diversion channels are only effective if they are properly installed. As diversion channels are mainly made of earth and soil, it is extremely important that this BMP is properly stabilised by proper earth compaction and surface cover. The side slope and top width criteria should be met to prevent erosion and slope failure.

12.4.3.3 Drainage Outlet Protection

(a) Description

Drainage outlet protection (Figure 12.11) is a physical device consisting of rock, grouted riprap, or concrete rubble that is placed at the outlet of a culvert, conduit, or channel to prevent scour caused by high flow velocities, and to absorb flow energies to produce non-erosive velocities. Energy dissipaters using ripraps or gabions are commonly used as temporary outlet protection. This BMP should be provided wherever discharge velocities and energies at the outlets of culverts, conduits, or channels are sufficient to erode the downstream reach. In cases where loose rocks or rip rap are used, it is important to carry out regular inspections to avoid material wash-off during large storm event. Lost materials should be immediately replaced. Materials used should be selected based on channel stability design.

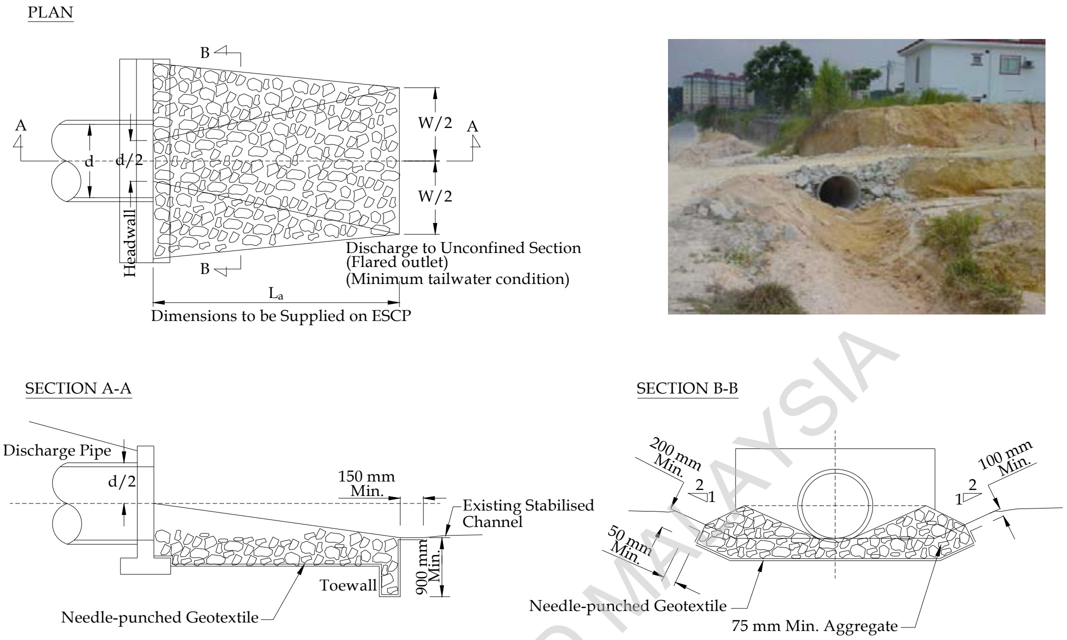


Figure 12.11 Drainage Outlet Protection

(b) Design Criteria

The hydraulic design of drainage outlet protection shall comply with erosion and scour protection recommended in Chapter 20 of this manual. The designed structure shall also be checked for seepage and structural stability. Rock outlet protection is effective when the rock is sized and placed properly. Rock size should be increased for high velocity flows.

12.4.3.4 Temporary Waterway Crossing

(a) Description

A temporary access waterway crossing is a culvert placed across a waterway to provide access for construction purposes for a period of less than one year. The purpose of a temporary crossing is to provide a safe, erosion-free access point across a waterway for vehicles. The concept of temporary crossing is slightly different from a permanent culvert, where it is supposed to convey only minor events and dry weather flows. During major events, the crossing will act as spill crest where larger flows overtop and overspill from the top of crossing while still contained in the conveyance system. Caution should be exercised when applying this BMP as it is a in-stream construction.

The crossing, though temporary might have adverse effect on upstream flooding if overlooked in design. Over a large watercourse, this may be an expensive undertaking for a temporary structure. Weekly inspection and maintenance is required to check for structural failure, debris removal, inlet and outlet protection maintenance etc.

(b) Design Criteria

The design of temporary crossings should generally comply with specifications outlined for culverts design in Chapter 18 of this manual and further incorporate the following design criteria as shown in Table 12.12.

Table 12.12: Design Criteria for Temporary Crossings

Parameter	Requirement
Design Storm	2-year ARI
Dry Weather Flow	To be prepared to allow existing natural flow regime
Overspill	All flow greater than 2 year ARI shall safely bypass the crossing
Flood Protection	Ensure upstream/ downstream flooding condition not aggravated
Hydraulic	Refer Culvert Design in Chapter 18
Scour Protection	Inlet and outlet protection shall be provided

12.4.4 Sediment Control BMPs

Sedimentation control BMPs trap sediment on the site in selected locations and minimize sediment transfer off the site. Sedimentation controls are generally passive systems that rely on filtering or settling of soil particles out of the water or air. Sedimentation control BMPs treat soil as waste products and work to remove it from the drainage system. Sediment control BMPs are the last line of defence against erosion and sedimentation before sediment reaches natural watercourses.

12.4.4.1 Silt Fences

(a) Description

A silt fence (Figure 12.12) is a temporary sediment barrier consisting of filter fabric stretched across and attached to supporting posts, entrenched, and, depending upon the strength of the fabric used, backed by a wire fence for support. This measure does not filter runoff, but acts as a linear barrier creating upstream ponding that allows soil particles to settle out, thereby reducing the amount of soil leaving a disturbed area. This BMP is relatively effective at retaining suspended solids coarser than 0.02 mm. Silt fences are simple to construct, relatively inexpensive and easily moved as development progresses.

Silt fence can be considered an on-site control as it caters to small overland sheet flow. It is most effective in securing site perimeter, protecting topsoil stock pile, and intercepting sheet flow along slope contours. Silt fence requires regular inspection and maintenance as it is easily damaged. Sediment built-up behind fence should be regularly removed. The fence is also not suitable for areas dominated by very fine (clayey) soil particles. The selection of material pore size is important and suppliers or manufacturers shall be consulted on this matter.

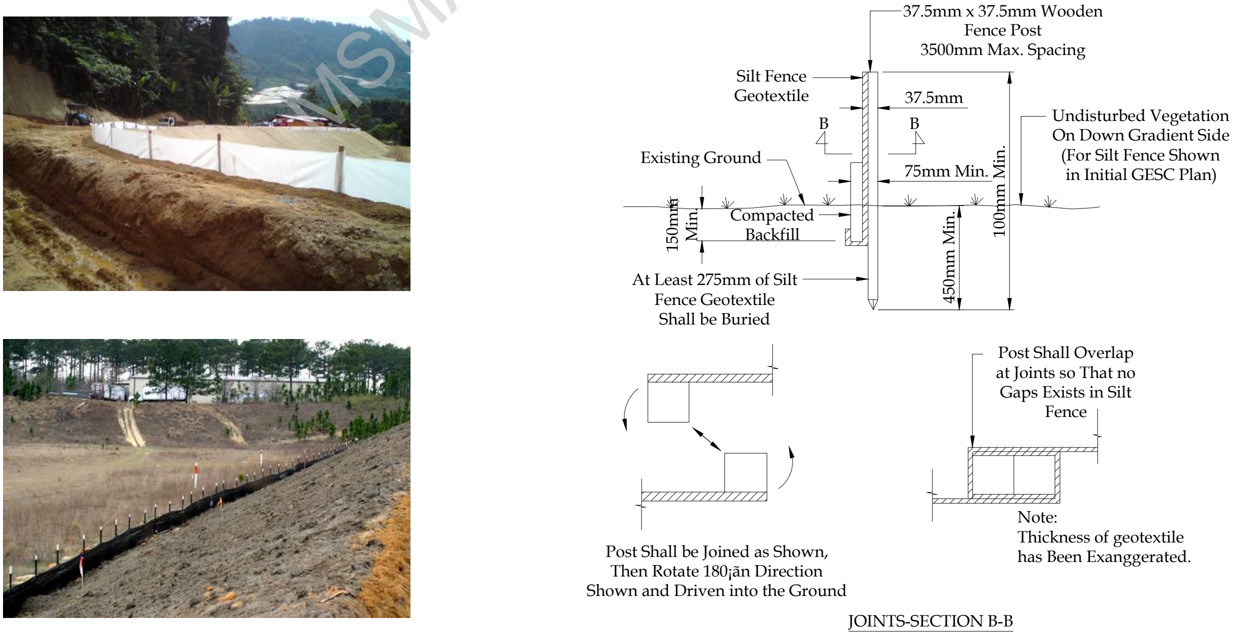


Figure 12.12: Silt Fence Application

(b) Design Criteria

Table 12.13: Design Criteria of Silt Fence

Parameter	Requirement
Design Storm (Both Quantity & Quality)	First 50 mm of rainfall over the contribution (equivalent impervious) catchment
Maximum Contributing Area	0.4 ha
Hydraulic	For any point along the fence, • Concentrated flow shall not exceed 50 L/s • Maximum water depth shall not exceed 600 mm
Sitting of facility	• Fences SHALL NOT be installed in areas receiving concentrated flow, i.e. streams or ditches • Maximum length of each fence segment shall not exceed 30 m • The at least 1 m from ends of each segment shall be turned uphill to prevent runoff flowing around the fence
Slope	• Slope draining to fence shall be 1(H):1(V) or flatter • Length of path draining to fence shall not exceed 60 m
Storage Area	Storage area to be provided behind fence Approximately 280 m² per ha of contributing area is required

(c) Installation and Application Criteria

- Designers should leave an undisturbed or stabilised area immediately downslope of the fence;
- Designers should select filter fabric which retains 85% of the soil, by weight, based on sieve analysis, but is not finer than an equivalent opening size of 70 (US Standard Sieve) or about 210 µm;
- Sediment fences should remain in place until the disturbed area is permanently stabilised;
- Posts should be spaced a maximum of 3.5 m apart and driven securely into the ground a minimum of 450 mm;
- A trench should be excavated approximately 200 mm wide and 300 mm deep along the line of posts and upslope from the barrier;
- The use of joints should be avoided. When joints are necessary, filter cloth should be spliced together only at a support post, with a minimum 150 mm overlap and both ends securely fastened to the post;
- The trench should be backfilled with 20 mm minimum diameter washed gravel or compacted native material;
- The ends of the filter fence should be turned uphill to prevent stormwater from flowing around the fence; and
- Designers should provide an undisturbed or stabilized outlet suitable for sheet flow.

12.4.4.2 Check Dams

(a) Description

A check dam (Figure 12.13) is a small temporary dam constructed across a diversion channel or swale. Check dams reduce the velocity of concentrated stormwater flows, thereby reducing erosion of the diversion channel or swale and promoting sedimentation behind the dam. Small barriers consisting of rocks or earth berms are suitable as for check dams. Many commercial products such as gabions and sand bags can also be used effectively as check dams.

Check dams are primarily used in small channels in steep terrain where velocities exceed 0.6 m/s. This BMP acts to prevent erosion by reducing the velocity of channel flow in small intermittent channels and temporary swales. In areas with high velocity, deep sump may be provided immediately upstream of the check dam to capture excessive sediment. Check dams are not to be used as a stand-alone substitute for other sediment control BMPs simply because trapping efficiency in check dams are relatively low, and high flows could potentially re-suspend settled sediment. Maintenance is therefore required to remove trapped sediment and to check for structural stability on regular basis. Check dams should only be placed in small open channels, and never on a flowing river or natural stream.

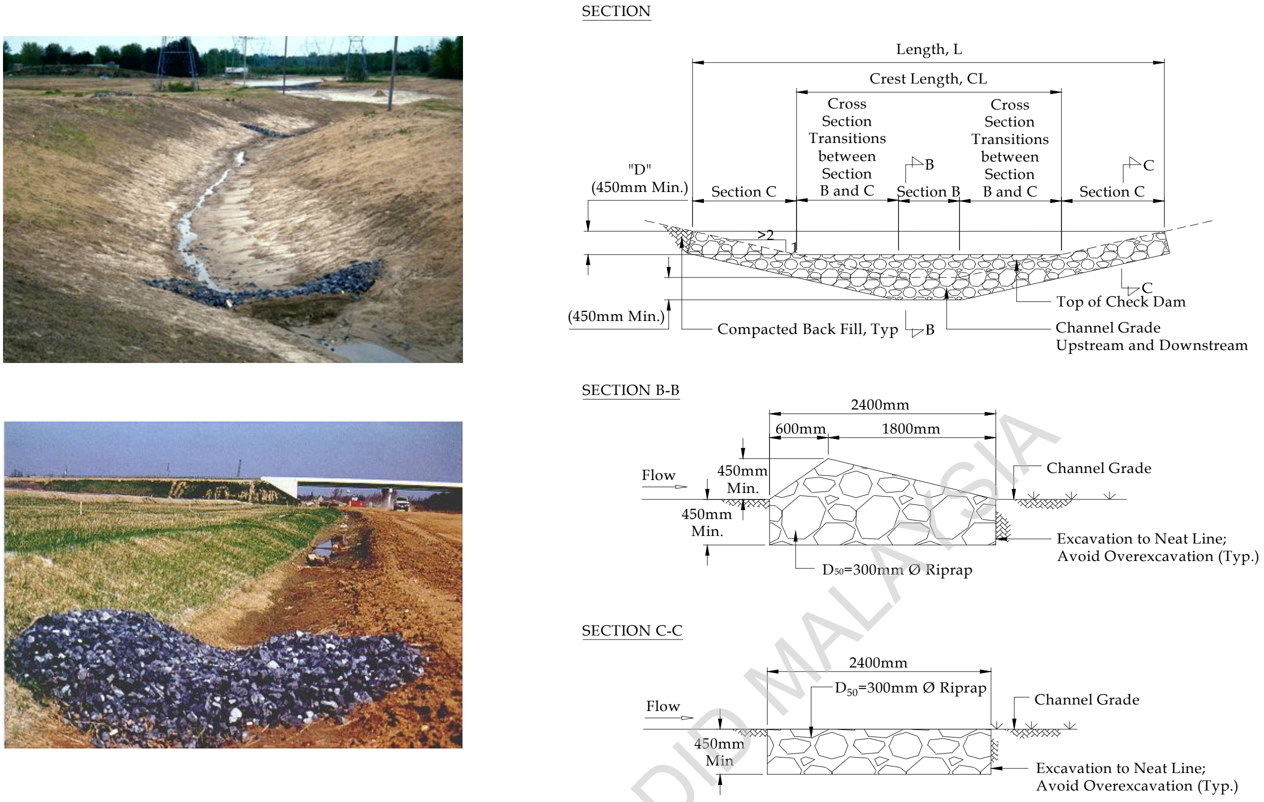


Figure 12.13: Check Dams

(b) Design Criteria

Table 12.14: Design Criteria of Check Dam

Parameter	Requirement
Design Storm	2-year ARI, unless specified otherwise by Authorities
Overspill	All flow greater than 2 year ARI shall safely bypass the crossing
Flood Protection	Ensure upstream/ downstream flooding condition not aggravated
Dimension	• Height (centre) of dam shall not exceed 1 m • For rock check dam: - Upstream slope: 2(H):1(V) or flatter - Downstream slope: 4(H):1(V) or flatter • Centres of the dam shall be notched to centre to promote concentrated flow (approx. 0.15 m) • Outer sides of dam shall be at least 0.5 m higher than centre to avoid undermining • Spill crest shall be of at least 100 mm in width parallel to flow
Intervals	A series of check dams can be placed such that the height of each subsequent check dam must be equal or lower than the base of the check dam before it
Geotextile	Check dam with height more than 450 mm shall be laid with geotextile to avoid seepage and structural failure
Scour Protection	• Structure shall withstand the shear force induced by a 2-year ARI flow. Materials (rocks, earth, and gabion) must be selected to meet this requirement • Additional scour protection downstream of check dam shall be provided if deemed necessary

12.4.4.3 Sediment Traps

(a) Description

A sediment trap is a small temporary ponding area, usually with a gravel outlet, formed by excavation and construction of an earthen embankment (Figure 12.14). The purpose of the trap is to detain runoff from disturbed areas for a long enough period of time to allow majority of the coarser suspended soil particles in the runoff to settle out. It is intended for use on small catchments (2 ha) areas with no complex drainage features, where construction will be completed in a reasonably short period of time.

This practice is one of the most efficient and cost effective methods of sediment control. When possible, sediment traps should be constructed as a first step in any land-disturbing activity. Sediment trap should never be location on-stream. In areas near to public, safety fence should be provided. It should be noted that since sediment traps are only effective in trapping coarse sediment, a chemical binder or coagulant may be required for fine particle trapping. The sediment trap requires monitoring during each storm event. It should drain within 36 hours after a storm event, failing which, mechanical dewatering is required. Regular maintenance should include sediment removal (after sediment is 300 mm thick), as well as structural and outlet protection inspection.

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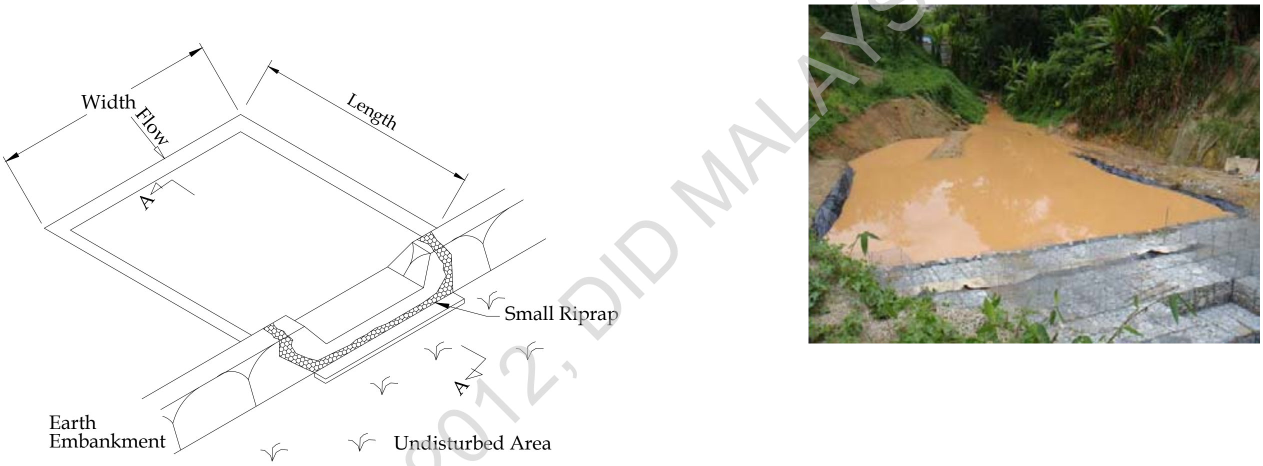

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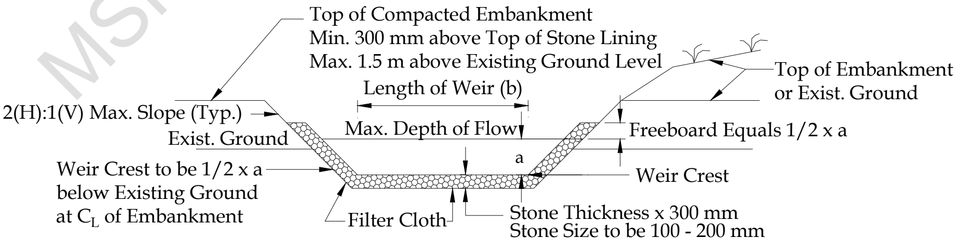


SECTION A-A

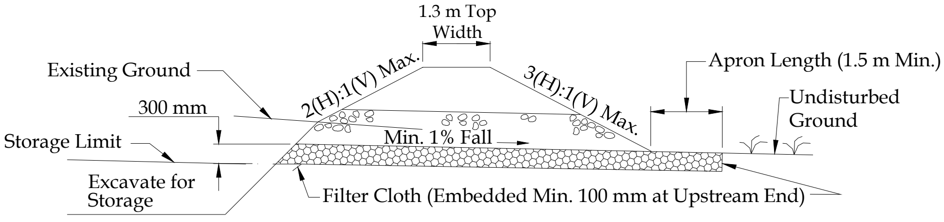


Figure 12.14: Example of a Sediment Trap

(b) Design Criteria

Table 12.15: Design Criteria of Sediment Traps for Sediment Control

Parameter	Requirement
Runoff Quantity Design	Up to 10-year ARI
Runoff Quality Design	First 50 mm rainfall over contributing (equivalent impervious) catchment
Overspill	All flow up to 10-year ARI shall safely bypass the trap
Runoff Retention	All flow up to runoff quality design flow shall be retained within basin. Extended drawdown can be permitted by authority when deem necessary.
Flood Protection	Ensure upstream/ downstream flooding condition not aggravated
Maximum Contributing Area	2 ha
Storage Volume	• Total Storage: 125 m^3/ ha of contributing area • Permanent Pool: half of total storage
Basin Dimension	• Minimum length to width ratio: 2:1 • Minimum depth of 1 m • Depths exceeding 2 m are not recommended. In unavoidable circumstances, provide perimeter fencing for safety
Embankment	• Inside embankment: 2(H):1(V) or flatter • Outside embankment: (3(H):1(V) or flatter • Maximum embankment height should not exceed 1.5 m
Lining Materials	Suitable size rocks or rip rap
Erosion Protection	Outlet protection shall be provided for the emergency spillway

(c) Installation and Application Criteria

- Traps should be located where sediment can be easily removed (Access roads are to be provided if required).
- The outlet of the trap must be stabilised with rock, vegetation, or another suitable material.
- The fill material for the embankment must be free of roots and other woody vegetation as well as oversized stones, rocks, organic material, or other objectionable matter. The embankment may be compacted with suitable equipment during construction.
- The spillway installation is critical to prevent failure of the structure during high flows and all specifications provided by the designer must be implemented.

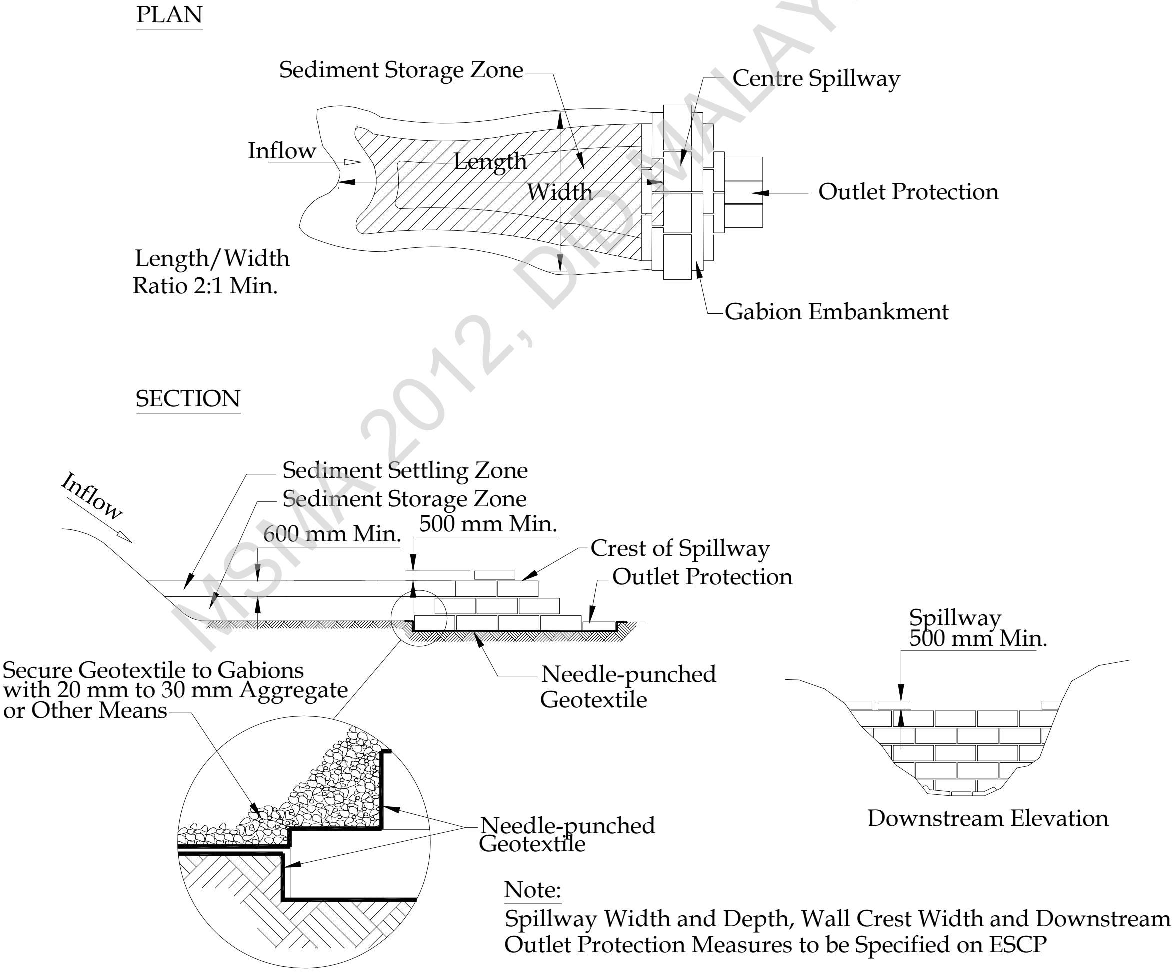
12.4.4.4 Sediment Basins

(a) Description

A sediment basin (Figure 12.15) typically consists of an impoundment, a dam, a riser pipe outlet, and an emergency spillway. It functions in the same way as a sediment trap but caters to a larger catchment. The basin is a temporary measure (with a design life of 12 to 18 months) and is to be maintained until the site area is permanently protected against erosion or a permanent detention basin or water quality control structure is constructed.

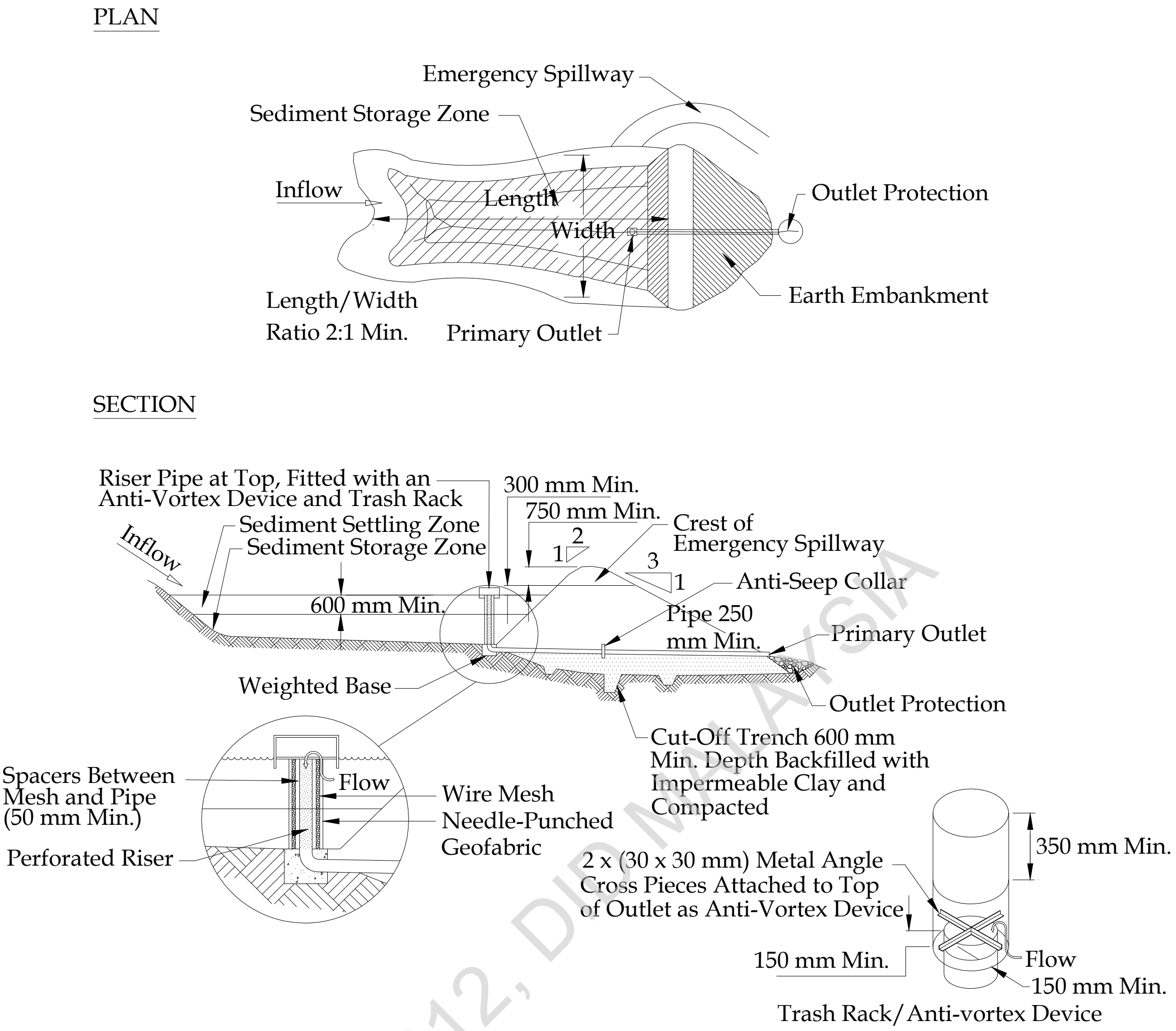
Sediment basins are suitable for nearly all types of construction projects. Wherever possible, they should be constructed before land clearing and grading work begins. The type of basin is to be determined using Table 12.16 below. The basin must not be located in a stream or natural waterway but should be located to trap sediment-laden runoff before it enters any stream. It is a common and encouraged practice to locate this structure at the location where permanent stormwater BMPs (mostly ponds) will be located. Like a sediment trap, sediment basin may pose a safety hazard and should be properly fenced if required by the local regulatory authority. Large sediment basins (dams higher than 3 m) shall be subject to Federal and/or State dam safety criteria.

Maintenance of sediment basins is similar to sediment trap maintenance. Removal of accumulated sediment should be carried out once the sediment storage zone is full. The basin must be able to drain in 36 hours of a rain event, failing which, mechanical dewatering is required. Regular inspections are to be carried out to ensure structural stability and functionality of the inlet, outlet and outlet protection works. If the basin is located at the final discharge point from site, periodic water quality samples shall be collected and tested for total suspended solids (TSS) and turbidity to comply with DOE water quality regulations.



a) Outlet Control-Gabion Spillway

Figure 12.15: Sediment Basins



b) Outlet Control-Riser

Figure 12.15: Sediment Basins (cont'd)

Table 12.16: Sediment Basin Types and Design Considerations

Category	Soil Description	Hydrological Soil Group	Basin Type	Design Considerations
I	Coarse-grained sand, sandy loam: less than 33% <0.02 mm	A	Dry	Settling velocity, sediment storage
II	Fine-grained loam, clay: more than 33% < 0.02 mm	B	Wet	Storm impoundment, sediment storage
III	Dispersible fine-grained clays: more than 10% of dispersible material	C, D	Wet	Storm impoundment, sediment storage, assisted flocculation

(b) Design Criteria

Table 12.17: Design Criteria of Sediment Basin for Sediment Control

Parameter	Requirement
Basin Type	Refer Table 12.16
Runoff Quantity Design	Up to 10-year ARI
Runoff Quality Design	First 50 mm rainfall over contributing (equivalent impervious) catchment
Runoff Control	• All flow up to runoff quality design shall be retained within the trap • The basin should drain within 24 hours (dry)/ 36 hours (wet) after the water quality design storm. • The primary outlet/riser should be used to control stormwater runoff. • The Emergency spillway should safely conveying flows up to 10-year ARI
Flood Protection	Ensure that upstream/ downstream flooding conditions do not aggravate possible failure of the embankment.
Minimum Contributing Area	2 ha
Storage Volume	Total Storage: Refer Table 12.18 (dry) or Table 12.19 (wet) Settling zone volume: half of total storage Sediment zone volume: half of total storage
Basin Dimension	• Minimum length to width ratio: 2:1 • Maximum length to settling depth ratio: 200:1 • Minimum settling zone depth: 0.6 m • Minimum sediment storage zone depth: 0.3 m
Embankment	Side slope: (2(H):1(V) or flatter
Erosion Protection	Outlet protection shall be provided for the emergency spillway
Sediment Trapping	90% of Total Suspended Solids Removal
Maintenance Frequency	Determined by dividing sediment storage capacity by the amount of sediment collected in a water quality design storm

Table 12.18: Dry Sediment Basin Sizing Criteria

Parameter	Time of Concentration of Basin Catchment (minutes)				
	10	20	30	45	60
Surface Area (m^2/ha)	333	250	200	158	121
Total Volume (m^3/ha)	400	300	240	190	145

Table 12.19: Wet Sediment Basin Sizing Volume (m^3/ha)

Parameter	Site Runoff Potential	Magnitude of Design Storm Event (mm)				
		20	30	40	50	60
Settling Zone Volume	Moderate to high runoff	70	127	200	290	380
	Very high runoff	100	167	260	340	440
Total Volume	Moderate to high runoff	105	190	300	435	570
	Very high runoff	150	250	390	510	660

(c) Installation and Application Criteria

- Sediment basins must be installed entirely within the limits of the site.
- Basins must be constructed before clearing and grading work on the overall site begins.

- Basins must not be located in a stream.
- All basins should be located where failure of the embankment would not result in loss of life, damage to homes or buildings, or interruption of use or service of public roads or utilities.
- Local ordinances regarding health and safety must be adhered to.
- Large basins may be subject to State and/or Federal dam safety requirements.
- Sediment basins are attractive to children and can be very dangerous. Adequate safety precautions must be provided by restricting access to the site or to the basin with suitable fencing.
- Contractor shall securely anchor the outlet pipe and riser, and install anti-seep collars on the outlet pipe.
- Sediment basins may be capable of trapping smaller sediment particles if sufficient detention time is provided. However, they are most effective when used in conjunction with other BMPs to minimise the amount of sediment mobilised and carried to the basin.

12.5 PREPARATION OF EROSION AND SEDIMENT CONTROL PLAN (ESCP)

The ESCP is to be prepared by a qualified consultant to manage erosion and sediment processes during the phases of earthworks and subsequent construction. The ESCP is literally the master plan for construction site management in terms of erosion, runoff and sedimentation control.

In this section, the general requirements, the generic principles governing ESCPs, plan preparation stages, the content of ESCPs, and the performance evaluation stages are laid out..

12.5.1 General Requirements of ESCPs

ESCPs must be submitted to local regulatory authority for developments that involve an area of more than 1 ha. However, ESCPs can be requested by local regulatory authorities for any development sites (including those less than 1 ha) as a supporting plan, as empowered by the Street, Drainage, and Building Act (1974).

In order to obtain the permit so that scheduled work can be started on time, it is suggested that the ESCP should be submitted to related local regulatory authority at least 2 months in advance, or a longer period according to the requirements of local regulatory authority. Approval for ESCPs must be obtained from related local regulatory authority at least 14 calendar days before the beginning of construction activity.

Water quality design criteria for all temporary BMPs is to control the first 50 mm of rainfall from the contributing (equivalent impervious) catchment.

12.5.2 Content of an ESCP

A complete ESCP consisting of 3 major components, i.e., report, site plans and engineering drawings, as well as an inspection and maintenance plan, shall be submitted as one document for evaluation by the relevant authorities for approval. The report will describe the preparation of the ESCP including the concept, assessment, and design. Plans and drawings provide visual interpretation of the ESCP, and the inspection and maintenance plan outlines the steps required to implement the ESCP during site development.

12.5.2.1 Report

(a) Site Description

A written report shall be prepared to describe the site, especially the location, climate, topology and current land use. Information gathered through site investigations, consultation with local regulatory authorities or any reliable source shall be furnished in this report. The report shall provide a clear picture of the existing site condition. The proposed development shall also be narrated herein. Details on size of development, the purpose and proposed layout shall be included.

The planner shall also illustrate how the ESCP is planned to convert the development from the existing site condition. Items below provide some important points that shall be included. Local regulatory authorities may request other information to facilitate in evaluation process. If more than one phase of activity is planned, a description of the following items must be provided for each phase of major earthworks (bulk grading).

- Earthwork phases;
- Securing of site perimeter;
- Access points and traffic control;
- Management of stockpiles;
- Slope stabilisation;
- Erosion control measures;
- Runoff management on site;
- Sediment control measures; and
- General inspection and maintenance planning.

(b) Site Assessments

The ESCP planner shall carry out assessments to evaluate the site while planning the ESCP. The assessment shall be presented as part of the submission to the local regulatory authority. The methodology used and results of assessment shall be presented to facilitate evaluation of the ESCP. Two assessments are required, i.e. hydrological & hydraulic assessment, and soil loss assessment.

(i) Hydrological & Hydraulic Assessment Tasks

- The planner (or designer) shall perform hydrological and hydraulic assessments for pre-construction, during construction, and post-construction conditions at the site.
- If earthworks are planned in phases, during construction conditions shall include individual assessment of site conditions for each phase of earthworks.
- The assessment shall be performed to examine the required design storm criteria mentioned below, or as instructed by plan evaluator.

(ii) Soil Loss Assessment

- Perform soil loss assessment using USLE model (or any approved method agreed by evaluator) to assess the soil loss conditions for the site. The assessment shall prove that soil loss is adequately controlled with implementation of ESCP. The assessment shall account for soil loss estimation during pre-development, during construction (with and without ESCP) and post construction conditions.
- Each earthwork and/or construction phase shall be considered separately.
- Assessment shall be provided for each design points, as determined by the site condition.

(c) Engineering Design and Calculation

While most erosion control BMPs can be applied without design, other ESC BMPs components (i.e. runoff management BMPs, sediment control BMPs, and slope protection structures) must be provided with design calculation to justify their application. The design must state clearly the dimensions and location of the facilities. Calculations must prove that the proposed facilities will be able to serve the site in accordance to standard design procedures.

(d) Other Supporting Documents

The ESCP report shall be furnished with the following documents to facilitate plan evaluation by the relevant authorities (DID, 2010).

- Bill of Quantities (BQ); and
- Specification/ Installation Instruction of materials/ commercial product proposed.

The BQ should be prepared such that each individual ESC facility is billed independently. This is to ensure all proposed facilities are included in budget and that contractors shall have no construction/ maintenance/ financial constrain in carrying out the ESCP.

12.5.2.2 Site Plans & Engineering Drawings

Site Plans are simple illustrations of the project site, showing key physical ESC-related features including levels, slopes, ESC facilities, site management (access roads) etc. Site plans shall provide a clear impression and interpretation of all ESC controls designed for the site. Essentially, site plans shall be provided for 2 stages, i.e. pre-bulk grading and post bulk grading. Engineering drawings shall be prepared for all ESC BMPs proposed. Standard symbols/legend for indication of ESC BMPs as given in Appendix 12.A shall be followed.

(a) Pre-Bulk Grading Site Plan

The pre-bulk grading site plan shall clearly portray the existing land condition and the planned grading activity to transform the terrain into the final development levels. This shall include the following information:

- Pre development topology – drainage pattern, contour, and catchment delineation;
- Areas (with quantity) in which grading (cut & fill) will be performed;
- Specify grading phasing;
- Specify stockpile management (location, protection etc);
- Perimeter controls including buffer, hoarding and site perimeter drains;
- Delineate new catchment area based on graded topology (to be used for ESC facilities design);
- Identify and delineate waterway buffers; and
- Specify ESC facilities (size, location etc) to be implemented at this stage.

(b) Post-Bulk Grading Site Plan

The pre-bulk grading site plan shall clearly portray actual construction site condition after the major earth work or grading is completed. The plan shall essentially show the ESC practices to be implemented, which includes:

- The graded contour (topology after major earthworks);
- Project development phasing;
- Proposed drainage patters and catchment delineation; and
- Specify ESC facilities (size, location etc) to be implemented at this stage.

(c) Engineering Drawings

Engineering drawing shall be prepared for all ESC facilities selected for the site. This shall consists but not limited to,

- Typical engineering drawings for erosion control facilities such as erosion blanket;
- Detailed engineering drawings for temporary and permanent (if the permanent components are used as ESC facilities) runoff management facilities such as diversion drain and swales;
- Detailed engineering drawings for sediment control facilities such as check dams and sediment basin; and
- Detailed engineering drawings for slope stabilization such as terracing and retaining wall.

12.5.2.3 Inspection and Maintenance Plan

Every BMP installed on a construction site must be checked periodically and maintained sufficiently to ensure proper performance. An Inspection and Maintenance Plan should be prepared and implemented. The purpose of the plan is to;

- Clearly specify personnel assigned/responsible for BMP inspection and maintenance;
- Determine maintenance requirements of any BMP and subsequently present regular maintenance schedule; and
- Determine and present methods/ procedures/ checklist/ record logs to be used for in-house BMPs inspections.

The plan shall provide the following details in regards of inspection and maintenance:

- *Schedule for Inspection and Maintenance* – a Gantt Chart indicating the scheduled date for regular inspection and expected facilities maintenance. Major maintenance such as servicing of sediment basin, must be included.
- *State responsibility of stakeholders* – a list of contacts to summarise person (or party) in charge of all ESCP aspects (from design, construction, maintenance, inspection, and operation).
- *Record Keeping* – The database used in record keeping shall be specified. Methods used to store engineering drawings, ESCP plans, inspection results & maintenance log etc, shall be clearly stated. If applicable, example of templates can be provided.

12.5.3 ESCP Preparation Stages

The ESCP must be prepared before construction begins, ideally during the project planning and design stages. It may be completed at the end of the design stage or early in the construction stage, as shown in Figure 12.16. Implementation of the ESCP begins when construction begins, typically before the initial clearing and grading operations since these activities usually increase erosion potential on the site. During construction, the ESCP should be referred to frequently and refined by the consultant and contractor as changes occur in construction operations, which have significant effects on the potential of pollutants discharge.

The ESCP may be very dynamic for large, complicated projects to be constructed in multiple stages over a long period of time. Planning, design, and construction of these projects may be occurring simultaneously. In such cases, it may be useful to prepare the ESCP in sections, with each section covering a stage or portion of the project and an overview section generally discussing the entire project.

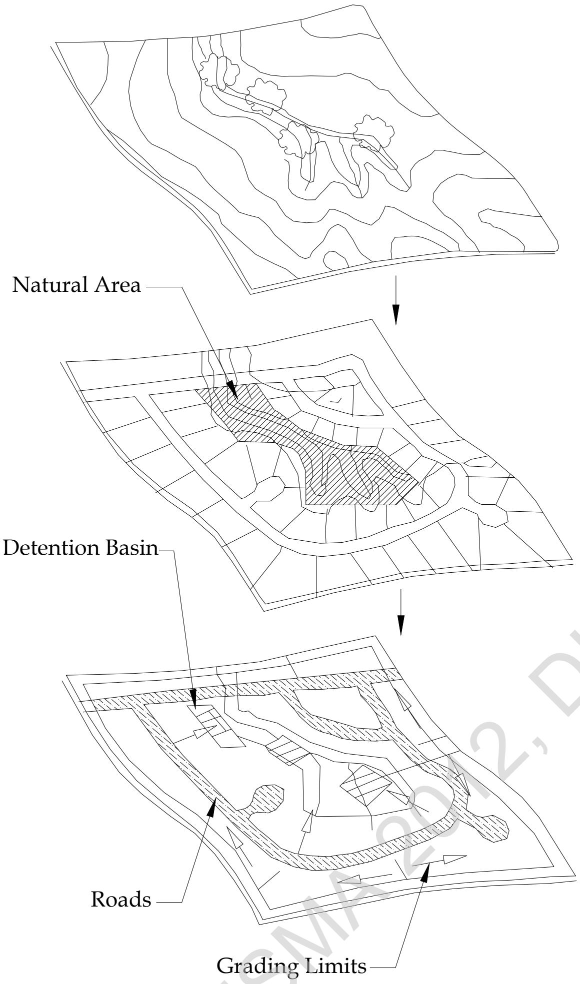
The following sections give guidance on how to incorporate ESCP preparation into the planning, design, and construction stages of a project.

12.5.3.1 Planning Phase

The planning phase is the source of most information needed for the ESCP. Conceptual and preliminary erosion and sediment control planning is also made at this phase via the normal review process with the local regulatory authority. Four activities which occur during planning that are important to the preparation of an ESCP are as follows:

- Assessing site conditions;
- Developing a base plan(s);
- Selecting post-construction measures; and
- Establishing long-term maintenance agreements.

ESCP preparation begins as early as planning phase of a development. Once the basic site condition is ascertained (i.e. topology, hydrology, development layout etc), erosion risk assessment can be carried out as the first step of ESCP preparation. The objective of the assessment is to determine the existing erosion risk and how the risk will be affected by the development. The additional information on erosion and hydrology from assessments will then allow Engineer to select and plan the post-construction (permanent) stormwater BMPs. In the process, a maintenance agreement shall be established at this stage regarding to the party in-charge of post-construction operation and maintenance of the selected BMPs



Planning Phase

- Asess Site Conditions
- Develop Base Plan
- Select Permanent BMPs
- Establish Maintenance Agreements

Design Phase

- Prepare Project Site Plan
- Select BMP Objectives
- Define Permananent

Construction Phase

Bidding and Mobilisation
- Select BMPs
- Complete the ESCP
- Submit the Notice of Commencement

Constructions Operations
- Inspect and Maintain BMPs for Construction Activities
- Construct Permanent BMPs
- Revise the ESCP as Construction Evolves or Property is Sold

Complete Constructions
- Permanent Site Stabilisation
- Maintenance Agreement for Permanent BMPs
- Submit Notice of Termination

Figure 12.16: Plan Preparation Stages for ESCP (CDM, 1993)

12.5.3.2 Design Phase

There are three principal activities that are typically incorporated into the ESCP during the design phase, i.e. Preparation of Site Plan, Define Objectives for Control Measures, and Designing Permanent Control Measures. During the preparation of site plan, numerous studies will be produced to provide information for detailed design of the development. The site plan will then contain final information on drainage, grading, structural and landscape layouts. Hence the plan forms the foundation for ESCP planning. Based on the grading plan, proper earthwork and construction phasing can be planned, while the drainage and landscape plan will aid in selection and placing of temporary ESC BMPs.

During the final project design process, the consultant will prepare detailed grading plans, paving and drainage plans, landscape plans, and other plans as necessary for the successful construction of the project. These plans provide the construction design requirements, specifications, and other construction documents necessary for the construction bidding, permitting, and inspection. For the ESCP to be compatible with the other engineering plans, the most practical process may be for the consultant to develop BMPs objectives for the construction period based on contractor activities and the grading and drainage plans for the site.

This step can occur as part of the preparation of the grading and drainage plan and be included in the bid package and/or construction documents for consideration of the potential contractors. This allows the developer to explicitly address unique site conditions, which may affect the stormwater pollution control during construction. Alternatively, the developer could require the contractor to prepare such a plan to justify the selection of BMPs.

12.5.3.3 Construction Phase

ESCP is heavily involved in construction phase of a development. This involvement can be further divided into bidding and mobilization stage, construction operation stage, and construction completion stage. During bidding and mobilization, ESCP plan should be completed with erosion and sediment control for each development phases being properly laid out. Contractor(s) selected by developer should be able to initiate construction activity. Physical setup for ESCP implementation including BMPs construction, record keeping system, and personnel training should take place as soon as development starts.

During construction operation, the developer should ensure that the selected contractor is responsible for implementing the BMPs according to the ESCP. Because site conditions will inevitably vary during construction, the ESCP should be revised as necessary, with any changes highlighted on the ESCP copy maintained at the construction site. The inspection and maintenance plan (part of ESCP) will play a major role in ensuring all BMPs are implemented as planned and that the overall ESCP is servicing the development at a satisfactory level throughout the entire project.

Upon completion of construction, it is the responsibility of the developer and contractor to ensure that:

(a) All treatment techniques or structures that are no longer required are removed using method approved by authorities;

(b) Sediment and other unwanted materials are disposed off in an approved manner;

(c) Slopes, embankments, vegetation and planting areas have been properly established;

(d) Site access is returned to its original condition or approved final layout depending on site-specific circumstances;

(e) All permanent structures constructed to serve as part of ESCP are restored to its designed condition;

(f) All safety standards are conformed and that site is safe to be occupied;

(g) All drainage ways, pond areas, and reserves are properly gazetted in the final site plan; and

(h) Standard operation procedure (SOP) or maintenance plan for stormwater structures is produced whenever deemed necessary by authorities.

12.5.4 Performance Evaluation of ESCP

The final step in the preparation of an ESCP is to develop a program to monitor how well the BMPs work and to evaluate whether additional BMPs are required. To meet these objectives, ESCP implementer is required to conduct site inspection and monitoring, systematic record keeping and regular review and modification (if necessary) on the ESCP.

12.5.4.1 Site Inspection & Monitoring

An inspector should be identified and specified in the submitted ESCP. The inspector is responsible to plan for regular inspection, monitoring, and record keeping of the ESCP. This person shall provide any on-site details on the implementation and performance of ESCP to relevant authorities or participate in joint inspection with local regulatory authorities whenever required.

Site inspections should be carried out on two bases. Inspection should be carried out on a fixed interval (e.g. weekly, fortnight, monthly). On top of that, event-based inspection should be carried out before and after storm events.

Inspection shall gauge and monitor the performance of the components of ESCP, and should cover construction management as well as erosion and sediment control BMPs. For contractor activity BMPs, inspection would include:

- looking for evidence of spills and resulting clean-up procedures (e.g. supplies of spill cleanup material);
- examining the integrity of containment structures;
- verifying the use of employee education programmes for the various activities;
- noting the location of activity (e.g. outdoor vs. indoor, concrete vs. grass);
- verifying adequacy of trash receptacles; and
- verifying waste disposal practices (e.g. recycle vs. hazardous waste bins).

For sediment and erosion control BMPs, the monitoring program should consist of regular inspection to determine the following:

- Changes in drainage patterns. Ensure all runoff, i.e. disturbed or natural flows are well managed. Changes of drainage pattern due to earthwork must be accounted for in ESCP. In the event of drainage pattern change in unanticipated manner, appropriate modification on ESCP shall be performed to rectify situation;
- Installation of BMPs. Installation of BMPs shall be as per designed. Any site adjustment shall be referred back to the consultant for approval;
- Site stabilization. Inactive areas (no activity for more than 30 days) must be stabilized with erosion control BMPs or structural method within 7 days;
- Effectiveness of BMPs. Indicators of underperformed BMPs include structural failure, sediment outside of control area, formation of rills and gullies, runoff overspill (flash flood), murky site discharge, etc.; and
- Maintenance of BMPs. All BMPs required to be serviced in accordance to design specification. Routine maintenance shall include vegetation watering, replacement of protection materials (vegetation, silt fence, rip rap etc), structural repair, and sediment removal from sediment control BMPs.

12.5.4.2 Record Keeping

- Results of the inspection shall be recorded in the site inspection checklist form and in inspection log book. Particulars such as date of the inspection, the inspectors and important results or observations must be included. Significant changes such as additional BMPs, change in capacity/structure, location etc, shall also be involved.
- All records are to be retained for at least 3 years by the developer.
- It is suggested that details of incidents such as spills or BMPs failures (flood, structural collapse or drain clogging) be kept. This information can be particularly useful during review of BMPs.
- Photographs may be useful and powerful description of incidents and proof.

12.5.4.3 Plan Review and Modification

During the course of construction, unexpected schedule changes, phasing changes, staging area modifications, off-site drainage impacts, and repeated failures of designed controls may affect the implementation and performance of ESCP. These changes must be made known and the ESCP revised accordingly. During the preparation and review of the modified ESCP, construction may continue with temporary modifications to the erosion and sediment control BMPs. Revisions to the ESCP are also required when the properly installed systems are ineffective in preventing silt transport off the site. Modifications to the ESCP shall be carried out by qualified engineers, preferably the same party which design the ESCP initially.

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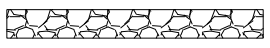
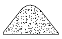
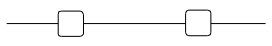
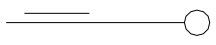
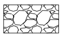
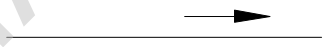
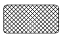
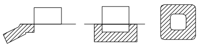
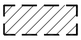
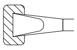
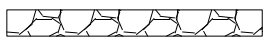
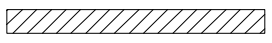
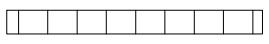
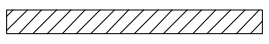
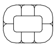
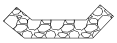
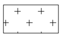
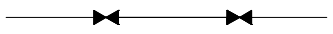
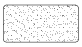
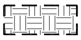
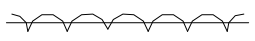
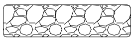
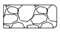
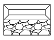
APPENDIX 12.A BMPs LEGEND AND PLAN SYMBOLS

Item	BMPs Names	Legend	Symbol
1	Check Dam	CD	
2	Compost Blanket	CB	
3	Compost Filter Berm	CFB	
4	Concrete Washout Area	CWA	
5	Construction Fence	CF	
6	Dewatering	DW	
7	Diversion Ditch	DD	
8	Drainage Outlet Protection	OP	
9	Earth Bank	EB	
10	Erosion Control Blanket	ECB	
11	Inlet Protection	IP	
12	Limits Of Construction	LOC	
13	Materials Storage Area	MSA	
14	RRB For Culvert Protection	RRC	
15	Reinforced Check Dam	RCD	
16	Reinforced Rock Berm	RRB	
17	Sand Bag Barrier	SBB	
18	Sediment Control Log	SCL	
19	Sediment Basin	SB	
20	Sediment Trap	ST	
21	Seeding And Mulching	SM	
22	Silt Fence	SF	
23	Stabilized Staging Area	SSA	
24	Stockpile Area	SPA	
25	Surface Roughening	SR	
26	Temporary Slope Drain	TSD	
27	Temporary Stream Crossing	TSC	
28	Terracing	TER	
29	Vehicle Tracking Control	VTC	
30	VTC With Wheel Wash	WW	

APPENDIX 12.B EXAMPLE – MODEL ESCP PREPARATION

Problem:

This example illustrates a proper ESCP Plan prepared for a agricultural land development in Cameron Highlands. The information from this example is repetitively used in the following examples to demonstrate engineering analysis and design for ESCP.

The proposed development on Lot 1587, Blue Valley, Cameron Highland is to transform the undeveloped into a high-tech hydroponic farm. The initial development covers over 28ha of steep terrain. The current example concentrate on one of the subdivided plot (Plot 1, hereafter referred to as the site) of the development, which only covers 2.74 ha of land. The challenge in this project is to transform the steep and undulating terrain into almost flat platforms to house the hydroponic greenhouse structures. In the final design, rainwater from the roofs of the greenhouses will be partially stored in underground on-site-detention tanks, and will be recycled for irrigation use.

Solution:

The ESCP must be prepared before construction begins, during the project planning and design phase. It was completed at the end of the design phase or early in the construction phase. In this project, once the final design platform, building footprint and infrastructures alignment are obtained, the design of earthwork and ESCP then took place.

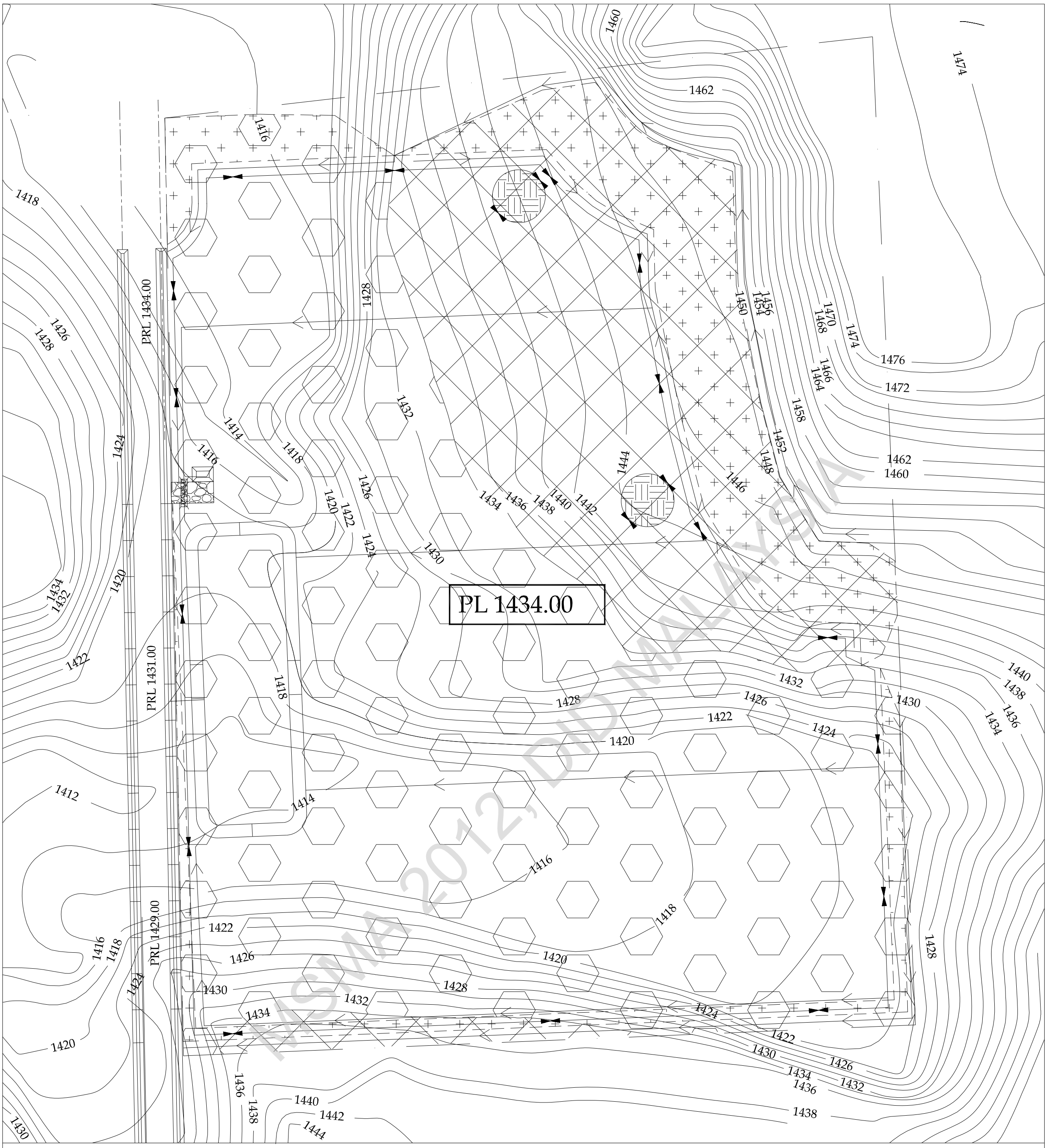
First, the erosion risk analysis is carried out to determine annual soil loss for existing and during construction condition at site. Then, preliminary ESC BMPs are tested to achieve required soil loss control on a site-scale basis. This assessment is presented in Appendix 12.C.

The ESCP prepared for the site is based on the 8 ESC principles given in this manual which include:

- Minimizing Soil Erosion – The north-eastern corner of the site consists of very steep slope, which is unsuitable for development. Therefore the area is being preserved as natural green. In order to further reduce erosion, earthwork is carried out in two phases. In the first phase, the proposed sediment basin is first constructed, together with the site access. Earthwork will progress by filling the access entrance, basin area advancing into the north-eastern area. Temporary diversion drains will be constructed as planned as earthwork progresses further into the site. The slope is trimmed and cut to desired terrace and platform level. As the first phase earthwork is completed and stabilized, second phase begins by continuing filling of the southern side of the site. Diversion drain is supplied to convey runoff from undisturbed land into the service drain, away from disturbed site. Additional diversion drain is constructed to convey runoff into sediment basin as required.
- Preserving Top Soil – Topsoil removed from site clearing are stockpiled and protected from erosion within the site for reuse in landscaping and erosion control turfing.
- Access Route – Access Route is provided near to the sediment basin. A fully paved (10m) access point is used as the only entry point to the site and is equipped with a washing bay.
- Drainage Control – Generally, runoff from undisturbed areas are diverted into service drain directly without additional treatment. Runoff from disturbed areas is collected by a diversion channel network which will be treated by sediment basin before being discharged into service drain.
- Erosion Control – Erosion control are provided via vegetation. At terraced slope, hydroseeding are provided. After earthwork is completed, areas not involved in construction are to be provided with close turfing.
- Sediment Prevention – Sediment control BMPs are provided via silt fence and sediment basin. The site perimeter is temporarily secured by using silt fence to avoid sediment leakage from site prior during earthwork. Runoff from the entire disturbed area is collected and treated by a wet sediment basin. Design of sediment basin is presented in Appendix 12.E.

- Slope Stabilization – All steep slopes in the site are provided with proper engineering stabilization. Terracing is provided for high slopes, with proper cut-off and cascade drains. All slope surfaces are protected with hydroseeding.
- Maintenance – The entire ESCP is designed with maintenance in mind. All structures are designed for minimal maintenance requirements. Detailed BMPs maintenance requirements and overall site maintenance are provided together with the ESCP report.

The final ESCP are presented in two layout plans, i.e. Pre-bulk Grading and Post-bulk Grading plans. In the pre-bulk grading plan (Figure 12.B1), the cut and fill areas are delineated, including areas to be preserved. In this plan, the temporary ESC BMPs to be implemented during the earthwork are presented. The post-bulk grading plan (Figure 12.B2) presents the ESC after major earthwork is completed. First of all the treated slope is converted into grassed terrace slope with proper cut-off and cascade drains. Runoff from this area are combined with flow from undisturbed areas and diverted away from site. The plan also include proposed building footprint of the development. Some runoff management structures such as culvert crossing are converted into permanent structures at this stage of development.



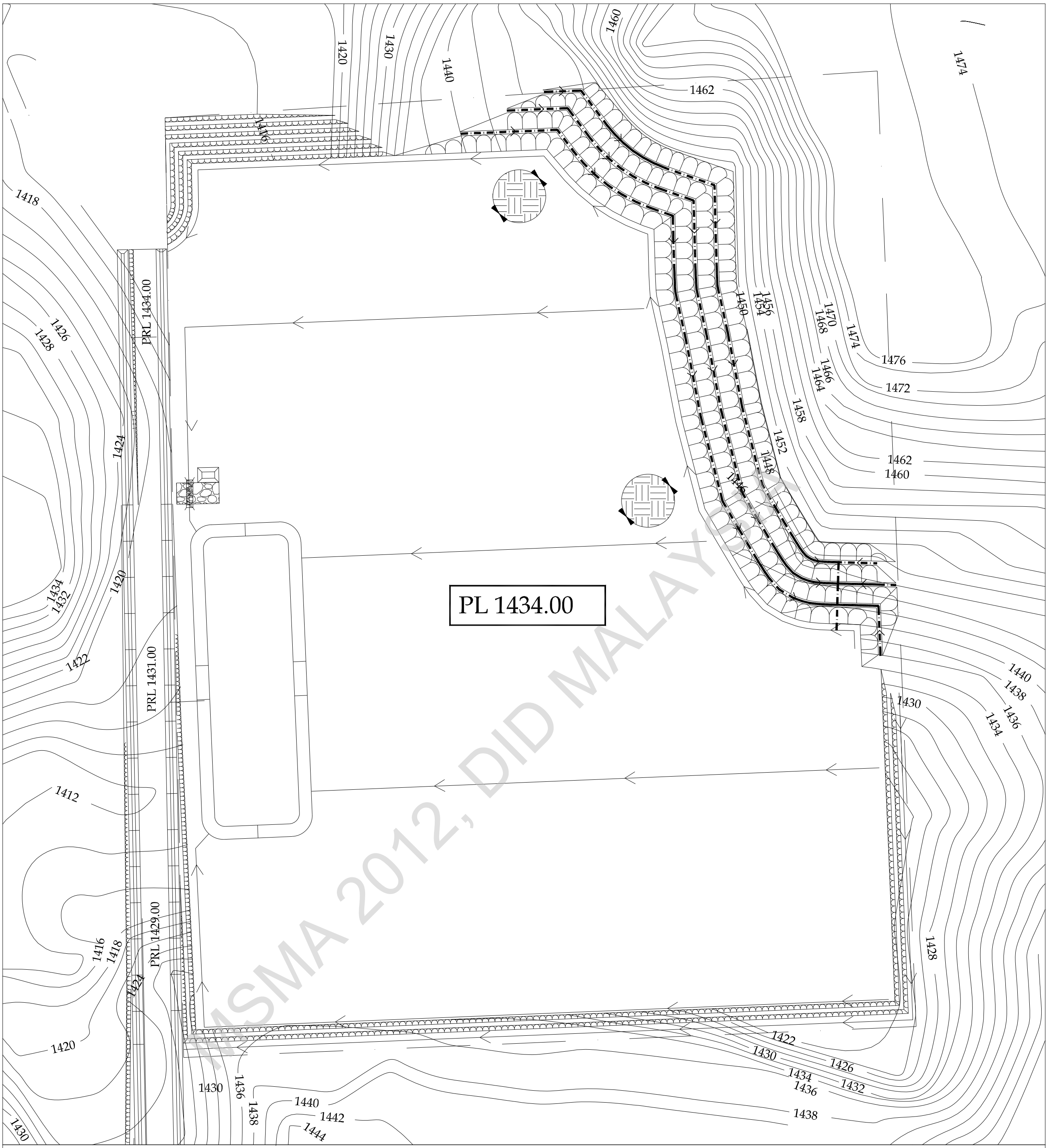
LEGEND

Note:
All Dimensions are in meters

⬡	Fill
▨	Cut
▸◂	Silt Fence
⬚	Site Boundary
←	Diversion Channel (Disturbed Flow)
←	Diversion Channel (Natural Flow)

▦	Site Access Point with Wash Bay
▭	Temporary Waterway Crossing
▦	Stockpile
▭	Sediment Basin
⬚	Terracing with Hydroseeding

Figure 12.B1: Pre-Bulk Grading ESCP



LEGEND

Note:
All Dimensions are in meters

▩	Fill
▩	Cut
—▸◂—	Silt Fence
⬚	Site Boundary
←	Diversion Channel (Disturbed Flow)
←	Diversion Channel (Natural Flow)

▦	Site Access Point with Wash Bay
▭	Temporary Waterway Crossing
▦	Stockpile
▭	Sediment Basin
⬚	Terracing with Hydroseeding

Figure 12.B2: Post-Bulk Grading ESCP

APPENDIX 12.C EXAMPLE – SOIL LOSS ESTIMATION WITH USLE

Problem:

Using details from the development project mentioned in Appendix 12.B, a soil loss assessment is to be carried out to determine the erosion risk at site. The assessment shall examine erosion risk before development took place, during earthwork without control and during earthwork with control to justify effective BMPs selected for the site.

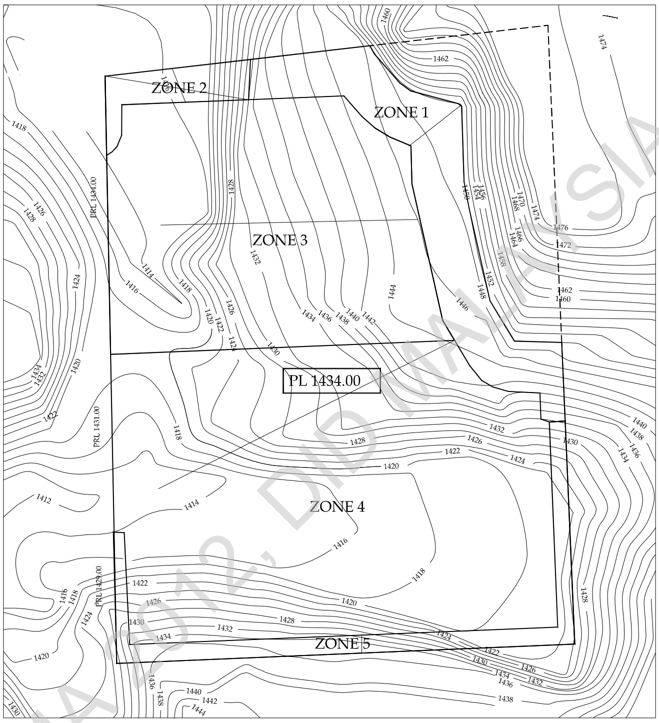
Solution:

<table>
<tr><th>Solution</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>In order to provide better assessment of the various conditions at site, the site is divided into 5 zones, which are considered to be homogenous in terms of erosion risk. The zone delineations are given in Figure 12C.1. Universal Soil Loss Equation (USLE) will be used to assess the erosion risk of the site under three conditions, i.e. existing (undisturbed), disturbed and uncontrolled (no ESC), and disturbed but controlled (with ESC). The example below shows step by step guide to obtain necessary USLE parameters for the Zone 3 under existing condition. The procedures to obtain USLE parameters for other zones and development conditions are exactly the same as that shown in this example.</td><td></td></tr>
<tr><td>Figure 12.5</td><td>(1) Determination of Rainfall Erosivity, <i>R</i> Factor:
<i>R</i> factor for the area of Tanah Rata (Pahang) falls in the range of 15,000 to 17,500 MJ.mm/ha.hr.yr. For evaluation purpose, the higher limit is used, therefore,

<i>R</i> Factor = 17,500 MJ.mm/ha.hr.yr</td><td><i>R</i> = 17,500 MJ.mm/ha.hr.yr</td></tr>
<tr><td>Equation 12.3</td><td>(2) Determination of Soil Erodibility, <i>K</i> Factor :
In determining the K factor of the develop area, soil data obtained from hand auger method for the site is used. The soil samples are tested for grain analysis and the results are converted to 100% of sand, sit, clay and organic matter (excluding larger particles), as shown in Table 12C.1.

Table 12.C1: Summary of Laboratory Test Result of Soil Data
<table>
<tr><th rowspan="2">Hand Auger No.</th><th rowspan="2">Sample Number</th><th rowspan="2">Depth (m)</th><th colspan="3">Particle Size Distribution (%)</th></tr>
<tr><th>Clay</th><th>Silt</th><th>Sand</th></tr>
<tr><td>HA 1</td><td>A</td><td>0.5</td><td>12.83</td><td>35.99</td><td>51.19</td></tr>
<tr><td></td><td>B</td><td>1.0</td><td>15.94</td><td>42.11</td><td>41.95</td></tr>
</table></td><td></td></tr>
<tr><td>Table 12.2</td><td><table>
<tr><th>Hand Auger No.</th><th>Sample Number</th><th>Structure Code (<i>S</i>)</th><th>Permeability Code (<i>P</i>)</th><th>K Factor</th></tr>
<tr><td>HA 1</td><td>A</td><td>2</td><td>3</td><td>0.0244</td></tr>
<tr><td></td><td>B</td><td>2</td><td>3</td><td>0.0266</td></tr>
</table></td><td></td></tr>
<tr><td></td><td>For existing (undisturbed) condition, erosion took place on ground surface and hence the concerned soil layer is Sample A. For disturbed condition (uncontrolled and controlled), the site has been disturbed and hence Sample B (representing soil at 1m depth) is used, assuming earth work removed the top soil and exposed the lower soil layer. Therefore,

<i>K</i> Factor = 0.0244</td><td><i>K</i> = 0.0244</td></tr>
</table>

Reference	Calculation	Output
	(3) Determination of *LS* Factor: In order to obtained LS factor, the horizontal slope length (m) and slope steepness (%) needs to be determined through elevation information available on site. For existing condition, the contours from topographic maps or survey works can be used. For disturbed conditions, designed platform levels can be used. It is reminded that the assessment evaluate the average soil loss and hence, the determination of slope length and slope steepness shall not be based on most critical, but values that represents the average of the entire zone. Referring to Figure 12C.1 the length and slope for Zone 3 (existing condition) can be worked out as below,	
	 Figure 12.C1: Zone Delineation and Slope Length for Development Site	
	Horizontal slope length, λ = 105m	λ = 105m
	Slope steepness, s = (1446-1413)/105 x 100 = 31%	s = 31%
Table 12.3	Matching λ and s in Table 12.3, the value for LS factor can be obtained using linear interpolation. Therefore,	
	LS Factor = 19.7	*LS* = 19.7
Table 12.4	(4) Determination of *C* Factor: For existing condition, the entire site is covered with thick forest, and hence 100% forest coverage was selected. Therefore,	
	C Factor = 0.03	*C* = 0.03

Reference	Calculation	Output
Table 12.5	(5) Determination of CP Factor: For existing condition, no management support practice is provided (undisturbed site) and therefore, *P* Factor = 1.00	*P* = 1.00
Equation 12.1	(6) Determination of Soil Loss: Estimation of soil loss for Zone 3 can then be completed: *A* = 17,500 x 0.0244 x 19.7 x 0.03 x 1.00 = 252 tonne/ha/yr	*A* = 252 tonne/ha/yr

Using same procedure, the soil loss for other zones and development conditions can be determine, as shown in Table 12C.2. It can be observed that uncontrolled land disturbance can cause significant increase in erosion risk. Annual soil loss for Zone 1, 2 and 5 increase due to the increase in elevation factors where cut and fill has cause dramatic increment in elevation difference compared to existing condition. However, the ESCP provides corrective measures to mitigate the situation by providing terracing practise and hydroseeding. For Zone 3 and 4, soil loss are significantly reduced even in uncontrolled condition compared to exiting condition because of the flatten land for development. Eventually, the implementation of ESCP manages to minimise increment of soil loss to a satisfactory level.

Table 12.C2: Summary of Soil Loss Assessment for Development Site

Condition	Parameters	Zone				
		1	2	3	4	5
Existing	*R*	17,500	17,500	17,500	17,500	17,500
	K	0.0244	0.0244	0.0244	0.0244	0.0244
	LS	2.2836	16.6415	19.6936	14.9137	3.4415
	C	0.03	0.03	0.03	0.03	0.03
	P	1.00	1.00	1.00	1.00	1.00
	A (ton/ha/yr)	29	213	252	191	44
Earthwork-Uncontrolled	*R*	17,500	17,500	17,500	17,500	17,500
	K	0.0266	0.0266	0.0266	0.0266	0.0266
	LS	27.4147	50.6281	0.1224	0.1267	0.994
	C	1.00	1.00	1.00	1.00	1.00
	P	1.00	1.00	1.00	1.00	1.00
	A (ton/ha/yr)	12,762	23,567	57	59	463
Earthwork-Controlled	*R*	17,500	17,500	17,500	17,500	17,500
	K	0.0266	0.0266	0.0266	0.0266	0.0266
	LS	27.4147	50.6281	0.1224	0.1267	0.994
	C	0.05	0.05	1	1	0.05
	P	0.18	0.18	0.5	0.5	0.18
	A (ton/ha/yr)	115	212	28	29	4

APPENDIX 12.D EXAMPLE – SEDIMENT YIELD ESTIMATION

Problem:

The design of all sediment control BMPs requires the information on the quantity of sediment it is expected to trap. Therefore, determination of sediment yield for sediment basin catchment area is required for this site. The design rainfall is set to be 50mm as per requirement of this manual.

Solution:

Reference	Calculation	Output
	The catchment area for the proposed sediment basin includes Zone 1 (during pre-bulk grading only), Zone 3 and Zone 4. Therefore, the sediment yield from these 3 catchments should be determined for possible highest sediment yield condition, i.e. during the earthwork (pre-bulk grading plan). The Modified Universal Soil Loss Equation is used to determined the sediment yield for sediment basin.	
	1. Determination of Runoff Parameters Runoff parameters required for sediment yield includes runoff peak discharge and runoff volume. Both parameters, however requires the knowledge of design rainfall.	
	(a) Design storm:	
	Zone 3: Design Storm = 50 mm Catchment Area, *A* = 1.84 ha	
Table 1.3	Time of Concentration, t_c = 20 minutes Duration of storm, *D* = 60 minutes(Assume 1hour)	
Section 2.2.2	Intensity of design storm, *I* = 50 mm/h	
	Zone 4: Design Storm = 50 mm Catchment Area, *A* = 1.27 ha	
Table 1.3	Time of Concentration, t_c = 25 minutes Duration of storm, *D* = 60 minutes(Assume 1 hour)	
Section 2.2.2	Intensity of design storm, *I* = 50 mm/h	
	(b) Calculate Peak Discharge:	
	Zone 3: Rational Method	
Section 2.3.1 Table 2.6	Runoff Coefficient, *C* = 0.50 (Bare soil) Intensity of design storm, *I* = 50 mm/h Catchment Area, *A* = 1.84 ha	
	Q_p = C x *I* x *A*/360 = (0.5 x 50 x 1.84)/360 = 0.128 m^3/s	*Zone 3:* Q_p =0.128m^3/s
	Zone 4: Rational Method	
Section 2.3.1 Table 2.6	Runoff Coefficient, *C* = 0.50 (Bare soil) Intensity of design storm, *I* = 50 mm/h Catchment Area, *A* = 1.27 ha	

Reference | Calculation | Output

$Q_p = C \times I \times A/360 = (0.5 \times 50 \times 1.27)/360$
$= 0.088\text{m}^3/\text{s}$

Zone 4:
$Q_p = 0.088\text{m}^3/\text{s}$

(c) Calculate Runoff Volume:

Zone 3:

Section 2.3.2 — Rational Method Hydrograph Method (Type 2)
Time of Concentration, t_c = 20 min
Peak Discharge, Q_p = $0.128\text{m}^3/\text{s}$

Therefore,
$V = 0.5 \times (2 \times T_c) \times (Q_p)$
(Area below hydrograph)
$= 0.5 \times (2 \times 20 \times 60) \times (0.128)$
$= 153.6\text{m}^3$

Zone 3:
$V = 153.6\text{m}^3$

Zone 4:

Section 2.3.2 — Rational Method Hydrograph Method (Type 2)
Time of Concentration, t_c = 25 min
Peak Discharge, Q_p = $0.088\text{m}^3/\text{s}$

Therefore,
$V = 0.5 \times (2 \times t_c) \times (Q_p)$
(Area below hydrograph)
$= 0.5 \times (2 \times 25 \times 60) \times (0.088)$
$= 132.0\text{m}^3$

Zone 4:
$V = 132.0\text{m}^3$

2. <u>Calculation of Sediment Yield</u>

In this case, the value of K, LS, C, and P factors are assumed the same as those used for soil loss estimation for disturbed (uncontrolled) condition. Using (MUSLE), the sediment yield can be calculated as shown below:

Equation 12.4

Zone	Runoff Volume, V (m^3)	Peak Discharge, Q_p (m^3/s)	K Factor	LS Factor	C Factor	P Factor	Sediment Yield, Y (tonne)
3	153.6	0.128	0.0266	5.3709	1.00	1.00	67.87
4	132.0	0.088	0.0266	0.1267	1.00	1.00	1.19
Total sediment yield to sediment basin							69.06

Note: LS for Zone 3 is taken as weighted average of LS factor for Zone 3 and Zone 1 shown previously,
where $LS = (LS_{zone3}(\lambda_{zone3}) + LS_{zone3}(\lambda_{zone3}))/(\lambda_{zone3} + \lambda_{zone3})$

Thus, the total sediment yield to sediment basin is: Y = 69.06 tonne (for the design storm)

Y = 69.06 tonne

APPENDIX 12.E EXAMPLE – WET SEDIMENT BASIN

Problem:

Right after the final selection of BMPs is identified through soil loss assessment, the detailed design of the BMPs is to be carried out. The dimension and outlet sizing of the proposed sediment basin should be determined. Frequency of maintenance should also be estimated using available information.

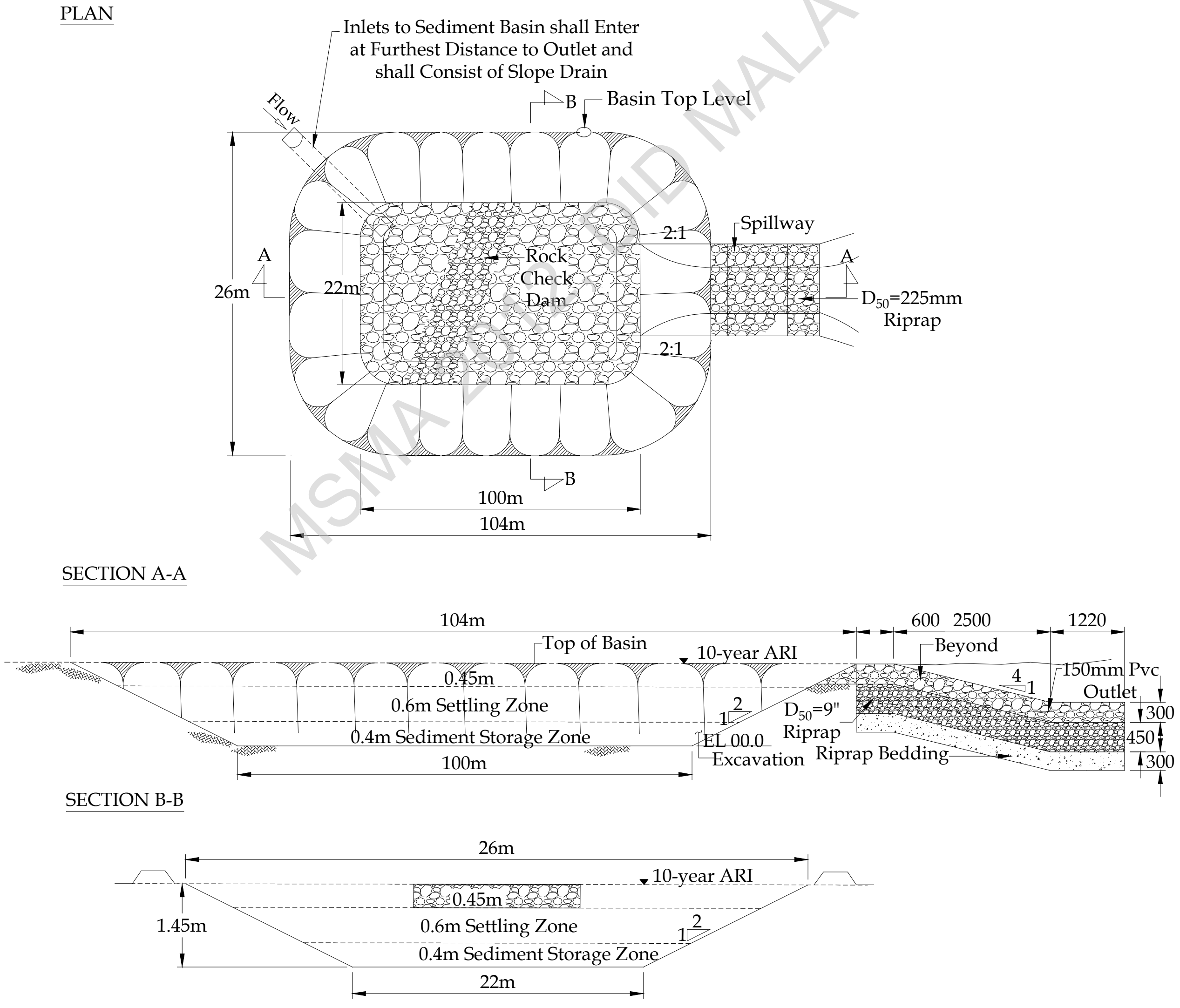
Solution:

Reference	Calculation	Output
	The detailed design of the sediment basin is presented in this section. Detailed design of temporary crossing and diversion drains are similar to those of their permanent counter parts, as presented in other chapters of this manual and therefore are not shown here.	
	(1) Determination Type of soil:	
Table 12.16	According to Table 12.A1, the soil in the Plot 1 is loam. Hence, a wet sediment basin is chosen.	
	(2) Determination of Basin Dimension:	
Table 12.19	The required surface area is 340m²/ha and the required total volume is 510m³/ha (high runoff is selected due to bare soil condition). Figure 12.E1 illustrates the final dimensions of the sediment basin.	
	The surface area required for the site = 340 x 3.11 = 1,057.4m² (Note: this is the average surface area for the settling zone volume, i.e. at mid-depth)	Required Surface Area = 1057.4m²
	The total basin volume required for the site = 510 x 3.11 = 1,586.1m³	Required Volume = 1,586.1m³
	(a) Settling Zone:	
Table 12.17	The required settling zone, V_1 = 793.05m³ (half the total volume) and the selected settling zone depth, y_1 = 0.75m.	
	Try a settling zone average width, W_1 = 16m Required settling zone average length $L_1 = \frac{V_1}{W_1 \text{ x } y_1} = \frac{793.05}{16 \text{ x } 0.75}$ = 66.09m , say 67m	
	Average surface area = 16 x 67 =	1,072m² > 1,057.4m² ; OK
	Check settling zone dimensions (Table 12.17: Basin Dimension): $\frac{L_1}{y_1}$ ratio = $\frac{67}{0.75}$ =	89.33 < 200; OK
	$\frac{L_1}{W_1}$ ratio = $\frac{67}{16}$ =	4.19 > 2; OK

Reference	Calculation	Output
	(b) Sediment Storage Zone: The required sediment storage zone volume is half the total volume, V_2 = 793.05m^3 For a side slope Z = 2(H):1(V), the dimensions at the top of the sediment storage zone are, $W_2 = W_1 - 2 \times \frac{d_1}{2} \times Z = 16 - 2 \times 0.375 \times 2 = 15\text{m}$ $L_2 = L_1 - 2 \times \frac{d_1}{2} \times Z = 67 - 2 \times 0.375 \times 2 = 66\text{m}$ The required depth for the sediment storage zone, which must be at least 0.3 m, can be calculated from the following relationship, $V_2 = Z^2 y_2^3 - Z y_2^2 (W_2 + L_2) + y_2 (W_2 L_2)$ which gives, $793.05 = 4y_2^3 - 162y_2^2 + 990y_2$ Use trial and error to find y_2 ,	
	For y_2 = 0.8m, V_2 = 690m^3 For y_2 = 0.9m, V_2 = 762m^3 For y_2 = 1m, V_2 = 832m^3	$y_2 >$ 0.3m; $V_2 >$ 793.05m^3 ; OK
	(c) Overall Basin Dimensions: Base: $W_B = W_1 - 2 \times Z \times \left(\frac{y_1}{2} + y_2\right) = 11\text{m}$ $L_B = L_1 - 2 \times Z \times \left(\frac{y_1}{2} + y_2\right) = 62\text{m}$	Total Basin Dimension: W_B= 11m L_B = 62m
	Depth: Settling Zone, y_1 = 0.75m Sediment Storage Zone, y_2 = 1.00m Side Slope Z = 2(H):1(V)	y_1=0.75m y_2=1.00m Z = 2(H):1(V)
Table 12.17	(3) <u>Sizing of Basin Outlet:</u> The spillway for this sediment basin must be design for 10-years ARI. The proposed spillway dimension is 1.5m wide x 0.3m high. The sill level must be set a minimum 300 mm above the basin top water level. To simplify the calculations, the following assumptions are made: • assume riser pipe flow is orifice flow through the top of the pipe only • riser pipe head is 300 mm, i.e. the height between the top of the pipe and the spillway crest level $Q_{\text{required}} = Q_{10} - Q_{\text{riser}}$	

<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>(a) Determination of Q_{10}</td><td></td></tr>
<tr><td>Equation 2.2</td><td>$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$$
where:
i = the average rainfall intensity (mm/hr) for selected ARI (T) and storm duration (d);
T = average recurrence interval, ARI (years);
d = storm duration (hours); $0.20 \le d \le 72$and
λ, κ, θ and η = fitting constants dependent on the raingauge location</td><td></td></tr>
<tr><td>Appendix 2.B
Table 2.B2</td><td>
<table>
<tr><th rowspan="2">Location & Station ID</th><th rowspan="2">ARI, T (years)</th><th rowspan="2">Storm duration d</th><th colspan="4">Derived Parameters</th></tr>
<tr><th>λ</th><th>κ</th><th>θ</th><th>η</th></tr>
<tr><td>Gunung Brinchang (4513033)</td><td>10</td><td>25</td><td>42.004</td><td>0.164</td><td>0.046</td><td>0.802</td></tr>
</table>
</td><td></td></tr>
<tr><td></td><td>$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} = \frac{42.004(10)^{0.164}}{\left(\left(\frac{25}{60}\right)+0.164\right)^{0.802}} = 133.32\text{mm/hr}$$</td><td>$^{10}I_{20}$ = 133.32 mm/hr</td></tr>
<tr><td>Section 2.3.1
Table 2.6</td><td>Rational Method,
Coefficient of Runoff, C = 0.5
Catchment Area, A = 3.11ha (Zone 3 and Zone 4)</td><td>C = 0.5
A = 3.11ha</td></tr>
<tr><td>Equation 2.3</td><td>$$Q_{10} = \frac{C.I.A}{360} = \frac{0.5 \times 133.32 \times 3.11}{360} = 0.576\text{m}^3/\text{s}$$
Try with 1 orifice with diameter 0.45 m at same level. Allow head of 0.3m from centroid of orifice.</td><td>Q_{10} = 0.58m^3/s</td></tr>
<tr><td></td><td>(b) Determination of Q_{riser}
$$Q_{riser} = C_o A_o \sqrt{2gH_o} = 0.6 \text{ x } \left(\frac{\pi(0.45)^2}{4}\right) \times \sqrt{2 \times 9.81 \times 0.3}$$
$$= 0.23\text{m}^3/\text{s}$$</td><td>Q_{riser} = 0.23m^3/s</td></tr>
<tr><td>Equation 2.6</td><td>Therefore, allowing for the riser pipe flow the required spillway capacity is:</td><td></td></tr>
<tr><td></td><td>(c) Sizing Spillway
$$Q_{required} = 0.58 - 0.23 = 0.35\text{m}^3/\text{s}$$</td><td>$Q_{required}$ = 0.25m^3/s</td></tr>
<tr><td>Equation 2.10</td><td>$$Q_{spillway} = C_{sp}\ B\ H_p^{1.5}.$$
Trial dimensions: B = 1.5m , H_p = 0.3m and C_{sp} = 1.48 from Table 2.7,
$$Q_{spillway} = 1.48 \text{ x } 1.5 \text{ x } 0.3^{1.5}$$ =</td><td>0.36m^3/s
> 0.35m^3/s; OK</td></tr>
</table>

Reference	Calculation	Output
	Therefore, the total basin depth including the spillway is, 0.75 + 1.0 + 0.3 + 0.3 = **2.35m** (4) <u>Trapping Efficiency:</u> From previous calculation in Appendix 12.D, the sediment yield is estimated at 69.06 tonnes for the design storm. With the design sediment trapping efficiency of 90%, the total sediment trapped for the design event is 62.154 tonne or 38.84 m^3 (converted from soil bulk density 1600kg/m^3). The sediment storage zone for the basin is 793.05m^3. Hence, the provided sediment basin can contain the settled sediment from Plot 1. It would require 793.05/38.84 = 20 events of same magnitude to fill up the sediment storage zone. Assuming the design storm is equivalent to 3-month ARI, the storage zone is likely to be filled up in 20/4 = 5 years.	



Engineering Drawing for Wet Sediment Basin for Development Site

APPENDIX 12.F EXAMPLE – DRY SEDIMENT BASIN

Problem:

Using information for the same sediment basin designed in Appendix 12.E, assume that the area is covered by sandy soil, re-design the basin into a dry sediment basin.

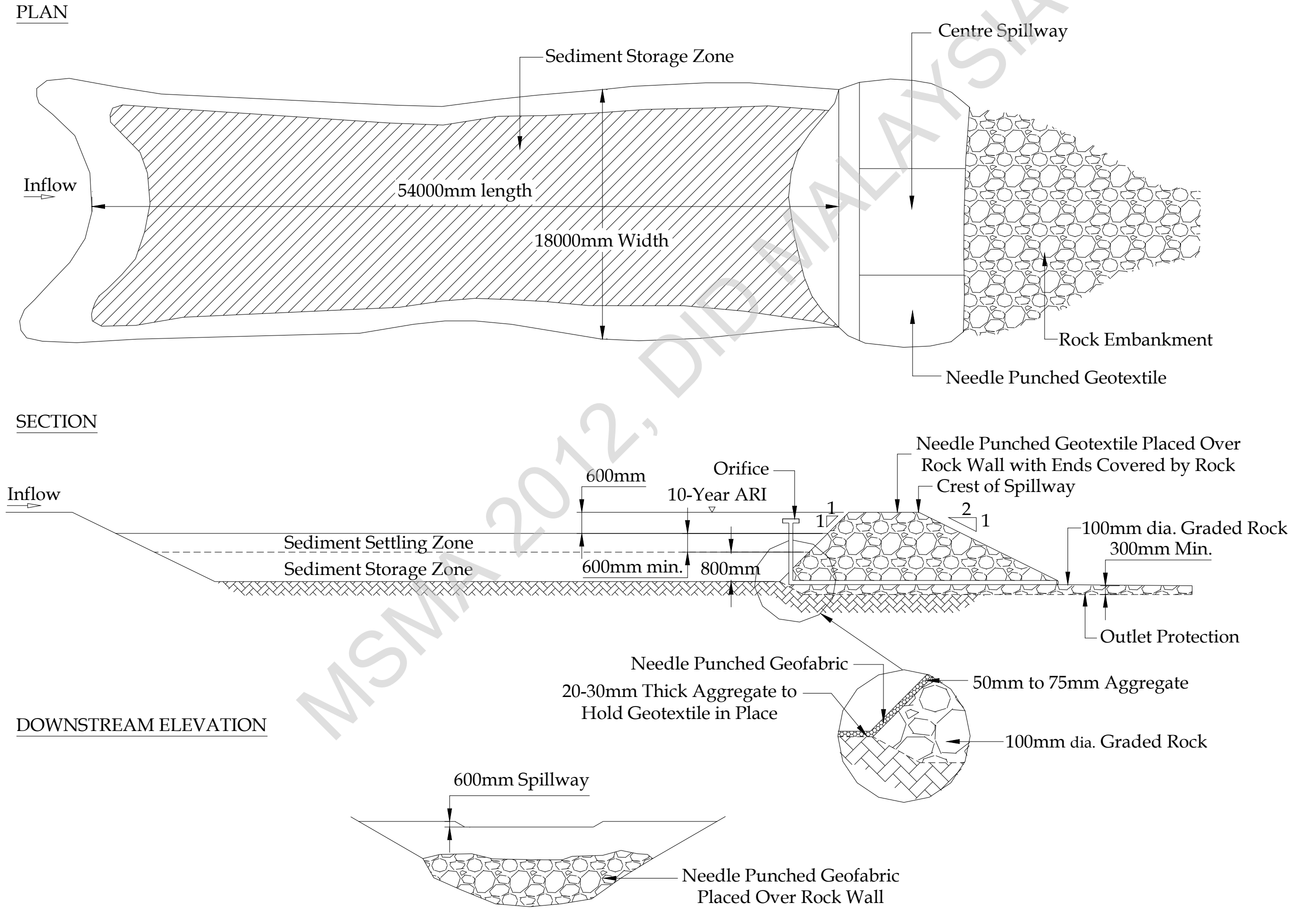
Solution:

Reference	Calculation	Output
	Design procedure of dry sediment basin is the same with wet sediment basin, except for the requirement of surface area and total basin volume. Calculations below show the design procedure of a dry sediment basin.	
	(1) Determination Type of soil:	
Table 12.16	Dry Sediment Basin is selected.	
	(2) Determination of Basin Dimension:	
Table 12.18	The required surface area is 250 m^2/ha and the required total volume is 300 m^3/ha (high runoff is selected due to bare soil condition). Figure 12.F1 illustrates the final dimensions of the sediment basin.	
	The surface area required for the site = 250 x 3.11 = 777.5m^2 (Note: this is the average surface area for the settling zone volume, i.e. at mid-depth)	Required Surface Area = 777.5m^2
	The total basin volume required for the site = 300 x 3.11 = 933m^3	Required Volume = 933m^3
	(a) Settling Zone:	
Table 12.17	The required settling zone, V_1 = 466.5m^3 (half the total volume) and the selected settling zone depth, y_1 = 0.60m.	
	Try a settling zone average width, W_1 = 15m Required settling zone average length	
	$L_1 = \frac{V_1}{W_1 \times y_1} = \frac{466.5}{15 \times 0.60}$ = 51.83m , say 52m	
	Average surface area = 15 x 52 =	780m^2 > 777.5m^2 ; OK
	Check settling zone dimensions (Table 12.17: Basin Dimension):	
	$\frac{L_1}{y_1}$ ratio = $\frac{52}{0.60}$ =	86.67 < 200; OK
	$\frac{L_1}{W_1}$ ratio = $\frac{52}{15}$ =	3.47 > 2; OK

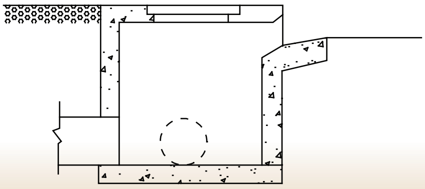
Reference	Calculation	Output
	(b) Sediment Storage Zone:	
	The required sediment storage zone volume is half the total volume, V_2 = 466.5m^3	
	For a side slope Z = 2(H):1(V), the dimensions at the top of the sediment storage zone are, $W_2 = W_1 - 2 \times \frac{d_1}{2} \times Z = 15 - 2 \times 0.30 \times 2 = 13.8$m say, 14m $L_2 = L_1 - 2 \times \frac{d_1}{2} \times Z = 52 - 2 \times 0.30 \times 2 = 50.8$m say, 51m	
	The required depth for the sediment storage zone, which must be at least 0.3m, can be calculated from the following relationship, $V_2 = Z^2 y_2^3 - Z y_2^2 \left(W_2 + L_2\right) + y_2 \left(W_2 L_2\right)$	
	which gives, $466.5 = 4y_2^3 - 130y_2^2 + 714y_2$	
	Use trial and error to find y_2,	
	For y_2 = 0.6 m, V_2 = 382 m^3 For y_2 = 0.7 m, V_2 = 437 m^3 **For y_2 = 0.8 m,** V_2 = 490 m^3	$y_2 >$ 0.3m; $V_2 >$ 466.5m^3; OK
	(c) Overall Basin Dimensions:	
	Base:	
	$W_B = W_1 - 2 \times Z \times \left(\frac{y_1}{2} + y_2\right) = 10.8$m say, 11m	W_B= 11m
	$L_B = L_1 - 2 \times Z \times \left(\frac{y_1}{2} + y_2\right) = 47.8$m say, 48m	L_B= 48m
	Depth: Settling Zone, y_1 = 0.75m Sediment Storage Zone, y_2 = 1.00m Side Slope Z = 2(H):1(V)	y_1=0.60m y_2=0.80m Z = 2(H):1(V)
	(3) Sizing of Basin Outlet:	
Table 12.17	The spillway for this sediment basin must be design for 10-years ARI. The proposed spillway dimension is 1.5m wide x 0.3m high.	
	The sill level must be set a minimum 300mm above the basin top water level. To simplify the calculations, the following assumptions are made: • assume riser pipe flow is orifice flow through the top of the pipe only • riser pipe head is 300mm, i.e. the height between the top of the pipe and the spillway crest level	
	$Q_{required} = Q_{10} - Q_{riser}$	

Reference	Calculation	Output
	(a) Determination of Q_{10}	
Equation 2.2	Using IDF Coefficient of Cameron Highlands and Duration of 20 minutes (see APPENDIX 12.E), $^{10}I_{20}$ = 133.32mm/h	$^{10}I_{20}$ = 133.32 mm/h
Section 2.3.1 Table 2.6	Rational Method, Coefficient of Runoff, C = 0.5 Catchment Area, A = 3.11ha (Zone 3 and Zone 4)	C = 0.5 A = 3.11ha
Equation 2.3	$Q_{10} = \frac{C.^{10}I_{20}.A}{360} = \frac{0.5\,x\,133.32\,x\,3.11}{360} = 0.576\text{m}^3/\text{s}$	$Q_{10} = 0.58\text{m}^3/\text{s}$
	Try with 1 orifice with diameter 0.45 m at same level. Allow head of 0.3m from centroid of orifice.	
	(b) Determination of Q_{riser}	
Equation 2.6	$Q_{riser} = C_o A_o \sqrt{2gH_o} = 0.6 \text{ x } \left(\frac{\pi(0.45)^2}{4}\right) \text{ x } \sqrt{2 \text{ x } 9.81 \text{ x } 0.3}$ $= 0.23\text{m}^3/\text{s}$	Q_{riser} = $0.23\text{m}^3/\text{s}$
	Therefore, allowing for the riser pipe flow the required spillway capacity is:	
	(c) Sizing Spillway	
	$Q_{required}$ = 0.48 – 0.23 = $0.25\text{m}^3/\text{s}$	$Q_{required}$ = $0.25\text{m}^3/\text{s}$
Equation 2.10	$Q_{spillway} = C_{sp}\ B\ H_p^{1.5}$.	
	Trial dimensions: B = 1.5m , H_p = 0.3m and C_{sp} = 1.48 ,	
Table 2.7	$Q_{spillway} = 1.48 \text{ x } 1.5 \text{ x } 0.3^{1.5}$	$Q_{spillway}$ = $0.36\text{m}^3/\text{s}$ > $0.25\text{m}^3/\text{s}$; OK
	Therefore, the total basin depth including the spillway is, 0.60 + 0.80 + 0.30 + 0.30 = 2.00m	

Reference	Calculation	Output
	(4) <u>Trapping Efficiency:</u> From previous calculation in Appendix 12.D, the sediment yield is estimated at 69.06 tonnes for the design storm. With the design sediment trapping efficiency of 90%, the total sediment trapped for the design event is 62.154 tonne or 38.84m^3 (converted from soil bulk density 1600kg/m^3). The sediment storage zone for the basin is 466.5 m^3. Hence, the provided sediment basin can contain the settled sediment from Plot 1. It would require 466.5/38.84 = 12 events of same magnitude to fill up the sediment storage zone. Assuming the design storm is equivalent to 3-month ARI, the storage zone is likely to be filled up in 12/4 = 3 years.	



Engineering Drawing for Dry Sediment Basin for Development Site

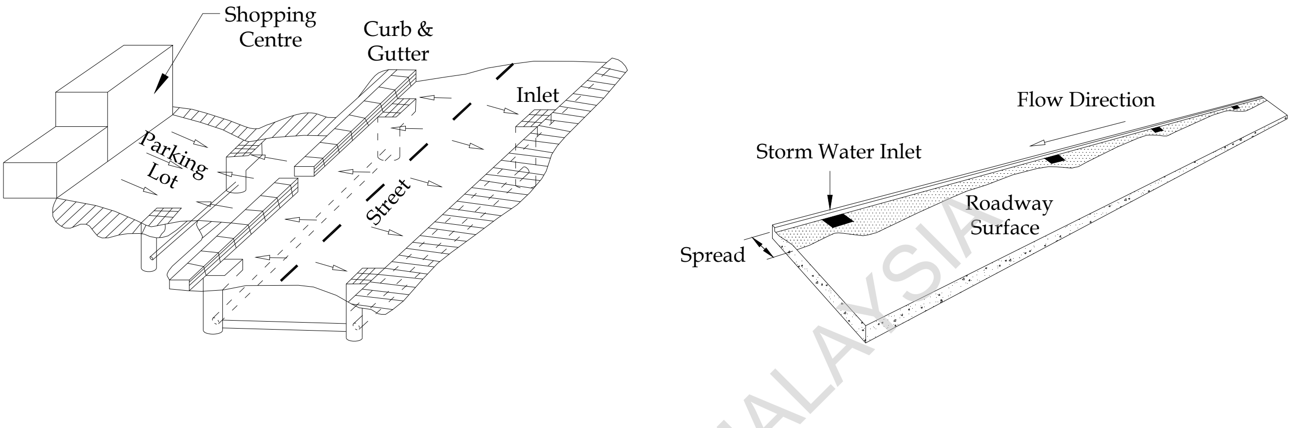


PAVEMENT DRAINAGE 13

CHAPTER 13 PAVEMENT DRAINAGE

13.1 INTRODUCTION

Effective drainage of pavements is essential to the maintenance of road service levels and to traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity (Figure 13.1). The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface. This chapter presents guidance for the design of these elements. Most of the information presented are adapted from Hydraulic Engineering Circular No. 22, Third Edition; Urban Drainage Design Manual (FHWA, 2009).



a) Drainage System b) Variation of Spread

Figure 13.1: Road Drainage Practices

13.2 HYDROPLANING REDUCTION

When rain falls on a sloped pavement surface, it forms a thin film of water that increases in thickness as it flows to the edge of the pavement. This wedge produces a hydrodynamic force which can lift the tyre off the pavement surface or hydroplaning. It has been shown that hydroplaning can occur typically at speeds of 90 km/hr with a water depth of 2mm. The hydroplaning potential of a roadway surface can be reduced by the followings:

- Design the roadway geometries to reduce the drainage path lengths of the water flowing over the pavement; and
- Provide drainage structures along the roadway to capture the flow of water over the pavement.

Table 13.1 provides design ARIs and spreads for different types of road and speeds. They have been set to limit the potential for hydroplaning at high speeds, as well as the potential for vehicles to float or be washed off roads at lower speeds. Risk associated with hydroplaning is high in tropical countries such as Malaysia.

13.3 DESIGN CONSIDERATION

13.3.1 Longitudinal Slope

The recommended minimum values of roadway longitudinal slopes (AASHTO, 1990) will provide safe, acceptable pavement drainage. In addition, the following general guidelines are presented:

- A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.
- Desirable gutter grades should not be less than 0.5% for curbed pavements with an absolute minimum of 0.3%. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.

- For adequate drainage in sag vertical curves, a minimum slope of 0.3 percent should be maintained within 15m of the low point of the curve.

Table 13.1: Design ARIs and Spreads (FHWA, 2009)

Road Classification		ARIs	Spread
High Volume or Divided or Bi-directional	< 70 km/hr	10 year	1 m
	> 70 km/hr	10 year	No Spread
	Sag Point	50 year	1 m
Collector	< 70 km/hr	10 year	½ Lane
	> 70 km/hr	10 year	No Spread
	Sag Point	10 year	½ Lane
Local Streets	Low Traffic	5 year	½ Lane
	High Traffic	10 year	½ Lane
	Sag Point	10 year	½ Lane

13.3.2 Cross / Transverse Slope

Table 13.2 presents an acceptable range of cross slopes with various pavement surface types (FHWA, 2009) . These cross slopes are a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort and safety. In areas of intense rainfall, a somewhat steeper cross slope, at 2.5% may be used to facilitate drainage.

Where three lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1%. The maximum pavement cross slope should be limited to 4%.

Table 13.2: Normal Pavement Cross Slopes (FHWA, 2009)

Surface Type	Cross Slope (%)
High-Type Surface	
2 lanes	1.5 – 2.0
3 or more lanes, each direction	1.5 minimum; increase 0.5 to 1.0 per lane; 4.0 maximum
Intermediate Surface	1.5 – 3.0
Low-Type Surface	2.0 – 6.0
Shoulders	
Bituminous or Concrete	2.0 – 6.0
With Curbs	≥ 4.0

Additional guidelines related to cross slope are:

- Although not widely encouraged, inside lanes can be sloped toward the median if conditions warrant;
- Median areas should not be drained across travel lanes;

- The number and length of flat pavement sections in cross slope transition areas should be minimised. Consideration should be given to increasing cross slope in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades; and
- Shoulders should be sloped to drain away from the pavement, except with raised, narrow medians and superelevations.

13.3.3 Curb and Gutter

Roads in urban areas shall generally be provided with an integral curb and gutter. However, where the volume of gutter flow is negligible as in car parks and on the high side of single-crossfall roads, a curb only is acceptable.

Curbs are normally used at the outside edge of pavement for low-speed, and in some instances adjacent to shoulders on moderate to high-speed roads. They serve the following purposes:

- Containment of the surface runoff within the roadway and away from adjacent properties;
- Prevention of erosion on fill slopes;
- Provision of pavement delineation; and
- Enable the orderly development of property adjacent to the roadway.

Gutters formed in combination with curbs are available in 0.3m through 1.0m width. Gutter cross slopes may be same as that of the pavement or may be designed with a steeper cross slope, usually 80 mm per metre steeper than the shoulder or parking lane (if used). An 8% slope is a common maximum cross slope (FHWA, 2009).

A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the travelled surface. Spread is what concerns the hydraulic engineer in curb and gutter flow. The distance of the spread is measured perpendicularly from the curb face to the extent of the water on the roadway and is shown in Figure 13.2.

The curb and gutter shall be a standard size to facilitate economical construction. The standard curb height of 150mm is based upon access considerations for pedestrians, vehicle safety including the opening of car doors, and drainage requirements.

If a local authority decides to adapt a different standard, the design curves given in this chapter will need to be adjusted accordingly.

13.3.4 Roadside and Median Channels

Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the pavement and from areas which drain toward the road. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent.

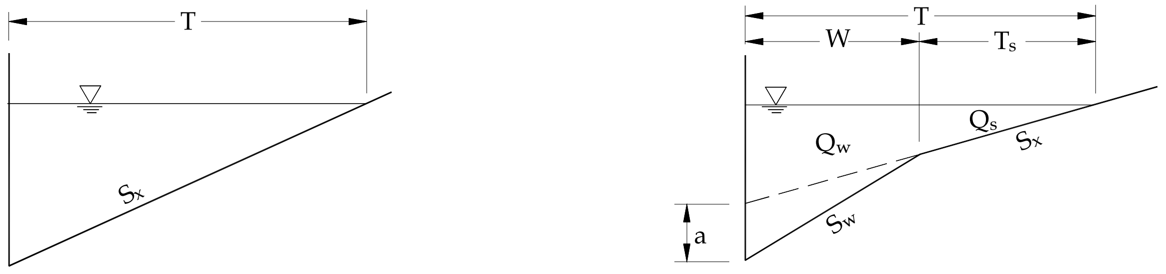
To prevent drainage from the median areas from running across the travel lanes, designers should slope median areas and inside shoulders to a center swale. This design is particularly important for high speed roads and for roads with more than two lanes of traffic in each direction.

13.3.5 Median Barriers

Designers should slope the shoulder areas adjacent to median barriers to the center to prevent drainage from running across the traveled pavement. Where median barriers are used, and particularly on horizontal curves with associated superelevations, it is necessary to provide inlets or slotted drains to collect the water accumulated against the barrier. Additionally, some road department agencies use a piping system to convey water through the barrier.

13.4 GUTTER DESIGN

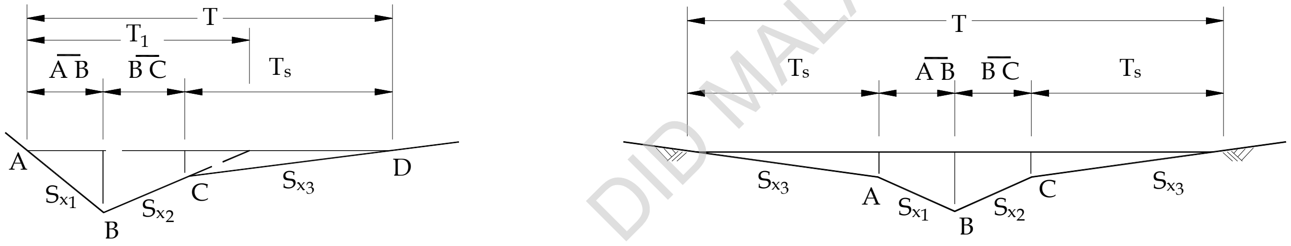
A pavement gutter is defined as a section of pavement of the roadway which conveys storm runoff. It may include a portion or all of a travel lane. Gutter sections can be categorized as conventional or shallow swale type as illustrated in Figure 13.2. Conventional curb and gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. Shallow swale gutters typically have V-shaped or circular sections and are often used in paved median areas on roadways with inverted crowns.



i) Uniform

ii) Composite

a) Curb Type



i) "V"-Shape-Shoulder

ii) "V"-Shape-Median

b) Shallow Swale Type

Figure 13.2: Typical Gutter Sections

13.4.1 Capacity Relationship

Gutter flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is:

$$Q = \frac{K_u}{n} S_X{}^{1.67} S_L{}^{0.5} T^{2.67} \qquad (13.1)$$

where,

K_u = 0.376
n = Manning's roughness coefficient (Table 13.3);
Q = Flow rate (m^3/s);
T = Width of flow or spread (m);
S_X = Cross slope (m/m); and
S_L = Longitudinal slope (m/m)

Spread on the pavement and flow depth at the curb are often used as criteria for spacing pavement drainage inlets. Chart 13.A1 in Appendix 13.A is a nomograph for solving Equation 13.1. The chart can be used for either criterion with the relationship:

$$d = T S_X \tag{13.2}$$

where,

d = Depth of flow (m).

Chart 13.A1 can be used for direct solution of gutter flow where the Manning's n value is 0.016. For other values of n (Table 13.3), designers should divide the value of Q_n by n.

Table 13.3: Pavement Roughness Coefficients (FHWA, 2009)

Gutter / Pavement Materials	Manning's Roughness, *n*
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016

Note:For gutters with small slope, where sediment may accumulate, increase above values of "*n*" by 0.002

13.4.2 Uniform Cross Slope Gutter Section

The nomograph in Chart 13.A1 solves Equation 13.1 for gutters having triangular cross sections. Example in Appendix 13.B illustrates its use for the analysis of conventional gutters with uniform cross slope.

13.4.3 Composite Gutter Sections

The design of composite gutter sections requires consideration of flow in the depressed segment of the gutter, Q_w. Equation 13.3a, displayed graphically as Chart 13.A2, is provided for use with Equations 13.3b and 13.3c below and Chart 13.A1 to determine the flow in a width of gutter in a composite cross section, *W*, less than the total spread, *T*. The procedure for analysing composite gutter sections is demonstrated in Example in Appendix 13.C.

$$E_o = 1/\left\{1 + \frac{S_w/S_x}{\left[1 + \frac{S_w/S_x}{\frac{T}{W} - 1}\right]^{8/3} - 1}\right\} \tag{13.3a}$$

$$Q_w = Q - Q_S \tag{13.3b}$$

$$Q = \frac{Q_S}{1 - E_0} \tag{13.3c}$$

where,

Q_W = Flow rate in the depressed section of the gutter (m^3/s);
Q = Gutter flow rate (m^3/s);
Q_S = Flow capacity of the gutter section above the depressed section (m^3/s);
E_o = Ratio of flow in a chosen width (usually the width of a grate) to total gutter flow *(Q_w/Q);*

S_W = $S_X + a/W$ (Figure 13.2aii);
S_X = Cross slope (m/m) ; and
a = Gutter depression, (m).

Chart 13.A3 illustrates a design chart for a composite gutter with a 0.60m width section and a 50mm depression at the curb that begins at the projection of the uniform cross slope at the curb face. A series of charts similar to Chart 13.A3 for "typical" gutter configurations can be developed.

13.4.4 Shallow Swale Gutter Sections

Where curbs are not needed for traffic control, a small swale section of circular or V-shape may be used to convey runoff from the pavement. As an example, the control of pavement runoff on fills may be needed to protect the embankment from erosion. Small swale sections may have sufficient capacity to convey the flow to a location suitable for interception.

Chart 13.A1 can be used to compute the flow in a shallow V-shaped section. When using Chart 13.A1 for V-shaped channels, the cross slope, S_X is determined by the following equation:

$$S_X = \frac{S_{X_1} S_{X_2}}{S_{X_1} + S_{X_2}} \tag{13.4}$$

Example in Appendix 13.D demonstrates the use of Chart 13.A1 to analyze a V-shaped shoulder gutter.

13.4.5 Gutter Flow Time

The flow time in gutters is an important component of the time of concentration for the contributing drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a reach of gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter, i.e., both the velocity and flow rate in a gutter are spatially varied. The time of flow can be estimated by use of an average velocity obtained by integration of the Manning's equation for the gutter section with respect to time.

Table 13.4 and Chart 13.A4 can be used to determine the average velocity in triangular gutter sections. In Table 13.4, T_1 and T_2 are the spread at the upstream and downstream ends of the gutter section respectively. T_a is the spread at the average velocity. Chart 13.A4 is a nomograph to solve Equation 13.5 for the velocity in a triangular channel with known cross slope, gutter slope, and spread.

$$V = \frac{K_u}{n} S_L^{\ 0.5} S_X^{\ 0.67} T^{\ 0.67} \tag{13.5}$$

where,

K_u = 0.752; and
V = Velocity in the triangular channel (m/s).

Table 13.4: Spread at Average Velocity in a Reach of Triangle Gutter

Spread Ratio	Value								
T_1/T_2	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80
T_a/T_2	0.65	0.66	0.68	0.70	0.74	0.77	0.82	0.86	0.90

13.5 INLET DESIGN

Storm drain inlets are used to collect runoff and discharge it to downstream storm drainage system. Inlets are typically located in gutter sections, paved medians, and roadside and median ditches.

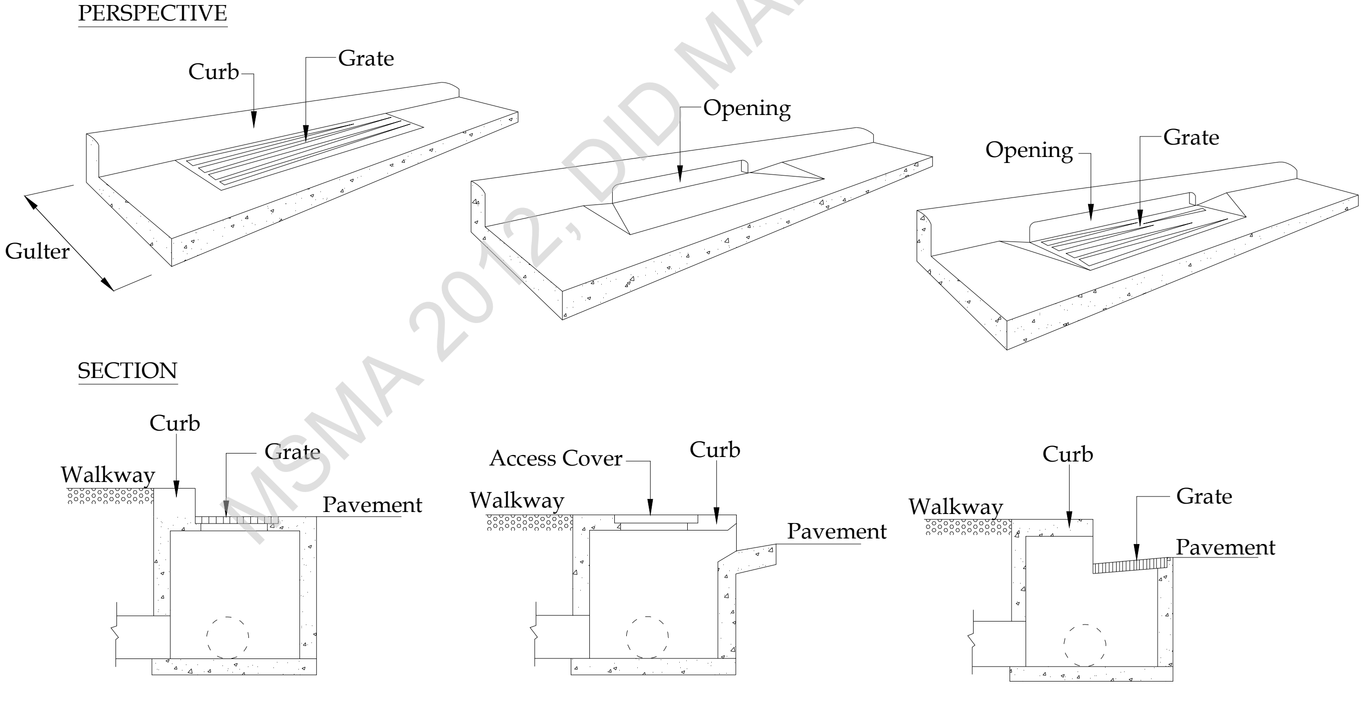
The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

13.5.1 Inlet Types

Inlets used for the pavement drainage of surfaces can be divided into the following classes:

- Grate inlets;
- Curb-opening inlets;
- Slotted inlets; and
- Combination inlets.

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Curb-opening inlets are vertical openings in the curb covered by a top slab. Slotted inlets consist of a pipe cut along the longitudinal axis with bars perpendicular to the opening to maintain the slotted opening. Combination inlets consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb-opening may be located in part upstream of the grate. Figure 13.3 illustrates the three (3) major inlets, grate, curb-opening and combination. Slotted drains may also be used with grates and each type of inlet may be installed with or without a depression of the gutter.



a) Grate b) Curb Opening c) Combination

Figure 13.3: Major Inlet Types

13.5.2 Interception Capacity of Inlets on Grade

In this section, design charts for inlets on grade and procedures for using the charts are presented for the various inlet configurations. On-grade inlets that are located in a sloping gutter, so that any flows bypassing the inlet will continue along the gutter flowpath to another inlet downstream or an outfall. For locally depressed inlets, the quantity of flow reaching the inlet would be dependent on the upstream gutter section geometry and not the depressed section geometry.

The chart for frontal flow interception is based on test results which show that grates intercept all of the frontal flow until a velocity is reached at which water begins to splash over the grate. At velocities greater than "splash-over" velocity, grate efficiency in intercepting frontal flow is diminished. Grates also intercept a portion of the flow along the length of the grate, or the side flow. A chart is provided to determine side-flow interception.

One set of charts is provided for slotted inlets and curb-opening inlets, because these inlets are both side-flow weirs. The equation developed for determining the length of inlet required for total interception fits the test data for both types of inlets. A procedure for determining the interception capacity of combination inlets is also presented.

13.5.2.1 Grate Inlets

Grates are effective road pavement drainage inlets where clogging with debris is not a problem. Where clogging may be a problem, see Table 13.5 where grates are ranked for susceptibility to clogging based on laboratory tests using simulated "leaves" (FHWA, 2009). This table should be used for relative comparisons only. When the velocity approaching the grate is less than the "splash-over" velocity, the grate will intercept essentially all of the frontal flow. Conversely, when the gutter flow velocity exceeds the "splash-over" velocity for the grate, only part of the flow will be intercepted. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

Table 13.5: Average Debris Handling Efficiencies of Grates (FHWA, 2009)

Rank	Grate	Longitudinal Slope	
		0.005	0.040
1	Curved Vane	46	61
2	30° 85 Tilt Bar	44	55
3	45° 85 Tilt Bar	43	48
4	P - 50	32	32
5	P – 50 x 100	18	28
6	45° 60 Tilt Bar	16	23
7	Reticuline	12	16
8	P - 30	9	20

The ratio of frontal flow to total gutter flow, E_o, for a uniform cross slope is expressed by Equation 13.6.

$$E_0 = \frac{Q_w}{Q} = 1 - \left[1 - \frac{W}{T}\right]^{2.67} \qquad (13.6)$$

where,

Q = Total gutter flow (m^3/s);
Q_w = Flow in width W (m^3/s);
W = Width of depressed gutter or grate (m); and
T = Total spread of water (m).

The ratio of side flow, Q_s, to total gutter flow, Q is:

$$\frac{Q_S}{Q} = 1 - \frac{Q_{Vw}}{Q} = 1 - E_o \qquad (13.7)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by Equation 13.8. The value of R_f cannot be more than 1.0

$$R_f = 1 - K_u\,(V - V_o) \qquad (13.8)$$

where,

K_u = 0.295;
V = Velocity of flow in the gutter (m/s); and
V_o = Gutter velocity where splash-over first occurs (m/s).

This ratio is equivalent to frontal flow interception efficiency. Chart 13.A5 provides a solution for Equation 13.8 which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs. The average gutter velocity (total gutter flow divided by the area of flow) is needed to use Chart 13.A5. This velocity can also be obtained from Chart 13.A4. The ratio of side flow intercepted to total side flow, R_S, or side flow interception efficiency, is expressed by Equation 13.9. Chart 13.A6 provides a solution to Equation 13.9.

$$R_S = 1/(1 + \frac{K_u V^{1.8}}{S_x L^{2.3}}) \qquad (13.9)$$

where,

K_u = 0.0828

A deficiency in developing empirical equations and charts from experimental data is evident in Chart 13.A6. The fact that a grate will intercept all or almost all of the side flow where the velocity is low and the spread only slightly exceeds the grate width is not reflected in the chart. However, the error due to this deficiency is very small. In fact, where velocities are high, side flow interception may be neglected without significant error.

The efficiency, E, of a grate is expressed in Equation 13.10.

$$E = R_f E_o + R_s (1 - E_o) \qquad (13.10)$$

The first term on the right side of Equation 13.10 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates. The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow.

$$Q_i = E\,Q = Q\,[R_f E_o + R_s (1 - E_o)] \qquad (13.11)$$

13.5.2.2 Curb-Opening Inlets

Curb-opening inlets are effective in the drainage of road pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or cyclists.

Curb opening heights vary in dimension, however, a typical maximum height is approximately 100 to 150mm. The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by Equation 13.12;

$$L_T = K_u Q^{0.42} S_L{}^{0.3} (\frac{1}{nS_X})^{0.6} \qquad (13.12)$$

where,

K_u = 0.817
L_T = Curb opening length required to intercept 100 percent of the gutter flow (m);
S_L = Longitudinal slope; and
Q = Gutter flow (m^3/s).

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by Equation 13.13.

$$E = 1 - (1 - \frac{L}{L_T})^{1.8} \qquad (13.13)$$

where,

L = Curb-opening length (m).

Chart 13.A7 is a nomograph for the solution of Equation 13.12, and Chart 13.A8 provides a solution of Equation 13.13.

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in Equation 13.12 in place of S_x and S_e can be computed using Equation 13.14.

$$S_e = S_X + S'_W E_o \qquad (13.14a)$$

$$S'_W = a \,/\, [1000\ W], \text{ for W in m; or } = S_w - S_x; \qquad (13.14b)$$

where,

S'_W = Cross slope of the gutter measured from the cross slope of the pavement S_x (m/m);
a = Gutter depression mm; and
E_o = Ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet.

Figure 13.4 shows the depressed curb inlet for Equation 13.14. E_o is the same ratio as that used to compute the frontal flow interception of a grate inlet.

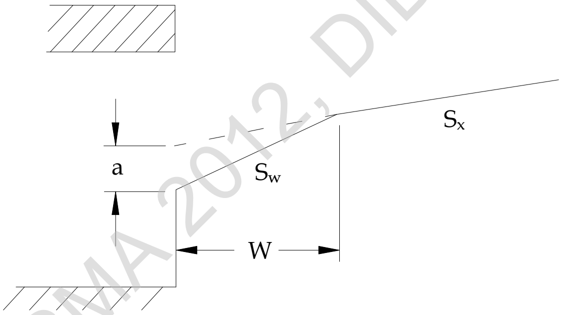


Figure 13.4: Depressed Curb Opening Inlet

As seen from Chart 13.A7, the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e, Equation 13.12 becomes,

$$L_T = K_T Q^{0.42} S_L^{\ 0.3} (\frac{1}{nS_e})^{0.6} \qquad (13.15)$$

where,

K_T = 0.817.

Equation 13.15 is applicable with either straight cross slopes or composite cross slopes. Charts 13.A7 and 13.A8 are applicable to depressed curb-opening inlets using S_e rather than S_x. Equation 13.14 uses the ratio, E_o, in the computation of the equivalent cross slope, S_e. Example in Appendix 13.E demonstrates the procedure to determine spread and then the example uses Chart 13.A2 to determine E_o. Example in Appendix 13.E demonstrates the use of these relationships to design length of a curb opening inlet.

13.5.2.3 Combination Inlets

The interception capacity of a combination inlet consisting of a curb opening and grate placed side-by-side, as shown in Figure 13.5, is no greater than that of the grate alone. Capacity is computed by neglecting the curb opening.

A combination inlet is sometimes used with a part of the curb opening placed upstream of the grate as illustrated in Figure 13.6. The curb opening in such an installation intercepts debris which might otherwise clog the grate and is called a "sweeper" inlet. A sweeper combination inlet has an interception capacity equal to the sum of the curb opening upstream of the grate plus the grate capacity, except that the frontal flow and thus the interception capacity of the grate is reduced by interception by the curb opening.

Figure 13.5: Combination Curb-opening, 45 Degree Tilt-bar Grate Inlet

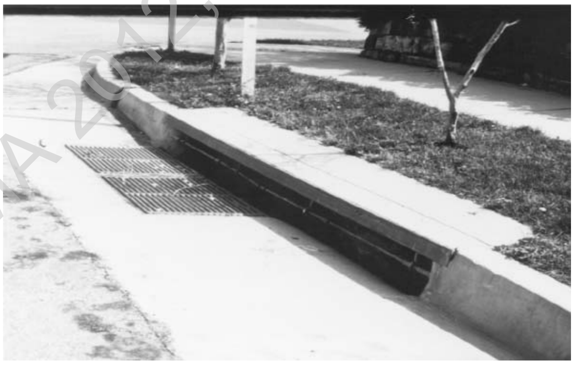


Figure 13.6: Sweeper Combination Inlet

The use of depressed inlets and combination inlets enhances the interception capacity of the inlet. The geometries of the inlets and the gutter slopes were consistent in the examples and Table 13.6 summarizes a comparison of the intercepted flow of the various configurations.

13.5.3 Interception Capacity of Inlets in Sag Location

Inlets in sag locations operate as weirs under low head conditions and as orifices at greater depths. Orifice flow begins at depths dependent on the grate size, the curb opening height, or the slot width of the inlet. At depths between those at which weir flow definitely prevails and those at which orifice prevails, flow is in transition stage. At these depths, control is ill-defined and flow may fluctuate between weir and orifice control. Design procedures presented here are based on a conservative approach to estimate the capacity of inlets in sump locations.

The efficiency of inlets in passing debris is critical in sag locations because all runoff which enter the sag must passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponded conditions. Grate inlets alone are not recommended for use in sag locations because of the tendencies of grates to become clogged. Combination inlets or curb-opening inlets are recommended for use in these locations.

13.5.3.1 Curb-Opening Inlets

The capacity of a curb-opening inlet in sag depends on water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in transition stage.

Spread on the pavement is the usual criterion for judging the adequacy of a pavement drainage inlet design. It is also convenient and practical in the laboratory to measure depth at the curb upstream of the inlet at the point of maximum spread on the pavement. Therefore, depth at the curb measurements from experiments coincide with the depth at curb of interest to designers. The weir coefficient for a curb-opening inlet is less than the usual weir coefficient for several reasons, the most obvious of which is that depth measurements from experimental tests were not taken at the weir, and drawdown occurs between the point where measurement were made and the weir.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb opening. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb opening, and its length is equal to that of the inlet, as shown in Chart 13.A9.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is;

$$Q_i = C_w(L+1.8\,W)\,d^{1.5} \qquad (13.16)$$

where,

Q_i = Interception capacity of a grate inlet on grade (m³/s);
C_W = 1.25;
L = Length of the curb opening (m);
W = Lateral width of depression (m);
d = Depth at curb measured from the normal cross slope (m) i.e., $d = TS_x$.

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation 13.16 for a depressed curb opening inlet is;

$$d \leq h + a/(1000) \qquad (13.17)$$

where,

h = Height of curb opening inlet (m); and
a = Depth of depression (mm).

Experiment have not been conducted for curb-opening inlets with a continuously depressed gutter, but it is reasonable to expect that the effective weir length would be as great as that for an inlet in a local depression. Use of Equation 13.16 will yield conservative estimates of the interception capacity.

The weir equation for curb–opening inlets without depression becomes;

$$Q_i = C_w\,L\,d^{1.5} \qquad (13.18)$$

Without depression of the gutter section, the weir coefficient, C_w, becomes 1.6. The depth limitation for operation as weir becomes $d \leq h$.

At curb-opening lengths greater than 3.6m, Equation 13.18 for non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using Equation 13.16. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, Equation 13.18 should be used for all curb opening inlets having lengths greater than 3.6m.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by Equation 13.19a and Equation 13.19b. These equations are applicable to depressed and undepressed curb-opening inlets. The depth at the inlet includes any gutter depression.

$$Q_i = C_o\, h\, L(2\, g\, d_o)^{0.5} \qquad (13.19a)$$

or

$$Q_i = C_o\, A_g\, \{2g[d_i - (h/2)]\}^{0.5} \qquad (13.19b)$$

where,

Q_i = Interception capacity of a grate inlet on grade (m³/s);
C_o = Orifice coefficient (0.67);
d_o = Effective head on the centre of the orifice throat (m);
L = Length of orifice opening (m);
A_g = Clear area of opening (m²);
d_i = Depth at lip of curb opening (m); and
h = Height of curb opening orifice (m).

The height of the orifice in Equations13.19a and 13.19b assumes as vertical orifice opening. As illustrated in Figure 13.7, other orifice throat locations can change the effective depth on the orifice and the dimension (d_i–$h/2$). A limited throat width could reduce the capacity of the curb-opening inlet by causing the inlet to go into orifice flow at depths less than the heights of the opening.

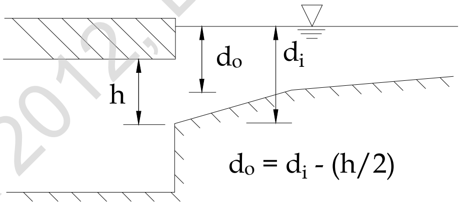


Figure 13.7: Curb-Opening Inlets

Chart 13.A9 provides solutions for Equations 13.16 and 13.19 for depressed curb-opening inlets, and Chart 13.A10 provides solutions for Equations 13.18 and 13.19 for curb-opening inlets without depression.

13.5.3.2 Combination Inlets

Combination inlets consisting of a grate and a curb opening are considered advisable for use in sag where hazardous ponding can occur. Equal length inlets refer to a grate inlet placed along side a curb opening inlet, both of which have the same length. A sweeper inlet refers to a grate inlet placed at the downstream end of a curb opening inlet. The curb opening inlet is longer than the grate inlet and intercepts the flow before the flow reaches the grate. The sweeper inlet is more efficient than the equal length combination inlet and the curb opening has the ability to intercept any debris which may clog the grate inlet. The interception capacity of the equal length combination inlet is essentially equal to that of a grate alone in weir flow. In orifice flow, the capacity of the equal length combination inlet is equal to the capacity of the grate plus the capacity of the curb opening. Equation 13.20 and Chart 13.A11 can be used for grates in weir flow of combination inlets in sag locations.

$$Q_i = C_W\, P\, d^{1.5} \qquad (13.20)$$

where,

Q_i = Interception capacity of a grate inlet on grade (m^3/s);
C_W = 1.66;
P = Perimeter of the grate (m); and
d = Average depth across the grate, $0.5(d_1 + d_2)$ (m).

Assuming complete clogging of the grate, Equation 13.16, 13.18, and 13.19 and Charts 13.A9, and 13.A10 for curb opening inlets are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the inlet is computed by using Equation 13.21:

$$Q_i = 0.67\, A_g\, (2\, g\, d)^{0.5} + 0.67\, h\, L\, (2\, g\, d_o)^{0.5} \tag{13.21}$$

where,

Q_i = Interception capacity of a grate inlet on grade (m^3/s);
A_g = Clear area of the grate (m^2);
g = 9.81 (m/s^2);
d = Average depth over the grate (m);
h = Height of curb opening orifice (m);
L = Length of curb opening (m); and
d_o = Effective depth at the center of the curb opening orifice (m).

13.5.4 Locating Inlets

The location of inlets is determined by geometric controls which require inlets at specific locations, the use and location of flanking inlets in sag vertical curves, and the criterion of spread on the pavement (Figure 13.8). In order to adequately design the location of the inlets for a given project, the following information is needed:

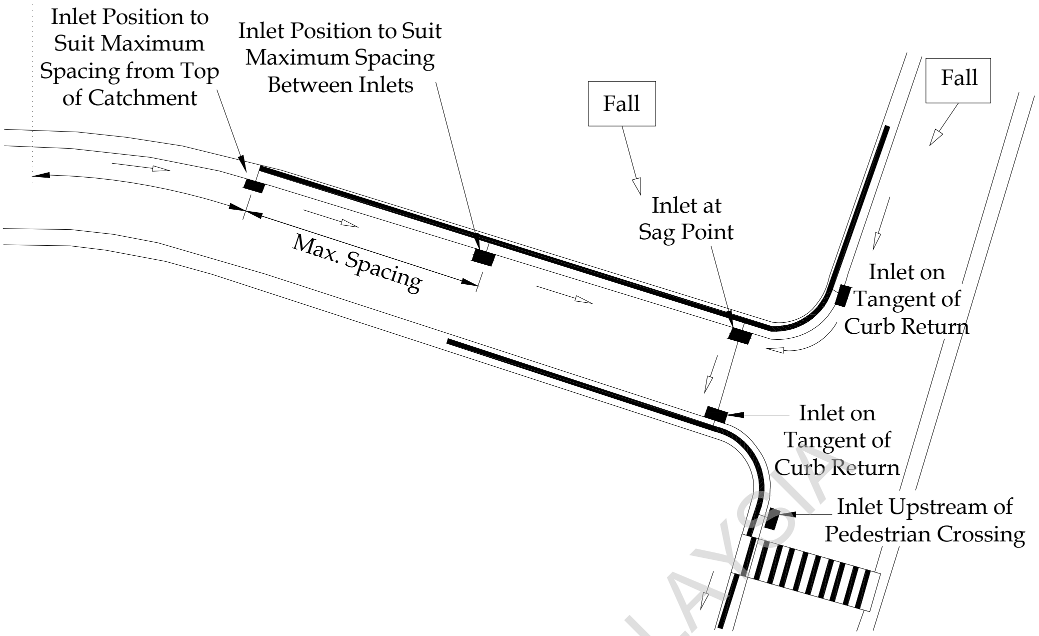
- A layout or plan sheet suitable for outlining drainage areas;
- Road profiles;
- Typical cross sections;
- Grading cross sections;
- Superelevation diagrams; and
- Contour maps.

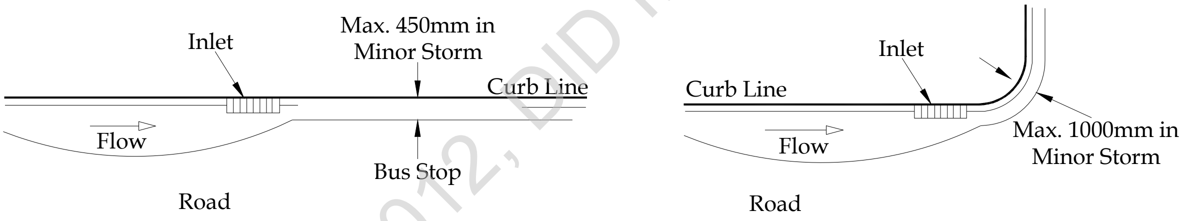
13.5.4.1 Geometric Controls

There are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations follow:

- At all low points in the gutter grade;
- Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street intersections., i.e., at any location where water could flow onto the travelway;
- Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks);
- Immediately downstream of bridges (to intercept bridge deck drainage);
- Immediately up grade of cross slope reversals;
- Immediately up grade from pedestrian cross walks;
- At the end of channels in cut sections;
- On side streets immediately up grade from intersections; and
- Behind curbs, shoulders or sidewalks to drain low area.

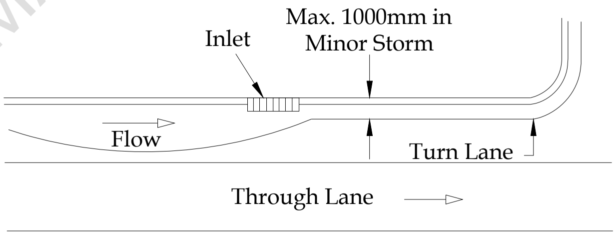
In addition to the areas identified above, runoff from areas draining towards the highway pavement should be intercepted by roadside channels or inlets before it reaches the roadway. This applies to drainage from cut





a) at Bus Stop

b) at Curb Return



c) at Decelaration Lane

Figure 13.8: Typical Location of Inlet on Roads

slopes, side streets, and other areas alongside the pavement. Curbed pavement sections and pavement drainage inlets are inefficient means of handling extraneous drainage.

13.5.4.2 Inlet Spacing on Continuous Grades

Design spread is the criterion used for locating storm drain inlets between those required by geometric or other controls. The interception capacity of the upstream inlet will define the initial spread. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at the point where the spread in the gutter reaches the design spread. Therefore, the spacing of inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry.

For a continuous slope, the designer may establish the uniform design spacing between inlets of a given design if the drainage area consists of pavement only or has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the time of concentration is assumed to be the same for all inlets. The following procedure and example illustrate the effects of inlet efficiency on inlet spacing.

13.5.5 Inlet Spacing Design Procedure

In order to design the location of inlets on a continuous grade, the computation sheet shown in Table 13.6 may be used to document the analysis. A step by step procedure for the use of Table 13.6 is presented as follows:-

Step 1: Complete the blanks at the top of the sheet to identify the job by state project number, route, date, and your initials.

Step 2: Mark on a plan the location of inlets which are necessary even without considering any specific drainage area, such as the locations described in Section 13.5.4.1.

Step 3: Start at a high point, at one end of the job if possible, and work towards the low point. Then begin at the next high point and work backwards toward the same low point.

Step 4: To begin the process, select a drainage area below the highest point and outline the area on the plan. Include any area that drain over the curb, onto the roadway.

Step 5: Describe the location of the proposed inlet by number and station and record this information in Columns 1 and 2. Identify the curb and gutter type in Column 19 (Remarks). A sketch of the cross section should be prepared.

Step 6: Compute the drainage area (hectares) outlined in Step 4 and record this in Column 3.

Step 7: Determine the runoff coefficient, *C*, for the drainage area. Select a *C* value provided in Table 2.5 and record the value in Column 4.

Step 8. Compute the time of concentration, t_c, in minutes, for the first inlet and record in Column 5. The minimum time of concentration is 5 minutes.

Step 9: Using the time of concentration, determine the rainfall intensity from the Intensity-Duration-Frequency (IDF) curve for the design frequency. Enter the value in Column 6.

Step10: Calculate the flow in the gutter using Equation 2.3, $Q=CIA/K_c$. The flow is calculated by multiplying Column 3 times Column 4 times Column 6 divided by K_c. Using the SI system of units, $K_u = 360$. Enter the flow value in Column 7.

Step 11: From the roadway profile, enter in Column 8 the gutter longitudinal slope, S_L, at the inlet, taking into account any superelevation.

Step12: From the cross section, enter the cross slope, S_x, in Column 9 and the grate or gutter width, *W*, in Column 13.

Step13: For the first inlet in a series, enter the value from Column 7 into Column 11, since there was no previous bypass flow. Additionally, if the inlet is the first in a series, enter 0 into Column10.

Step14: Determine the spread, *T*, by using Equations 13.1 and 13.3a or Charts 13.A1 and enter, the value in Column 14. Also, determine the depth at the curb, d, by multiplying the spread by the appropriate cross slope, and enter the value in Column 12. Compare the calculated spread with the allowable spread as determined by the design criteria outlined in Table 13.1. Additionally, compare the depth at the curb with the actual curb height in Column 19. If the calculated spread, Column 14, is near the allowable spread and the depth at the curb is less than the actual curb height, continue on to Step 15. Else, expand or decrease the drainage area up to the first inlet to increase or decrease the spread, respectively. The drainage area can be expanded by increasing the length to the inlet and it can be decreased by decreasing the distance to the inlet. Then, repeat Steps 6 through 14 until appropriate values are obtained.

Step 15: Calculate W/T and enter the value in Column 15.

Step 16: Select the inlet type and dimensions and enter the values in Column 16.

Step 17: Calculate the flow intercepted by the grate, Q_i, and enter the value in Column 17. Use Equations 13.6 and 13.5 or Charts 13.A2 and 13.A4 to define the gutter flow. Use Chart 13.A5 and Equation 13.9 or Chart 13.A6 to define the flow intercepted by the grate. Use Equations 13.12 and 13.13 or Charts 13.A7 and 13.A8 for curb opening inlets. Finally, use Equation 13.11 to determine the intercepted flow.

Step 18: Determine the bypass flow, Q_b, and enter into Column 18. The bypass flow is the value in Column 11 minus that in Column 17.

Step 19: Proceed to the next inlet down the grade. To begin the procedure, select a drainage area appropriately. Repeat Steps 5 through 7 considering only the area between the inlets, entering values in Columns 1 to 4.

Step 20: Compute the time of concentration for the next inlet based upon the area between the consecutive inlets and record this value in Column 5.

Step 21: Determine the rainfall intensity from the IDF curve based upon the time of concentration determined in Step 19 and record the value in Column 6.

Step 22: Determine the flow in the gutter by using Equation 2.3 and record the value in Column 7.

Step 23: Record the value from Column 18 of the previous line into Column 10 of the current line. Determine the total gutter flow by adding Column 7 and Column 10 and record this in Column 11.

Step 24: Determine the spread and the depth at the curb as outlined in Step 14, entering the depth in Column 12 and the spread in Column 14. Repeat Steps 18 through 24 until the spread and the depth at the curb are within the design criteria.

Step 25: Select the inlet type and record this in Column 16.

Step 26: Determine the intercepted flow in accordance with Step 17, entering this in Column 17.

Step 27: Calculate the bypass flow by subtracting Column 17 from Column 11. Enter this in Column 18. This completes the spacing design for the inlet.

Step 28: Repeat Steps 19 through 27 for each subsequent inlet down to the low point.

Example in Appendix 13.F illustrates the use of this procedure, referring to Table 13.6.

Table 13.6: Inlet Spacing Computation Worksheet

INLET SPACING COMPUTATION SHEET

Date :
Computed By:

Route/Location :
Sheet 1 of 1

Inlet		Gutter Discharge Design Frequency				Gutter Discharge Allowable Spread									Inlet Discharge			
No. (1)	Station (2)	Drainage Area(ha)(3)	Runoff Coefficient, C (4)	Time of Concentration, t_c (min) (5)	Rainfall Intensity, *i* (mm/hr) (6)	$Q = CIA/K_c$ (m^3/s) (7)	Longitudinal Slope, S_L (m/m) (8)	Cross Slope, S_x or S_w (m/m) (9)	Previous By-pass Flow (m^3/s)(10)	Total Gutter Flow (m^3/s)(11)	Depth, d (m) (12)	Grate or Gutter Width, W (m) (13)	Spread, T (m)(14)	W/T (15)	Inlet Type (16)	Intercept Flow, Q_i (m^3/s)(17)	By-pass Flow, Q_b (m^3/s) (18)	Remarks (19)

REFERENCES

1. American Association of State Highway and Transportation Officials, AASHTO (1990). *Policy on Geometric Design* . Washington DC.
2. American Association of State Highway and Transportation Officials, AASHTO (1987). *Drainage Handbook.* Washington DC.
3. U.S. Federal Highway Administration (2009). *Urban Drainage Design Manual.* U.S. Department of Transportation, Washington DC.

APPENDIX 13.A DESIGN CHARTS

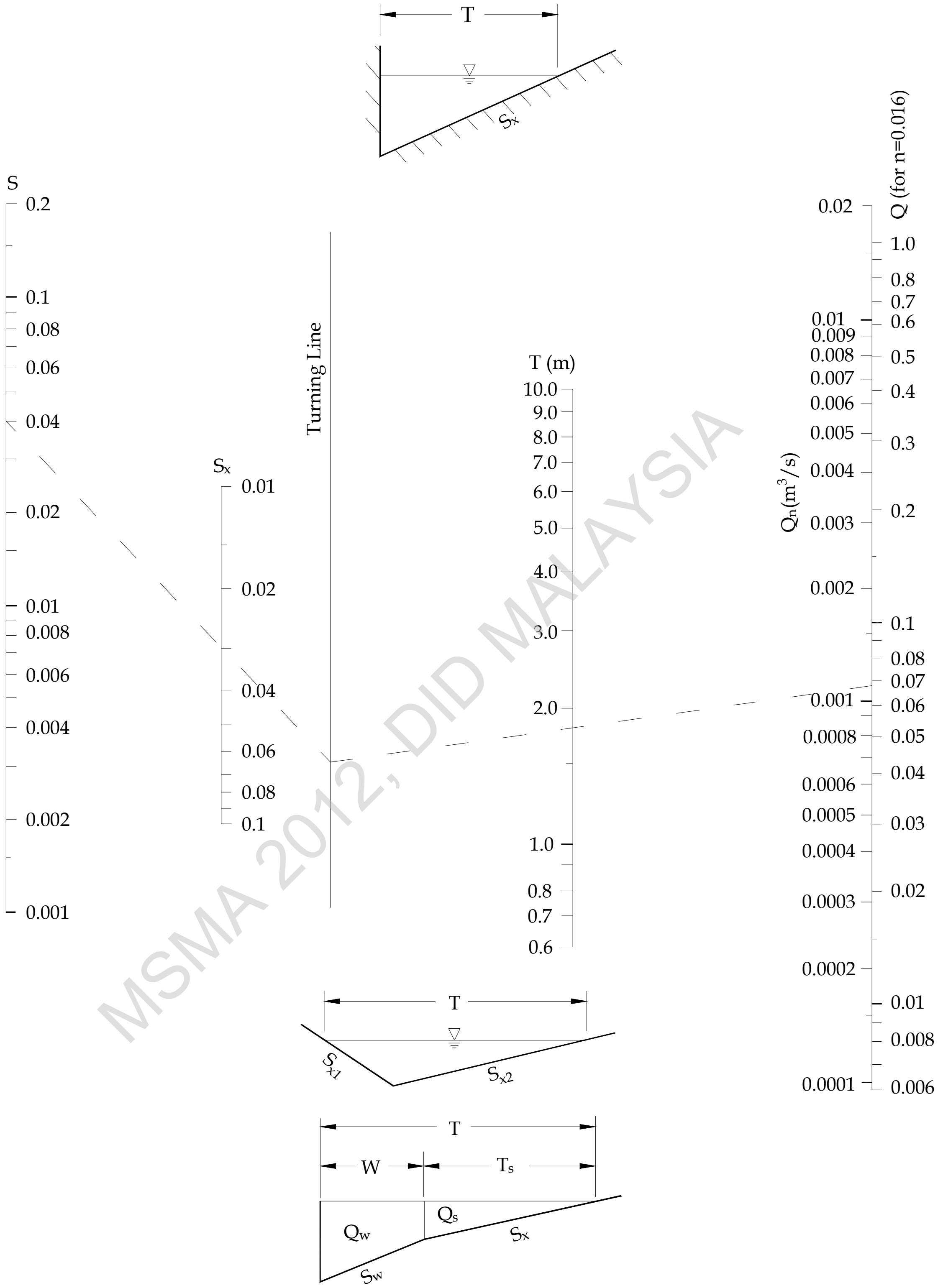


Chart 13.A1: Flow Estimation in Triangular Gutter Section (FHWA, 2009)

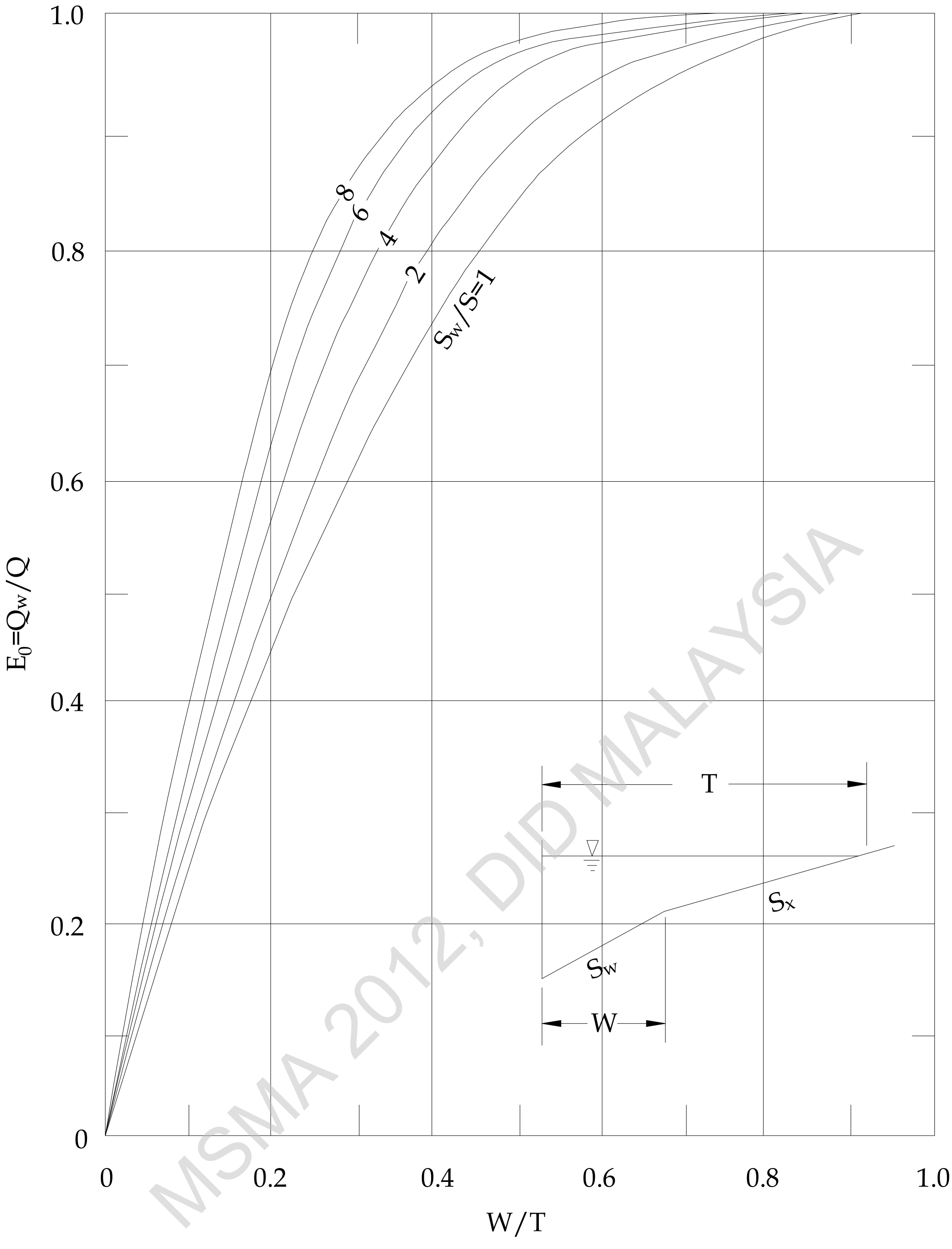


Chart 13.A2: Ratio of Frontal Flow to Total Gutter Flow (FHWA, 2009)

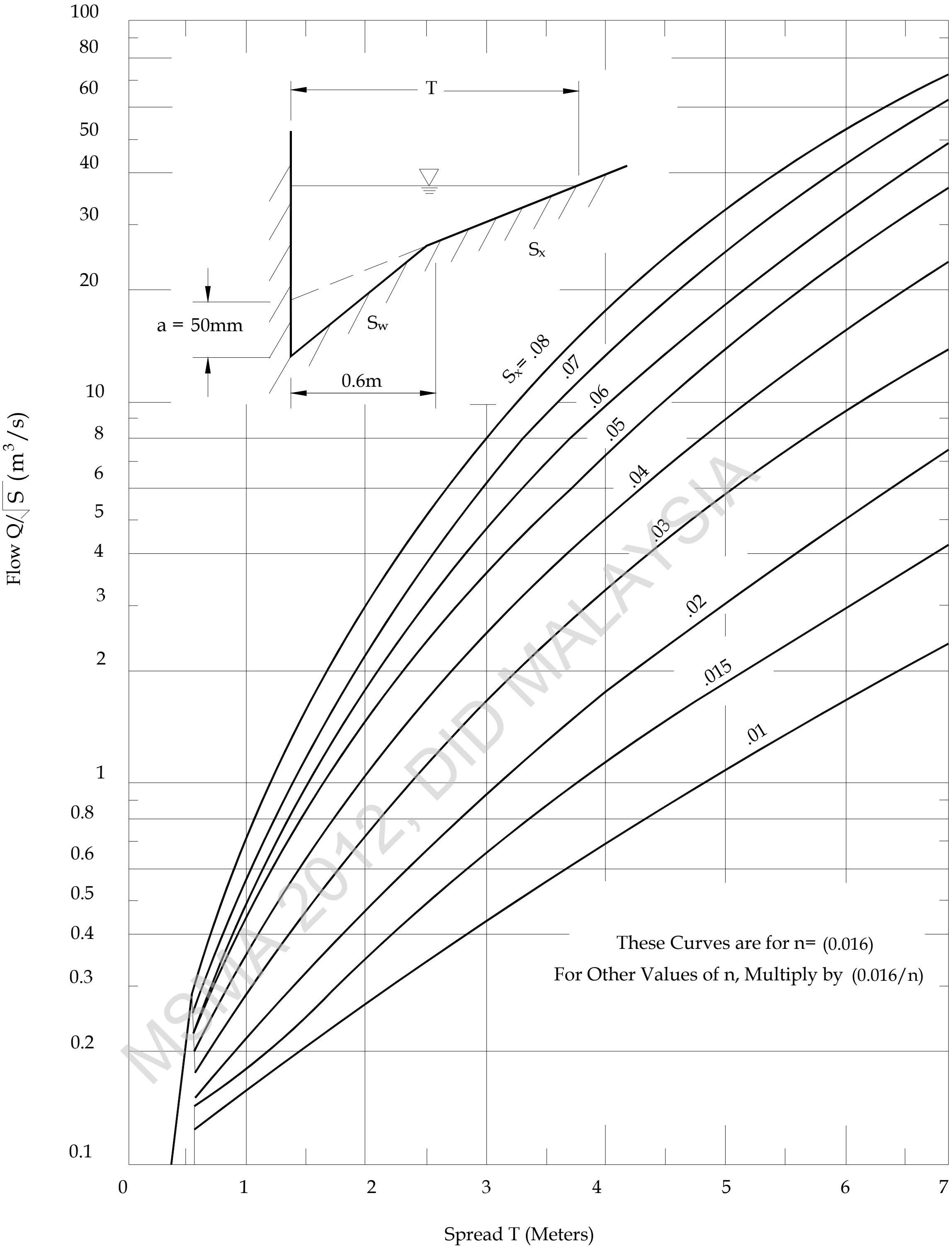


Chart 13.A3: Conveyance – Spread Curves for a Composite Gutter Section (FHWA, 2009)

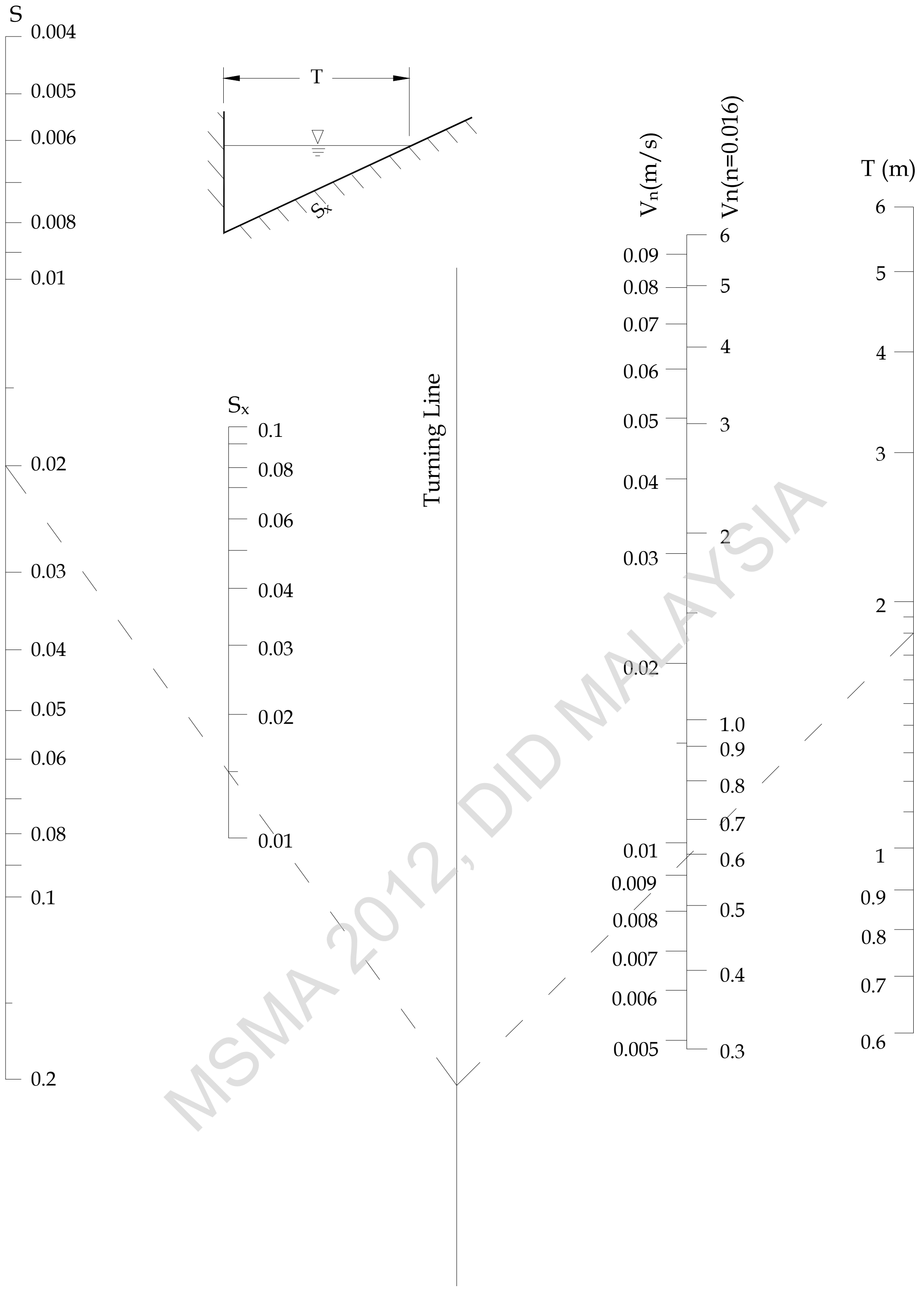


Chart 13.A4: Velocity in Triangular Gutter Sections (FHWA, 2009)

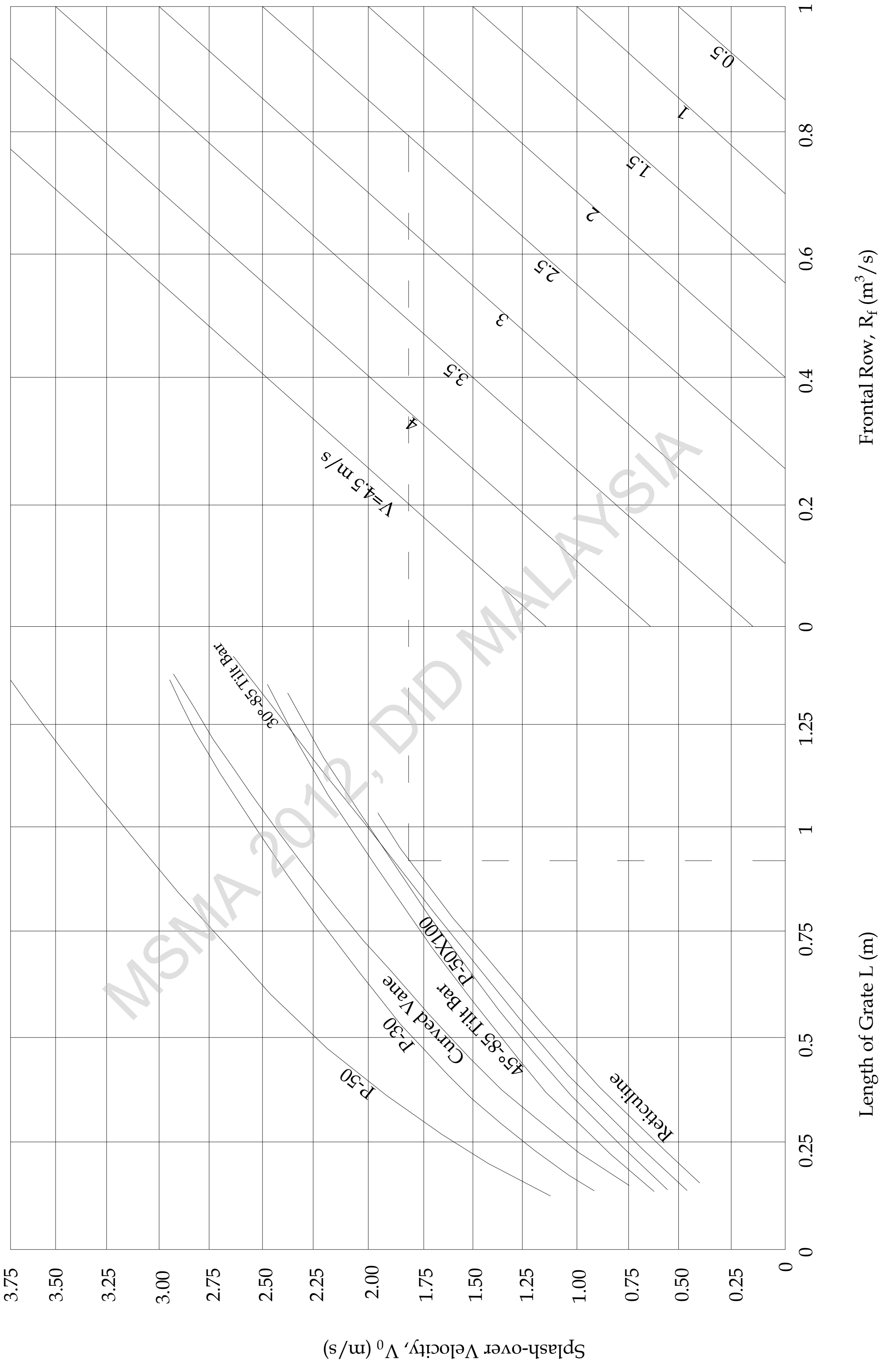


Chart 13.A5: Grate Inlet Frontal Flow Interception Efficiency (FHWA, 2009)

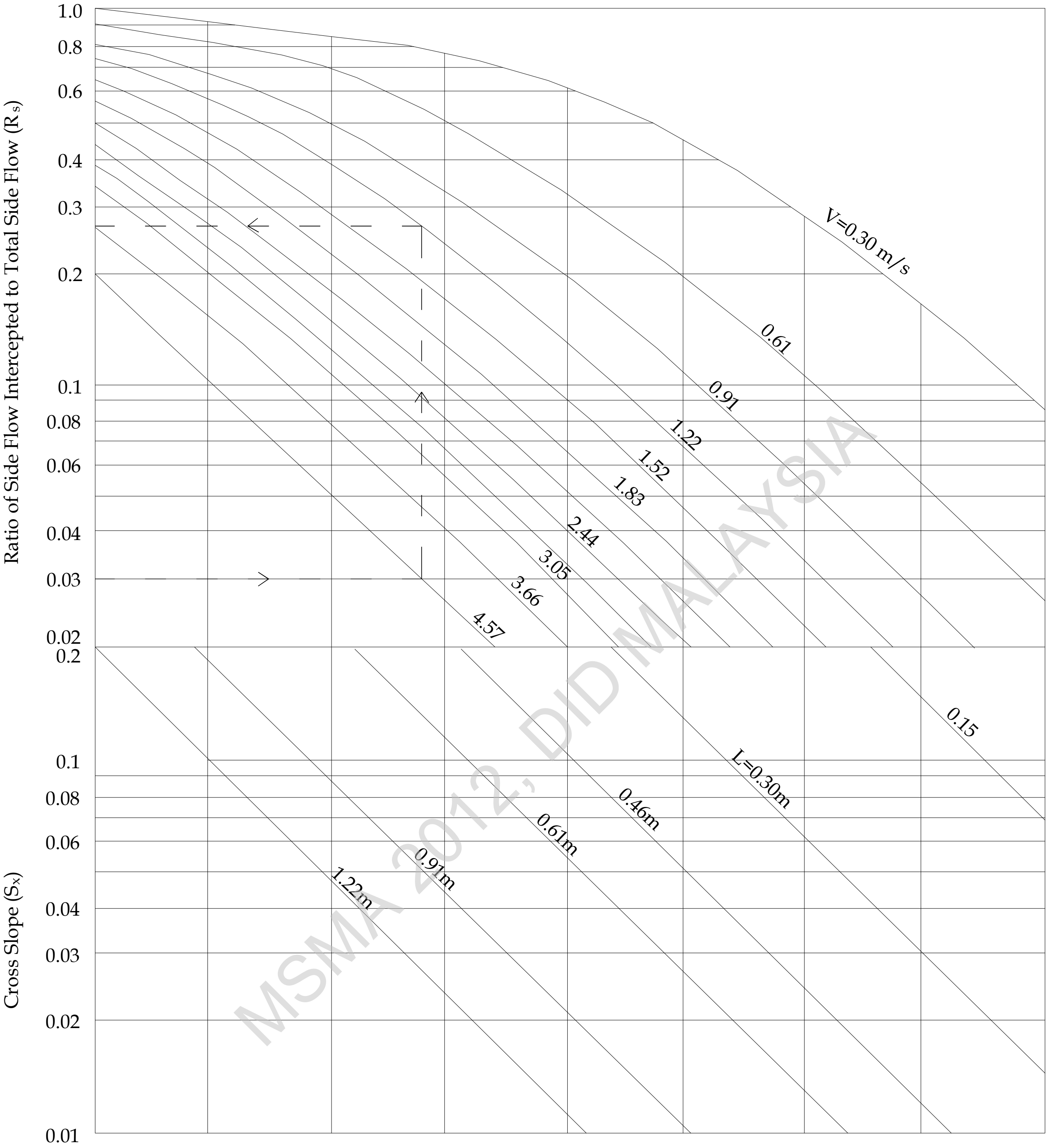


Chart 13.A6: Grate Inlet Side Flow Intercept Efficiency (FHWA, 2009)

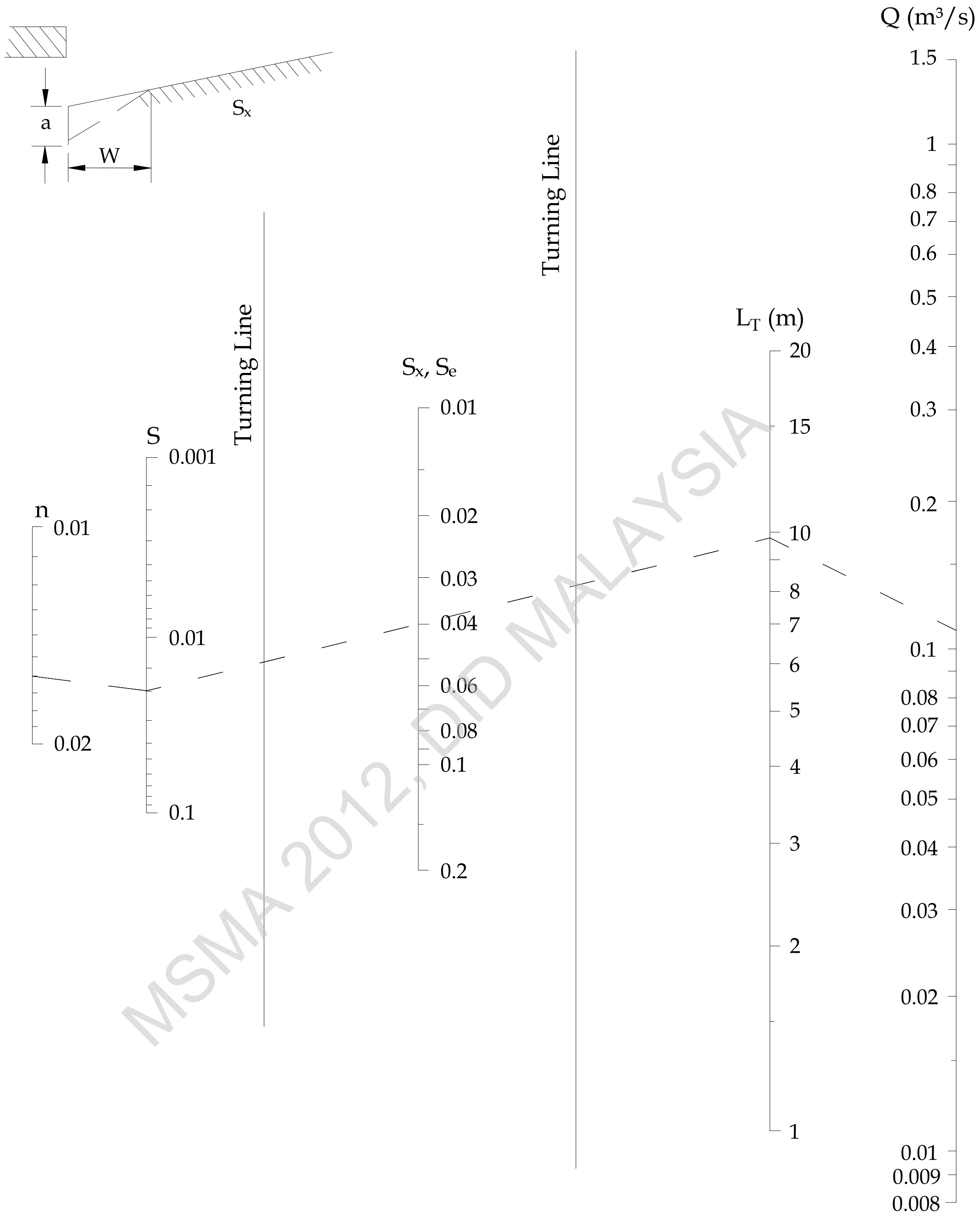


Chart 13.A7: Curb Opening and Slotted Drain Inlet Length for Total Interception (FHWA, 2009)

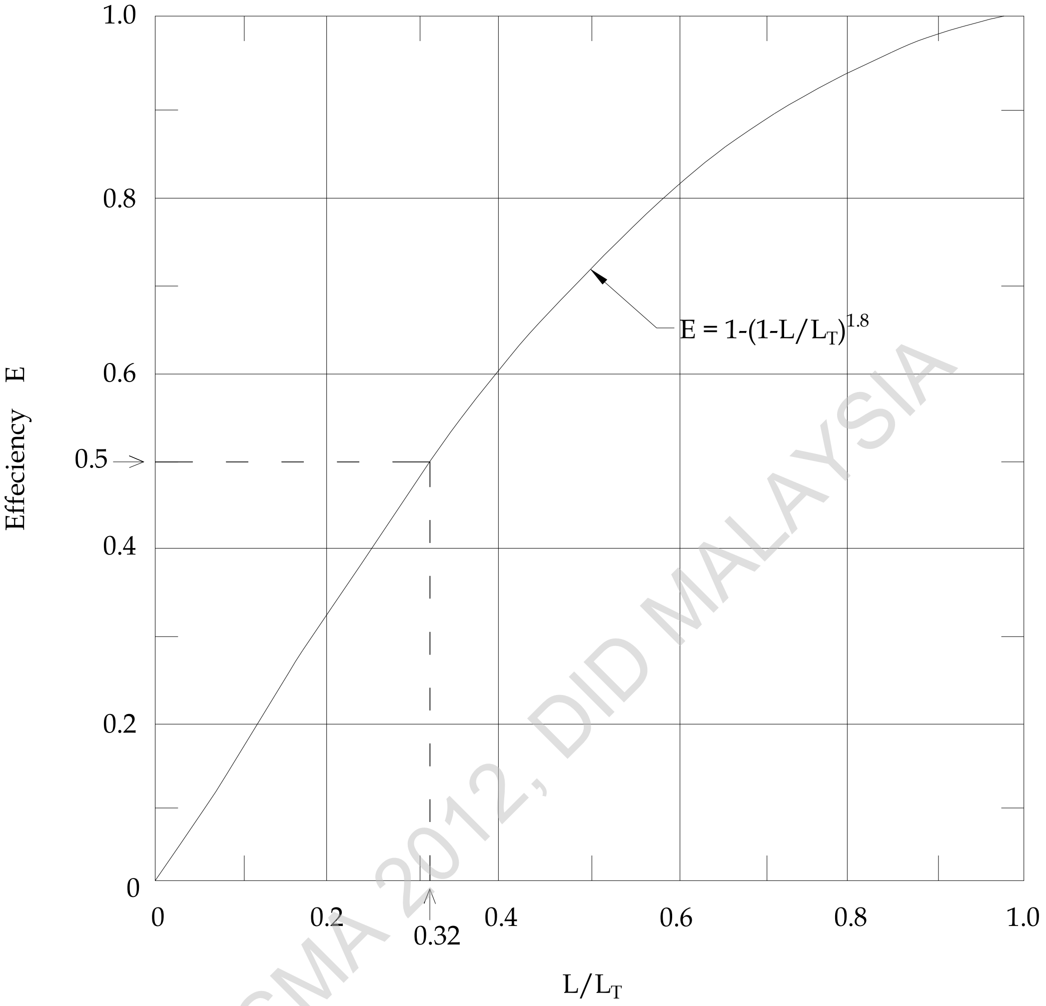


Chart 13.A8: Curb-Opening and Slotted Drain Inlet Interception Efficiency (FHWA, 2009)

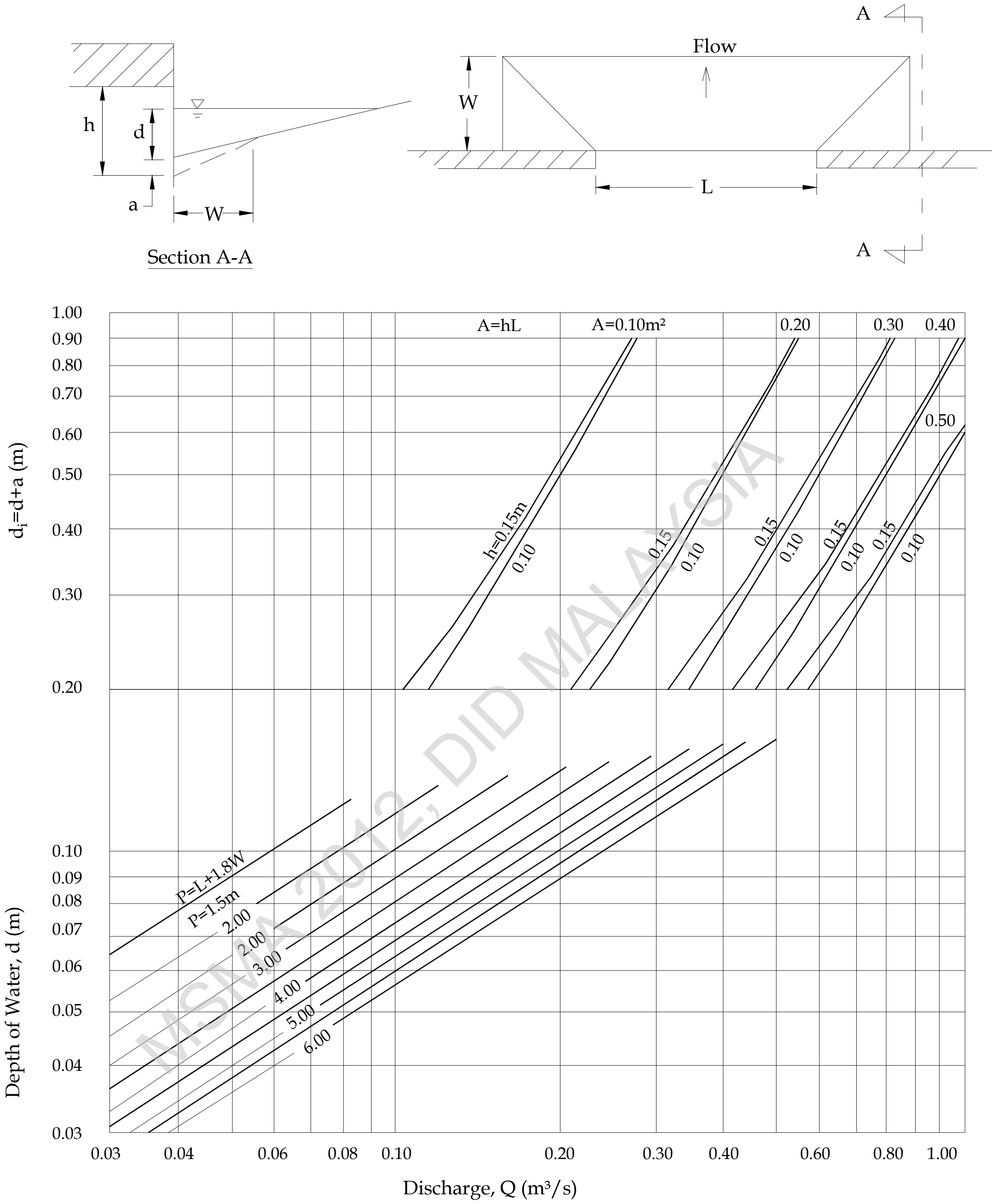


Chart 13.A9: Curb-Opening Inlet Capacity in Sump Location (FHWA, 2009)

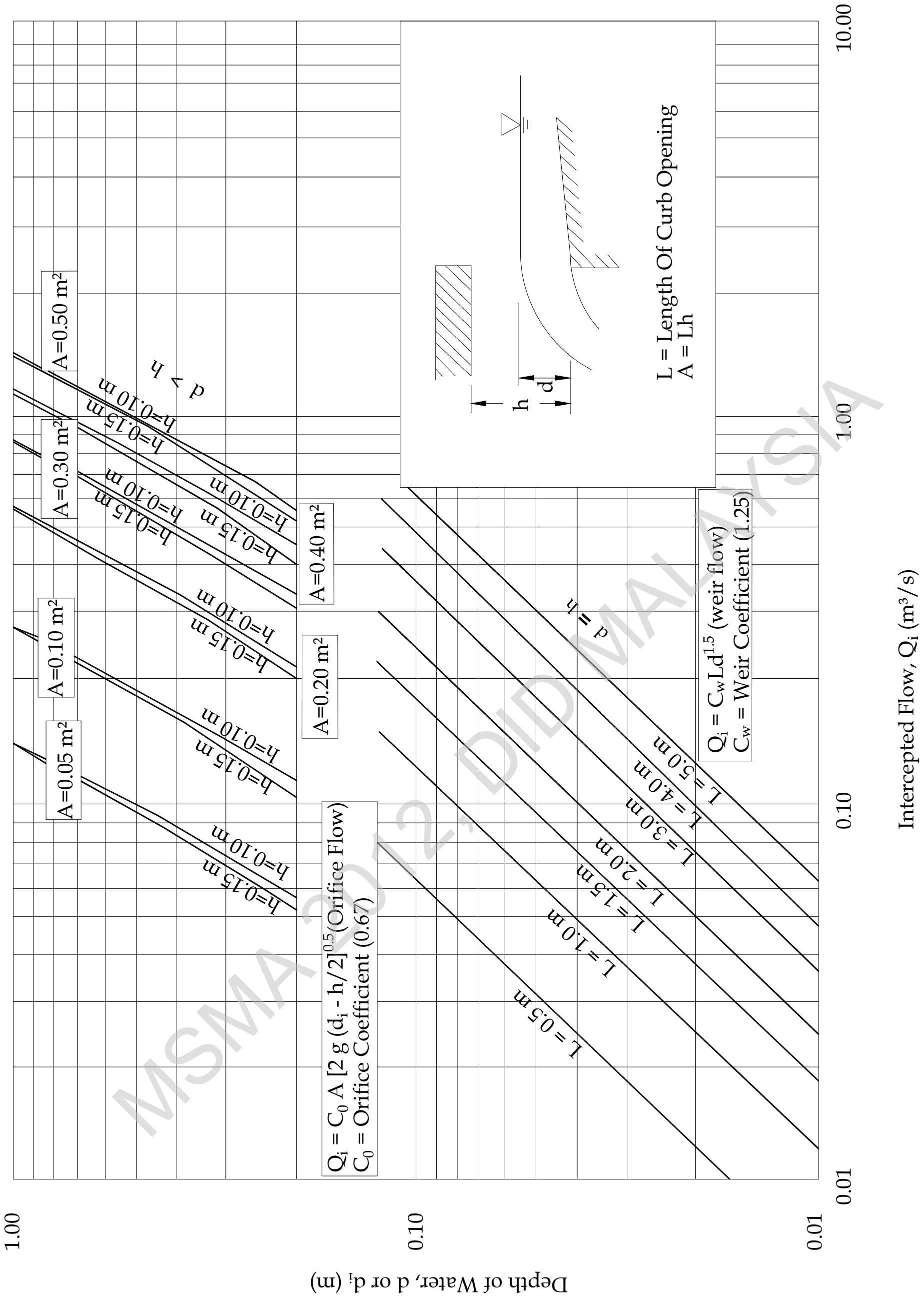


Chart 13.A10: Undepressed Curb-Opening Inlet Capacity in Sump Conditions (FHWA, 2009)

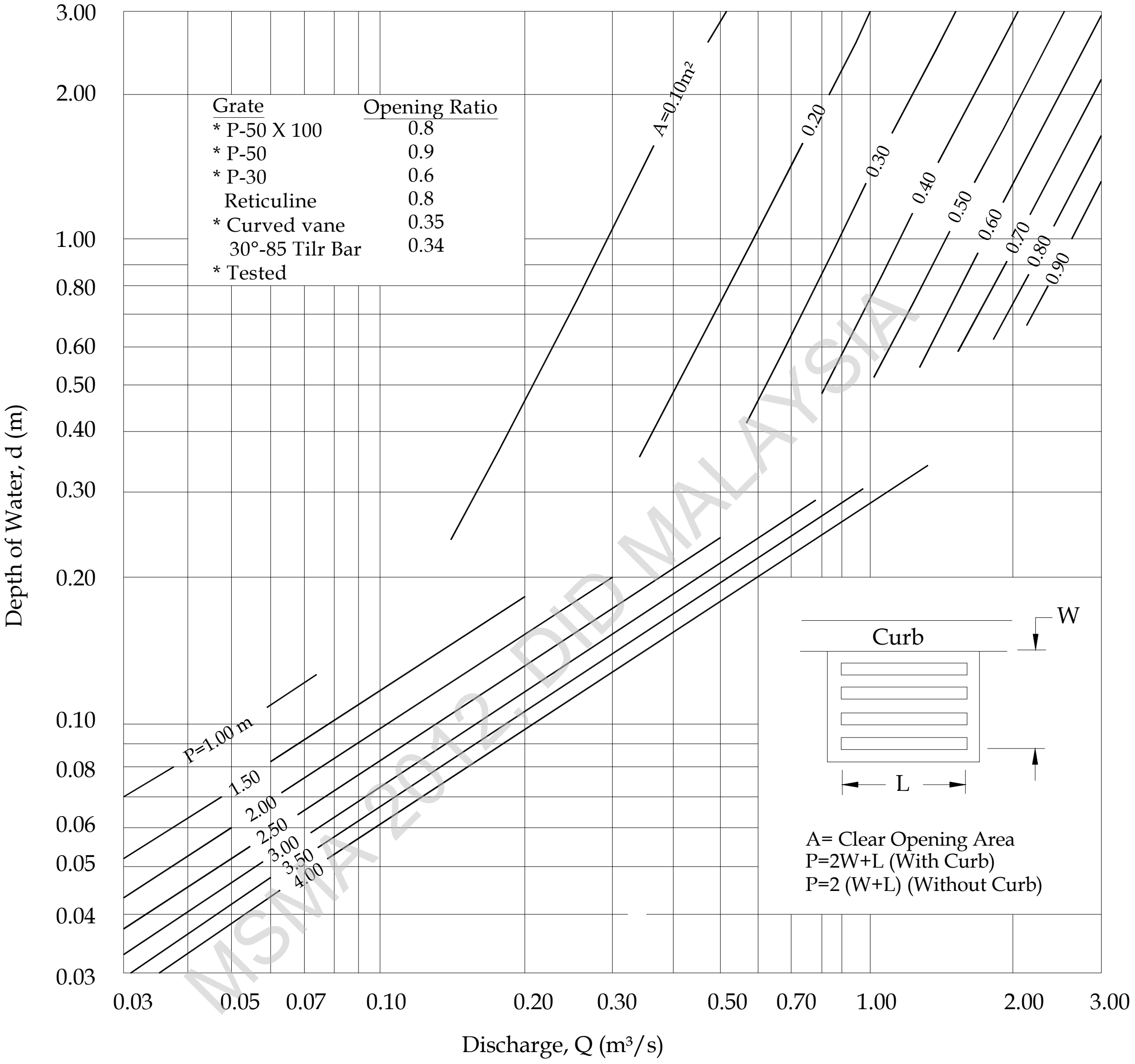
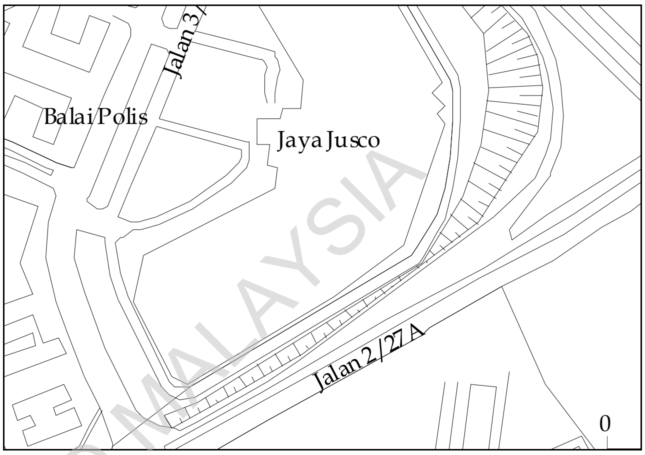


Chart 13.A11: Curb Opening Inlet Orifice Capacity (FHWA, 2009)

APPENDIX 13.B EXAMPLE – UNIFORM GUTTER

Problem:

A rough alphalt roadway (Manning n = 0.016) is to be designed with a cross slope of 0.02, a longitudinal slope of 0.03 and triangular gutter is to be used for pavement drainage. The proposed roadway site is located in Wangsa Maju, Kuala Lumpur. Determine the spread of water on the roadway if the roadway section considered consist of a 13m wide with a length of 100m.



Solution

Reference	Calculation		Output
	Step 1. Calculate velocity, V To determine shallow concentrated flow velocity, assume the spread of gutter, T is 1m $V = 1/n\, R^{2/3}S^{1/2}$ $R=A/P$		
	$R=[(0.5\text{x}1\text{x}0.02)/(0.02+(0.02^2+1^2)^{0.5})]$	=	0.0098m
	$V = (1/0.016)(0.0098)^{2/3}(0.03)^{1/2}$	=	0.496m/s
	Step 2. Calculate time of concentration, t_c. Time of concentration, t_c $= L / 60V$		
	$= 100/(60)(0.496)$	=	3.36 minutes (use 5 min. minimum)

Reference	Calculation	Output
	Step 3. Calculate rainfall intensity, I for 5 minutes duration and 10 ARI. Years	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$	
Table 2.B1	λ = 63.24 K = 0.162 θ = 0.137 η = 0.856 d = 0.0833	
	= $(63.24)(10)^{0.162}/(0.0833 + 0.137)^{0.856}$ =	335.25mm/hr
	Step 4. Calculate area of the location Area = (100)(13) =	1300m^2 0.13ha
	Step 5. Calculate flow During 10 year storm event.	
Table 2.5	Runoff coefficient, C = 0.95	
Equation 2.3	Q = $CIA/360$ = (0.95)(335.25)(0.13)/360 =	0.115m^3/s
	Step 6. Calculate spread, T	
Equation 13.1	$T = \sqrt[2.67]{\frac{[Q \times n]}{KS_x^{1.67}S_L^{0.5}}}$ $= \sqrt[2.67]{\frac{0.115 \times 0.016}{(0.376)(0.02)^{1.67}(0.03)^{0.5}}}$ =	3.04m

APPENDIX 13.C EXAMPLE - COMPOSITE GUTTER

Problem:

Given gutter width, W = 0.6m, S_L = 0.03, S_X= 0.04, n = 0.016, gutter depression, a = 50mm. Then, find spread, T based on flow, Q from Example in Appendix 13.B.

Solution:

Reference	Calculation	Output
	Step 1. Calculate the cross slope of the depressed gutter, S_w	
	$S_W = \frac{a}{W} + S_X$	
	$= \frac{0.05}{0.6} + 0.04$ =	0.123
	Step 2. Calculate Q_W	
	Try $Q_S = 0.033 m^3/s$	
Equation13. 3(b)	$Q_W = Q - Q_S$	
	$= 0.115 - 0.033$ =	$0.082 m^3/s$
	Step 3. Determine W/T ratio :	
Equation 13.6	$E_o = Q_W / Q$	
	$= 0.082/0.115$ =	0.713
	=	3.08
	S_W / S_X = 0.123 / 0.04	
Chart 13.A2	With the value of E_o and S_W / S_X, use chart 13.A2 to get the value of W/T	
	W/T =	0.30
	$T = W/0.30$	
	$= 0.6/0.30$ =	2.00m
	Step 4. T_s based on assumed Q_S	
	$T_s = T - W$	
	$= 2.00 - 0.6$ =	1.40m
	Step 5. Q_S for calculated T_S	
Equation 13.1	$Q_S n = K S_X^{1.67} S_L^{0.5} T^{2.67}$	
	$=(0.376)(0.04)^{1.67}(0.03)^{0.5}(1.40)^{2.67}$ =	$0.00074 m^3/s$
	$Q_s = 0.00074/0.016$ =	$0.046 m^3/s$
	Step 6. Compare assumed Q_s with calculated Q_s	
	assumed $Q_s = 0.033 m^3/s$ < calculated $Q_s = 0.046 m^3/s$	
	Step 7. Try new assumed Q_S	
	$Q_S = 0.022 m^3/s$	
	$Q_W = 0.115 - 0.022$ =	$0.093 m^3/s$
	$E_o = Q_W/Q = 0.093/0.115$ =	0.809

Reference	Calculation		Output
	S_W/S_X = 0.123/0.04	=	3.08
Chart 13.A2	W/T	=	0.36
	T = 0.6/0.36	=	1.67m
	T_s = 1.67 – 0.6	=	1.07m
Equation 13.1	Q_{sn} = $KS_X^{1.67}S_L^{0.5}T_s^{2.67}$		
	= $(0.376)(0.04)^{1.67}(0.03)^{0.5}(1.67)^{2.67}$	=	0.000352m^3/s
	Q_s = 0.000352/0.016	=	0.022m^3/s
	<u>Step 8.</u>		
	Assumed Q_s of 0.022m^3/s equals calculated Q_s		
	Therefore, T	=	1.67m

APPENDIX 13.D EXAMPLE - V–SHAPED ROADSIDE GUTTER

Problem:

Given S_L = 0.01, n = 0.016, S_{X1} = 0.25 (Refer to Figure 13.2), S_{X2} = 0.04, S_{X3} = 0.02, distance $\overline{BC}$ = 0.6m (Refer to Figure 13.2). Then, find spread at flow based on Example in Appendix 13.B.

Solution:

Reference	Calculation	Output
	Step 1. Calculate S_x	
Equation 13.4	$S_X = \frac{S_{X1}S_{X2}}{S_{X1}+S_{X2}}$	
	$S_X = \frac{(0.25)(0.04)}{0.25+0.04}$	= 0.0345
	Step 2. Calculate T_1	
Equation 13.1 or Chart 13.A1	$T_1 = \sqrt[2.67]{\frac{[Q \times n]}{KS_X^{1.67} S_L^{0.5}}}$	
	$T_1 = \sqrt[2.67]{\frac{0.115 \times 0.016}{(0.376)(0.0345)^{1.67}(0.01)^{0.5}}}$	= 2.65m
	Step 3. Calculate $\overline{AC}$	
	To determine if T_1 is within S_{X1} and S_{X2} , calculate depth at point B in the V-shaped gutter, knowing BC and S_{X2}. Then, knowing the depth at B, distance AB can be computed.	
	S_{X2} = $dB/\overline{BC}$	
	dB = $\overline{BC}\ S_{X2}$	
	= (0.6)(0.04)	= 0.024m
	$\overline{AB}$ = dB/S_{X1}	
	= (0.024) / 0.25	= 0.096m
	$\overline{AC}$ = $\overline{AB} + \overline{BC}$	
	= 0.096 + 0.6	= 0.696m ~ 0.7m
	0.7m < T_1 therefore, spread falls outside V-shaped gutter section.	
	Step 4. Calculate spread T (Refer to Figure 13.2)	
	Assumed $\overline{BD}$ = 3m.	
	dC = $\overline{CD}\ S_{X3}$	
	= (3.0-0.6) x 0.02	
	$\overline{AB}$ $= \frac{0.048+0.024}{0.25}$	= 0.048m
	T = $\overline{AB}$ +$\overline{BD}$	= 0.0288m
	= 0.288 + 3	= 3.288m

Reference	Calculation	Output
	Step 5. Calculate T based on Equation 13.1	
	Develope a weighted slope for S_{X2} and S_{X3} based on assumed $\overline{BD}$ = 3m	
	$S_{X_2} = \frac{0.024+0.048}{3}$ =	0.024
Equation 13.4	$S_X = \frac{S_{X1}S_{X2}}{S_{X1}+S_{X2}}$	
	$= \frac{(0.25)(0.024)}{(0.25+0.024)}$ =	0.0219
Equation 13.1	$T = \sqrt[2.67]{\frac{[Q\times n]}{KS_X^{1.67}S_L^{0.5}}}$	
	$= \sqrt[2.67]{\frac{(0.115)(0.016)}{(0.376)(0.0219)^{1.67}(0.01)^{0.5}}}$ =	3.53m
	The spread T computed in step 5 in higher than T computed in step 4	
	Step 6	
	Assume $\overline{BD}$ = 3.24m	
	dC = $\overline{CD}\ S_{X3}$	
	= (3.24-0.6) x 0.02 =	0.0528
	$AB = \frac{0.0528+0.024}{0.25}$ =	0.3072
	T = $\overline{AB}$ + $\overline{BD}$	
	= 0.3072 + 3.24 =	3.55m
	Step 7.	
	$S_X = \frac{S_{X1}S_{X2}}{S_{X1}+S_{X2}}$	
	$Sx = \frac{(0.25)(0.023)}{(0.25+0.023)}$ =	0.0216
Equation 13.1	$T = \sqrt[2.67]{\frac{[Q\times n]}{KS_X^{1.67}S_L^{0.5}}}$	
	$= \sqrt[2.67]{\frac{(0.115)(0.016)}{(0.376)(0.0216)^{1.67}(0.01)^{0.5}}}$ =	3.56m
	T= 3.56m close to T= 3.55m computed based on assumed $\overline{BD}$ of 3.24m	
	Therefore T is =	3.56m

APPENDIX 13.E EXAMPLE – CURB OPENING INLET

Problem:

Given a curb-opening inlet with the following characteristics, S_L= 0.03, S_X= 0.04, Q = 0.115m^3/s (from Example in Appendix 13.B) n = 0.016. Find

(i) Q_i for 3m curb-opening
(ii) Q_i for a depressed 3m curb opening inlet with a continuous curb section.
a = 25mm
W = 0.6m

Solution:

Reference	Calculation	Output
	Solution (i) :	
Equation 13.12 or Chart 13.A7	Step 1. Determine the length of curb opening required for total interception of gutter flow	
	$L_T = 0.817\, Q^{0.42}\, S_L^{0.3}\, (1/(n.S_X))^{0.6}$	
	$= (0.817)(0.115)^{0.42}(0.03)^{0.3}[1/(0.016 \times 0.04)]^{0.6}$ =	9.49m
Equation 13.13 or Chart 13.A8	Step 2. Calculate the curb-opening efficiency.	
	$L/L_t = 3/9.49$ =	0.32
	$E = 1 - (1 - L/L_t)^{1.8}$	
	$= 1 - (1- 0.32)^{1.8}$ =	0.5
Equation 13.11	Step 3. Calculate the interception capacity.	
	$Q_i = E\, Q$	
	= (0.5)(0.115) =	0.058m^3/s
	Solution (ii) :	
	Step 1. Determine W/T ratio.	
	Determine spread, T, (Procedure from Example 1 in Appendix 13.C	
Equation 13.3a or Chart 13.A2 and Equation 13.1 or Chart 13.A1	Assume Q_s = 0.036m^3/s	
	$Q_W = Q - Q_s$	
	= 0.115 – 0.036 =	0.079m^3/s
	$E_o = Q_W/Q$	
	= 0.079 / 0.115 =	0.687
	$S_W = S_X + a/W$	
	= (0.04) + (0.025/0.6) =	0.082
	S_W/S_x = 0.082/0.04 =	2.05
Chart 13.A2	Determine W/T	
	W/T = 0.32	
Equation 13.3a or	T = 0.6 / 0.32 =	1. 88m
	$T_s = T - W$	
	= 1.88 – 0.6 =	1.28m

Reference	Calculation	Output
Equation 13.1	Calculate value of Q_s $Q_s = \frac{K_u}{n} S_X^{1.67} S_L^{0.5} T^{2.67}$ $= \frac{0.376}{0.016}(0.04)^{1.67}(0.03)^{0.5}(1.28)^{2.67}$ =	$0.036m^3/s$ (equals to Q_s assumed)
Equation 13.14	Step 2. Determine efficiency of curb opening $S_e = S_X + S'_W E_o = S_x + (a/W)E_o$ $= 0.04 + (0.025/0.6)(0.079)$ =	0.0686
Equation 13.15 or Chart 13.7	$L_T = 0.817\ Q^{0.42}\ S_L^{\ 0.3}\ (1/nS_X)^{0.6}$ $= (0.817)(0.115)^{0.42}\ (0.03)^{0.3}\ [1/(0.016 \times 0.0686)]^{0.6}$ =	6.86m
Equation 13.13 or Chart 13.A8	Determine curb inlet efficiency. $L/L_T = 3/6.86 = 0.44$ $E = 1 - (1 - L/L_T)^{1.8}$ $= 1 - (1 - 0.44)^{1.8}$ =	0.65
Equation 13.11	Step 3. Calculate curb opening inflow $Q_i = Q\ E = (0.115)(0.65)$ =	$0.075m^3/s$

APPENDIX 13.F EXAMPLE –INLET SPACING

Problem:

Given the storm drainage system illustrated in Figure 13.F1 has the roadway characteristics which are n = 0.016, Pavement cross slope = 0.02, S_L = 0.03, allowable spread = 2.0m, gutter and shoulder cross slope, Sx = 0.04, and curb height = 0.3m. Then, find the design inlet spacing for a 0.60m wide by 0.9m long P50 x 100 grate, during a 10 years storm event.

Solution:

Reference	Calculation	Output
Figure 13.8	Steps 1-4 The calculation can begin at the the inlet located at station 21+00 (First inlet from the crest of the roadway. The crest which is the top of the drainage basin is located at station 22+00). The initial drainage area for locating the first inlet consist of a 13m wide roadway section with a length of 100m.	
	Step 5 Column 1 Inlet 1 Column 2 Station 21+00 Column 19 Gutter with a curb height =	0.3m
	Step 6 Column 3 Distance from top of drainage area to first inlet = 22+00– 21+00 =	100m
	Width = 13m Drainage area = (100)(13) =	$1300m^2$ 0.13ha
Table 2.5	Step 7 Column 4 Runoff coefficient, C = 0.95	
	Step 8 Column 5 Firstly calculates velocity of gutter flow, assumed the spread of gutter, T is 1 m $V = 1/n R^{2/3}S^{1/2}$ $V = (1/0.016)(0.0192)^{2/3}(0.03)^{1/2}$ =	0.776m/s
Table 2.1	Calculate the time of concentration, t_c $t_c = L / (60 V) = 100 / [(60)(0.776)]$ =	2.15 minutes (Use 5min. minimum)
	Step 9 Column 6 Determine rainfall intensity, I for Wilayah Persekutuan Kuala Lumpur t_c = 5 minutes Find $^{10}I_5$	
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$ $= (63.24)(10)^{0.162}/(0.0833 + 0.137)^{0.856}$ =	335.25mm/hr

Reference	Calculation	Output
Equation 2.3	Step 10. Column 7 Determine Gutter flow rate, Q $Q = CiA/360 = (0.95)(335.25)(0.13)/360$	= $0.115 m^3/s$
	Step 11 Column 8 S_L	= 0.03
	Step 12 Column 9 S_x	= 0.04
	Step 13 Column 13 W	= 0.60m
Equation 13.1 or Chart 13.A1	Step 14 Column 14 Determine spread, T $T = \sqrt[2.67]{\frac{[Q \times n]}{KS_x^{1.67} S_L^{0.5}}}$ $= \sqrt[2.67]{\frac{(0.115)(0.016)}{(0.376)(0.04)^{1.67}(0.03)^{0.5}}}$ $= 1.969\ m$ (Less than allowable is 2 m, so proceed Next step.)	
Equation 13.2	Column 12 Determine depth at curb, d, using $d = TS_x = (1.969)(0.04)$	= 0.079m
	Step 15 Column 15 $W/T = 0.60/1.969$	= 0.305
	Step 16 Column 16 Select P50 x 100 grate measuring 0.60m wide by 0.9m long.	
Equation 13.6	Step 17 Column 17 Calculate intercepted flow, Q_t $E_o = 1-(1-W/T)^{2.67}$ $= 1 – (1-0.305)^{2.67}$	= 0.621
Equation 13.5	Calculate velocity, V $V = 0.752/n\ S_L^{0.5}\ S_x^{0.67}\ T^{0.67}$ $V = (0.752/0.016)(0.03)^{0.5}(0.04)^{0.67}(1.969)^{0.67}$	= 1.489m/s
Chart 13.A5	Find R_f $R_f = 1.0$	
Equation 13.9	Find R_s $R_s = 1/(1+ K_u\ V^{1.8} / (S_x\ L^{2.3}))$ $= 1 / (1+0.0828\ V^{1.8} / (S_x\ L^{2.3}))$ $= 1 / 1 +((0.0828)(1.489)^{1.8} / (0.04)(0.9)^{2.3})$	= 0.156
	Calculate value Q_t by using $Q_t = Q[R_f\ E_o + R_s\ (1-E_o)]$ $= 0.115[(1.0)(0.621) + 0.156\ (1-0.621)]$	= $0.0782 m^3/s$
Equation 13.11	Step 18 Column 18 $Q_b = Q – Q_t$ $= 0.115 – 0.0782$	= $0.0368 m^3/s$
	Step 19 Column 1 Inlet 2 Column 2 Station 20+ 32 (taking distance to successive inlets =68m Column 3 Drainage area = 68x13 Column 4 Runoff coefficient, $C = 0.95$	= $884 m^2$ 0.0884ha
	Step 20 Column 5 $V = 0.776 m/s$ (same as step 8 Col.5) $T_c = L/[60V] = (68)/(60)(0.776)$	= 1.46min (use 5min minimum)
Table 2.1	Step 21 Column 6 $I = 335.21 mm/hr$ (same as step 10)	
Equation 2.3	Step 22 Column 7 $Q = CiA/360 =(0.95)(335.25)(0.0884)/360$	= $0.0782 m^3/s$

Reference	Calculation	Output
	Step 23 Column 11 = Col. 18 + Col. 7 = 0.0368 + 0.0782 =	0.115m^3/s
	Step 24 Column14 T = 1.969m $T < T$ allowable = 2m Column 12 d = *(1.969)(0.04)* =	0.079 < d curb = 0.3m
	Step 25 Column 16 Select P50 x 100 grate 0.60m wide by 0.9m long.	
	Step 26 Column 17 Q_t = 0.0782m^3/s	
	Step 27 Column 18 Q_b = Q – Q_t = Column 11 – Column 17 = 0.115 – 0.0782 =	0. 0368m^3/s
	Step 28 Repeat steps 19 through 28 for each subsequent inlet down to the low point. This will end up with maximum distance to successive inlets of about 68m.	
	The required spacing of successive inlets is therefore 68m.	

Table 13.F1 Inlet Spacing Computation Sheet

INLET SPACING COMPUTATION SHEET												Date : JUN 2010 Computed By: Ramli		Route/Location : Sheet 1 of 1		Wilayah Persekutuan Kuala Lumpur		
Inlet		Gutter Discharge Design Frequency					Gutter Discharge Allowable Spread								Inlet Discharge			
No. (1)	Station (2)	Drainage Area(ha)(3)	Runoff Coefficient, C (4)	Time of Concentration, t_c (min) (5)	Rainfall Intensity, *i* (mm/hr) (6)	$Q = CIA/K_c$ (m^3/s) (7)	Longitudinal Slope, S_L (m/m) (8)	Cross Slope, S_x or S_w (m/m) (9)	Previous By-pass Flow (m^3/s)(10)	Total Gutter Flow (m^3/s)(11)	Depth, d (m) (12)	Grate or Gutter Width, W (m) (13)	Spread, T (m)(14)	W/T (15)	Inlet Type (16)	Intercept Flow, Q_i (m^3/s)(17)	By-pass Flow, Q_b (m^3/s) (18)	Remarks (19)
1	21+00	0.1300	0.95	5.00	335.25	0.1150	0.03	0.04	0.000	0.115	0.079	0.60	1.969	0.30	P50X100	0.078	0.037	0.3 (curb height)
2	20+32	0.0884	0.95	5.00	335.25	0.0782	0.03	0.04	0.037	0.115	0.079	0.60	1.969	0.30	P50X100	0.078	0.037	0.3 (curb height)

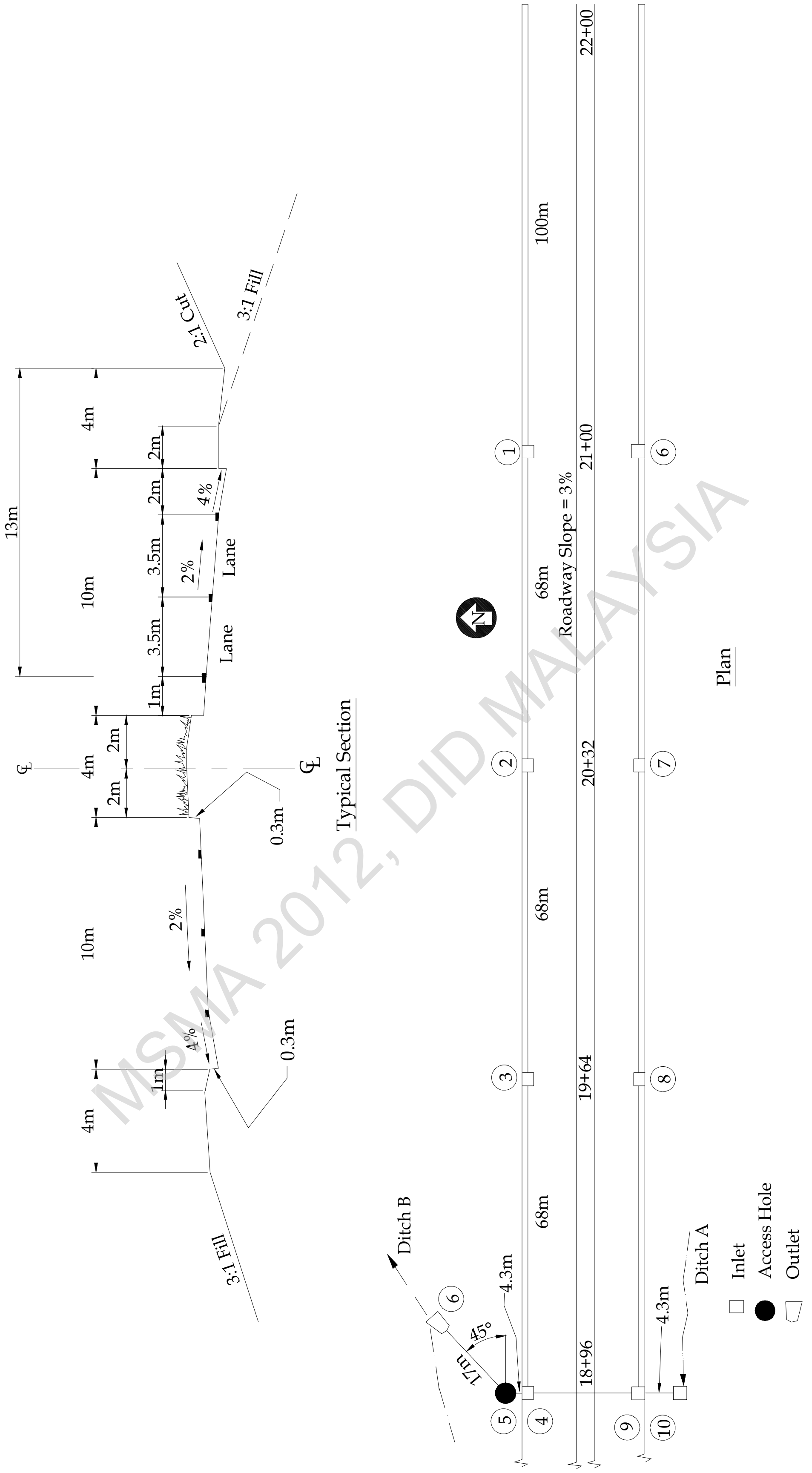
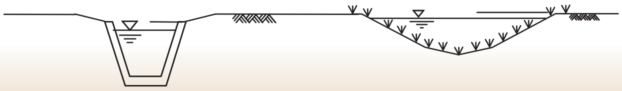


Figure 13.F1: Storm Drainage System for Example in Appendix 13.F



DRAINS AND SWALES 14

CHAPTER 14 DRAINS AND SWALES

14.1 INTRODUCTION

This chapter provides guidelines for the design of open drainage system, such as lined drains and grassed swales. These facilities, along with stormwater inlets are components of the minor drainage system designed to collect minor flood flows from roads, properties and open space, and convey them to the major drainage system.

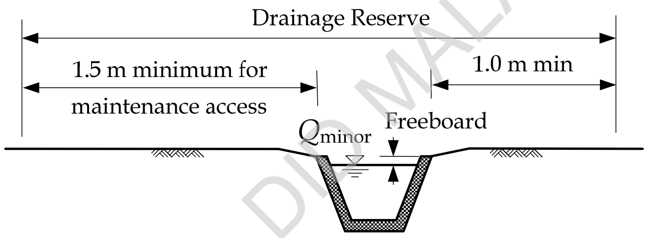
It should be noted that fully lined drains are not encouraged anymore in local practice while grass lined ones as encouraged. Developers and designers shall seek approval from the local regulatory authority if such needs arise. Much of procedures and experience that deal with open drainage system have been established in Malaysian practice since late 1970s.

14.1.1 Design Storm

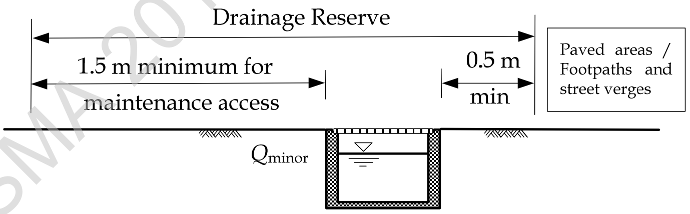
Drains and swales should have the capacity to convey the flow up to and including the minor system design ARI.

14.1.2 Drainage Reserves

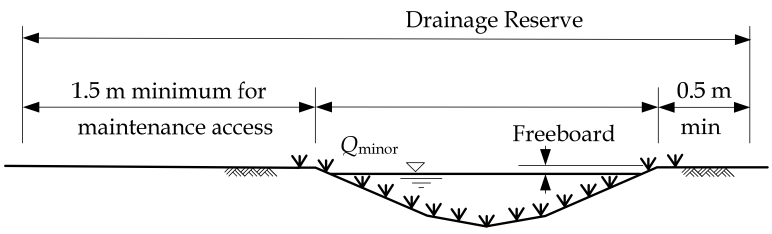
Most open drains will be located within road reserve and therefore do not require a separate reserve to allow access for maintenance. However, open drains and swales located outside of road reserves, such as in public walkways and open space areas, should be provided with a drainage reserve in accordance with Figure 14.1.



(a) Lined Open Drain



(b) Covered Lined Open Drain



(c) Swale

Figure 14.1: Drainage Reserve for Minor Drain and Swale

In new development areas, the edge of a swale should generally be located 0.5 m from the road reserve or property boundary. In existing areas, this alignment may be varied depending on the alignment and depth of existing underground services within the road verge. The designer should consult the local regulatory authority for appropriate alignments in existing areas.

14.2 STRUCTURAL AND COMPOSITE DRAIN

14.2.1 Description

A lined drain is highly resistant to erosion. This type of drain is expensive to construct, although it should have a very low maintenance cost if properly designed. Non-erodible lining should be used when stability cannot be achieved with a swale. Its principal disadvantages are high initial cost, susceptibility to failure if undermined by scour and the tendency for scour to occur downstream due to high flow velocities and acceleration of the flow on a steep slope or in critical locations where erosion would cause extensive damage.

A composite drain is combination of a grassed section and a lined drain that may be provided in locations subject to dry-weather base flows which would otherwise damage the invert of a grassed swale, or in areas with highly erodible soils. The composite drain components shall comply with the relevant design requirements specified for grassed swales and lined drains.

14.2.2 Lining Materials

Lined drains shall be constructed from materials proven to be structurally sound and durable and have satisfactory jointing systems.

Lined open drains may be constructed with any of the following materials:

- plain concrete;
- reinforced concrete;
- stone pitching;
- plastered brickwork; and
- precast masonry blocks.

Alternative drain materials may be acceptable. Proposals for the use of other materials shall be referred to the Local Authority for consideration on a case-by-case basis, especially for steep sloped high velocity applications.

14.2.3 Design Consideration and Requirements

The design should take into consideration of site conditions as described below.

(a) Drainage Area

Determination of drainage types (earth/concrete/composite) based on space availability, site suitability, environment conditions (aesthetic, conservation values, etc.) and maintenance advantages and disadvantages. Standardised locations for lined drains are provided to limit the negotiations needed when other services are involved.

(b) Roadway Reserves

The outer edge of a lined drain should be located 0.5 m (minimum) from the property boundary on the high side of road reserves to permit relatively short connections to service adjacent properties. Lined drains may also be located within road median strips.

The local regulatory authority should be consulted for standard alignments of public utility services within street verges.

Where there is significant advantage in placing a lined drain on an alignment reserved for another authority, it may be so placed provided that both the authorities agree upon the responsibility for maintenance of the stormwater conveyance. The other authority concerned shall provide written approval to release the reservation.

Curved alignments are preferred on curved roadways. However, in areas such as culs-de-sac and narrow street verges, straight alignments may be acceptable, where there are significant advantages in doing so.

(c) Privately Owned Lots

Municipal lined drains shall not be located within privately owned properties. Where lined drains are to be provided at the side or rear of private properties, they shall be placed within a separate drainage reserve in accordance with Figure 14.1(a) or Figure 14.1(b).

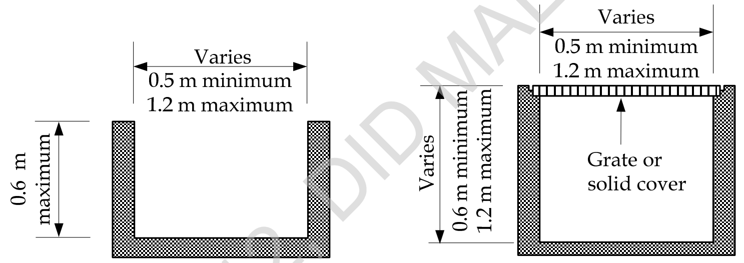
(d) Public Open Space

The location of lined drains within public land such as open space shall be brought to the attention of the local regulatory authority for consideration. As a guide, unless directed otherwise, lined drains shall be located as close as possible to the nearest property boundary with due consideration for public safety. Appropriate safety measures shall be provided to protect the public from being trapped within a drain during flash flooding.

14.2.4 Design Criteria

14.2.4.1 Geometry

The dimensions of lined open drains have been limited in the interests of public safety and to facilitate ease of maintenance. The minimum and maximum permissible cross-sectional dimensions are illustrated in Figure 14.2.



(a) Uncovered Open Drain (b) Covered Open Drain

Figure 14.2: Dimension Limits for Open Lined Drains

The preferred shape for a composite drain is shown in Figure 14.3. The lined drain section is provided at the drain invert to carry dry-weather base flows and minor flows up to a those resulting from a 10 mm rainfall depth over the contributing catchment. The lining section shall be sized to provide additional flow capacity up to and including the design storm ARI.

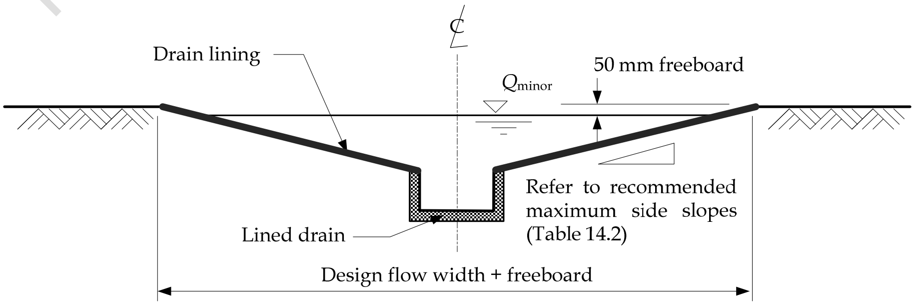


Figure 14.3: Recommended Composite Drain Cross Section

(a) Depth

The maximum depth for lined open drains shall be in accordance with Table 14.1. A reinforced concrete drain shall be provided for lined open drains that exceed 0.9 m in depth.

Table 14.1: Recommended Maximum Depths

Cover/Handrail Fence Condition	Maximum Depth (m)
Without protective covering	0.6
With solid or grated cover	1.2

(b) Width

The width of lined open drains may vary between a minimum width of 0.5 m and a maximum of 1.2 m.

(c) Side slope

The recommended maximum side slopes for lined open drains is indicated in Table 14.2.

Table 14.2: Recommended Maximum Side Slopes

Drain Lining	Maximum Side Slope
Concrete, brickwork and blockwork	Vertical
Stone pitching	1.5(H):1(V)
Grassed/vegetated, rock riprap	2(H):1(V)

14.2.4.2 Freeboard

The depth of an open lined drain shall include a minimum freeboard of 50 mm above the design storm water level in the minor drain.

14.2.4.3 Velocities and Longitudinal Slope

To prevent sedimentation and vegetative growth, the minimum average flow velocity for minor drain shall not be less than 0.6 m/s. The maximum flow velocity in open drain should be restricted to a maximum of 2 m/s. However, for flow velocities in excess of 2 m/s and less than 4 m/s, drains shall be provided with a 1.2 m high handrail fence, or covered with metal grates or solid plates for the entire length of the drain for public safety.

As longitudinal slope increase the velocity increases proportionally. Open drains longitudinal slope should be constant and no steeper than 0.2%. Drop structures may be required to reduce the longitudinal slope in order to control flow velocities.

14.2.4.4 Roughness Coefficients

Lined drain materials for proposed systems typically include concrete, stone pitching, riprap, gabions, brickworks and precast masonry. The suggested Manning's roughness coefficients for these drain materials are indicated in Chapter 2 (Table 2.3).

14.2.4.5 Drainage Sumps

Drainage sumps (with a minimum size of 450 mm x 450 mm) shall be provided along covered drains with a maximum interval spacing of every 100 m and a minimum depth from drain invert is 600 mm.

14.2.4.6 Safety Requirements

Open drains in locations exposed to pedestrian access or sited close to carriageway shall be covered if the drain exceeds 0.6 m in depth. The type of drain cover used will depend on the expected live loadings and whether or not the drain is required to accept surface flow. The acceptable types of drain covering are as follows:

(a) Precast Reinforced Concrete Covers

Drains not subject to traffic loads or inflow of surface runoff may be covered using precast reinforced concrete covers. Covers should be sized such that the weight is limited to what can be easily lifted by two workmen to gain access for maintenance.

(b) Metal Grates and Solid Plates

Drains subject to vehicular traffic loads or inflow of surface runoff shall be covered using metal grates or solid plates. Metal covers shall be designed in accordance with the latest editions of relevant Malaysian Standards or equivalent.

(c) Cover Levels

Covers for lined open drains shall be set at the finished cover levels given in Table 14.3.

Table 14.3: Cover Levels

Location	Cover Level
Paved areas	Flush with finished surface
Footpaths and street verges	Flush with finished surface
Elsewhere	100 mm above the surface to allow for top soiling and grassing

(d) Handrail Fence

The maximum depth for cover or handrail fence condition shall be in accordance with Table 14.1.

Adequate safety measures such as 1.2 m high handrail fence shall be provided at populated areas or areas exposed to pedestrian access. Vertical drops should be avoided. Fencing or railings may also need to be considered if side slopes are to be steeper than 3(H):1(V) in the design.

14.2.5 Design Procedure

The preliminary sizing estimation procedure for minor drain is given below:

Step 1: Estimate the design discharge, Q_{minor} based on the design minor ARI using suitable methods from those outlined in Chapter 2 (Section 2.3).

Step 2: Estimate Manning's n of the lining material.

Step 3: Select the design cross-section. Determine the depth and the minimum base width for the proposed system. Determine the proposed drain capacity using Manning's Equation.

Step 4: Compare the estimated drain capacity with the calculated design discharge, Q_{minor}. If the drain capacity is found to be inadequate, then the drain cross section should be modified to increase the capacity. Likewise a reduction in the cross section may also be required if the drain is not to be overdesigned. In the case of any modifications to drain cross section, repeat Step 3.

Step 5: Calculate the average flow velocity from $V = Q/A$ and check that it is within the maximum and minimum velocity criteria for the open drain. If not, adjust the drain dimensions and return to Step 3.

Step 6: Determine the flow depth, y and check if y is within required limits for the open drain type. If not, adjust the drain dimensions and return to Step 3.

Step 7: Add the required freeboard. If required, calculate the top width of drain for drains with sloping sides.

Step 8: Calculate the width of the drainage reserve.

14.3 SWALES

14.3.1 Description

Swales are broad and shallow channels designed to store and/or convey runoff at a non-erosive velocity, as well as enhance its water quality through infiltration, sedimentation and filtration. Swales may be covered by dense vegetation, usually grass to slow down flows and trap particles and remove pollutant.

(a) Universiti Tun Hussein Onn Malaysia (b) Taiping Health Clinic (Type 2)

Figure 14.4: Grassed Swales in Malaysia

14.3.2 Advantages

- easy to incorporate into landscaping;
- good removal of urban pollutants;
- reduces runoff rates and volumes;
- low capital cost;
- maintenance can be incorporated into general landscape management; and
- good option for small area retrofits.

14.3.3 Disadvantages

- not suitable for steep areas;
- limited to small areas;
- risks of blockages in connecting pipework/culverts;
- sufficient land may not be available for suitable swale designs to be incorporated; and
- standing water in vegetated swales can result in potential safety, odour, and mosquito problems.

14.3.4 Design Consideration and Requirements

14.3.4.1 Drainage Area

Grassed swales engineered for enhancing water quality cannot effectively convey large flows. Therefore, swales are generally appropriate for catchments with small, flat impermeable areas. If used in areas with steep slopes, grassed swales must generally run parallel to contours in order to be effective.

14.3.4.2 Space Requirement

Grassed swales must be effectively incorporated into landscaping and public open spaces as they demand significant land-take due to their shallow side-slopes. Grassed swales are generally difficult to be incorporated into dense urban developments where limited space may be available (CIRIA, 2007).

14.3.4.3 Location and Site Suitability

Swales should be integrated into the site planning and should take account of the location and use of other site features. The siting of a swale should be such that the topography allows for the design of a channel with sufficiently mild slope and cross-sectional area to maintain non-erosive velocities. Swales should not be sited on unstable ground and ground stability should be verified by assessing site soil and groundwater conditions. Maintenance access should be easy and good growth of vegetation should be ensured by siting swales in areas that receive sufficient sunlight (CIRIA, 2007).

14.3.4.4 Site Slope

Grassed swales are usually restricted to sites with significant slopes, though careful planning should enable their use in steeper areas by considering the contours of the site (CIRIA, 2007). The longitudinal terrain slope should not exceed 2% as low runoff velocities are required for pollutant removal and to prevent erosion. Longitudinal slopes can be maintained at the desired gradient and water can flow into swales laterally from impermeable areas.

14.3.4.5 Subsurface Soils and Groundwater

Where grassed swales are designed to encourage infiltration, the seasonally high groundwater table must be more than 1 m below the base of the swale. Where infiltration is not required, the seasonally high groundwater level should be below any underdrain provided with the swales (CIRIA, 2007).

14.3.5 Design Criteria

14.3.5.1 Alignment

Standardised alignments for grassed swales are provided to limit the negotiations needed when other services are involved, such as privately owned lots and public open space.

Municipal grassed swales shall not be located within privately owned properties. If grassed swales are to be provided at the side or rear of private properties, they shall be placed within a separate drainage reserve of minimum dimensions in accordance with Figure 14.1. The location of swales within public land such as open space should generally conform to natural drainage paths wherever practical. The designer should consult with the local regulatory authority for appropriate alignments with due consideration for public and aesthetic amenity.

14.3.5.2 Geometry

The preferred shapes for swales are shown in Figure 14.5. The depth shall not exceed 1.2 m. A ‘vee’ or triangular shaped section will generally be sufficient for most applications; however, a trapezoidal or parabolic swale shape may be used for additional capacity or to limit the depth of the swale. Swales with trapezoidal cross sections shall be recommended for ease of construction. A parabolic shape is best for erosion control, but is hard to construct.

For a trapezoidal shape, the bottom width should be between 0.5 m and 3.0 m. The 0.5 m minimum bottom width allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The 3.0 m maximum bottom width prevents shallow flows from concentrating and potentially eroding channels, thereby maximizing the filtering by vegetation.

Side slope shall not be steeper than 2(H):1(V) while side slope 4(H):1(V) or flatter is recommended for safety reason. However, side slope of 2(H):1(V) in residential areas are strongly discouraged. The larger the wetted area of the swale, the slower the flow and the more effective it is in removing pollutants.

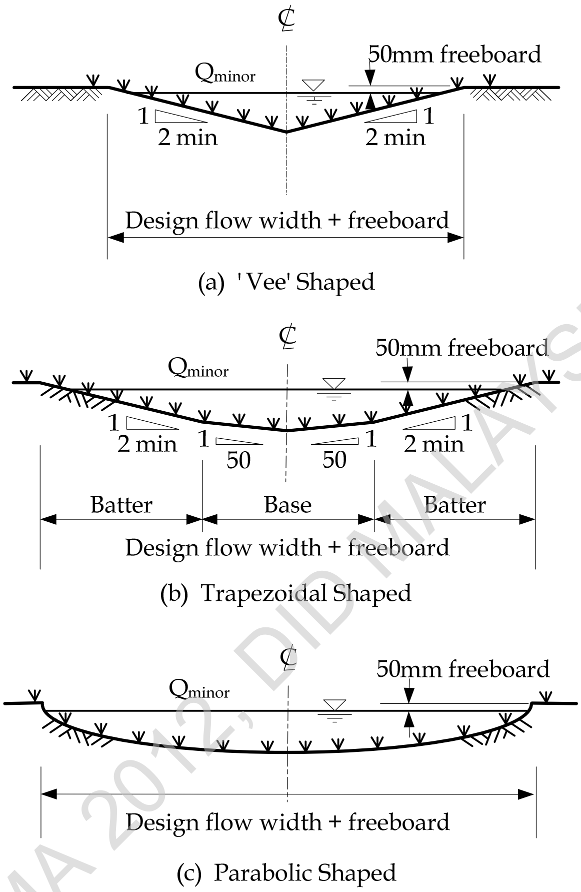


Figure 14.5: Recommended Swale Cross Sections

14.3.5.3 Longitudinal Slope

Slope of swales should normally be between 0.1% (1 in 1000) and no greater than 0.5% (1 in 200). Underdrains may be required for slopes below 0.2% (1 in 500), while drop structures such as rock check dams in the channel may be required for slopes greater than 0.2% to reduce the drainage longitudinal slope such that the design flow velocities do not exceed the permissible limits.

14.3.5.4 Freeboard

The depth of a swale shall include a minimum freeboard of 50 mm above the design stormwater level (based on maximum design flows) in the swale to allow for blockages.

14.3.5.5 Velocities

Maximum acceptable flow rate velocities for conveyance of peak design flow (maximum flood flow design) along the swale shall not exceed the recommended maximum scour velocity for various ground covers and values of soil erodibility, or ideally be less than 2 m/s, unless additional erosion protection is provided.

14.3.5.6 Grass Cover

A dense planting of grass provides the filtering mechanism responsible for water quality treatment in swales. Therefore, the grass species chosen for lining of grassed swales must be sturdy, drought resistant, easy to establish, and able to spread and develop a strong turf layer after establishment. A thick root structure is necessary to control weed growth and erosion.

Grass is by far the most effective choice of plant material in swales, however not all grass species are best for vegetative cover. The grasses recommended for permanent seed mixes shall be selected for those in Annex 1.

Compacted soils will need to be tilled before grass seeding or planting. At least 100 mm thick of good quality topsoil is required. General guidelines for establishing an effective grass lining shall be in accordance with Annex 1.

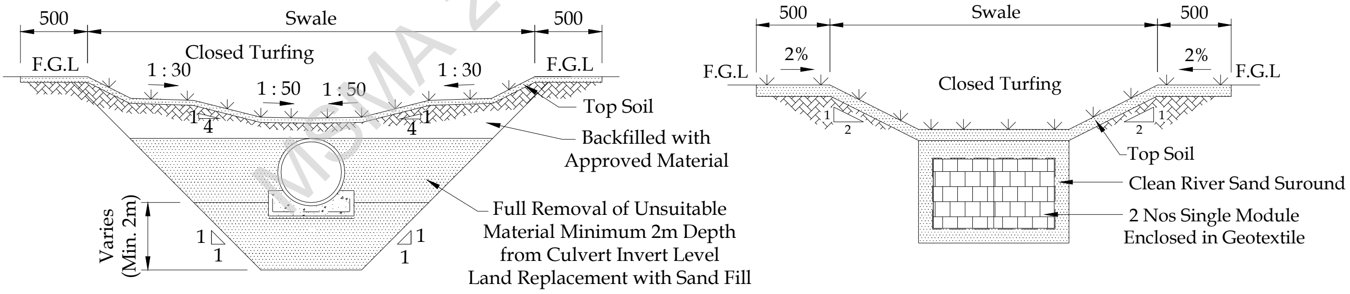
14.3.5.7 Roughness Coefficients

The roughness coefficient, *n*, varies with the type of vegetative cover, longitudinal slope and average flow depth. The *n* value must be adjusted for varying flow depths. Manning's roughness coefficients for swales are provided in Chapter 2 (Table 2.3).

14.3.5.8 Underdrain

A swale should have the capacity to convey the peak flows from the design minor ARI without exceeding the maximum permissible velocities. If this is not practical or there is insufficient space for a swale, designer should consider dividing the flow into surface and subsurface conduits where underground pipedrains or drainage modules can be provided (Figure 14.6). Underdrains can also be placed beneath the channel to prevent ponding.

It is important for biofiltration swales to maximise water contact with vegetation and the soil surface. Gravely and coarse sandy soils will not provide water quality treatment unless the bottom of the swale is lined to prevent infiltration. (Note: sites that have relatively coarse soils may be more appropriate for stormwater quantity infiltration purposes after runoff treatment has been accomplished). Therefore, the bed of a biofiltration swale shall consist of a permeable soil layer above the underdrain material.



(a) Underground Pipedrain

(b) Drainage Module

Figure 14.6 Example of Underdrains Provided in the Subsurface of a Swale

A composite swale is a combination system of underground pipedrain and swale. The underdrain caters to the minor storm event for transporting runoff from roadway and other inlets to the outfalls and receiving waters. Drainage sumps shall be provided along swales with a maximum interval spacing of every 100 m. The design of underground pipedrain system should conform to the design criteria as shown in Chapter 15.

14.3.5.9 Low Flow Provision Channel

For swales that will be subjected to dry weather flows, an underdrain or surface invert should be provided.

14.3.5.10 Water Quality Treatment and Flood Flow Design

Swales should be sized as both a treatment facility to slow the stormwater as much as possible to encourage pollutant removal, and as a conveyance system to pass the peak hydraulic flows of the design storms.

(a) Design Water Quality Flow Depth

The sizing calculations for water quality treatment design should check flow velocity, depth and residence times during water quality design storm (3-month ARI). To be effective, the depth of the stormwater during treatment must not exceed the height of the grass. A grass height of 150mm or more and a flow depth of less than 150mm should be selected for the water quality design storm (Figure 14.5). Grasses over that height tend to flatten down when water is flowing over them, which prevents sedimentation. To attain this height requires regular maintenance.

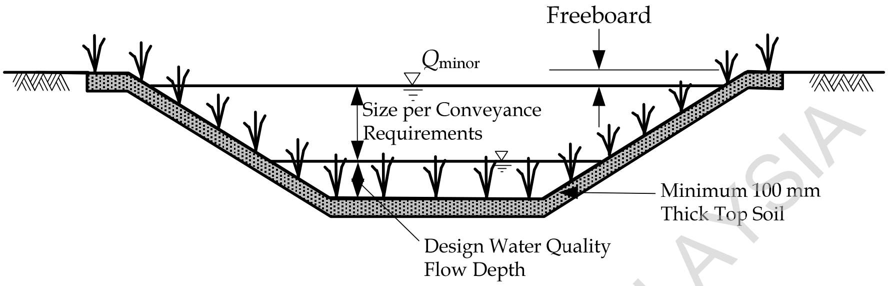


Figure 14.5: Water Quality Treatment and Flood Flow Design for Swale

The velocity of runoff shall not exceed 0.5 m/s along a swale during the water quality design storm to avoid flattening of the vegetation; however, lower velocities are preferable. Design calculation should allow the water quality storm and capacity check for the design storm.

(b) Length

The flow velocity shall not exceed 0.5 m/s along a swale of 60 m length during the water quality design storm. Swales are recommended to have a minimum length of 60 m, which can be achieved by specifying a wide-radius curved path where the available land is not adequate for a linear swale (Sharp bends should be avoided to reduce erosion and to provide for erosion protection). If a shorter length must be used, designers should increase the swale cross sectional area by an amount proportional to the reduction in length below 60 m, to obtain the same water residence time. The recommended minimum residence time is 2 minutes.

14.3.5.11 Safety

Public safety should be maintained by providing sufficient conveyance capacity and appropriate flow depth and velocity to satisfy design requirements for adjacent pedestrian access; the safety of children and adults wading in the swale should be considered.

14.3.5.12 Erosion Protection

Once the maximum flow depth calculations have been performed and safety has been considered, erosion protection calculations should be performed. For any design storm event, the flow velocities are required to be less than 2 m/s, although higher velocities may be allowed if erosion protection is provided.

14.3.6 Design Procedure

The following design steps are recommended when designing grassed swales:

Step 1: Estimate the design discharge, Q_{minor} based on the design ARI for conveyance within the swale based on the site specific characteristics such as catchment area, topography and impervious area.

Step 2: Adopt a trial swale cross section. Calculate the swale geometry for either a triangular, trapezoidal or parabolic swale shape.

Step 3: Determine the Manning's roughness coefficient (n) value based on the type of swale vegetation, longitudinal slope (S) based on-site conditions and average depth of flow for the design flow rate (Q_{minor}).

Step 4: Calculate a peak flow (Q) and determine the flow depth, velocity and flow rate for the trial swale using Manning's equation.

Step 5: Compare Q with the Q_{minor}. Perform iterations through Steps 2 to 4 by changing flow depth, D until $Q \geq Q_{minor}$.

Step 6: Check the minimum swale length (L)

Step 7: Check the flow velocity for the design flowrate. If this is greater than 2.0 m/s, increase the flow width or reduce the depth of flow. Allow higher velocities if erosion protection is provided.

Step 8: Evaluate water quality parameters. The water quality flow depth should be a maximum of 150 mm. Adjust the swale geometry and re-evaluate as needed.

Step 9: Evaluate the swale geometry for the water quality design storm (40 mm rainfall depth over the contributing catchment), peak discharge velocity and capacity.

Step 10: Check that the flow velocity does not exceed 0.5 m/s along the swale during the water quality design storm for the design flow rate.

Step 11: Establish a construction sequence and construction specifications.

Step 12: Establish maintenance and inspection requirements.

REFERENCES

1. Chow V.T. (1959). *Open Channel Hydraulics.* McGraw-Hill Book Company, New York, USA.
2. Construction Industry Research and Information Association or CIRIA. (2007). *The SUDS Manual*. CIRIA Report C697. London, UK.

APPENDIX 14.A EXAMPLE – LINED DRAIN

Problem:

Determine the size of a lined rectangular drain to convey a 5-year ARI minor system design flow from a proposed 5 hectare bungalow development in Kuala Lumpur. The post-development time of concentration, t_c at the development outlet is estimated to be 20 minutes.

Solution:

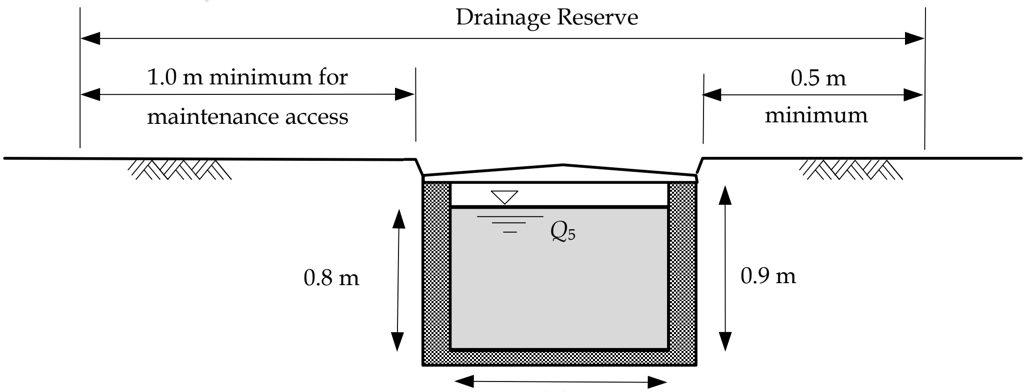
<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>Determine design flows for the drain:</td><td></td></tr>
<tr><td>Equation 2.2</td><td>$i=\frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$</td><td></td></tr>
<tr><td>Table 1.1</td><td>where:
i = the average rainfall intensity (mm/hr) for selected ARI (T) and storm duration (d);
T = average recurrence interval, ARI (years); =
d = storm duration (hours); $0.20 \le d \le 72$and =
λ, κ, θ and η = fitting constants dependent on the raingauge location</td><td>5-year ARI
20 minutes</td></tr>
<tr><td>Appendix 2.B
Table 2.B2</td><td>
<table>
<tr><th rowspan="2">Location & Station ID</th><th>ARI, T</th><th>Storm duration</th><th colspan="4">Derived Parameters</th></tr>
<tr><th>(years)</th><th>d</th><th>λ</th><th>κ</th><th>θ</th><th>η</th></tr>
<tr><td>Puchong Drop, KL (3015001)</td><td>5</td><td>0</td><td>69.650</td><td>0.151</td><td>0.223</td><td>0.880</td></tr>
</table>
</td><td></td></tr>
<tr><td></td><td>$i=\frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}=\frac{69.650(5)^{0.151}}{\left(\left(\frac{20}{60}\right)+0.223\right)^{0.880}}$ =</td><td>148.79 mm/h</td></tr>
<tr><td>Equation 2.3</td><td>$Q=\frac{C.I.A}{360}$</td><td></td></tr>
<tr><td>Table 2.6</td><td>where: Q = peak flow (m^3/s);
C = dimensionless runoff coefficient; =
I = average rainfall intensity over time of concentration, t_c (mm/hr); and =
A = drainage area (ha). =</td><td>0.65
148.79 mm/h
5 ha</td></tr>
<tr><td></td><td>$Q_5=\frac{0.65 \times 148.79 \times 5}{360}$ =</td><td>1.34 m^3/s</td></tr>
<tr><td></td><td>Calculate size of lined drain section</td><td></td></tr>
<tr><td>Table 2.3</td><td>Manning's n for concrete lining =</td><td>0.015</td></tr>
<tr><td></td><td>Assuming:
Drain longitudinal slope =
Width, B =
Side slope, Z =</td><td>0.4% (1 in 250)
0.90 m
0</td></tr>
</table>

Reference	Calculation	Output
	$Q = v \ x \ A$ where : $v = \frac{1}{n} S_o^{1/2} R^{2/3}$; $A = (B + ZD)D = 0.90D$ $P = B + 2D\sqrt{1+Z^2} = 0.90 + 2D$	

D (m)	A (m^2)	P (m)	R (m)	v m/s	Q (m^3/s)
0.60	0.540	2.10	0.26	1.70	0.921
0.70	0.630	2.30	0.27	1.78	1.120
0.80	0.720	2.50	0.29	1.84	1.324
0.85	**0.765**	**2.60**	**0.29**	**1.87**	**1.427**

Check (0.85) — OK (1.427) — Output: (>Q_5=1.34m^3/s)

Reference	Calculation		Output
Section 14.2.4.2	Allowing a minimum freeboard	=	50 mm
	Hence, required depth, $D_{minimum}$	=	0.85 m
	Provided dimension of lined drain (Refer to precast concrete manufacturers):	=	0.90 m
	Width, B	=	0.90 m
	Depth, D	=	1.89 m/s
	Velocity, v	=	1.530 m^3/s
	Discharge, Q		
	The drain dimensions are 0.90 m wide x 0.90 m deep, which is within the recommended limits.		
Section 14.2.4.3	Check flow velocity for lined drain	=	1.89 m/s (0.6<v<2 m/s); ...OK
Section 14.2.4.1	Depth between 0.6 m and 1.2m, lined drain shall be covered.		
	Service opening shall be provided along open drain with maximum interval spacing at every 100 m		



Proposed Lined Drain Design Dimensions

APPENDIX 14.B EXAMPLE – COMPOSITE DRAIN

Problem:

Determine the size of a composite drain to convey a 5-year ARI minor system design flow from a proposed 2 hectare bungalow development in Kuala Lumpur. The post-development time of concentration, t_c at the development outlet is estimated to be 25 minutes.

Solution:

<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>Determine design flows for the composite drain:</td><td></td></tr>
<tr><td>Equation 2.2</td><td>$i = \frac{\lambda T^{\kappa}}{(d + \theta)^{\eta}}$</td><td></td></tr>
<tr><td></td><td>where:
i = the average rainfall intensity (mm/hr) for selected ARI (T) and storm duration (d);</td><td></td></tr>
<tr><td>Table 1.1</td><td>T = average recurrence interval, ARI (years);
d = storm duration (hours); $0.20 \le d \le 72$; and
λ, κ, θ and η = fitting constants dependent on the raingauge location.</td><td>= 5-year ARI
= 25 minutes</td></tr>
<tr><td>Appendix 2.B
Table 2.B2</td><td>
<table>
<tr><th rowspan="2">Location & Station ID</th><th>ARI, T</th><th>Storm duration</th><th colspan="4">Derived Parameters</th></tr>
<tr><th>(years)</th><th>d</th><th>λ</th><th>κ</th><th>θ</th><th>η</th></tr>
<tr><td>Puchong Drop, KL (3015001)</td><td>5</td><td>25</td><td>69.650</td><td>0.151</td><td>0.223</td><td>0.880</td></tr>
</table>
</td><td></td></tr>
<tr><td></td><td>$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} = \frac{69.650(5)^{0.151}}{\left(\left(\frac{25}{60}\right)+0.223\right)^{0.880}}$</td><td>= 131.59 mm/h</td></tr>
<tr><td>Equation 2.3</td><td>$Q = \frac{C.I.A}{360}$</td><td></td></tr>
<tr><td>Table 2.6</td><td>where: Q = peak flow (m^3/s);
C = dimensionless runoff coefficient;
I = average rainfall intensity over time of concentration, t_c (mm/hr); and
A = drainage area (ha).</td><td>
= 0.65
= 131.59 mm/h
= 2 ha</td></tr>
<tr><td></td><td>$Q_5 = \frac{0.65 \, x \, 131.59 x \, 2}{360}$</td><td>= 0.475 m^3/s</td></tr>
<tr><td>14.2.4.1</td><td>For dry-weather base flows and minor flows up to a 10 mm rainfall depth,</td><td></td></tr>
<tr><td></td><td>Assume storm duration</td><td>= 25 minutes</td></tr>
<tr><td></td><td>Hence, the design flows are:</td><td></td></tr>
<tr><td></td><td>$Q_{dry-weather} = \frac{0.65 \, x \left(\frac{10}{25/60}\right) x2}{360}$</td><td>= 0.087 m^3/s</td></tr>
</table>

Reference | Calculation | Output

Calculate size of lined drain section (dry-weather base flows):

Table 2.3

Manning's n for concrete lining = 0.015

Assuming:

Drain longitudinal slope = 0.5% (1 in 200)
Drain width, *B* = 0.60 m
Side slope, *Z* = 0

Q= *v x A*

Where :

$$v = \frac{1}{n} S_o^{1/2} R^{2/3};$$

$$A = (B+ZD)D = 0.60D$$

$$P = B + 2D\sqrt{1+Z^2} = 0.60 + 2D$$

D (m)	*A* (m²)	*P* (m)	*R* (m)	*v* m/s	*Q* (m³/s)
0.15	0.090	0.90	0.10	1.02	0.091
0.20	**0.120**	**1.00**	**0.12**	**1.15**	**0.138**
0.25	0.150	1.10	0.14	1.25	0.187
0.30	0.180	1.20	0.15	1.33	0.240

(>0.087 m³/s)

Provided dimension of lined drain:
Drain width, *B* = 0.60 m
Depth, *D* = 0.20 m
Velocity, *v* = 1.15 m/s
Discharge, *Q* = 0.138 m³/s

The drain dimensions are 0.60 m wide x 0.20 m deep, which is within the recommended limits.

Section 14.2.4.3

Check flow velocity for lined drain. (0.6<v<2 m/s); …OK

Calculate size of total drain section:

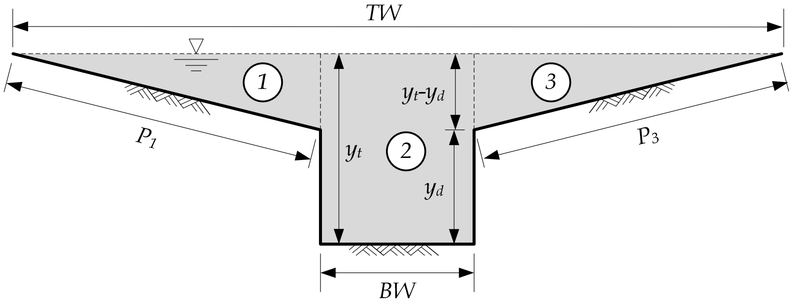
Table 2.3

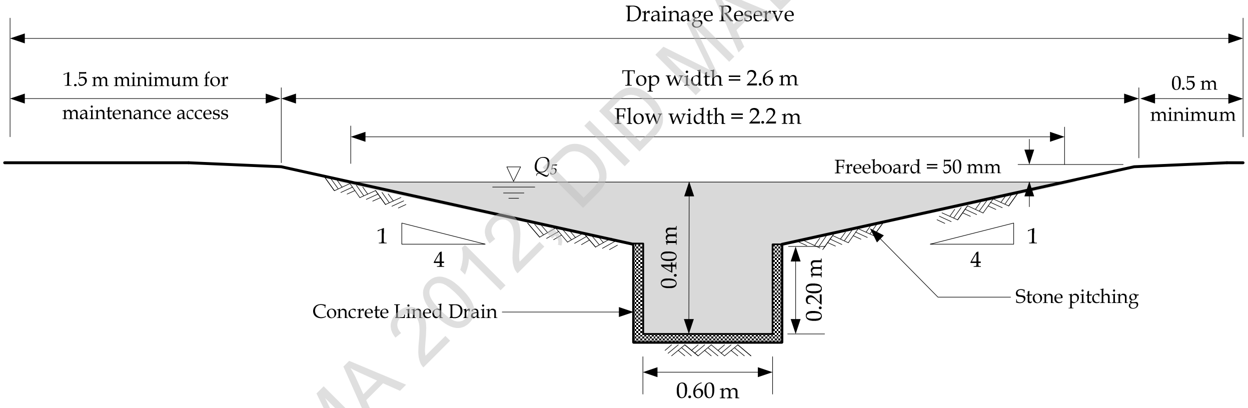
Manning's n for concrete lining = 0.015
Manning's n for stone pitching = 0.035

y_t (m)	*TW* (m)	*BW* (m)	y_d (m)	y_t -y_d (m)	$A_{1\&3}$ (m²)	$P_{1\&3}$ (m)	$R_{1\&3}$	$V_{1\&3}$ (m/s)	$Q_{1\&3}$ (m³/s)
0.50	3.0	0.6	0.2	0.30	0.36	2.47	0.146	0.559	0.201
0.45	2.6	0.6	0.2	0.25	0.25	2.06	0.121	0.495	0.124
0.40	**2.2**	**0.6**	**0.2**	**0.20**	**0.16**	**1.65**	**0.097**	**0.427**	**0.068**

y_t (m)	*TW* (m)	*BW* (m)	A_2 (m²)	P_2 (m)	R_2	V_2 (m/s)	Q_2 (m³/s)	Q_{total} = $Q_{1\&3}$ + Q_2 (m³/s)
0.50	3.0	0.6	0.30	1.00	0.300	2.113	0.634	0.835
0.45	2.6	0.6	0.27	1.00	0.270	1.969	0.532	0.655
0.40	**2.2**	**0.6**	**0.24**	**1.00**	**0.240**	**1.821**	**0.437**	**0.505**

(>Q_5 = 0.475 m³/s); …OK

<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>


Flow Segments for Calculating Composite Roughness</td><td></td></tr>
<tr><td>Section 14.2.4.2</td><td>Allowing a minimum freeboard =</td><td>50 mm</td></tr>
<tr><td></td><td>Provided dimension of composite drain:</td><td></td></tr>
<tr><td></td><td>Top width, <i>TW</i> =</td><td>2.6m</td></tr>
<tr><td></td><td>Bottom width, <i>BW</i> =</td><td>0.6 m</td></tr>
<tr><td></td><td>Depth, y_t =</td><td>0.45 m</td></tr>
<tr><td></td><td>Discharge, <i>Q</i> =</td><td>0.655 m^3/s (>Q_5); …OK</td></tr>
<tr><td colspan="3">


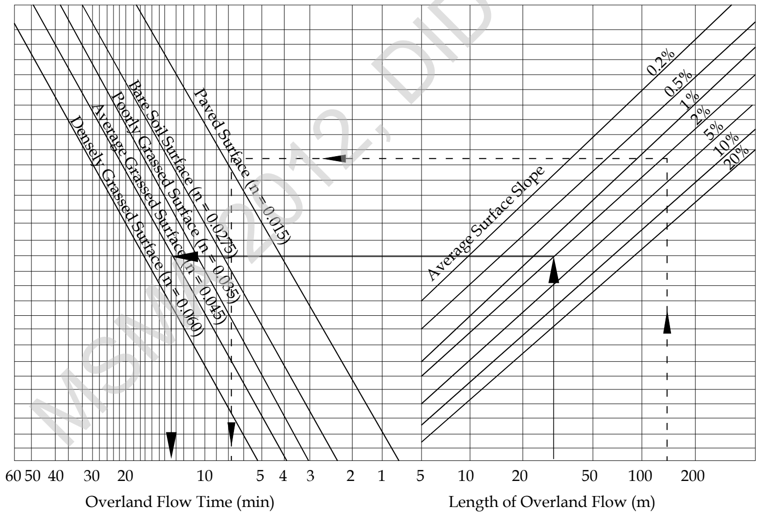
Composite Drain Design Dimensions</td></tr>
</table>

APPENDIX 14.C EXAMPLE – SWALES

Problem:

Determine the size of a grassed swale to convey a 10-year ARI minor system design flow from the project "Cadangan Mendirikan Sebuah Klinik Kesihatan Taiping – 2 Tingkat (Jenis KK2), Daerah Larut & Matang, Perak Darul Ridzuan".

Solution:

Reference	Calculation	Output
Relevant Layout Plan	Development project area: =	2.51 ha
	Sub-catchment area, A =	0.2325 ha
	Type of landuse =	Open Spaces Grass
	Estimate time of concentration, t_c:	
Table 2.1	Overland flow time, $t_o = (107.n.L^{1/3}) / S^{1/2}$ =	1.1 minutes
	Where: t_o = Overland sheet flow travel time (minutes);	
	L = Overland sheet flow path length (m); =	34.4 m
	n = Manning's roughness value for the surface; and =	0.045
	S = Slope of overland surface (%). =	2 %
Design Chart 2.A1		
	Time of travel over surface, t_o =	12.5 minutes
	Say, t_o =	12.5 minutes
Table 1.1	Design of average recurrence interval (ARI) =	10-year ARI
Equation 2.2	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$	
	where:	
	i = the average rainfall intensity (mm/hr) for selected ARI (T) and storm duration (d);	
Table 1.1	T = average recurrence interval, ARI (years); =	5-year ARI
	d = storm duration (hours); $0.20 \leq d \leq 72$; and =	12.5 minutes
	λ, κ, θ and η = fitting constants dependent on the rain gauge location	

Reference	Calculation	Output

Appendix 2.B
Table 2.B2

Location & Station ID	ARI, *T*	Storm duration	Derived Parameters			
	(years)	*d*	λ	κ	θ	η
Bukit Larut, Taiping (4511111)	10	12.50	87.236	0.165	0.258	0.842

$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} = \frac{87.236(5)^{0.165}}{\left(\left(\frac{12.50}{60}\right)+0.258\right)^{0.842}}$$ = 242.47 mm/h

Runoff Estimation:

Equation 2.3 — Design Flow, $Q = \frac{C.I.A}{360}$

Where: *Q* = peak flow (m³/s);

Table 2.6 — *C* = runoff coefficient; = 0.40

I = average rainfall intensity over time of concentration, t_c (mm/hr); and = 242.47 mm/hr

A = drainage area (ha). = 0.2325 ha

Design Flow, $Q_{10} = \frac{0.40 \text{x} 242.47 \text{x} 0.2325}{360}$ = 0.063 m³/s

Sizing for Swales:

Swale cross-section = Trapezoidal

Table 2.3 — Manning's roughness coefficient, *n* = 0.035

Section 14.3.5.2 — Swale longitudinal slope, *S* = 0.2%

Side slope, *Z* = 3

Depth, *D* (m)	Bottom width, *BW* (m)	Side slope, *Z* (m)	Top width, *TW* (m)	Area, *A* (m²)	Wet. Perimeter, *P* (m)	Hydraulic radius, *R* (m)	Velocity, *v* (m/s)	Discharge, *Q* (m³/s)
0.10	0.60	3	1.20	0.09	1.23	0.07	0.22	0.020
0.20	**0.60**	**3**	**1.80**	**0.24**	**1.86**	**0.13**	**0.32**	**0.078**
0.25	0.60	3	2.10	0.34	2.18	0.15	0.37	0.124
0.30	0.60	3	2.40	0.45	2.50	0.18	0.41	0.182
0.40	0.60	3	3.00	0.72	3.13	0.23	0.48	0.344

(>0.063 m³/s)

Section 14.2.4.2 — Provide a minimum freeboard = 50 mm

Hence, required swale depth, *D* = 0.25 m

Provided dimension of swale:
Bottom width, *BW* = 0.6 m
Top width, *TW* = 1.80 m
Velocity, *v* = 0.37 m/s
Discharge, *Q* = 0.124 m³/s

Reference	Calculation	Output
Section 14.3.5.5	Check flow velocity for swale =	(<2 m/s); …OK
	Water Quality Storm Design:	
	For water quality storm up to a 3-month ARI flow,	

Appendix 2.B Table 2.B2

Location & Station ID	ARI, *T* (years)	Storm duration *d*	Derived Parameters			
			λ	κ	θ	η
Bukit Larut, Taiping (4511111)	0.25	12.50	83.3964	0.3189	0.1767	0.8166

$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} = \frac{87.236(5)^{0.165}}{\left(\left(\frac{12.50}{60}\right)+0.258\right)^{0.842}}$$
= 116.85 mm/h

Hence, the design flows are:

$$Q_{water\ quality} = \frac{0.40\text{x}116.85\text{x}0.2325}{360}$$
= 0.030 m³/s

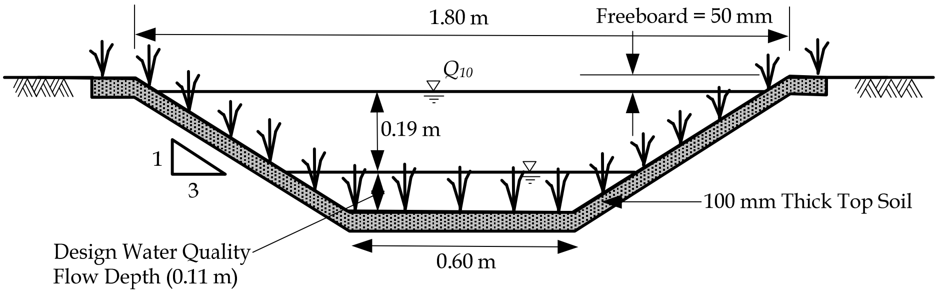
Calculate flow depth and velocity for the water quality flow:

Reference	Calculation	Output
Table 14.5	Swale cross-section =	Trapezoidal
	Manning's roughness coefficient, *n* =	0.035

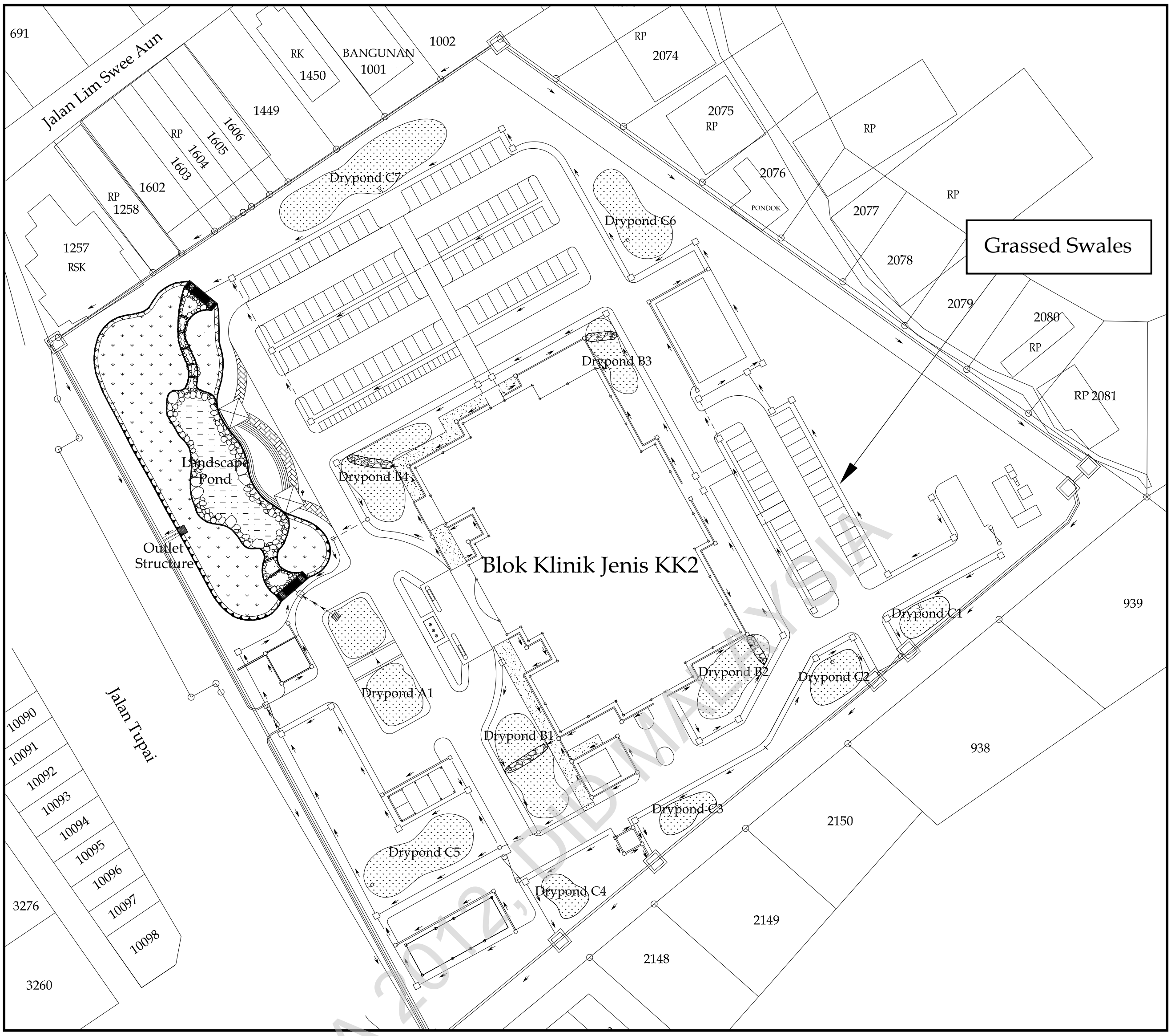
Depth, *D* (m)	Bottom width, *BW* (m)	Side slope, *Z* (m)	Top Width, *TW* (m)	Area, *A* (m²)	Wet. perimeter, *P* (m)	Hydraulic radius, *R* (m)	Velocity, *v* (m/s)	Discharge, *Q* (m³/s)
0.11	0.60	3	1.26	0.10	1.30	0.08	0.23	0.024
0.12	0.60	3	1.32	0.12	1.36	0.08	0.24	0.028
0.13	**0.60**	**3**	**1.38**	**0.13**	**1.42**	**0.09**	**0.26**	**0.033**
0.14	0.60	3	1.44	0.14	1.49	0.10	0.27	0.038

(> 0.030 m³/s)

Reference	Calculation	Output
Section 14.3.5.10	Check water quality flow depth; *D* =	0.13 m (≤ 150 mm); …OK
	Check velocity for water quality flow depth; *v* =	0.26 m/s (< 0.5 m/s); …OK

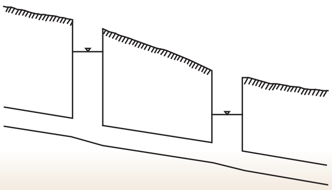


Grassed Swale Design Dimensions



Layout Plan

Examples of Grassed Swales at Project Area



PIPE DRAINS 15

CHAPTER 15 PIPE DRAINS

15.1 INTRODUCTION

This Chapter provides an introduction and outline of the design requirement and procedures for pipe drain, a subsurface conduit for conveying minor storm flows from impervious surfaces, such as streets, parking lots, buildings, etc. to major stormwater facilities within an urban catchment.

Pipe drain systems are recommended mainly for high density residential and commercial/industrial developments where the use of open drain and swales is not feasible and uneconomical. Since other services are involved, requirement for the locations and alignments of pipe drains will also be provided in the design procedure.

The pipes, differing in lengths and sizes, are connected by appurtenant structures below ground surface (Figure 15.1) using pits, junctions or manholes and other miscellaneous structures such as transitions, bends and branches.

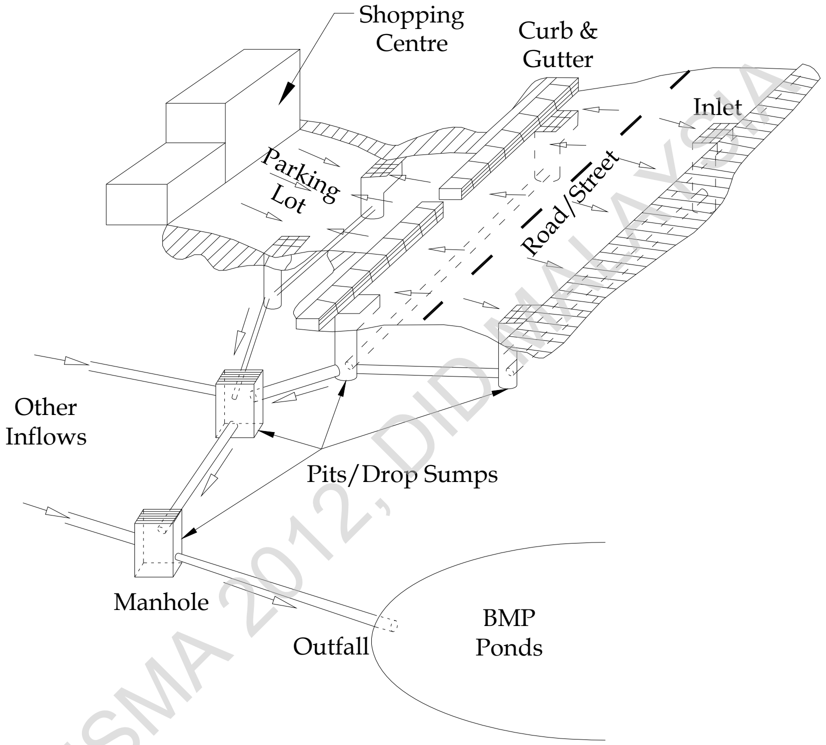


Figure 15.1: Elements of Pipe Drainage System

15.2 LOCATION AND ALIGNMENT

Standardised locations for stormwater pipelines are provided to limit the negotiations needed when other services are involved and permit ready location by maintenance crews.

15.2.1 Roadway Reserves.

Table 15.1 provides typical requirements for location of pipe drains and services within road reserves, however these may be varied for different authorities. The relevant authority should be consulted concerning its standard alignments for services.

In selecting pipeline locations, it is necessary to also consider inlet location preferences as outlined in Chapter 13.

Table 15.1: Alignments within Roadway Reserves

Pipe Diameter (mm)	Alignment
375 to 675	under curb line
750 to 1800	within median strip, or centreline of road

15.2.2 Privately Owned Properties

Stormwater pipelines are often constructed in parallel to sewers and as the sewerage system is usually deeper, pipes connecting to stormwater ties have less problems in crossing over the sewer.

Alignments shall offset with sufficient distance from building lines to allow working space for excavation equipment.

Acceptable centreline offset alignments from property boundaries in residential, commercial, and industrial areas shall be in accordance with Table 15.2.

Table 15.2: Offset Distance within Privately Owned Properties

Pipe Diameter (mm)	Rear Boundary	Side Boundary
375 to 450	1.8m	1.2m (see Note)
525 to 675	1.8m	1.5m (see Note)

Note: Where other hydraulic services or power poles are located on the same side of a property boundary, the centreline of the stormwater pipeline shall be located 1.8m from the property boundary.

15.2.3 Public Open Space

The location of stormwater pipelines within public land such as open space shall be brought to the attention of the operating Authority for consideration. As a guide, unless directed otherwise, stormwater pipelines shall be located not less than 3 m from the nearest property boundary.

15.2.4 Drainage Reserves

A drainage reserve shall be wide enough to contain the service and provide working space on each side of the service for future maintenance activities. Minimum drainage reserve widths shall be in accordance with Table 15.3.

Pipelines up to and including 675mm diameter may be located within privately owned properties if satisfactory arrangements are made for permanent access and maintenance. Larger diameter pipelines shall be located within public open space or outside privately owned properties in separate drainage reserves.

Consideration should be given to the multi-purpose use of drainage reserves such as open space or pedestrian corridors.

Table 15.3: Minimum Drainage Reserve Widths

Pipe Diameter, *D* (mm)	Minimum Reserve Width (m)
Invert < 3.0 m deep	
375 to 450	2.5
525 to 675	3.0
750 to 900	3.5
1050 to 1200	3.5
1350 to 1800	not less than 3 x *D*
Invert 3.0 - 6.0 m deep	
375 to 450	3.5
525 to 675	4.0
750 to 900	4.5
1050 to 1800	not less than 4 x *D*

Note: Where other hydraulic services or electricity services are laid on the same side of the property boundary, the required reserve width shall be increased by 500 mm to provide horizontal clearance between services.

15.2.5 Clearance from Other Services

Minimum clearances between stormwater pipelines and other services shall be in accordance with Table 15.4. The nominated clearance should make due allowance for pipe collars and fittings. Special protection may be provided to protect service crossings by concrete encasing the stormwater pipe for sufficient length to bridge the trench of the other service.

Table 15.4: Minimum Clearances

Service	Clearance (mm)
Horizontal	
All services	600
Vertical	
Sewers	150
Water Mains	75
Telephone	75
High Pressure Gas	300
Low Pressure Gas	75
High Voltage Electricity	300
Low Voltage Electricity	75

15.3 HYDRAULICS FUNDAMENTALS

15.3.1 Flow Type Assumptions

The design procedures presented in this chapter assume that flow within pipe drain is steady and uniform. The discharge and flow depth in each pipe segment are therefore assumed to be constant with respect to time and distance. For prismatic conduits, the average velocity throughout a segment is considered to be constant.

In actual storm water drainage systems, the flow entering at each pit is variable, and flow conditions are not steady or uniform. However, since the usual hydrologic methods employed in storm water drain design are based on computed peak discharges at the beginning of each segment, it is generally a conservative practice to design using the steady uniform flow assumption.

Two design philosophies exist for sizing storm water drains under steady uniform flow assumption. The first is referred to as open channel or gravity flow design. The pipe segments are sized so that the water surface within the conduit remains open to atmospheric pressure and the flow depth throughout the conduit is less than the height of the conduit. The second is the pressure flow design where the flow in the conduit is at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within conduit. The pressure head will be above the top of the conduit and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the Hydraulic Grade Line (HGL).

For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow but it provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. However, in situation where there is adequate headroom between the conduit and pit/junction elevations to tolerate pressure flow, significant cost savings may be realised.

15.3.2 Hydraulic Capacity

The flowrate through a storm water pipe depends on the upstream and downstream water levels, as well as the pipe characteristics. Several flow friction formulas, however, have been advanced which define the relationship between flow capacity and the pipe characteristics (size, shape, slope and friction resistance). The most common equations are from the Manning formula and Colebrook-White formula. For circular storm water drains flowing full, Manning formula is reduced to represent the followings:

$$V = (0.397/n)\ D^{0.67}\ S_o^{0.5} \qquad (15.1a)$$

$$Q = (0.312/n)\ D^{2.67}\ S_o^{0.5} \qquad (15.1b)$$

$$D = [(Q\ n/(0.312\ S_o^{\ 0.5})]^{0.375} \qquad (15.1c)$$

where,

V = Mean velocity (m/s);
Q = Flow rate (m^3/s);
D = Circular pipe diameter (m);
S_o = Slope of HGL; and
n = Manning coefficient.

The Colebrook-White equation, representing the flow mean velocity is

$$V = -\ 0.87[\sqrt{(2g.D.S)}]\ \log_e\ [(k/3.7D) + (2.51\upsilon/D\sqrt{(2g.D.S)}] \qquad (15.2)$$

where,

S = Longitudinal slope of pipe (m/m);
k = Pipe roughness height;
D = Circular pipe diameter (m); and
υ = Kinematic viscosity of water (m^2/s)

Chart 15.A1 and Chart 15.A2 in Appendix 15.A can be used respectively for solution of Manning formula and Colebrook-White formula for flow in circular conduits. The Hydraulic Elements graph Chart 15.A4 can be used to assist in the solution of the Manning’s equation for part full flow in storm water drains. Tables for the hydraulic design of pipe, sewer and channel are also provided (Wallingford and Barr, 2006).

15.3.3 The Energy and Hydraulic Grade Lines

The energy grade line (EGL) is a line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

The hydraulic grade line (HGL) is a line coinciding with the level of water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The HGL is used in determining the acceptability of a proposed storm water drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. The HGL can be determined by subtracting the velocity head from the EGL, as shown in Figure 15.2.

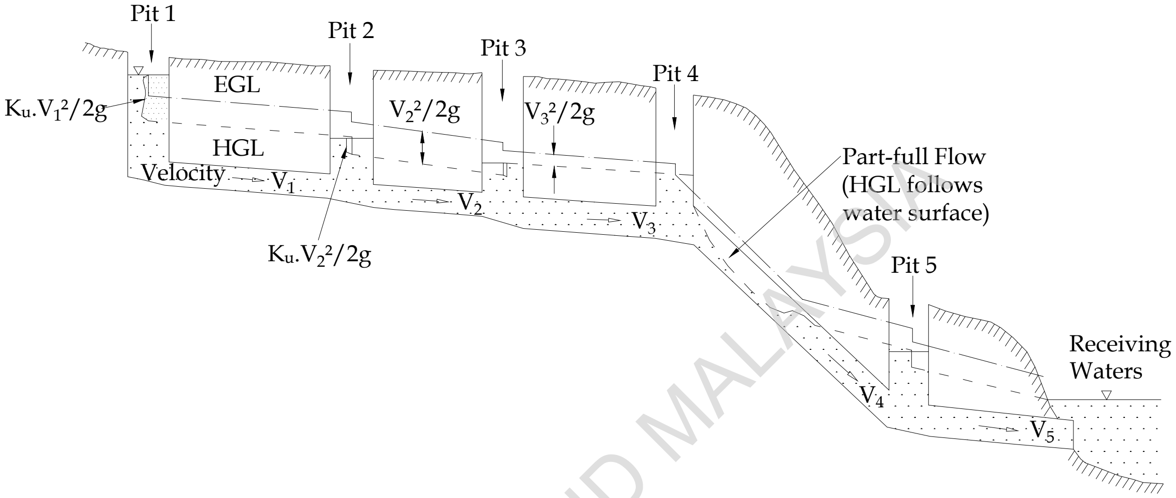


Figure 15.2: Hydraulic Grade Line (HGL) and Energy Grade Line (EGL) of Pipe Drainage
(Inst. Engrs., Aust., 1977)

Pressure head is normally lost in both pipes and pits due to friction and turbulence and the form of the HGL is therefore a series of downward sloping lines over line lengths, with steeper or vertical drops at manholes. In some circumstances there may be a pressure gain and therefore a rise in the HGL at a structure. In these cases the gain should be taken into account in the hydraulic calculations.

When the pipe is not flowing full, such flow is considered as open channel flow and the HGL is at the water surface. When pipe is flowing full under pressure flow, the HGL will be above the obvert of the pipe. Inlet/pit overflow can occur if the HGL rises above the ground surface.

Pipes which flow under pressure may be located at any grade and at any depth below the HGL without altering the velocity and flow in the pipe subject to the grade limitations. Hence, pipe grade may be flattened to provide cover under roads, or clearance under other services, without sacrificing flow capacity, provided sufficient head is available.

The HGL and the Water Surface Elevation (WSE) must be below the surface level at manholes and inlets, or the system will overflow. This depth below the surface is termed the *freeboard*. Minimum freeboard requirements are specified in the design criteria in Section 15.3.5 and 15.4 The level of the HGL for the design storm applied should be calculated at the upstream and downstream side of every inlet or manhole, at points along a pipe reach where obstructions, penetrations or bends occur, or where a branch joints.

It is recommended that designers check that the elevation of the total energy line falls progressively as flow passes down through the drainage system. This is an important check that should be undertaken where the drainage system is complex and where the configuration of pipes and structures does not conform to the structure loss charts available.

15.3.4 Energy Losses

Prior to computing the HGL, all the energy losses in pipe sections and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit section, energy or head is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions and manholes.

15.3.4.1 Friction Losses

The head loss due to friction in pipes is computed by the relationship Equation 15.3a and for full flow pipe it is given by Equation 15.3b derived from Manning's formula.

$$h_f = S_f L \tag{15.3a}$$

$$Sf = h_f / L = (Qn/0.312\ D^{2.67})^2\ L \tag{15.3b}$$

where,

h_f = Head loss in pipe due to friction (m);
n = Manning's roughness coefficient;
L = Length of the pipe (m);
S_f = Slope of HGL;
D = Diameter of the pipe (m); and
Q = Discharge in the pipe (m^3/s).

The Manning's roughness coefficients *n* and roughness height *k* for some pipe material are given in Table 15.5.

Table 15.5: Pipe Roughness Values

Pipe Material	*n*	*k* (mm)
Cost iron – cement lined	0.011	0.3
Concrete (Monolithic) – Smooth forms	0.012	0.6
Concrete (Monolithic) – rough forms	0.015	0.6
Concrete pipe	0.011	0.3
Plastic Pipe (smooth)	0.011	0.3
Vitrified clay pipe	0.011	0.3

Further sources of information on roughness values can be found elsewhere (French, 1985 and Chow, 1959).

15.3.4.2 Structure Losses

Losses due to obstructions, bends or junctions in pipelines may be expressed as a function of the velocity of flow in the pipe immediately downstream of the obstruction, bend or junction as follows:

$$h_s = K\ V_o^2 / 2g \tag{15.4}$$

where,

h_s = Head loss at structure (m);
K = Pressure change coefficient (dimensionless); and
V_0 = Velocity of flow in the downstream pipe (m/s).

Pressure change coefficients K (sometimes referred to as structure loss coefficients) are dependent on many factors, for example:

- Junction, manhole and inlet structure geometry;
- Pipe diameters change;
- Bend radius;
- Angle of change of direction; and
- Relative diameter of obstruction etc.

(a) Losses at Junctions

At a junction with an access structure such as manhole or pit, the static head loss or pressure head change, which is the drop of the HGL at the junction can be calculated from,

$$H = K_u \, V_o^2/2g \quad (15.5)$$

where,

H = Pressure head change at junction (m);
K_u = Pressure change coefficient (dimensionless); and
V_o = Velocity at the outlet pipe.

The value of the head loss coefficients, K_u for a limited range of configurations are presented by Chart 15.A5 and Chart 15.A6, part of the 'Missouri Charts'. The main difficulty in presenting information on pit losses is the almost infinite number of configurations which can occur. The Missouri Charts (Sangster et al., 1958) or similar information such as the charts in the Queensland Urban Drainage Manual, 2008 are too voluminous to present in this manual and they should be consulted if a great precision calculation is required. The design Chart 15.A7(a) and Chart 15.A7(b) is an attempt to simplify the pit loss calculation procedure by Mills (O'Loughlin (2009)). The data is greatly simplified and therefore is not highly accurate, and it is not intended that the chart be used with great precision. Though it is not highly accurate, the charts provide realistic head loss coefficients for most cases. The input data $(d/D_o, D_u/D_o)$ can be determined approximately without the need for iteration to give reasonable value with minimum of effort. For greater precision, the loss coefficient calculated iteratively as layout by Flowchart 15.B1 in Appendix 15.B for design of small reticulation systems where the values obtained are seen not to be critical, the Equation 15.6 can be used to estimate pit loss coefficient K.

$$K = 0.5 + 2(Q_m/Q_o) + 4(Q_g/Q_o) \quad (15.6)$$

where,

Q_o = Outlet flow rate (m^3/s);
Q_g = Inflow above the water level (pipes are assumed to flow full) (m^3/s); and
Q_m = Misaligned inflows that enters below the pit water level (m^3/s).

Notes;

- Any pipe inflow that is aligned with the outlet within D/4 is assumed not to cause losses;
- Subtract 0.5 to the result of Equation 15.6 if (i) there are deflectors, or (ii) the outlet diameter is larger than the biggest inlet; and
- Add 1.0 for opposed inlet.

Large energy losses and pressure changes can be avoided by attention to simple details in the design and construction supervision of pits and manholes (AR & R, 1998). One principle is to ensure that jets of water emerging from incoming pipes are directed to outlet pipes, rather than impinging on pit walls.

HEC – 22 Manual (USFHWA, 2009) introduced its latest method for estimating losses in access holes (junctions) and inlets (pits) by classifying access holes and their hydraulic conditions in a manner analogous to inlet control and full flow for culverts. The FHWA access hole method follows three fundamental steps:

Step 1: Determine an initial access hole energy level based on inlet control (weir and orifice) or outlet control (partial and full flow) equations.

Step 2: Adjust the initial access hole energy level based on benching, inflow angle (s), and plunging flows to compute the final calculated energy level.

Step 3: Calculate the exit loss from each inflow pipe and estimate the energy gradeline, which will then be used to continue calculations upstream.

For detail account of the method, readers and designers are referred to the publication.

(b) Inlets and Outlets

Where the inlet structures is an endwall (with or without wingwalls) to a pipe or culvert, an allowance for head loss should be made. Table 15.A1 in Appendix 15.A provides loss coefficients K_e to be applied to the velocity head.

$$h_e = K_e V_o^2/2g \quad (15.7)$$

where,

h_e = Head loss at entry or exit (m);

K_e = Entry or exit loss coefficient; and

V_0 = Velocity in pipe (m/s).

(c) Bends

Under certain circumstances it may be permissible to deflect a pipeline (either at the joints or using precast mitred sections) to avoid the cost of junction structures and to satisfy functional requirements. Where pipelines are deflected an allowance for energy loss in the bends should be made. The energy loss is a function of the velocity head and may be expressed as:

$$h_b = K_b V_o^2/2g \quad (15.8)$$

where,

h_b = Head loss through bend (m); and
K_b = Bend loss coefficient = 0.0033 A, where A is the angle of curvature in degrees.

Values of bend loss coefficients for gradual and mitred bends are given in the Design Chart 15.A8 and Table 15.A2 respectively in Appendix 15.A.

(d) Obstructions or Penetrations

An obstruction or penetration in a pipeline may be caused by a transverse (or near transverse) crossing of the pipe by a service or conduit e.g. sewer or water supply. Where possible, such obstructions should be avoided as they are likely sources of blockage by debris and damage to the service. To facilitate the removal of debris, a manhole should be provided at the obstruction or penetration.

The pressure change coefficient K_P at the penetration is a function of the blockage ratio. Design Chart 15.A9 in Appendix 15.A should be used to derive the pressure change coefficient, which is then applied to the velocity head.

$$H = K_p\ V_0^2/2g \quad (15.9)$$

where,

h_p = Head loss at penetration (m); and

K_p = Pressure change coefficient of the penetration.

Where a junction is provided at an obstruction or penetration it is necessary to add the structure loss and the loss due to the obstruction or penetration based upon the velocity, *V* in the downstream pipe.

(e) Pipe Branch losses

A pipe branch is the connection of a lateral pipe to a larger trunk pipe without the use of junction (Figure 15.3). The loss for a pipe branch can be estimated by Equation 15.8. Alternatively the formula relating the pressure loss coefficients and velocity head at the branch lines may produce estimate of losses. Design Chart 15.A10 in Appendix 15.A provides the pressure loss coefficients.

$$H_j = \{[(Q_o\ V_o)-(Q_u\ V_u)-(Q_L\ V_L \cos\theta)]/[0.5g(A_o-A_u)]\} + h_u-h_o \qquad (15.10)$$

where:

H_j = Branch loss (m);
Q_o, Q_u,, Q_L = Outlet, inlet, and lateral flows respectively (m^3/s);
V_o, V_u, V_L = Outlet, inlet, and lateral velocities respectively (m/s);
h_o, h_u = Outlet and inlet velocity heads (m);
A_o,, A_u = Outlet and inlet cross sectional areas (m$^{2)}$; and
θ = Angle between the inflow trunk pipe and inflow lateral pipe.

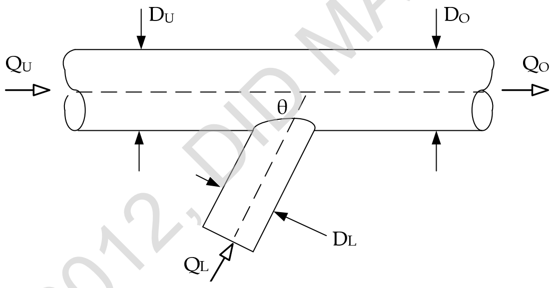


Figure 15.3: Branch /junction Line

(f) Transition losses (Expansion and Contractions)

Sudden expansions or contractions in stormwater pipelines should normally be avoided. They may however need to be installed as part of a temporary arrangement in a system being modified or upgraded, or in a relief drainage scheme.

Expansions and contractions also occur at the outlet and inlet, respectively, of stormwater pipelines The energy loss in expansions or contractions in non-pressure flow can be evaluated using Equation 15.11a for a sudden contraction and Equation 15.11b for sudden expansion,

$$H_c = C_u\ (V_2^2/2g - V_1^2/2g) \qquad (15.11a)$$

$$H_e = C_u\ (V_1^2/2g - V_2^2/2g) \qquad (15.11b)$$

where,

C_u = Expansion or contraction coefficient (Chart 15.A11, Appendix 15.A);
V_1, V_2 = Velocity upstream and downstream respectively (m/s); and
g = Acceleration due to gravity, 9.81 m^3/s.

The pressure change due to the expansion or contraction can be derived using the energy loss coefficients determined from the Design Charts 15.A11, in Appendix 15.A. The entrance loss coefficient should be applied to the absolute value of the difference between the two velocity heads.

15.3.5 Freeboard at Inlets and Junctions

For the design of underground drainage systems a freeboard should be provided above the calculated water surface elevation (WSE) to prevent surcharging and to ensure that unimpeded inflow can occur at gully inlets.

The maximum permitted WSE should allow for the head loss resulting from surface inflow through grates etc. into the structure being considered.

Where an appropriate chart is not available it is recommended that the WSE be arbitrarily adopted at the height above the calculated HGL (as described in Section 15.5) in accordance with Equation 15.12:

$$WSE - HGL = 0.3\ V_u^2/2g \tag{15.12}$$

where,

V_u = Upstream velocity (m/s).

The freeboard recommendations should be applied as detailed in Table 15.6.

Table 15.6: Minimum Freeboard Recommendations for Inlets and Junctions

Pit Types	Freeboard
Gully pit on Grade	0.15m below invert of curb and channel. (See Notes 1 and 2).
Gully pit in Sag	0.15m below invert of curb and channel. (See Note 1)
Field pit	0.15m below top of grate or lip of pit
Junction Structure (See Note 3)	0.15m below top of lid.

Notes:

1. Where the channel is depressed at a gully inlet the freeboard should be measured from the theoretical or projected invert of the channel.
2. Where an inlet is located on grade the freeboard should be measured at the centreline of the gully inlet chamber.
3. Where it is necessary for the HGL to be above the top of a manhole or junction structure, a bolt-down lid should be provided. This will, of course, prevent the use of the manhole as an inlet.

15.4 DESIGN CRITERIA

The design of pipe drainage system should conform to the following criteria:

- Pipe are designed by a 'Hydraulic Grade Line' (HGL) method using appropriate pipe friction loss and structure head loss;
- If the potential water surface elevation (in junction/pit) exceeds 0.15m below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line;
- The maximum hydraulic gradient should not produce a velocity that exceeds 6m/s and the minimum desirable pipe slope should be 1.0% to provide self cleansing and free from accumulation of silt;
- Minimum diameter for pipe draining a stormwater inlet and crossing a footpath alignment shall be no less than 300mm;
- To allow for passage of debris, the minimum diameter shall not be less than 381mm;

- For a non-self draining underpass, the pipe shall be sized for 10 year ARI and shall not be less than 450mm;
- The maximum pipe diameter to be used depends on the availability of pipe from manufacturers. Culverts or multiple pipes should be used in situation where large pipe interfering with clearance for other services;
- Curved stormwater pipelines are only permitted for diameters 1200mm and above; and
- Stormwater pipelines shall be designed for a minimum effective service life of 50 years.
- The most commonly used design ARI values for minor drainage systems are provided in Table 1.1.

The HGL is used to determine the acceptability of a proposed stormwater drainage system by establishing the elevation to which water level will rise when the system is operating under design conditions.

15.5 DESIGN PROCEDURE

The steps generally followed in the design of pipe drains are listed as follows:

Step 1: Prepare a tentative layout plan for the stormwater pipe drainage system which covers:

- Location of stormwater drains and inlets or pits;
- Direction of flow;
- Location of manholes or junction box; and
- Location of existing facilities such as water, gas, or underground cables.

The slope and the length of pipe segments are determined. The pipe lengths for each segment are entered in Column 2 in Table 15.C3 Hydraulic Design Sheet.

Step 2: Determine drainage areas, and by using the Rational Method the pit inlet capacities and the design discharges in each pipe in the system are established and entered into Column 3 in Table 15.C3.

Assume a trial pipe diameter and calculate the full pipe velocity as in Column 5 Table 15.C3. Calculate the friction loss in pipe segment and the pressure/head loss in the pit (Column 10 Table 15.C3).

The procedure of obtaining the pit inlet capacities have been addressed in Chapter 13 and the Rational Method have been described in Chapter 2.

Step 3: Carry out the HGL evaluation procedure (Section 15.6) for the system starting at a point where the HGL may be readily determined, either working upstream from the outlet or proceeding downstream from a starting permissible water level depending on the flow conditions of the system.

Step 4: Check whether the design criteria in Section 15.4 (minimum velocity/slope and freeboard) are complied with, and carry out the necessary adjustments.

Step 5: It is a good practice to test the adopted configuration and pipe sizing with discharge of higher ARI. The risk factor is therefore defined.

15.6 HGL CALCULATION PROCEDURE

A step-by- step procedure for manual calculation of the HGL using the energy loss method is presented in this section. Hydraulic Design Sheet Table 15.C3 in Appendix 15.C is used in the organisation of data and calculations. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe.

Columns 1 to 6 in Table 15.C3 present the basic design information, while Columns 7 to 16 are for calculation of HGL position. The remaining columns are used to determine pipe invert levels, allowing for hydraulic considerations, cover and positions of upstream pipes. Pipes slope are calculated to check for sedimentation problem. The table ends with remarks and actions taken.

Here the storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and the junction losses are summed to determine the upstream HGL levels or subtracted to determine the downstream HGL depending where we begin computation.

If supercritical flow occurs, pipe and junction losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. The process continues until the storm drain system returns to a subcritical flow regime (Hec-22, 2009). The flow is supercritical when the Froude Number of the flow is greater than one *(Fr>1)* or when the normal depth *(y_o)* of flow is less than its critical depth *(y_c)*. For illustration, the HGL computations begin at the outfall and are worked upstream taking each junction into consideration. The HGL at the outfall, in most cases are readily determined. Furthermore, the head losses in pits and junction are expressed as a function of the velocity in the downstream pipe.

The HGL computation for pipe flow is in accordance with the followings:

Step 1: Identify the tailwater elevation *(TW)* at the downstream storm drain outlet. If the outlet is submerged, the HGL will correspond to the water level at the outlet. If the outlet is not submerged, the HGL are computed as follows:

- If the TW at the pipe outlet is gr eater than $(d_c + D)/2$, the *TW* elevation taken as the HGL; and
- If the TW at the pipe outlet is less than $(d_c + D)/2$, use $(d_c + D)/2$ plus the invert elevation as HGL.

Where d_c is the critical depth determine from Chart 15.A3 for circular conduit.

Step 2: Calculate full flow velocity *(V)*, and hence the velocity head *($V^2/2g$)*.

Step 3: Compute the friction slope *(S_f)* for the pipe using Equation 15.2b.

Step 4: Compute the friction loss *(H_f)* by multiplying the length *(L)* by the friction slope.

Compute other losses along the pipe run such as bend losses *(h_b)*, transition Contraction *(H_c)* and expansion *(H_e)*, and junction losses *(H_j)* using Equations 15.5 to 15.11, or use Charts 15.A9 to 15.A11 in Appendix 15.A.

Step 5: Compute the EGL at the upstream end of the outlet pipe as the HGL plus the total pipe losses plus the velocity head.

Step 6: Estimate the depth of water in the access hole as the depth from the outlet pipe invert to HGL in pipe at outlet. It also can be computed as EGL minus the pipe velocity head minus the pipe invert.

Step 7: If the inflow storm drain invert is submerged by the water level in the access hole, compute the access hole losses using Equation 15.3 and 15.4. The value of pressure change coefficients *(K_u)* for various configurations can be obtained from Charts 15.A5 to 15.A7 or Equation 15.5. Iterative calculation may be required in high precision case to obtain the pressure change coefficient, since pressure loss depends on depth, Flowchart 15.B1 in Appendix 15.B.

If the inflow storm drain invert is not submerged by the water level in the access hole, the head in the manhole is computed using culvert techniques, depending whether the outflow pipe under outlet or inlet control. For outlet control, coefficient obtained from Table 15.A12 in Appendix 15.A.

Step 8: Compute the EGL at the structure by adding the structure losses to EGL at the upstream end of the outlet pipe (step 5).

Step 9: Compute the HGL at the structure by subtracting the velocity head from the EGL (Step 8).

Table 15.C3 in Appendix 15.C summarised the calculation steps.

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APPENDIX 15.A DESIGN CHART, NOMOGRAPHS AND TABLES

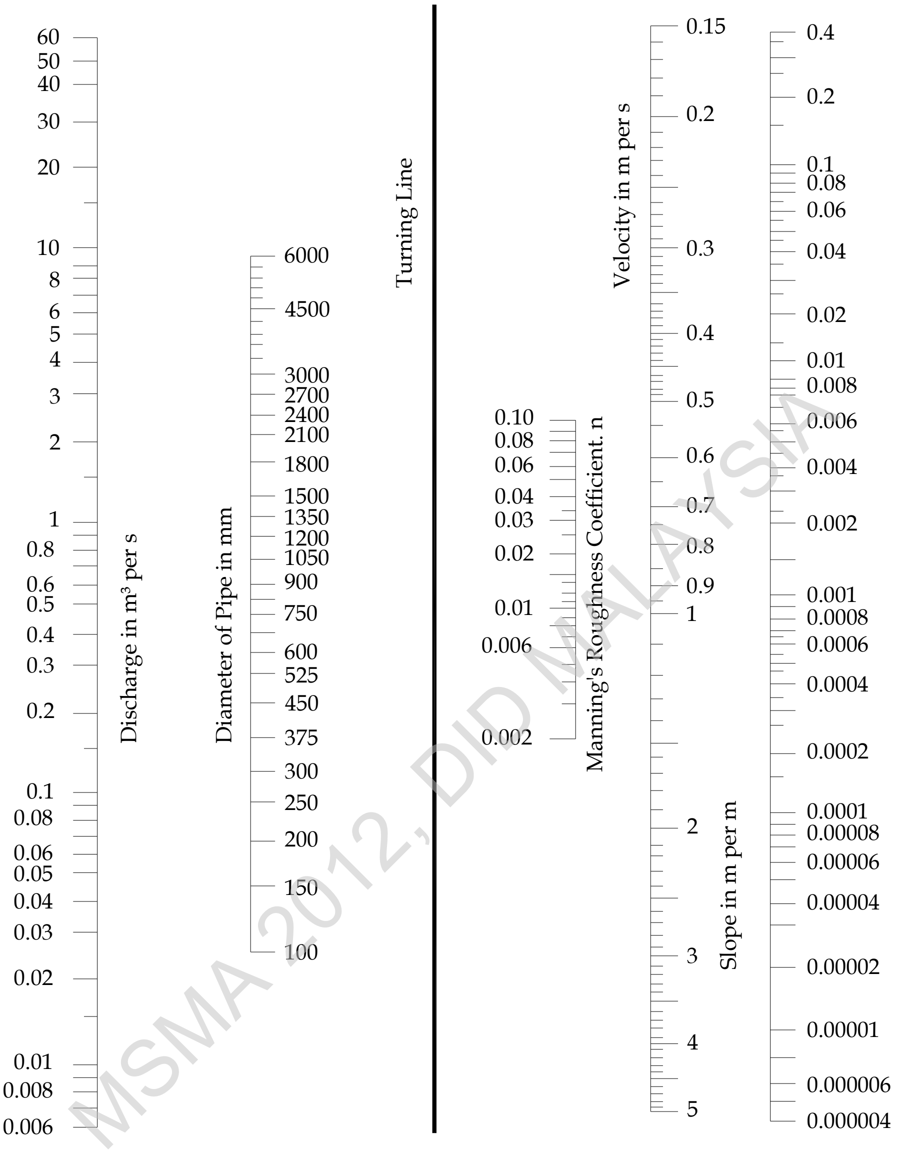


Chart 15.A1: Hydraulic Design of Pipe- Manning Formula

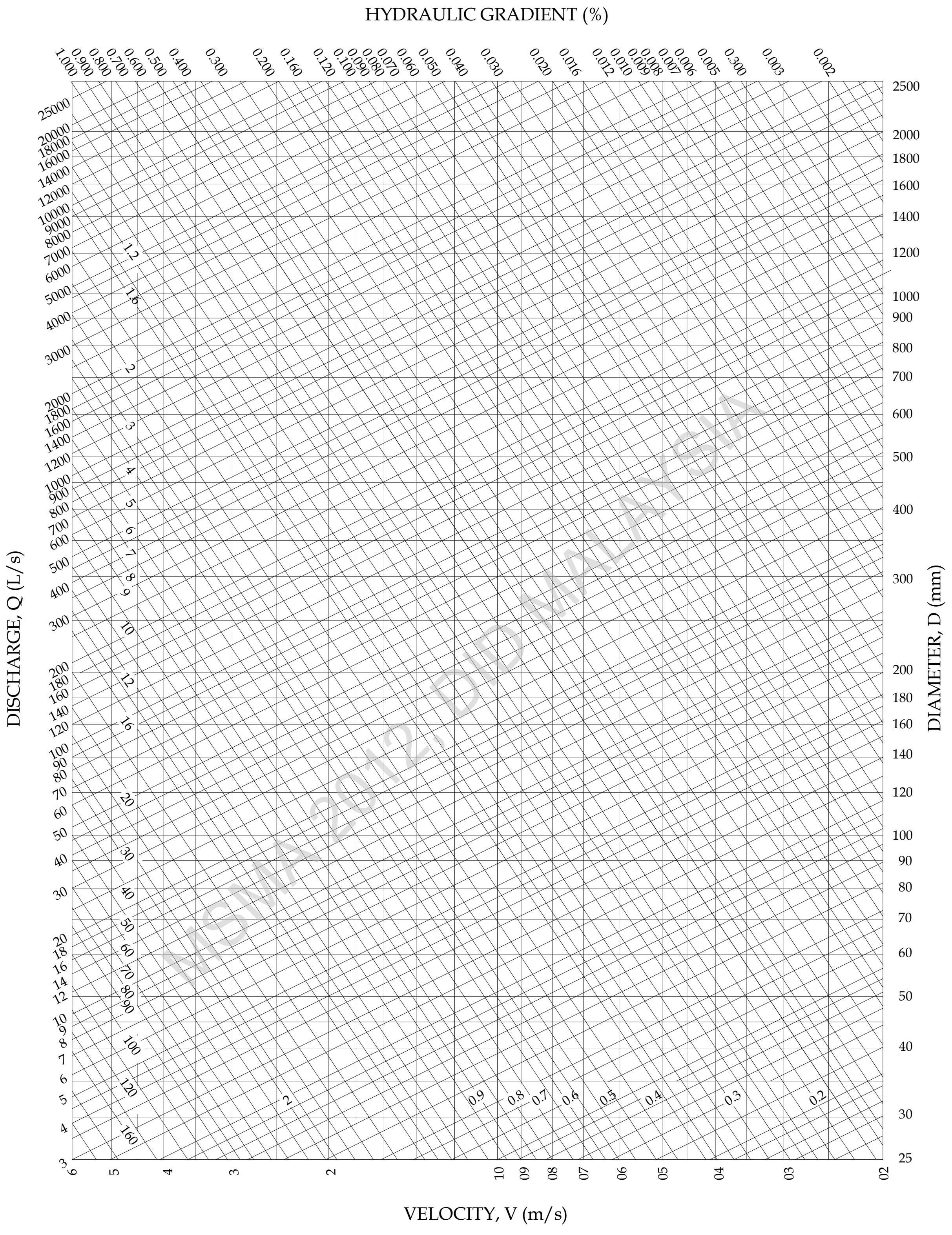


Chart 15.A2: Hydraulic Design of Pipes – Colebrook-White Formula – *k* = 0.30 mm

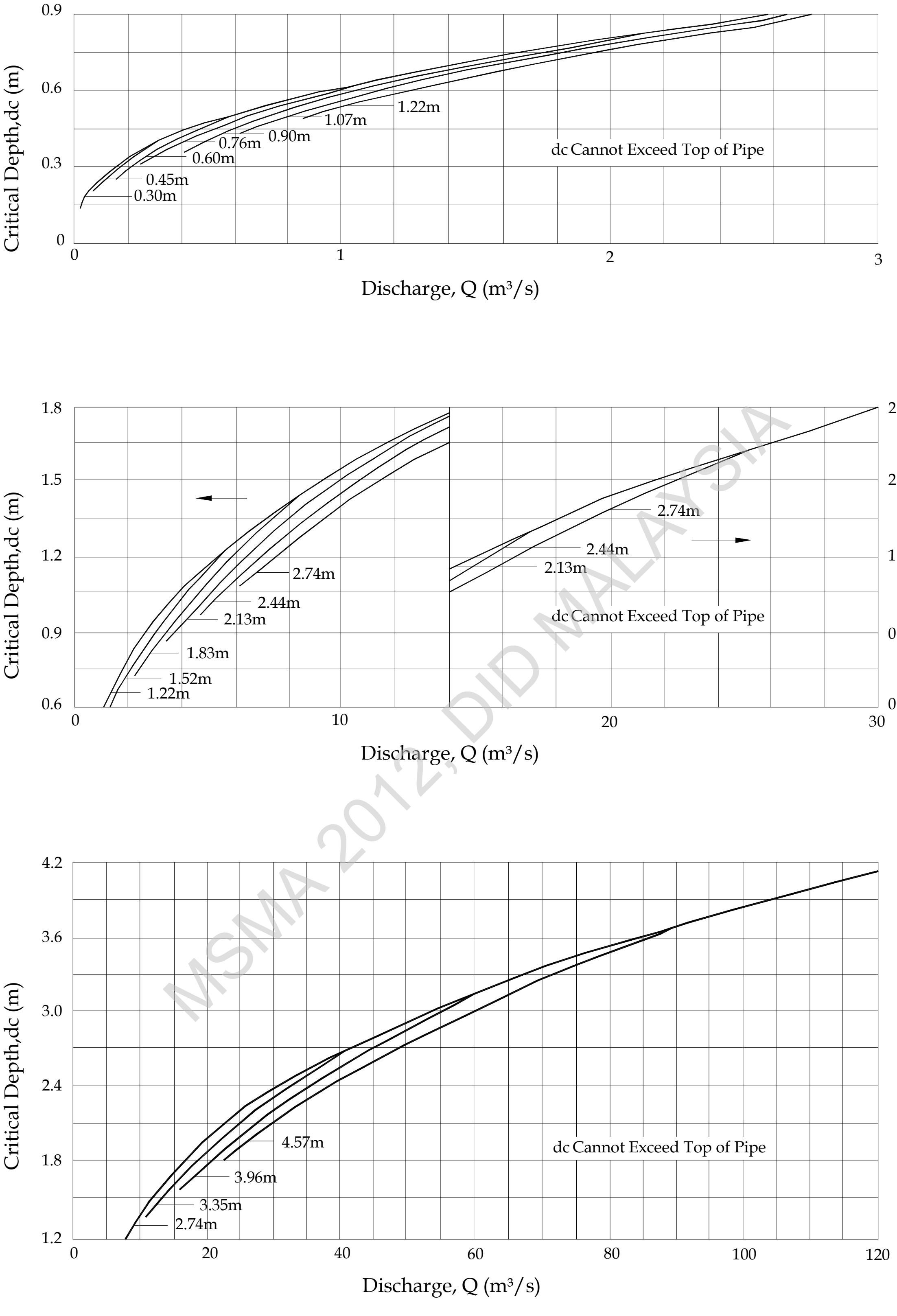


Chart 15.A3: Critical Depth in Circular Pipe

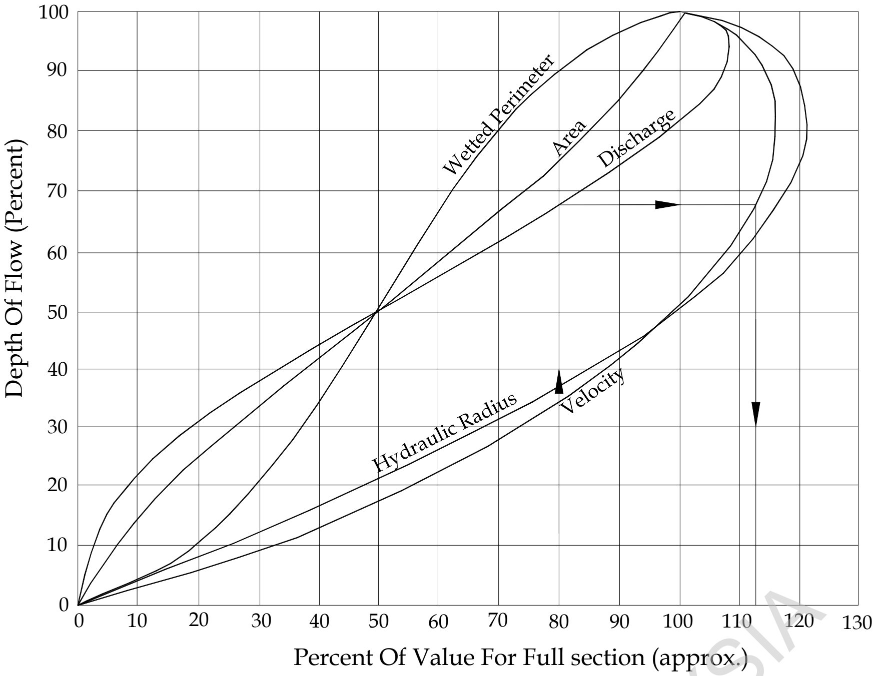
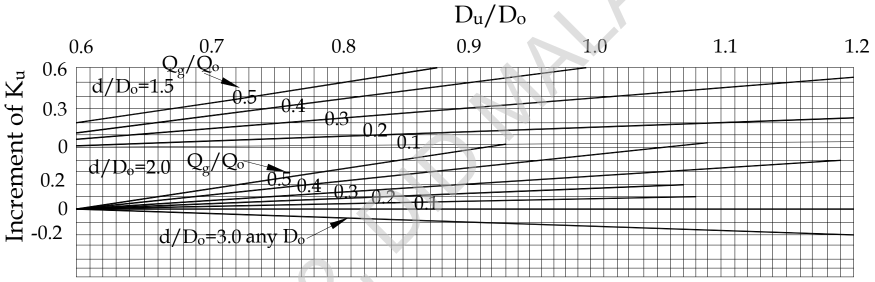


Chart 15.A4: Hydraulic Element Chart



Supplementary Chart for Modification of K_u
for Depth in Inlet other than $2.5D_o$

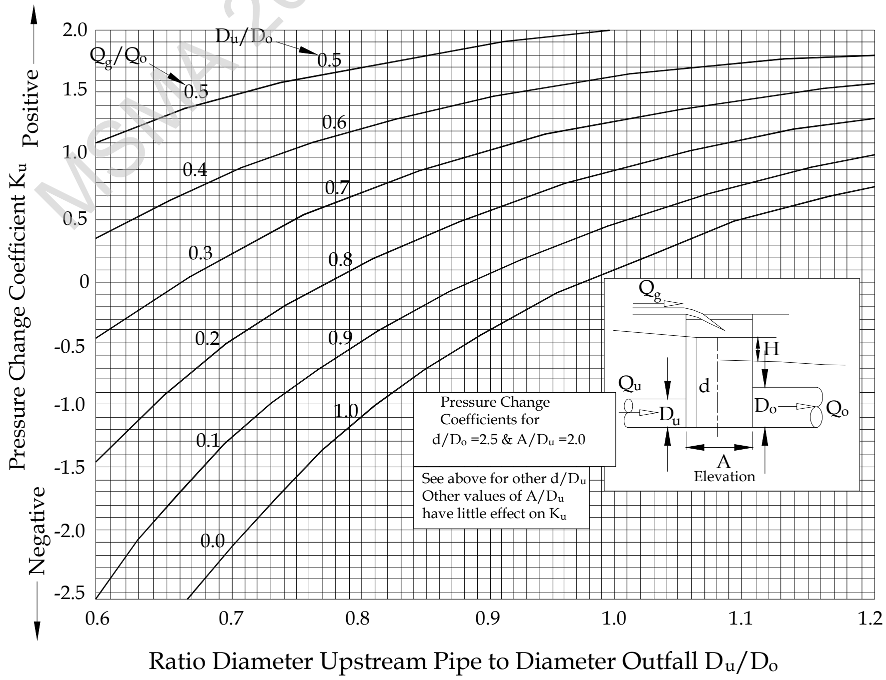
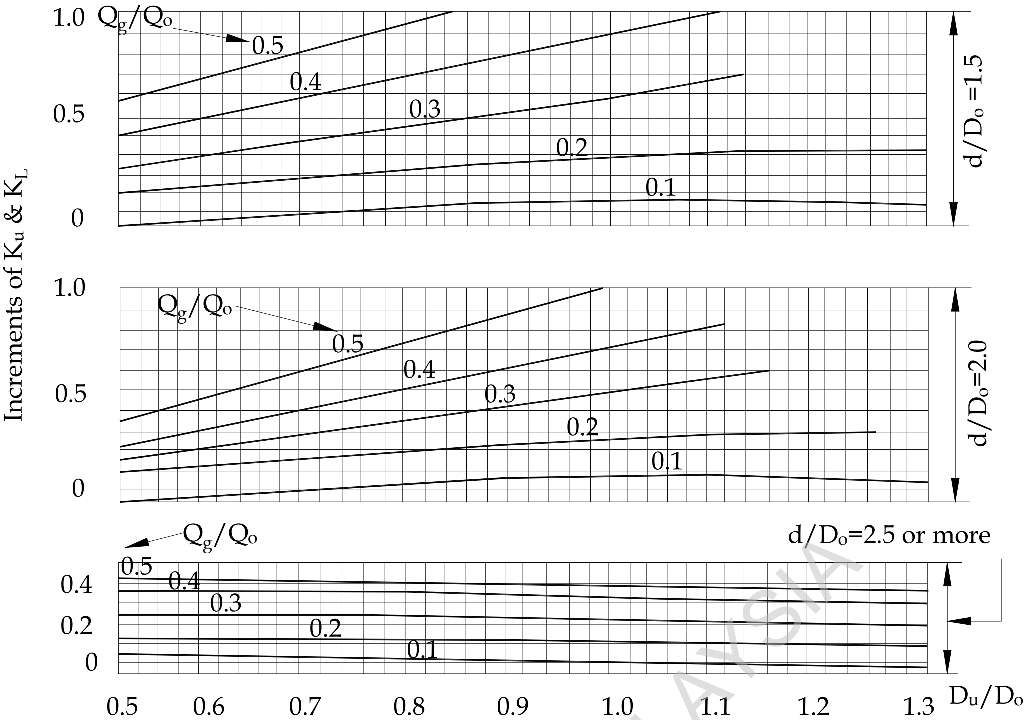


Chart 15.A5: Pressure Change Coefficient for Rectangular Inlet with Through Pipe and Grate Flow (Sangster et al, 1958)



Supplementary Chart for Modification of K_u & K_L for Grate Flow

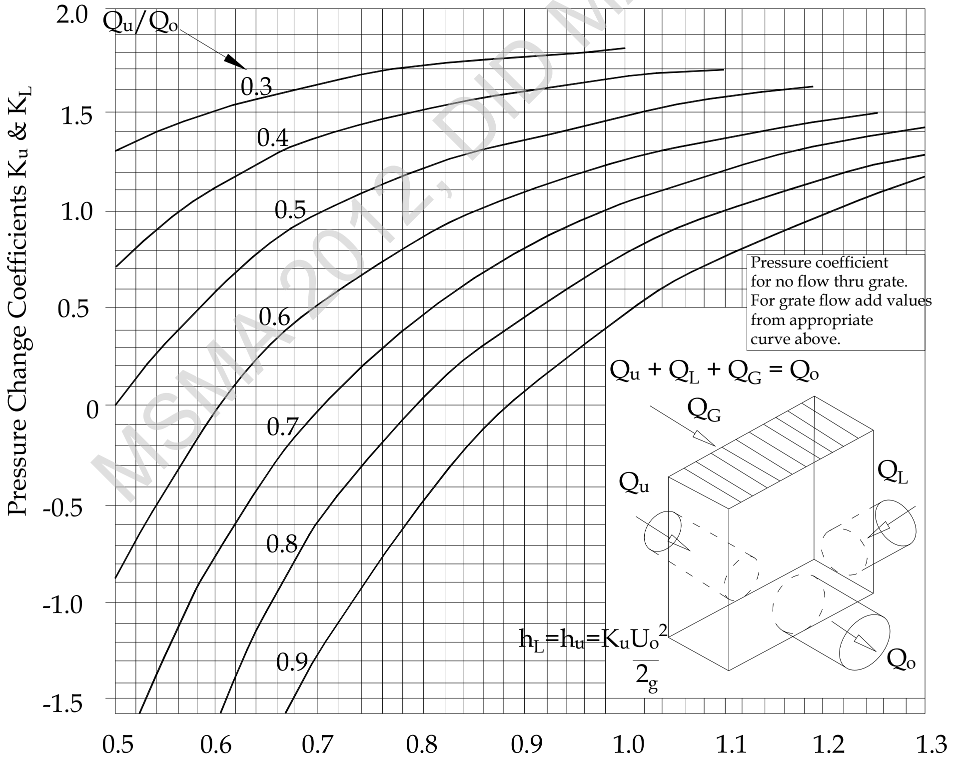


Chart 15.A6: Pressure Change Coefficients for Rectangular Inlets with In-line Upstream Main and 90° Lateral Pipe (with or without grate flow), (Sangster et al, 1958)

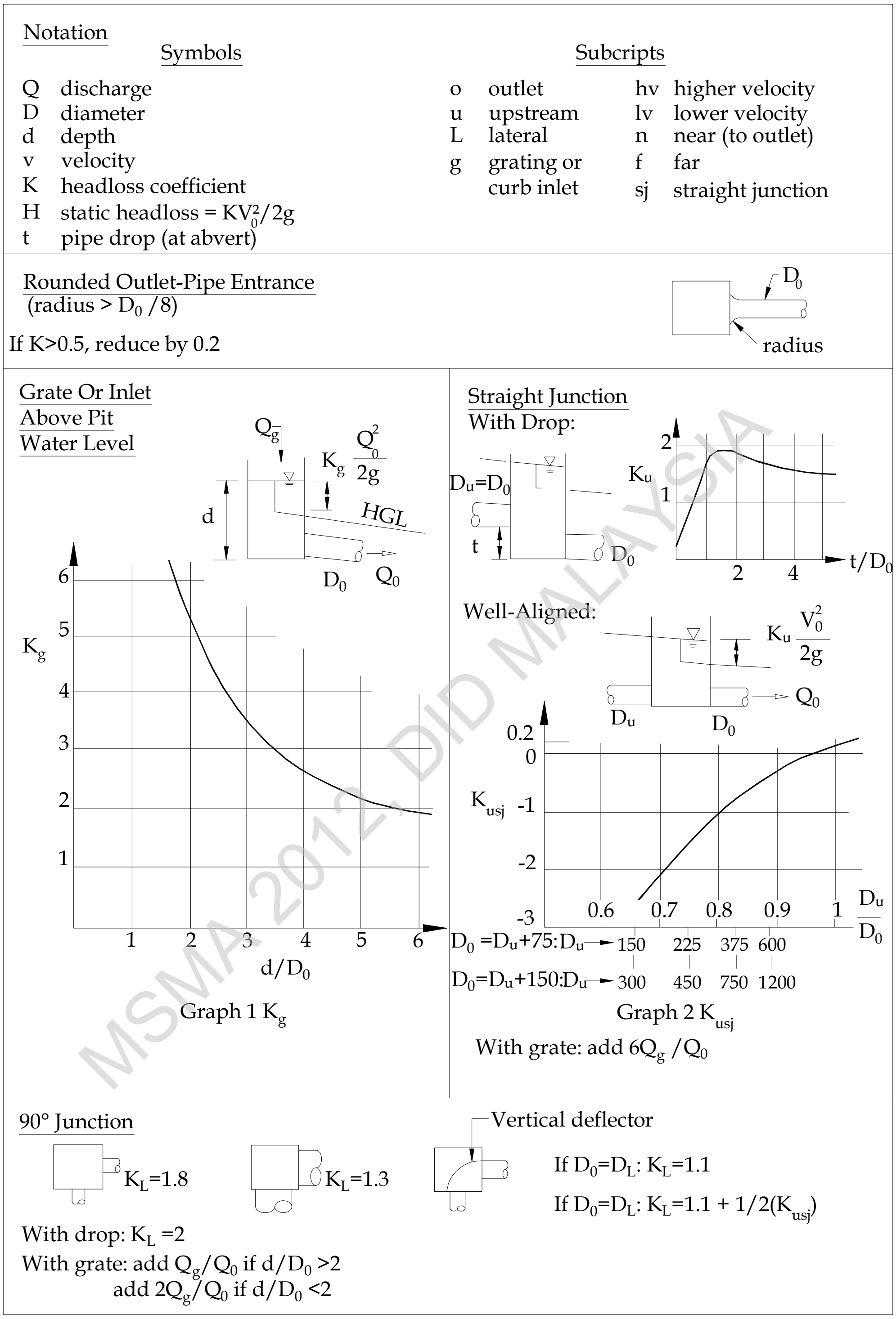
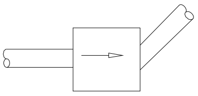
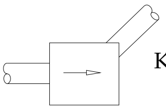


Chart 15.A7(a): Pit Static Head Loss Coefficients

45° Junction



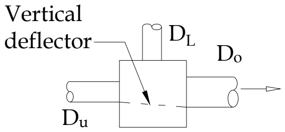
$K_u = 0.6 + \frac{1}{2}K_{usj}$



$K_u = 2.1$

With grate: add Q_g/Q_o if $d/D_o>2$
add $2Q_g/Q_o$ if $d/D_o<2$

Straight Junction With 90° Lateral



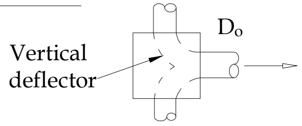
Q_u/Q_o		.3	.6	.9	1.0
$Q_o=Q_u$	no deflector: $K_u=K_L$ =	1.8	1.3	.5	.2
	with deflector: $K_u=K_L$ =	1.4	1.0	.4	.2

If $D_o>D_u$: add $\frac{1}{2}(Q_u/Q_o)(K_{usj})$

(actually reduces K)

With grate: add Q_g/Q_o if $d/D_o>2$
add $2Q_g/Q_o$ if $d/D_o<2$

Opposed Inlets

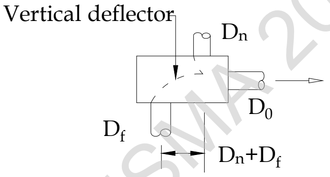


K_{hv} =1.8

Q_{hv}/Q_o	.3	.6	.9
$D_o=D_{hv}$:K_{1v}=	1.5	2.0	3.0
$D_o>D_{hv}$:K_{1v}=	1.5	3.0	4.5

With grate : no change.
With deflector : subtract 0.3, and $K_{1v} \ngtr 2$

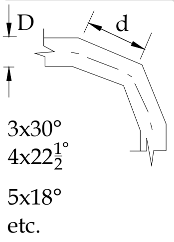
Opposed Offset Inlets

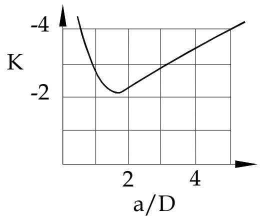


	K_n	K_f
Q_n is Q_{hv} :	1.6	1.9
Q_n is Q_{1v} :	1.6	2.4

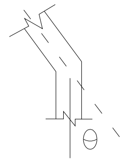
With grate : add 0.2
With deflector : subtract 0.3

Compound 90° Mitre Bend (Splayed)





Mitre Bend



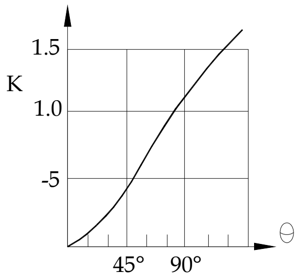


Chart 15.A7(b): Pit Static Head Loss Coefficient

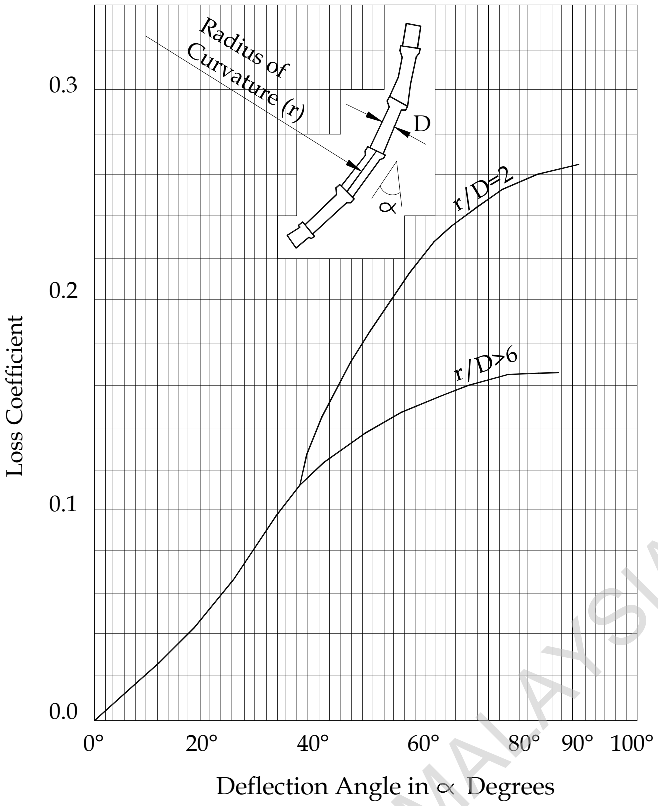


Chart 15.A8: Bend Loss Coefficients Source(D.O.T., 1992)

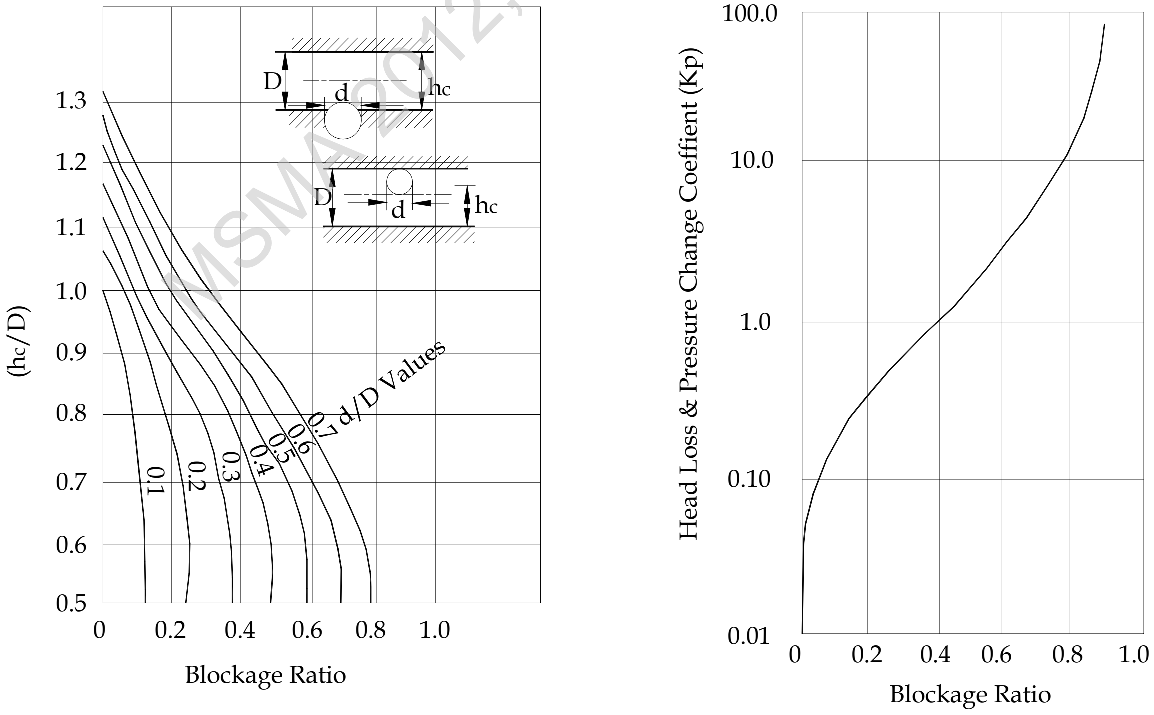


Chart 15.A9: Penetration Loss Coefficients (Black, 1987)

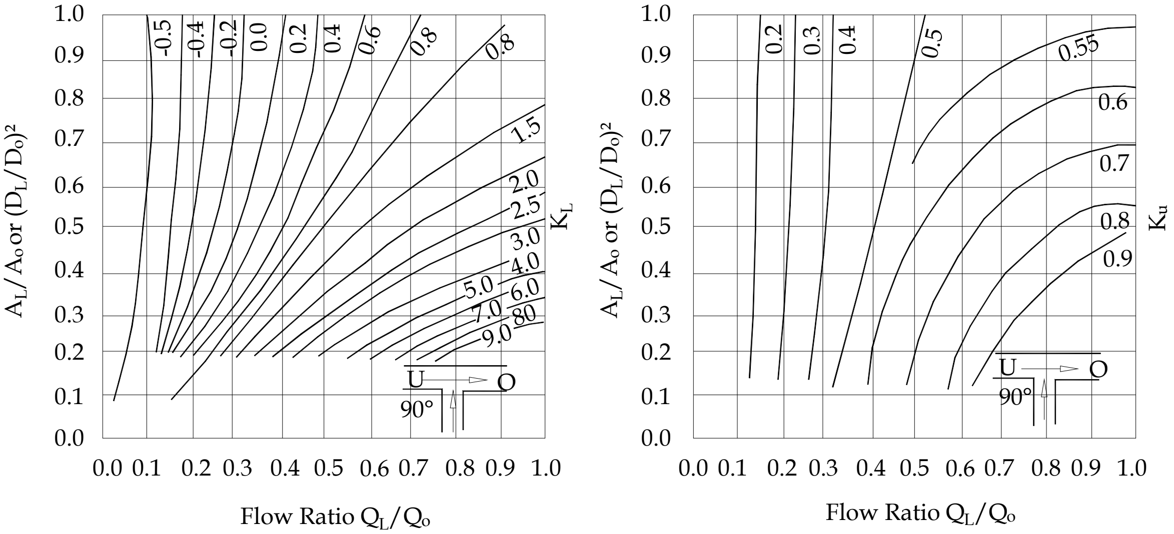


Chart 15.A10: Pressure Loss Coefficients at Branch Lines Source

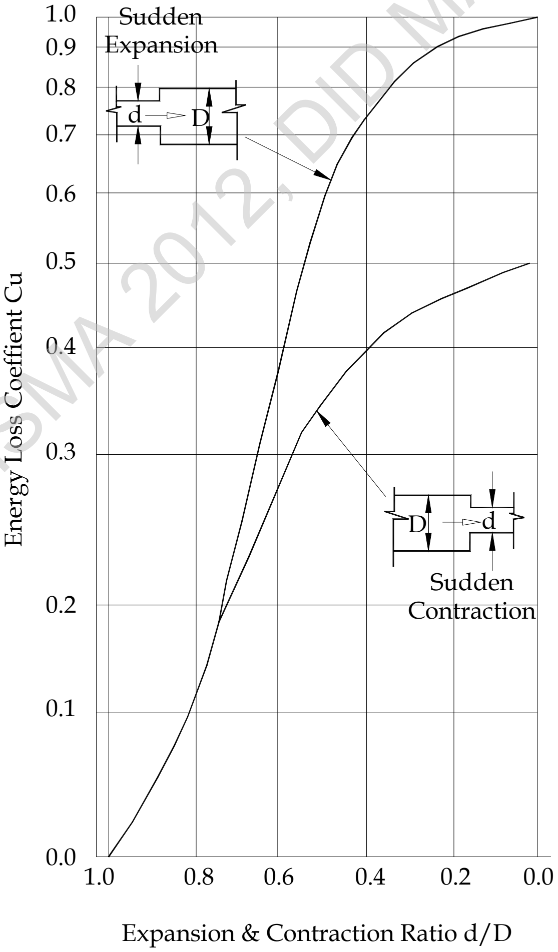


Chart 15.A11: Expansion and Contraction Loss Coefficients

Table 15.A1: Entrance Loss Coefficients

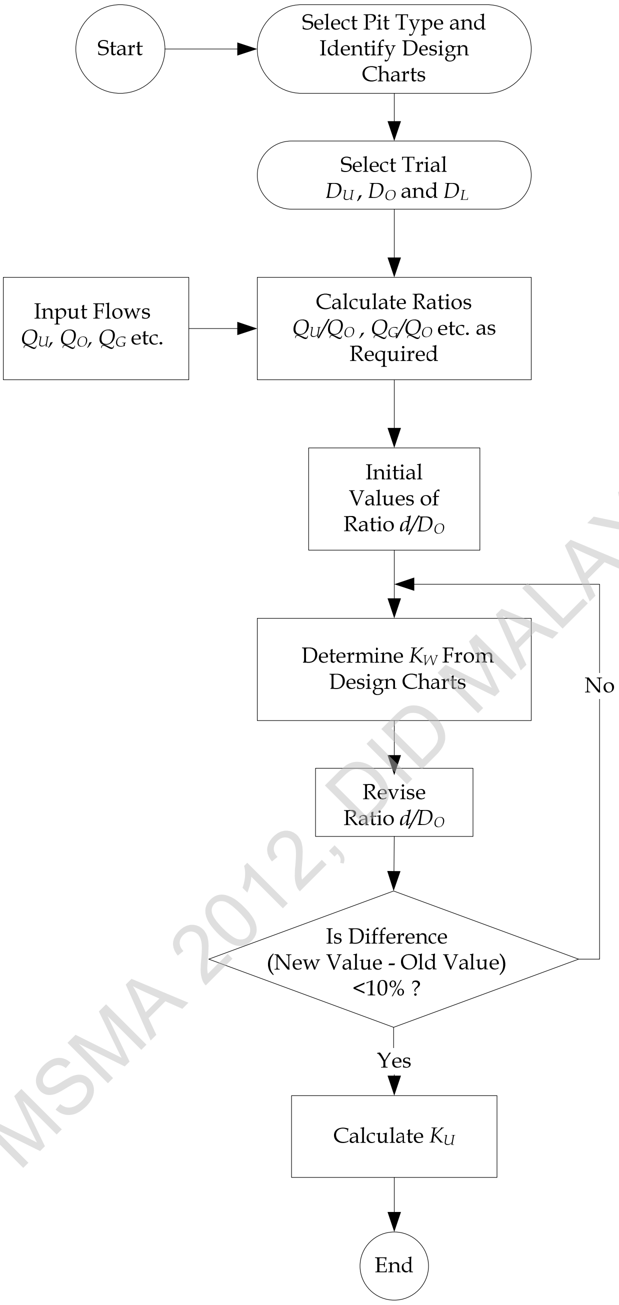
Type of Structure and Entrance Design	Coefficient K_e
Concrete Pipe	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls:	
Socket end of pipe (groove end)	0.2
Square edge	0.5
Rounded (radius = D/12)	0.2
Mitred to conforming to fill slope	0.7
End section conforming to fill slope	0.5
Hooded inlet projecting from headwall	See note
Corrugated Metal Pipe	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwall square edge	0.5
Mitred to comform to fill slope	0.7
End section conforming to fill slope	0.5
Reinforced Concrete Box	
Headwall parallel to embankment (no wingwalls)	
Square edges on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.4
Crown edge rounded to radius 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square edged at crown	0.7

Note : Refer Argue (1960) and O'Loughlin (1960)

Table 15.A2: Pressure Loss Coefficient at Mitred Fittings (ARR, 1987)

Type	K_b
90° double mitred bend	0.47
60° double mitred bend	0.25
45° single mitred bend	0.34
22.5° single mitred bend	0.12

APPENDIX 15.B DESIGN FLOW CHARTS



Flowchart 15.B1: Procedure for K_u and K_w Calculation

APPENDIX 15.C EXAMPLE – PIPE DRAINS DESIGN

Problem:

It is required to estimate the size and slope of the proposed pipe system to discharge the surface runoff from an urban area as shown in Figure 15.C1. The landuses of the area comprise of high density residentials, roads, open space and a service station. Through hydrologic data analysis performed from each pits the flow rates in the proposed pipelines are estimated for design use. Pits level and pipe segment flow rates are given in Table 15.C1 and 15.C2, respectively.

Table 15.C1: The Proposed Levels at Pits.

Pit No.	Surface Level (m)
1	28.02
2	26.23
3	24.51
4	24.48
5	24.38
6	24.38

Table 15.C2: Estimated Flow Rates

Pipe Segment	Flow Rates (L/s)
1 – 2	167
2 – 4	243
3 – 4	56
4 – 5	375
5 - 6	451

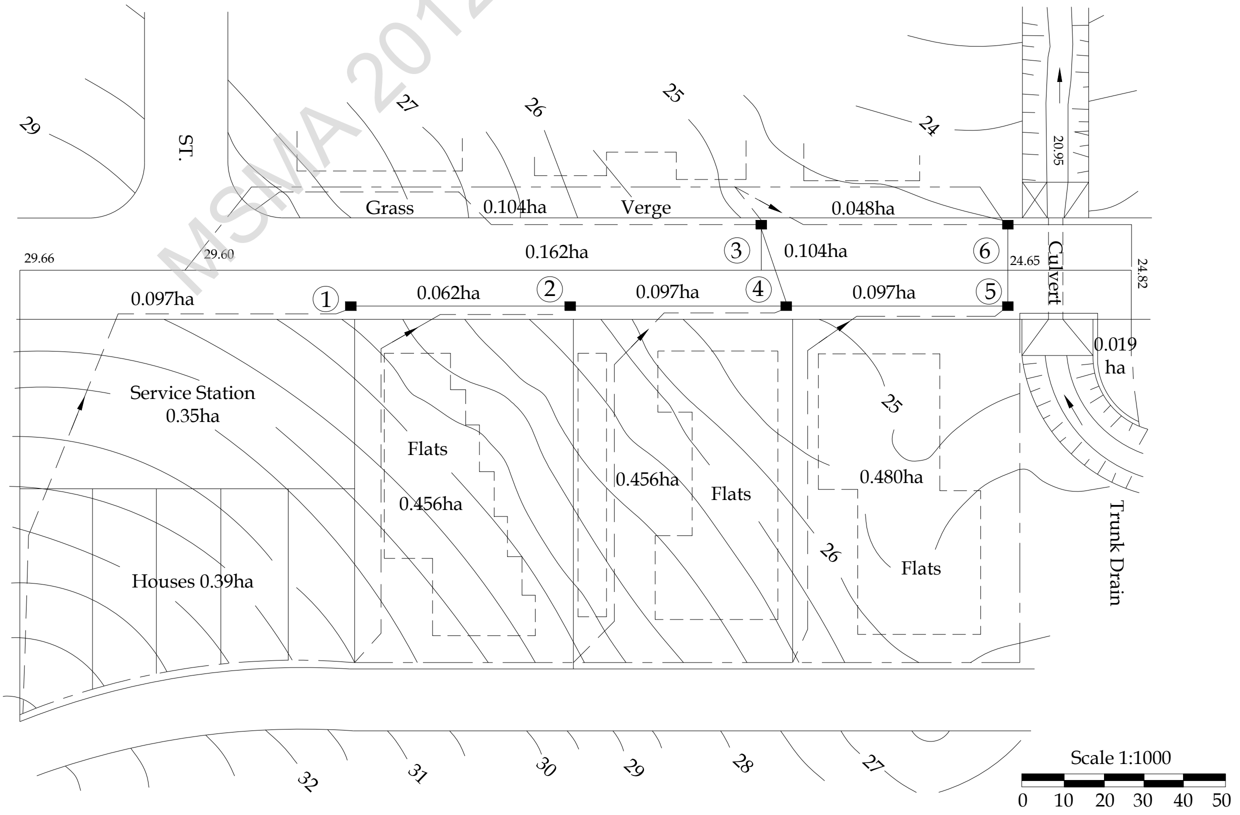


Figure 15.C1: Proposed Pipe Drains Layout

Solution:

The calculation starts at the upstream pit and moves downstream. Figure 15.C2 shows a pipe reach on which features are identified and linked to various column of the calculation sheet in Table 15.C3. Column 1 to 6 present the basic design information, while Column 7 to 16 are for calculation of the hydraulic grade line (HGL) position. The remaining columns are used to determine pipe invert levels, allowing for hydraulic considerations, cover and positions of upstream pipes. Pipes slopes are calculated to check for sedimentation problems. The calculation steps are given below.

The calculation start at the upstream pit with a trial pipe diameter of 381 mm being selected. This diameter is normally specified as minimum diameter to allow for passage of debris. The freeboard of 0.15m below the surface level is adopted for water levels in pits.

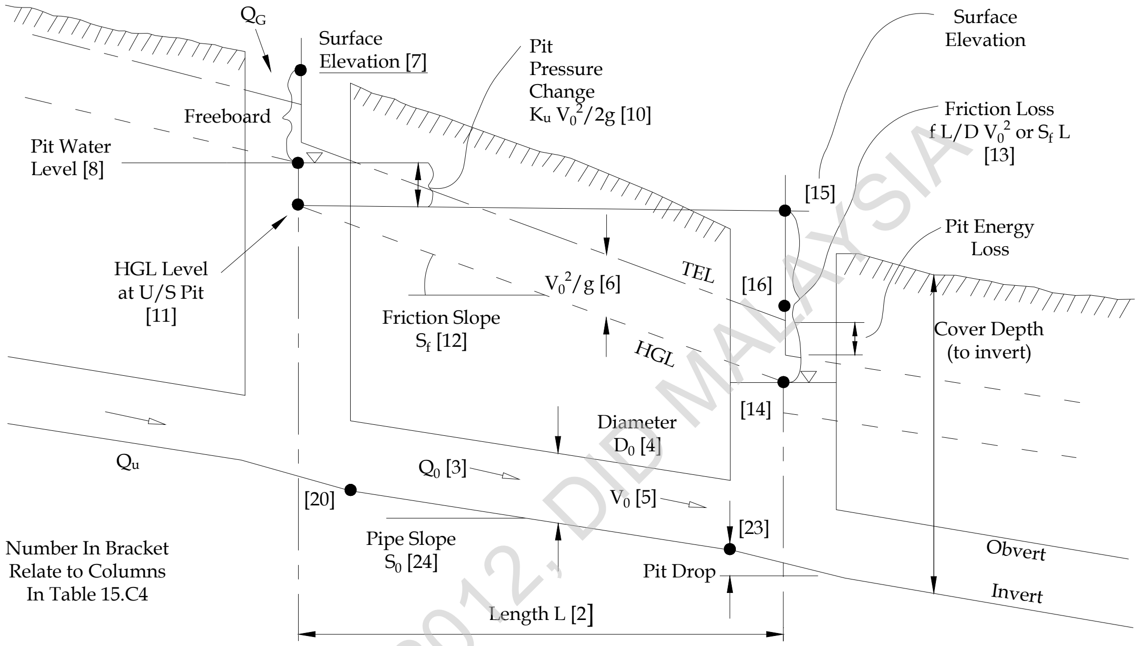


Figure 15.C2: Pipe Reach Showing Features Identified in Calculations

References	Calculation	Output
	Row 1 Column 1 : Pipeline 1-2 Column 2 : Pipe length L = 54.8 m Column 3 : Design Flow rate Q_{1-2} = 0.167 m³/s Column 4 : Trial diameter D_{1-2} = 0.381 m Column 5 : Full pipe velocity $V = Q/A = 0.167/\ 0.114$	$A=\frac{\pi\,D^2}{4}=\frac{\pi x(0.381)^2}{4}=0.114m^2$ V = 1.465m/s

	Column 6 : $V^2/2g = 1.465^2/2g$	$V^2/2g$ = 0.109m
	Column 7 : u/s Pit Surface = 28.02 m	u/s Pit Surface = 28.02 (Pit 1)
	Column 8 : Allowable water level in u/s Pit (Column 7 – freeboard) = u/s Pit Surface – Free board (0.15 m) = 28.02 – 0.15	WL in Pit 1 = 27.87m
	Column 9 : To find Pressure change coefficient K_u Assume depth in pit, d = 1.0	
Chart 15.A7(a)	$d/D_O = 1/0.381 = 2.63$	$K = K_g = K_u = K_w = 4.0$
Equation 15.4	Column 10: $H = \Delta P/\gamma = K_u V^2/2g = 4.0 \times 0.109$ Headloss in pit, H = 0.438m	H = 0.438m
	Column 11: HGL = Column 8 – Column 10 = 27.87 – 0.438	HGL = 27.432m at Pit 1
	Column 12: HGL Slope (S_f) for uniform subcritical flow from Manning's or Colebrook-White equation or charts.	
Equation 15.2	Using Equation 15.2, k = 0.3mm, Q = 167 L/s V = 1.465 m/s D = 381 mm	S_f = 0.553 % = 0.00553m/m
	Column 13: Friction energy loss $h_{f1\text{-}2} = S_f \, x \, L$ = 0.00553 x 54.8	$h_{f1\text{-}2}$ = 0.303m
	Column 14: HGL at the d/s pit = HGL at Pit 1 - h_f = 27.432 – 0.303	HGL at Pit 2 = 27.129m
	Column 15 : Surface level at the d/s pit	d/s Pit Surface = 26.23m
	Column 16 : Allowable water surface in d/s pit (Pit 2) is the lower of (i) Surface level – freeboard = 26.23 – 0.15 = 26.08m or (ii) HGL at pit = 27.129m	WL at pit 2 = 26.08m
	Column 17 : Calculation of pipe invert level for the u/s Pit (Pit 1) by hydraulic requirement; column 11 – column 4 = 27.432 – 0.381 = 27.051m	u/s invert level (Pit 1) Hydraulic requirement = 27.051m
	Column 18: Calculation of invert level for the u/s Pit (Pit 1) by cover requirement; Column 7– cover depth	u/s invert level (Pit 1) cover requirement = 27.007m

	(Note : cover depth = depth from surface to pipe crown + pipe thickness + internal diameter) = 28.02 – 0.6 – 0.032 – 0.381 = 27.007m	
	Column 19 : u/s Pit invert level minus any allowance for slope across the Pit, (drop) [Note : Recommended drop across pit = 0.03m]	no drop on slope of Pit 1
	Column 20 : Adopted u/s Pit invert, ie the lowest of [17], [18] and [19]	Adopted u/s Pit invert level (Pit 1) = 27.007m
	Column 21 : d/s pit invert level based on hydraulic requirement. Column 16 – Column 4 = 26.08 – 0.381 = 25.699 m	d/s Pit invert level by hydraulic requirement = 25.699m
	Column 22 : d/s pit invert level based on cover requirement. Column 15 – cover = 26.23 – 0.6 – 0.032 – 0.381 = 25.217 m	d/s pit invert level by cover requirement = 25.217m
	Column 23 : Adopted d/s pit invert level, i.e the lower of the values in Columns 21 & 22	Adopted d/s invert level (Pit 2) = 25.217m
	Column 24 : Calculate the pipe$_{1-2}$ Slope = (Column 20-Column 23) /Length = (27.007 – 25.217)/54.8 = 0.0327	Pipe$_{1-2}$ Slope = 0.327m/m
	Column 25 : Remarks	Pipe levels set by cover considerations
	Note : (1) The process are repeated for Pipe 2-4, 3-4,4-5 and 5-6 and the results are shown in Table 15.C3. (2) The assumptions made during calculation should now be re-checked. The assumed values of S and S/D$_o$ in this example were reasonable. If not, they should be amended and the calculations repeated.	

Table 15.C3: Hydraulic Design Sheet

[1] Pipe	[2] Length L (m)	[3] Design Flow-rate Q (L/s)	[4] Trial Pipe Diameter D (m)	[5] Full Pipe Velocity V (m/s)	[6] $V^2/2g$ (m)	[7] U/S Surface Level (m)	[8] U/S Pit Water Level Limit* (m)	[9] Pit Pressure Change Coeff. k_u or k_w	[10] $k.V^2/2g$ (m) [9] x [6]	[11] HGL At U/S Pit (m) [8] – [10]
1-2	54.8	167	0.381	1.465	0.109	28.02	27.870	4.0	0.438	27.432
2-4	55.0	243	0.381	2.131	0.232	26.23	26.080	0.2	0.046	26.034
3-4	18.4	56	0.381	0.491	0.012	24.51	24.360	4.5	0.055	24.305
4-5	56.2	375	0.381	3.289	0.552	24.48	24.292	0.5	0.276	24.016
			0.457	2.286	0.267	24.48	24.292	0.3	0.080	24.212
5-6	17.0	451	0.457	2.750	0.386	24.38	23.615	1.7	0.656	22.959
			0.533	2.021	0.208	24.38	23.615	1.5	0.313	23.302

Table 15.C3: Hydraulic Design Sheet (Cont.)

[1]	[12]	[13]	[14]	[15]	[16]	[17]	[18]	[19]	[20]
						U/S Invert Levels (m)			
Pipe	HGL Slope S_f (m/m)	Pipe Friction Loss $S_f . L$ [12] x [2]	HGL At D/S Pit (m) [11]-[13]	D/S Pit Surface Level (m)	D/S Pit Water Level Limit** (m)	Hydraulic [11]-[4]	Cover [7] - cover	U/S Pipe [23] – drop (0.03m)	Adopted Lowest of [17][18] & [19]
1-2	0.00553	0.303	27.129	26.23	26.080	27.051	27.007	-	27.007
2-4	0.01158	0.637	25.397	24.48	24.330	25.653	25.217	25.187	25.187
3-4	0.00066	0.012	24.292	24.48	24.292	23.924	23.497	-	23.497
4-5	0.02730	1.536	22.480	24.38	22.480	23.635	23.467	23.283	23.283
	0.01063	0.597	23.615	24.38	23.615	23.755	23.385	23.283	23.283
5-6	0.01531	0.260	22.699	24.38	22.699	22.502	23.285	22.691	22.502
	0.00690	0.117	23.185	24.38	23.185	23.769	23.205	22.691	22.691

** Lower of [14] or ([15] – Freeboard)

Table 15.C3: Hydraulic Design Sheet (Cont.)

[1]	[21]	[22]	[23]	[24]	[25]
	D/S Invert Levels (m)				
Pipe	Hydraulic [16]-[4]	Cover [15]- cover	Adopted Lower of [21] & [22]	Pipe Slope S_o $\frac{[20]-[23]}{[2]}$	Remarks
1-2	25.699	25.217	25.217	0.0327	Pipe levels set by cover consideration
2-4	23.949	23.467	23.467	0.0313	Cover still controls
3-4	23.911	23.467	23.467	0.0016	Too flat - adjust to 1% slope
			23.313	0.0100	
4-5	22.099	23.367	22.099	0.0211	Use 457mm, but adjust to 1% slope
	23.158	23.285	23.158	0.0022	
			22.721	0.0100	
5-6	22.242	23.285	22.242	0.0153	Use 533mm and adjust to 1% slope
	22.652	23.205	22.652	0.0023	
			22.521	0.0100	

ENGINEERED CHANNEL

16

CHAPTER 16 ENGINEERED CHANNEL

16.1 INTRODUCTION

Drainage should be designed in an environmentally responsible way to minimise disruption of the natural environment of the nation's streams and channels.

Engineered channels are a preferred means of meeting the above objective by providing a drainage system that is more closely resembles natural streams. The word "engineered" in the title indicates that the channels are carefully designed to achieve this condition.

Engineered channels is one of the components for the major drainage system designed to collect and convey flows from the minor drainage system and to provide for the safe passage of larger flows up to the major design storm.

16.1.1 Advantages

In urban settings, a wide variety of options are used for lining engineered channels. Although these engineered channels are more costly than the use of vegetation, but they do offer advantages such as stability under higher velocities and can accept runoff immediately after construction.

16.1.2 Disadvantages

Engineered channels with a more natural form will convey flows less efficiently than traditional straight-lined channels. As a result, a natural-form engineered channel will require a larger cross-sectional area to convey the same flow. Grassed channels generally need suitable soils for vegetation and adequate area for installation. It is critical during the vegetative establishment period to restrict outside water from flowing through the channel. Therefore, it may be necessary to delay construction until the engineered channel is well established. Grassed waterways are also used as filters to remove sediment, but may sometimes lose their effectiveness when there is excessive sediment build up in the engineered channel.

16.1.3 Engineered Channel Types

The types of engineered channels available for urban drainage systems are almost infinite, depending only upon good hydraulic practices, environmental design, sociological impact, and basic project requirements. However, from a practical standpoint, the basic choice to be made initially is whether or not the engineered channel is to be a hard-lined one for higher velocities, a grassed channel, or a natural channel already existing. The following types shown in Table 16.1 are applicable in urban areas.

16.2 DESIGN CONSIDERATIONS AND REQUIREMENTS

16.2.1 General

The design of constructed engineered channels should, wherever practicable, mimic the natural stream forms in the immediate region. The following shall be considered for the successful management of aquatic, ephemeral and terrestrial environments along constructed channels (Melbourne Water, 2009):

- the capacity and hydraulic functions of the channel;
- an integrated approach to planting and hydraulic planning to prevent or reduce flooding;
- consideration of roughness issues and potential impacts on localised flooding where Manning's n values are increased due to excessive use of shrubs in revegetation;
- planting for the purpose of mitigating problems associated with increased flows resulting from greater impervious surfaces due to urban development (i.e. for erosion and bank stabilisation control); and
- revegetation design also needs to consider the impact of hydraulic roughness and potential barriers to flow. The increased resistance to flow from denser vegetation growth tends to slow the passage of flood water, thereby reducing channel conveyance and raising water levels for a given flow.

Table 16.1: Engineered Channel Types Applicable in Urban Areas in Malaysia

Types and Descriptions	Example
Natural Channels: These are engineered channels carved or shaped by nature before urbanisation occurs and are the most desirable of the various types of constructed or modified channels. They often, but not always, have mild slopes and are reasonably stable. As the tributary catchment urbanises, natural channels often experience erosion and may need grade control checks and localised bank protection for channel stabilisation.	
Grassed Channels: These are soft-lined engineered channels designed to lower flow velocities, provide channel storage, and offer various multiple use benefits. Low flow areas generally need to be concrete or rock lined to minimise erosion and maintenance problems. Generally, grassed channels should be located to conform with and use the natural drainage system. Grassed channels may also be developed along roadways and property lines but should avoid sharp changes in flow direction and longitudinal slope.	
Concrete Lined Channels: Concrete lined channels are high velocity artificial channels that are not encouraged in urban areas. However, in retrofit situations where existing flooding problems need to be solved and where a drainage reserves are limited, concrete channels may offer advantages over other types of engineered channels.	
Other Channel Liners: A variety of artificial channel liners may be used to protect the channel walls and bottom from erosion at higher velocities. These include gabions, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fibre liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems, but are not recommended for new developments. Each type of liners has to be scrutinised for its merits, applicability, how it meets other community needs, its long-term integrity, maintenance needs and costs.	

16.2.2 Location

Continuous designated overland flow paths shall be provided from the top of the catchment through the entire urban area.

Engineered channels may be located within designated drainage reserves, roadways, parkland and open space areas, and pedestrian ways. All engineered channels shall be located wholly outside of privately owned lots. If circumstances arise where this arrangement cannot be provided, prior agreement to locate engineered channels within privately owned areas must be obtained from the local regulatory authority and the affected private landowners. Engineered channels are generally not recommended for construction within river floodplains. In these situations the aim is to ensure that flood flows spread across the floodplain as they would under natural conditions (NRW, 2004).

Engineered channels shall be provided along the alignment of existing watercourses and drainage depressions. Diversion of engineered channels away from their natural paths will only be permitted in exceptional circumstances and only with the approval of the local regulatory authority.

Wherever possible, landuse within engineered channel corridors should be designated as public open space. Other types of landuse may be considered, but they must be fully compatible with the primary role of the channel to convey flood flows up to and including the design storm.

16.3 DESIGN CRITERIA

16.3.1 Design Storm

Engineered channels shall be designed to cater to flows up to and including the major system design ARI. All new urban developments shall be provided with major drainage systems designed with sufficient capacity and freeboard to ensure that flood flows up to 100-year ARI do not encroach upon private leases. Extreme flood events should also be considered to allow for the investigation of the channel stability and performance under unusually large flood events. At times, it may be necessary to adjust design flows to match more realistic observed or historical flows (ACT Government, 1992).

Adjoining low-lying land may need to be acquired and/or reclaimed to ensure effective surface drainage and containment of the design ARI flow within an engineered channel.

16.3.2 Reserve

Reserves are required for all engineered channels. These must be clearly defined on all development plans to ensure that future developments do not encroach upon land inundated by flows up to and including the design storm.

The prime function of reserves is to give ready access to personnel, plant and materials, which may, from time to time, be required for channel and berm maintenance. No encroachment, especially earth fill that may inhibit such access or make such maintenance unduly difficult, shall be allowed on reserves.

Engineered channel easement shall be wide enough to contain the service and provide working space on each side of the service for future maintenance activities. The minimum channel reserve width shall be the top channel width for the major storm ARI flow plus a 300 mm freeboard requirement. Maintenance width requirements may be incorporated within this reserve width by benching. If this cannot be achieved, the reserve width must be increased to include maintenance width requirements. Minimum widths to be provided for maintenance access shall be in accordance with Table 16.2.

Table 16.2: Minimum Requirements for Maintenance Access

Top Width of Engineered Channel	Minimum Requirements for Maintenance Access
W ≤ 6m	One side 3.7 m, other side 1.0 m
W > 6m	Both sides 3.7 m

Where other hydraulic services or electrical services are located within the same reserve, the required reserve width shall be increased to provide adequate clearance between the services.

When planning development along an engineered channel for which a master plan is not yet available, a drainage reserve width shall be estimated based on the premise that the design storm flow will be catered to by a grassed channel. This premise ensures that sufficient land will be available for the design of the engineered channel when carried out in conjunction with detailed landuse planning at a later date.

16.3.3 Freeboard

The freeboard above the design storm water level for all engineered channels shall be a minimum of 300 mm, as shown in Table 16.3 unless suggested otherwise by the local regulatory authority. A higher freeboard should be considered at locations where superelevation or hydraulic jumps are anticipated.

16.3.4 Access Requirements

For engineered channels lined with concrete, stone pitching and rock mattresses, access ramps shall be provided for maintenance machinery to gain access to the channel bottom when necessary. Ramps shall be a minimum of 3.5 m wide with a longitudinal slope not steeper than 10(H):1(V) and have a non-slip surface.

16.3.5 Longitudinal Slope

Engineered channels shall be constructed with sufficient longitudinal slope to ensure that ponding and/or the accumulation of silt does not occur, particularly in locations where silt removal would be difficult.

The minimum longitudinal slope for engineered channels shall be as shown in Table 16.3. The longitudinal slope shall not produce velocities less than 0.6 m/s if low flow inverts flowing full.

16.3.6 Design Velocity

Engineered channels, either bare or lined with vegetation, should carry the design discharge at nonerosive velocities. Engineered channels also shall be designed with longitudinal grades that minimise the incidence of hydraulic jumps, dangerous conditions for the public, and erosion of surface linings and/or topsoil.

Longitudinal slopes shall be chosen such that the design storm average flow velocity will not exceed the limits shown in Table 16.3.

16.3.7 Side Slope

The recommended maximum side slopes for engineered channel is indicated in Table 16.3.

Table 16.3 Engineered Channel Types Design Criteria

Channel Type	Minimum Freeboard (mm)	Minimum Longitudinal Grade (%)	Maximum Average Flow Velocity (m/s)	Maximum Side Slope
Natural channels	300	0.1	2	1V:3H
Grassed channels	300	0.1	2	1V:3H
Soft lined channels with turf reinforcement mats (TRM)	300	0.1	4	1V:2H
Composite channels	300	0.4	4	1V:1.5H
Hard lined channels	300	0.4	4	Vertical

16.3.8 Drop Structures

Drop structures should be provided to reduce channel longitudinal slopes such that the design storm average flow velocities do not exceed the limits specified in the previous section. Drop structures shall be designed to ensure that the structures do not get 'drown out' due to high tailwater levels under the major system design flow plus freeboard. Design requirements for drop structures are provided in Chapter 20.

16.3.9 Roughness Coefficient

The Manning's roughness coefficient is dependent on a number of variables including surface roughness, channel irregularity, the presence of obstructions, channel alignment, and the likelihood of sedimentation or scour. Manning's roughness coefficients for the various types of engineered channels are provided in Table 16.4. The choice of an appropriate value for the roughness coefficient of an engineered channel is often critical in the overall design procedure and requires a considerable degree of judgment.

Estimation of an equivalent or composite Manning's roughness coefficient in a channel of varying roughness is often required when there is a marked variation in the boundary roughness across a cross section. In the case of an engineered channel designed within a typical urban environment, examples of this situation include a grassed channel containing a concrete low flow invert, or a channel containing a low level access road along one side of the channel. Equation 16.1 may be used to estimate the overall roughness coefficient in engineered channels of composite roughness. It is important for the designer to check if the composite Manning's roughness coefficient value obtained using Equation 16.1 is reasonable. A distorted or inaccurate value will result in inaccurate predictions of channel flow conditions.

$$n^* = \frac{\sum_{i=1}^{m} \frac{n_i A_i^{5/3}}{P_i^{2/3}}}{\sum_{i=1}^{m} \frac{A_i^{5/3}}{P_i^{2/3}}} \qquad (16.1)$$

where,

n^* = equivalent Manning's roughness coefficient for the whole cross section;

n_i = Manning's roughness coefficient for segment *i*;

A_i = flow area of segment *i* (m^2);

P_i = wetted perimeter of segment *i* (m); and

m = total number of segments.

16.3.10 Safety Requirements

Where the design storm flow depth within a lined channel exceeds 0.9 m, a 1.2 m high handrail fence shall be provided on both sides of the channel to discourage public access. Handrail fencing should be parallel to the channel as far as practicable and should be located within 1 to 3 m from the channel edge. Lockable gates shall be placed at appropriate locations to permit access for maintenance.

Where a lined channel is adjacent to a public road or sited close to carriageway and housing lots, guard railing shall be provided instead of a handrail fence. Guard rails shall be provided in accordance with the requirements of the relevant Authority and/or those for the Public Works Department (JKR) Roads. Alternatively, precast reinforced concrete covers may be also provided for the concrete lined channels. Service opening shall be provided along the channel with maximum interval spacing of 100 m.

Ladder-type steps or step irons should be provided where the channel side slope is steeper than 2(H):1(V), or where the channel depth exceeds 1.2 m. These should be installed no more than 120 m apart on alternating sides of the channel. The bottom rung of the ladder or bottom step iron shall be placed approximately 300 mm vertically above the channel invert. Steps of step irons shall be provided to the top of benching and toe holes provided in the benching.

16.4 NATURAL CHANNEL

Natural channels are either in the form of steeply banked streams, which have erodible banks and bottoms, or mild channels, which are reasonably stabilised. For either type of channel, if it is to be used for carrying storm runoff from an urbanised area, it can be assumed initially that the changed runoff regime will result in highly active erosional tendencies. In nearly all cases, some form of modification of the channel will be required to create a more stabilised condition for the channel (ACT Government, 1992).

The design guidelines for grassed channels do not necessarily apply to natural channels, but such criteria can be utilised in gauging the adequacy of a natural channel for future changes in runoff regime. Design criteria and techniques, which should be used in the design of natural channels, include the following:

- channel and overbank capacity shall be adequate for the design storm;
- channel velocity shall not exceed the lesser of 2 m/s or the critical velocity for any particular section. Manning's roughness coefficient, n , which are representative of maintained channel conditions shall be used for determining critical channel velocities;
- water surface limits shall be defined so that the floodplain can be zoned and protected. Manning's roughness coefficient, n , which represent unmaintained channel conditions shall be used for the analysis of water surface profiles; and
- drop structures or check dams should be constructed to limit flow velocities and control water surface profiles, particularly for the initial storm runoff.

Table 16.4 Suggested Values of Manning's Roughness Coefficient, n

Surface Cover	Suggested n Values
Natural Channels	
Small streams	
Straight, uniform and clean	0.033
Clean, winding with some pools and shoals	0.045
Sluggish weedy reaches with deep pools	0.080
Steep mountain streams with gravel, cobbles, and boulders	0.070
Large streams	
Regular cross-section with no boulders or brush	0.060
Irregular and rough cross-section	0.100
Overbank flow areas	
Short pasture grass, no brush	0.035
Long pasture grass, no brush	0.050
Light brush and trees	0.080
Medium to dense brush	0.160
Dense growth of trees and brush	0.200
Grassed Channels	
Grass cover only	
Short grass (< 150 mm)	0.035
Tall grass (≥ 150 mm)	0.050
Shrub cover	
Scattered	0.070
Medium to dense	0.160
Tree cover	
Scattered	0.050
Medium to dense	0.120
Lined Channels and Low Flow Inverts	
Concrete	0.015
Shotcrete	0.025
Stone Pitching	
Dressed stone in mortar	0.017
Random stones in mortar or rubble masonry	0.035
Rock Riprap	0.030

16.5 GRASSED CHANNEL

16.5.1 General

Grassed channels are engineered channels included in the soft-lined category. Engineered channels of this type are generally regarded as more aesthetically pleasing than the hard-lined channels and they can be designed to blend into the surrounding natural environment. Grassed channels can be used in situations where;

- flow velocities are insufficient to produce scour;
- the drainage reserve width is not a problem; and
- aesthetics is an important design consideration.

Whilst grassed channels generally have significant construction cost advantages over hard-lined channels, this benefit is in some cases offset by their additional land-take requirements. However, the effect of this additional land-take can be minimised by designing the channel for multi-purpose use. When a grassed channel is to be constructed in a poorly drained area, a dry period must be chosen in which to do the work. In some instances, it may be necessary to direct water away from a new channel until all the work has been completed and the vegetative cover stabilized.

Grassed channels shall conform to the following general requirements:

- channels shall be grassed with provision for low flows; and
- access road and footpath paving within a channel shall be designed to withstand the design discharge in areas of high velocity such as adjacent to bridges and underpasses.

16.5.2 Geometry

Side slopes for grassed channels areas must be adequate to ensure drainage without localised ponding occurring. Batter side slopes should not be milder than 6(H):1(V) for reasons of public safety. However, steeper side slopes up to a maximum of 3(H):1(V) may be provided in special circumstances, such as to preserve existing trees and other natural features, or in existing areas where the land available for a grassed channel is limited.

The base width of a grassed channel should be designed to accommodate the hydraulic capacity of the floodway with due consideration given to limitations on flow velocity. The width must be sufficient to allow access to the base for maintenance purposes. The channel base side slopes shall not be less than 50(H):1(V). Alternative arrangements may be provided to widen a grassed channel adjacent to public open space or recreational areas. In such cases, the minimum side slope for the channel shall be 50(H):1(V).

The preferred cross-section for grassed channels is shown in Figure 16.1.

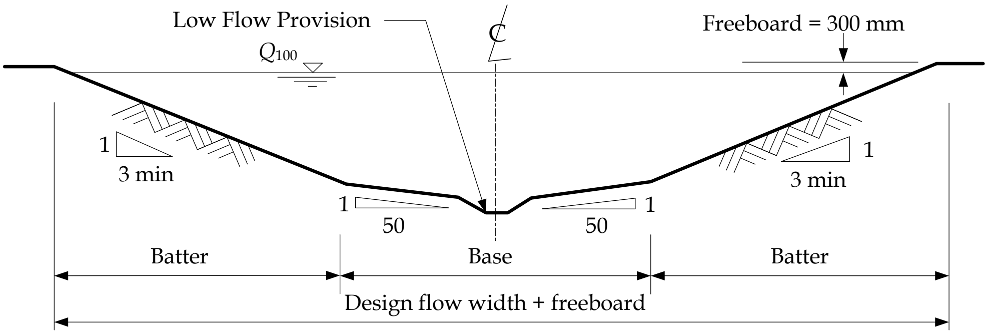


Figure 16.1: Typical Grassed Channel Cross-Section (Modified from ACT Government, 1992)

Terracing, as indicated in Figure 16.2, may be introduced across the grassed channel to contain more frequent flood flows and to localize much of the maintenance to the frequently flooded zone. Terraces may be adjacent public open space and recreational areas or dedicated right of way. This type of section provides for flood storage and to reduced downstream flood levels and lower average velocities for the major system design flow and more natural green space for wildlife and public recreation. Flooding of the adjacent open space and recreational areas should only be incorporated in such areas where flood protection less than the design storm is satisfactory. The minimum side slope for the terrace base shall be 50(H):1(V).

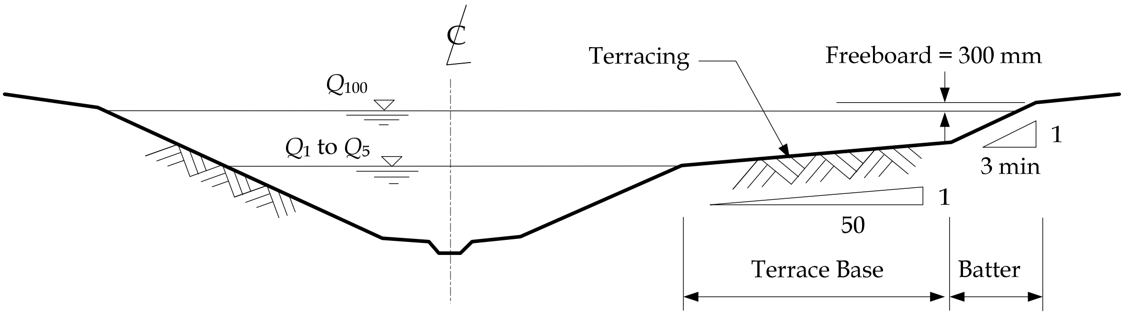


Figure 16.2: Typical Grassed Channel Terracing (Modified from ACT Government, 1992)

16.5.3 Grass Cover

The grass species chosen for lining of channels must be sturdy, drought resistant, easy to establish, and able to spread and develop a strong turf layer after establishment. A thick root structure is necessary to control weed growth and erosion.

A protective cover consisting of mulch and grass seeding is necessary to protect newly constructed engineered channels immediately after construction. If possible, disturbed areas should be seeded with a permanent grass seed mix. Permanent grassing for channels and planting plan shall be in accordance with Annex 1.

16.5.4 Low Flow Provision

The primary function of low-flow channels within open channels is to (NRW, 2007):

- allow efficient drainage of the greater channel or floodway area to minimise the risk of undesirable waterlogging of the soil;
- control erosion along the invert of large drainage channels;
- provide a hydraulic regime that allows the flushing of regular sediment;
- flows towards specified instream sediment traps; and
- provide necessary ecological features (e.g. habitat and passage) within channel habitats.

A grassed channel or floodway must be provided with a low flow system to facilitate drainage, pest control, and maintenance. Urban engineered channels normally have a continuous dry-weather base flow mainly due to runoff from domestic water usage. Continuous flow over the grass lining in the waterway invert will destroy the grass stand and promote weed growth. The invert may also become eroded and form pools of water, which may cause soggy ground conditions, a breeding ground for mosquitoes.

(a) Design Capacity

Low flow inverts shall be sized for a minimum capacity of the 3-month ARI flow.

(b) Invert

Careful consideration shall be given to minimising the possibility of scour at the interface between the invert edge and the grassed surface of the channel. It may be necessary to provide a transition zone using a stabilisation system such as reinforced grass.

16.6 LINED AND COMPOSITE CHANNEL

16.6.1 General

Engineered channel lined with concrete, stone pitching, and rock mattresses are typical of the type of channel included in this category. Lined channels of this type should only be used for upgrading works in existing areas where:

- the flow velocity would cause scour in a grassed channel;
- the drainage reserve width is restricted;
- continual maintenance of the channel is a problem; and
- other factors dictate that a grassed channel is not practicable.

16.6.2 Geometry

The slope of the composite channel sides shall be no steeper than 1.5(H):1(V) unless designed to act as a structurally reinforced wall to withstand soil and groundwater forces.

The inverts of lined channels should have a nominal invert 'vee' of at least 10(H):1(V) or a precast 'pudu-cut' section, such that low flows remain concentrated along a single location within the channel invert (Figure 16.3). The low flow invert may be either centrally located or offset to one side of the channel. Channels lined with rock mattresses should be provided with a cast-in-situ or precast concrete low flow drain located within the channel invert. The requirements for this low flow drain are the same as those for grassed channels.

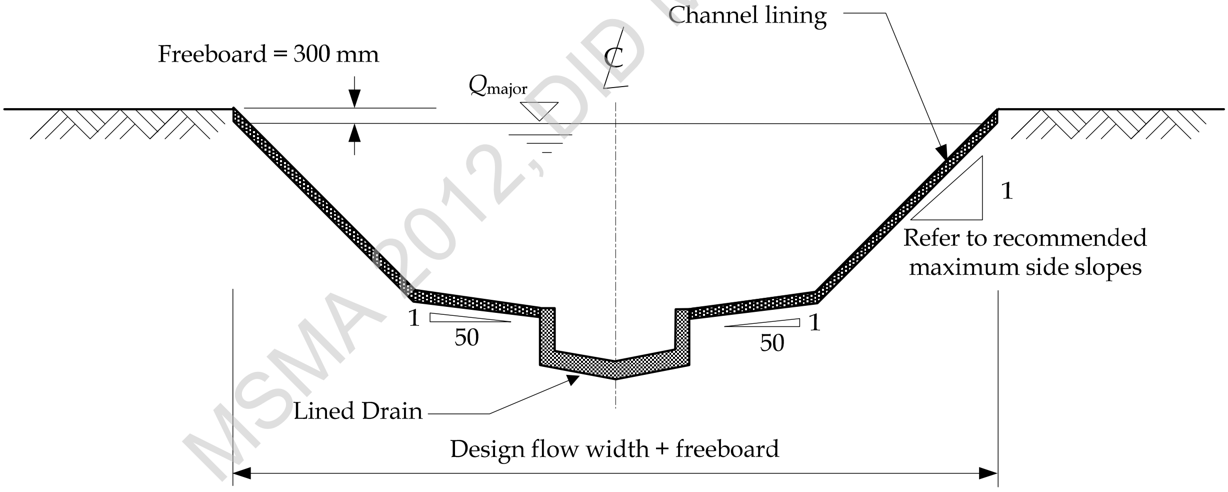


Figure 16.3: Typical Composite Channel

16.6.3 Superelevation

When flow moves around a channel bend, a rise in the water surface elevation occurs along the outer radius of the bend, whilst a corresponding lowering in the water surface elevation occurs along the inner radius of the bend. This difference in water levels is known as the superelevation, and in some cases may be an important factor in channel design (NRW, 2007).

Superelevation of the water surface must be determined at horizontal curves and the design of the channel cross-section adjusted accordingly. An approximation of the superelevation can be obtained from the following equation:

$$h = \frac{V^2 W_T}{g r_c} \tag{16.2}$$

where,

h = required superelevation (m);
V = velocity (m/s);
r_c = centreline radius of curvature (m);
WT = top width of channel (m); and
g = acceleration due to gravity (9.81 m/s^2).

16.7 EROSION AND SCOUR PROTECTION

16.7.1 General

The design average flow velocity limits specified in Section 16.3.5.2 have been selected to prevent erosion and scour of channel surfaces under normal conditions. However, engineered channels may be subject to intense local erosion or scour at obstructions (e.g. bridge piers, pipe headwalls), sudden changes in engineered channel cross-sections, drops, regions of changes in engineered channel bed materials, and other similar conditions (ACT Government, 1992). The following factors should be considered wherever significant changes in flow regime occur and appropriate measures provided to protect the engineered channel surface from local scour.

The main factors that provide favourable conditions for erosion and scour in a engineered channel are high flow velocities, particularly at shallow depths, and soft and/or fine bed materials. Velocities are higher in steep waterways, at changes in engineered channel configuration, in smooth waterways, and at higher discharges. Soil type largely determines the erosion potential of bed materials.

Local scour occurs in non-uniform flow regions where pressure forces, lift forces, and shear forces fluctuate. For example, local scour around bridge piers is caused by the vortex resulting from water piling up on the upstream edge and subsequent acceleration of flow around the nose of the pier. Local scour is a function of a combination of several of the following factors:

- slope of the engineered channel;
- characteristics of the bed materials;
- characteristics of the flood hydrograph;
- direction of the flow in relation to its depth;
- direction of the flow in relation to its structures; and
- characteristics of the transported materials.

The following locations are the most common areas where localised erosion can occur and will require careful consideration of the need for erosion protection measures (ACT Government, 1992):

- Transitions: Any changes in cross-section or changes in engineered channel lining material. Particular attention should be paid to the region immediately alongside low flow inverts;
- Bends: The outside bank of bends will be subject to higher flow velocities;
- Drain tributaries: Engineered channels usually have many small capacity tributary drain and pipe connections. Flows from these tributary connections will normally be of relatively high velocity and the angle of entry will cause turbulence in the engineered channel;
- Engineered channel tributaries: Other engineered channels entering the main channel system may cause turbulence and erosion of the engineered channel bottom and opposing bank;
- Energy dissipater structures: Changes in the flow regime will usually occur immediately upstream and downstream of drop structures and energy dissipation basins;
- Culverts: Exit velocities from culvert crossings will normally be supercritical;

- Bridges: Flow velocities around bridge piers and abutments may be higher than the engineered channel limit; and
- Localised scour or general degradation can quickly lower the bottom of a channel. Erosion protection facilities must have deep toe protection or they can fail by being undermined.

16.7.2 The Froude Number

The Froude Number (Fr) characterises the conditions in flowing water in terms of its velocity and depth. An understanding of critical flow conditions and the appreciation of Froude Numbers can assist in the design of channels to prevent erosive damage to the channel does not occur. The Froude Number provides a means for determining whether a given flow is subcritical, critical or supercritical.

In natural and grassed channels, channels become unstable when a Froude Number of 1.0 is approached. Flows at Froude numbers between 0.8 and 1.2 are unstable and unpredictable and should be avoided (UDFCD, 2008). For safe design of vegetated channels, the Froude Number of the design flow should be less than 0.8 (subcritical flow) depending on the degree of erosion resistance provided by the vegetation. Where values exceed unity it would be necessary to ensure that the channel lining had a very high degree of erosion resistance (refer to Chapter 20).

16.7.3 Protection Measures

Engineered channel protection must be provided to suit the local physical and scour characteristics. Erosion protection is required for waterway linings in reaches where the maximum permissible flow velocities or critical tractive forces are exceeded under the design storm flow conditions.

Waterway vegetation is perhaps the simplest erosion and scour control measure. However, where the flow velocities will exceed the velocities at which the vegetation is effective, other erosion protection measures will need to be considered (see Chapters 17 and 20).

16.8 DESIGN PROCEDURE

After an engineered channel type has been selected, the general procedure outlined in Figure 16.4 may be used for locating and sizing the engineered channel.

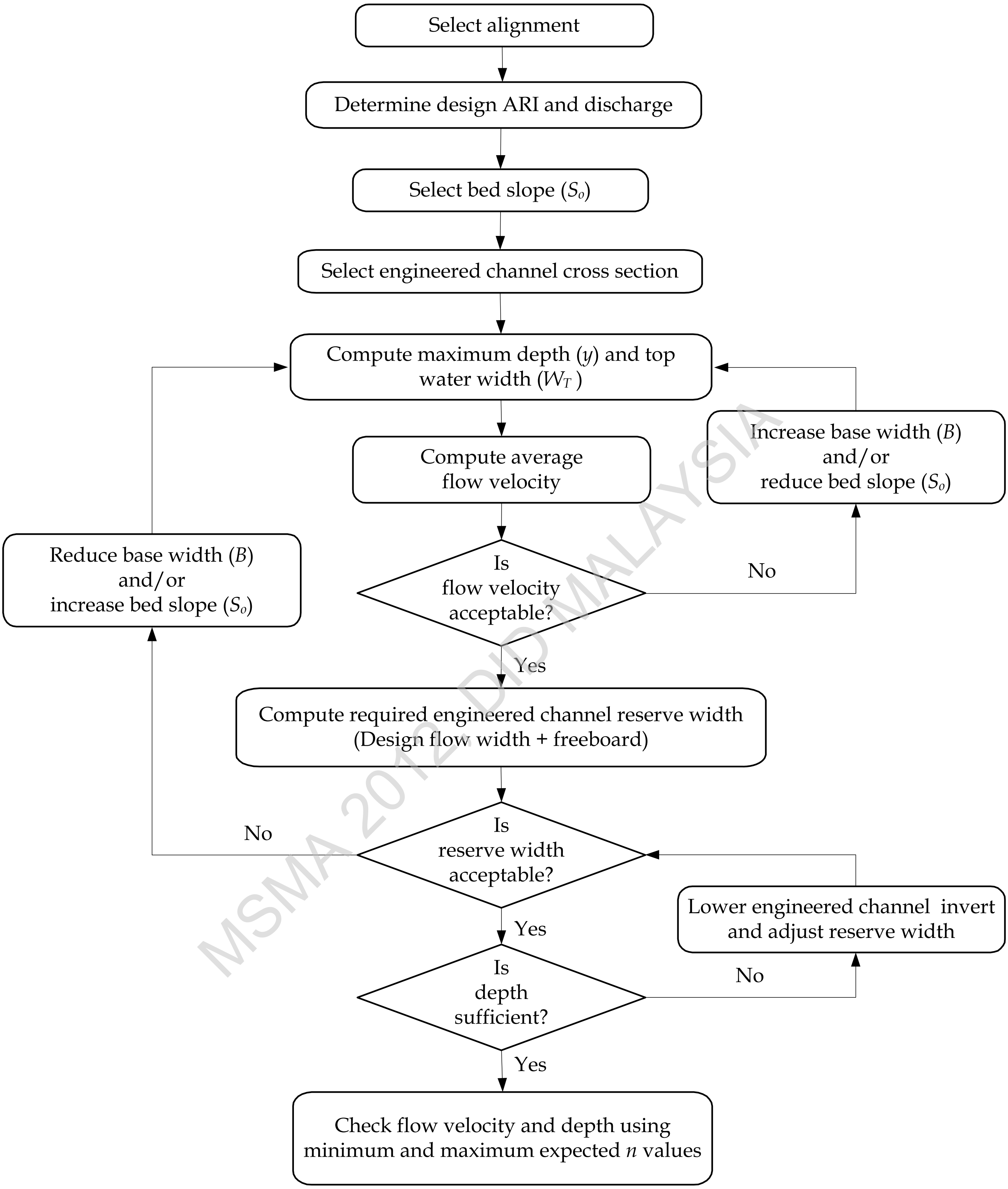


Figure 16.4: General Design Procedure for Engineered Channels

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APPENDIX 16.A EXAMPLE – GRASSED CHANNEL

Problem:

Determine the size of a grassed channel at the downstream end of a proposed 150 hectare residential area (link and terrace house) in Ipoh, Perak based on the following assumptions:

- the engineered channel is to be designed to carry the 100-year ARI flow with no freeboard;
- the post-development time of concentration t_c is estimated to be 100 minutes; and
- the engineered channel will be well maintained with an estimated design Manning's roughness coefficient of 0.035.

Solution:

<table>
<tr><th>Reference</th><th>Calculation</th><th>Output</th></tr>
<tr><td></td><td>Determine design flows for the engineered channel:</td><td></td></tr>
<tr><td>Equation 2.2</td><td>$$i=\frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$$</td><td></td></tr>
<tr><td></td><td>where:
i = the average rainfall intensity (mm/hr) for selected ARI (T) and storm duration (d);</td><td></td></tr>
<tr><td>Table 1.1</td><td>T = average recurrence interval, ARI (years); =</td><td>100-year ARI</td></tr>
<tr><td></td><td>d = storm duration (hours); $0.20 \le d \le 72$; and =</td><td>100 minutes</td></tr>
<tr><td></td><td>λ, κ, θ and η = fitting constants dependent on the raingauge location</td><td></td></tr>
<tr><td>Appendix 2.B
Table 2.B2</td><td>
<table>
<tr><th rowspan="2">Location & Station ID</th><th rowspan="2">ARI, T (years)</th><th rowspan="2">Storm duration d</th><th colspan="4">Derived Parameters</th></tr>
<tr><th>λ</th><th>κ</th><th>θ</th><th>η</th></tr>
<tr><td>Politeknik Ungku Omar (4409091)</td><td>100</td><td>100</td><td>70.238</td><td>0.164</td><td>0.288</td><td>0.872</td></tr>
</table>
</td><td></td></tr>
<tr><td></td><td>$$i=\frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}=\frac{70.238(100)^{0.164}}{\left(\left(\frac{100}{60}\right)+0.288\right)^{0.872}}$$ =</td><td>83.32 mm/h</td></tr>
<tr><td>Equation 2.3</td><td>$$Q=\frac{C.I.A}{360}$$</td><td></td></tr>
<tr><td></td><td>where: Q = peak flow (m^3/s)</td><td></td></tr>
<tr><td>Table 2.6</td><td>C = dimensionless runoff coefficient =</td><td>0.90</td></tr>
<tr><td></td><td>I = average rainfall intensity over time of concentration, t_c (mm/hr); and =</td><td>83.32 mm/h</td></tr>
<tr><td></td><td>A = drainage area (ha) =</td><td>150 ha</td></tr>
<tr><td></td><td>$$Q_{100}=\frac{0.90 \times 83.32 x 150}{360}$$ =</td><td>31.25 m^3/s</td></tr>
<tr><td>Appendix 2.B
Table 2.B2</td><td>
<table>
<tr><th rowspan="2">Location & Station ID</th><th rowspan="2">ARI, T (years)</th><th rowspan="2">Storm duration d</th><th colspan="4">Derived Parameters</th></tr>
<tr><th>λ</th><th>κ</th><th>θ</th><th>η</th></tr>
<tr><td>Politeknik Ungku Omar (4409091)</td><td>0.25</td><td>100</td><td>62.9315</td><td>0.3439</td><td>0.1703</td><td>0.8229</td></tr>
</table>
</td><td></td></tr>
</table>

Reference	Calculation	Output
	$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} = \frac{62.9315(100)^{0.3439}}{\left(\left(\frac{100}{60}\right)+0.1703\right)^{0.8229}}$	= 23.69 mm/h
	Hence, the design flows are: $Q_{low\ flow} = \frac{0.90 x 23.69 x 150}{360}$	= 8.88 m³/s
	Select grassed channel cross section:	
Table 16.6	The low flow invert cross section	= Trapezoidal
	Manning's roughness coefficient, *n*	= 0.035
	Calculate size of low flow invert:	
	Assuming:	
	Channel longitudinal slope	= 0.8% (1 in 125)
	Width, B	= 2.20 m
	Side slope, Z	= 3

Depth, *y*	Bottom width, *BW*	Side slope, *Z*	Top Width, *TW*	Area, *A*	Wet. perimeter, *P*	Hydraulic radius, *R*	Velocity, *v*	Discharge, *Q*
(m)	(m)	(m)	(m)	(m²)	(m)	(m)	(m/s)	(m³/s)
0.80	2.5	3	7.30	3.92	7.56	0.52	1.65	6.466
0.85	2.5	3	7.60	4.29	7.88	0.55	1.70	7.319
0.90	2.5	3	7.90	4.68	8.19	0.57	1.76	8.234
0.95	**2.5**	**3**	**8.20**	**5.08**	**8.51**	**0.60**	**1.81**	**9.212**

Output: >$Q_{low\ flow}$ (=9.05 m³/s)

Q = v x A

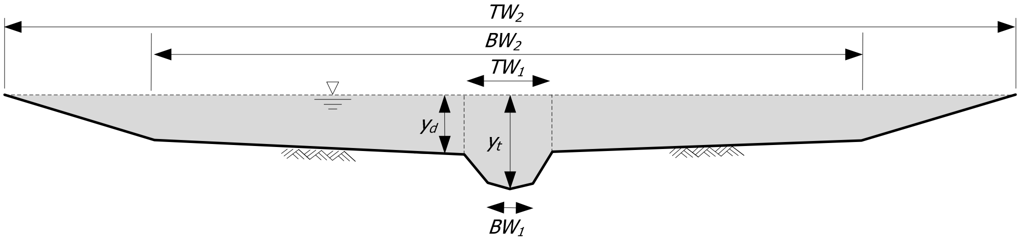
where : $v = \frac{1}{n} S_o^{1/2} R^{2/3}$;

$$A = (B + Zy)y = 2.50y + 3y^2$$

$$P = B + 2y\sqrt{1+Z^2} = 2.50 + 2y\sqrt{10}$$

	Calculation	Output
	Provided dimension of low flow invert:	
	Bottom width, *BW*	= 2.50 m
	Depth, *y*	= 0.95 m
	Top width, *TW*	= 8.20m
	Velocity, *v*	= 1.81 m/s
	Discharge, *Q*	= 9.212 m³/s
	Check flow velocity for lined drain.	(0.6<v<2 m/s); ...OK

Reference	Calculation	Output



Flow Segments

Calculate size of grassed channel section:

Table 16.6 — Manning's roughness coefficient, n = 0.035

y_t (m)	y_t-y_d (m)	TW_1 (m)	BW_2 (m)	Side slope, Z	y_d (m)	TW_2 (m)	A (m²)	P (m)	R (m)	v (m/s)	Q (m³/s)
1.40	0.95	8.20	14.20	6	0.45	19.60	12.69	19.98	0.63	1.89	23.95
1.50	0.95	8.20	14.20	6	0.55	20.80	14.71	21.20	0.69	2.00	29.46
1.60	**0.95**	**8.20**	**14.20**	**6**	**0.65**	**22.00**	**16.85**	**22.42**	**0.75**	**2.11**	**35.59**
1.90	0.95	8.20	14.20	6	0.95	25.60	23.99	26.07	0.92	2.42	57.99

Output: >Q_{100} (= 31.25 m³/s); …OK

Section 16.3.2 — Allowing a minimum freeboard = 300 mm

Provided dimension of grassed channel:

Top width, TW_2 = 25.60

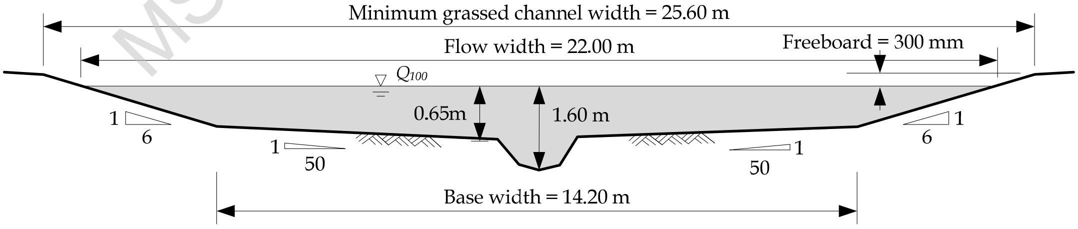
Depth, y_t = 1.90

Discharge, Q = 57.99 m³/s

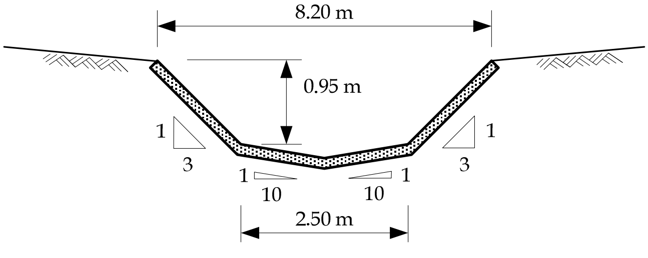
Section 16.8.2 — Check Froude Number for grassed channel:

$$Fr = \sqrt{\frac{Q^2 B}{g A^3}} = \sqrt{\frac{35.59^2(22.00)}{9.81(16.85)^3}}$$

= 0.77 (< 0.8; …OK) (Subcritical Flow)

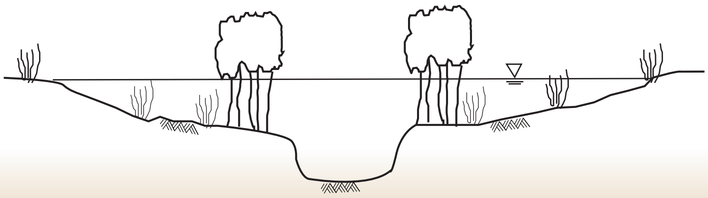


(a) Grassed Channel



(b) Low Flow Invert

Grassed Channel and Low Flow Invert Dimensions



BIOENGINEERED CHANNEL 17

CHAPTER 17 BIOENGINEERED CHANNEL

17.1 INTRODUCTION

One of the tools available for protecting stream banks and channels in an environmentally sensitive area from the negative consequences of bank erosion and sloughing is the application of bioengineering techniques. Bioengineering is the combination of biological, mechanical, and ecological concepts to control channel degradations and bank erosion through the use of vegetation or a combination of vegetation and construction materials.

17.2 RIPARIAN MANAGEMENT

Riparian vegetation plays an important role in the maintenance of stable watercourse morphology as well as in the preservation of ecological values such as terrestrial and avian wildlife habitat. For these reasons the preservation riparian zones along watercourses needs to be a part of stormwater management planning

17.2.1 Approach

The principal approaches to managing riparian and floodplain vegetation are retention and replanting with appropriate native species that are indigenous to the area. In practical terms a riparian zone at either side of the channel should be reserved for retention of exiting vegetation or replanting when it is disturbed. The width of this zone is influenced by a number of factors including channel width, the nature of existing development, the importance of local flora and fauna bio-diversity and the need to maintain wildlife corridors. The nature and size of riparian reserves should be determined early during the planning process.

17.2.2 Objectives

a) Vegetation Retention

The most important principle is to maintain existing indigenous riparian vegetation where possible.

b) Vegetation Replacement

Restoration of riparian zones can generally be undertaken by planting indigenous vegetation. If indigenous vegetation is scarce or limited in numbers, appropriate species can be imported from nearby catchments with similar climatic and soil conditions. In some circumstances, changed conditions such as increased wind exposure, temperature fluctuations (due to removal of adjacent vegetation) and loss of topsoil may preclude the use of indigenous vegetation. In these cases the use of non-invasive and easy to maintain species from other Malaysian catchments may be necessary at least in the initial stages. A diversity of species is recommended to replicate the pre-development condition of the riparian zone and to encourage terrestrial and aquatic bio-diversity.

17.3 RIPARIAN ZONES

The riparian zone (Figure 17.1) is the area of land (including floodplains) adjacent to a watercourse. Riparian vegetation can include emergent aquatic and semi-aquatic plants, terrestrial over storey (canopy) and terrestrial under storey (cover). The riparian zones contribute to the ecological value and geomorphologic stability of a watercourse through a number of processes. These values and processes are discussed in the following sections.

17.3.1 Bank Stability and Channel Integrity

Vegetation can exert a significant control over fluvial processes through two main mechanisms: resistance to flow (through the vegetation increasing roughness) and bank strength (through the binding of soil by root systems). Bank vegetation disturbances can include unrestricted access by people and animals. Human disturbance in riparian zone includes removal of vegetation or logging, agricultural development or cultivation, and urban development. These activities can lead to bank destabilisation, soil compaction and higher overbank flow velocities.

Examples of bank stability project for stream channel using bioengineering techniques are shown in Table 17.1.

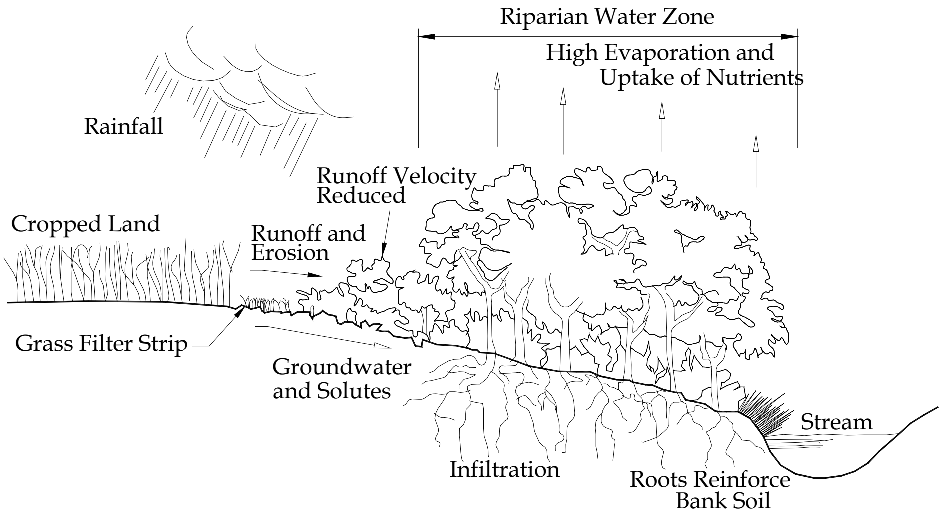


Figure 17.1: Example of Riparian Buffer Zone (DID, 2009)

Table 17.1: Bioengineering Technique Projects

Bioengineering Techniques	Bank Stability Projects	
	During Installation	After Installation
TRM Reinforced Grass		
Sand Filled Mattress Reinforced Grass		
Gabion Mattress Reinforced Grass		

17.3.2 Habitat Values

Broadly, riparian reserves can be described as having a number of habitat functions:

- habitat island. This is a zone that has an adequate width of retention of natural vegetation to be able to permanently sustain communities of natural terrestrial fauna;
- wildlife corridor. This is a zone that has an adequate width of retention of natural vegetation to facilitate the movement of natural fauna from one "habitat island" to the next;
- food source. Riparian vegetation contributes to the ecology of freshwater communities by providing food in the form of fallen organic material and fallen insects and invertebrates (Figure 17.2); and
- aquatic habitat. Trees and branches that fall into the stream and exposed tree roots provide shelter from high velocity flow, predators and sunlight.

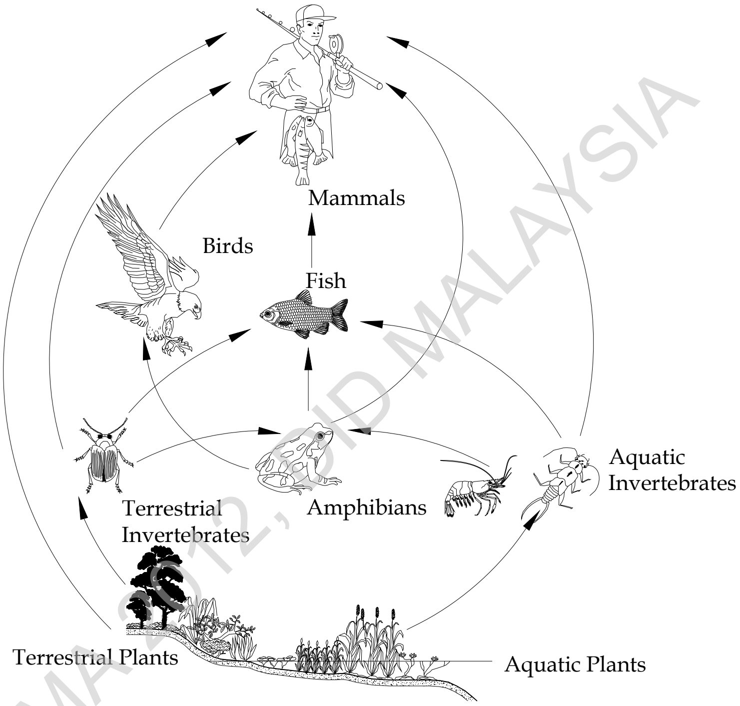


Figure 17.2: Riparian Food Chain

17.3.3 Importance of Diverse Natural Vegetation

The basis for maintaining a natural food chain is natural vegetation. Maintaining natural vegetation is also important in maintaining appropriate levels of shading for the watercourse. This directly affects water temperature, the rate of photosynthesis and shelter from predators such as birds.

17.3.4 Buffer Filters

The riparian zone provides a buffer strip between land use and watercourses. This buffer can provide a filter zone for sediments, nutrients and to some degree pollutants. Buffer zone width as well as the type and magnitude of pollutant loads will determine the long-term effectiveness of the buffer. The filter buffer works by slowing surface sheet runoff, intercepting sediment particles and trapping them around stems and roots. Nutrients in sediments can be taken up into the riparian vegetation, whilst non-organic pollutants may be bound into the soil system. The effectiveness of riparian filter buffers can also depend on site-specific factors such as soil type, permeability, vegetation type and density, and terrain slope. Without such function, sediments and pollutant loads can have long term degrading effects on the overall well being of the watercourse.

17.3.5 Buffer Width

The required width of riparian vegetation buffers zones is dependent on all of the factors outlined above and is a major determinant in the effectiveness of the buffer. The primary function of the buffer depends on its width. Minimum widths suggested for different functions are shows in Table 17.2.

Table 17.2: Recommended Widths for Riparian Buffer Zones (Fischer and Fischenich, 2000; Price, Lovett and Lovett, 2005)

Management Objectives	Recommended Width	Remarks
Protect Water Quality	5 – 30 m	Low slope (0-10%) – Dense grassy or herbaceous buffers intercept runoff, trap sediments, remove pollutants and promote ground water recharge. Moderate slope (10-20%) – most filtering occur within the first 10 m. Greater widths are required for: steeper slopes; where the buffer comprise mainly trees and shrubs; where soils have low permeability; or where non-point source pollutions are significant.
Reduce Bank Erosion	10 – 20 m	Riparian vegetation enhances bank stability by moderating soil moisture and providing tensile strength through the root system. Greater width may be necessary where there is active bank erosion.
Provide Food Input/Aquatic Habitat and Maintain Light/Temperature Level	5 – 10 m	Fallen leaves, twigs and branches are important sources of nutrients and aquatic habitats. Native riparian vegetation provides shade which is crucial for maintaining natural levels of light intensity and temperature for healthy in-stream ecosystem.
Provide Terrestrial Habitat	30 – 500 m	Buffers comprising diverse stands of shrubs and trees provide food and shelter for a wide range of riparian and aquatic wildlife.
Enable Agriculture Production	10 – 30 m	Riparian land is often a highly productive part of the landscape. It can be managed directly for commercial products such as timber or honey, or indirectly so that it improves production by providing habitat for pollinators or acting as windbreak for commercial crops and domestic stock. When acting as windbreak, the length should be at least 20 times the width.
Downstream Flood Attenuation	20 – 150 m	Riparian buffers promote floodplain storage through backwater effects. They increase water flow time by interception, resulting in reduced flood peaks.

The others criteria to be considered are:

- the slope of the site and the erodibility of the soils are the factors having the largest influence in determining buffer widths for the purpose of sediment control;
- site specific investigations will need to take into account factors such as channel and floodplain geometry, vegetation, flora and fauna requirements, adjacent land uses and soil erodibility; and
- indigenous species will inhibit weed growth, although weed management may be required until the indigenous vegetation is established and windbreak plantings may also be necessary to assist with riparian vegetation establishment in exposed areas.

17.3.6 Flow Management

Management of flow velocities and flow depths, in addition to the frequency and extent of inundation will enhance the viability of riparian vegetation. Generally, natural vegetation has adapted to the natural cycles of runoff and the frequency of inundation. The alterations to the hydrological regime leads to the displacement of riparian vegetation from its natural level on the stream bank and may lead to erosion and reduced bank stability.

17.4 STREAM PROCESSES

a) Introduction

Stream management problems may arise from either underlying processes of change in the river system or localised perturbations. Stream instability can be the result of natural processes or human activities. It is therefore important to identify the dominant stream processes present if stream management strategies are to be implemented, which are appropriate and unlikely to cause adverse responses elsewhere in the system.

Stream instability can be initiated by natural and human induced causes such as:

- long term alteration to the hydrologic and/or sediment regime;
- a catastrophic flood or sequence of major floods;
- crossing of a geomorphic threshold; and
- direct or indirect human interference.

b) Bank Erosion Processes

Bank erosion can be the effect of morphological processes such as:

- meander processes;
- channel avulsion; and
- bed degradation, or
- combination of the above, or
- the product of localised processes unrelated to the more general morphological changes in the river system.

The mechanism of bank failure will generally involve more than one failure modes. It normally involves mass failure such as collapse caused by undermining and slumping (sloughing), rotational or slip circle failure, and initial detachment of individual particles involving attrition or fretting. Other modes of failure include erosion by overland flow entering or leaving the main channel creating a headward erosion gully and tunnel erosion (piping failure).

c) Causes of Bank Erosion

Typical factors, which may contribute to bank erosion, include:

- altered water-sediment ratio in the watercourse;
- altered flow patterns including tidal currents and heights;
- general or local steam bed degradation (i.e., lowering) resulting from altered flows;
- changes in stream flow velocities;
- loss of bank vegetation;
- wave action; and

- soil pore water pressure.

Some typical failure modes are illustrated in Figures 17.3 and 17.4.

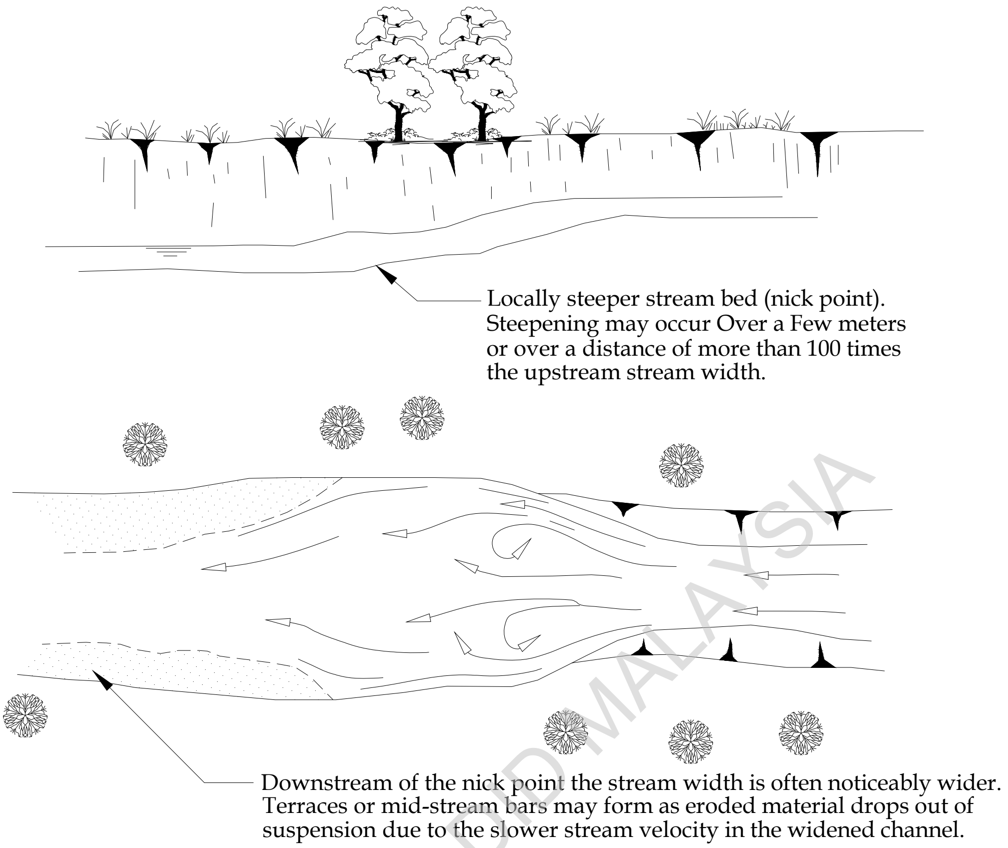
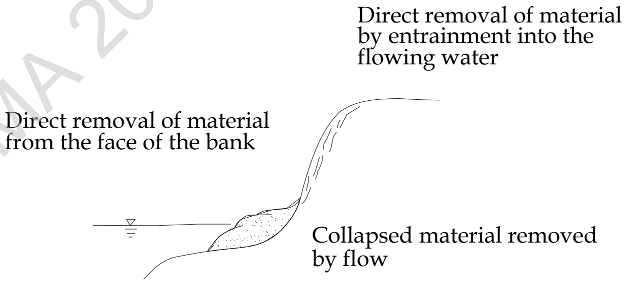
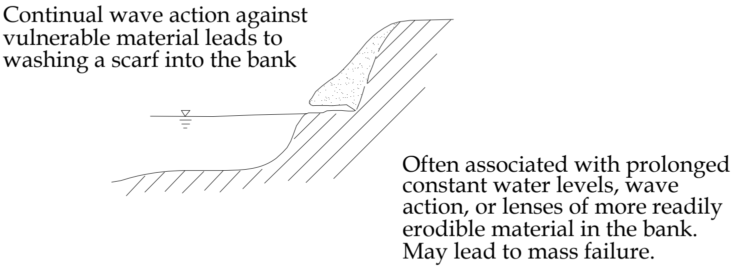


Figure 17.3: Typical Characteristics of Bed Scour



(a) Lateral Bank Erosion



(b) Bank Undercutting

Figure 17.4: Bank Failures by Attrition and Fretting

17.5 WATERCOURSE MANAGEMENT TYPES

Erosion mechanisms include bed scour, bank attrition and bank collapse caused by undermining, piping, slumping, or rotational failure.

17.5.1 Bed Scour (Degradation)

a) Description

A deepening of the stream bed that propagates in an upstream direction. The deepening moves upstream by an advancing erosion head that may take the form of a small waterfall or locally steeper section of stream bed (head cut or nick point) (Figure 17.3). In some cases the location of the erosion head is not easy to identify.

b) Indicators/ Symptoms

- a visible waterfall;
- steep section of stream bed at head of scour hole;
- bell shaped scour hole immediately downstream of erosion head;
- exposure of foundations on structures such as bridge piers and culverts;
- steep raw banks caused by lowering of the bed and the consequential collapse of the adjacent banks;
- relatively sudden decrease in bank height in upstream direction;
- choking of downstream reaches by sediment deposits;
- steep banks; and
- downstream flooding through loss of flood storage resulting from concentration of streamflow within the incised stream channel.

c) Possible Causes

- Clearing and/or urbanisation of the catchment resulting in an increase in flow entering the stream at the original head cut location, or
- Direct human modification to the stream such as:
  - channelisation works causing a sudden increase in mean flow velocity which has a knock on upstream effect;
  - in-stream gravel and sand mining operations;
  - de-snagging; and
  - unstable drop inlets upstream of culverts.

17.5.2 Bank Attrition

a) Description

Bank attrition is the direct removal of material from the face and toe of the bank by entrainment into the stream flow (Figure 17.4). It may be caused by channel flow and/or runoff from the surrounding area flowing down the face of the bank.

b) Indicators/ Symptoms

- steepening of the stream banks;
- absence of bank vegetation;

- lateral gullies and/or columnisation (this is a special case, refer to Section 17.5.7);
- widening of the stream;
- meander migration; and
- downstream sedimentation.

c) Possible Causes

High velocity flow in contact with the bank which maybe the result of one or more of the following:

- increase in the rate and volume of flow entering the stream caused by land clearing and/or urbanisation of the catchment;
- obstructions in the stream channel such as fallen trees, dumped material or bridge piers; and
- a lack of vegetation on the bank due to shading, trampling from animals or humans, wave action, or direct removal by human activities.

17.5.3 Bank Undermining (Fretting)

a) Description

Fretting is the direct removal of material from an exposed underlying vulnerable soil layer by the continual movement of water (flow or waves) against the layer (Figure 17.4). An erosion scarf is formed which can lead to mass failure of the overhanging bank material.

b) Indicators/ Symptoms

- overhanging bank; and
- a sharp steeping of the bank with a near vertical face close to the waterline.

c) Possible Causes

- increase in water level that makes continual or frequent contact with an exposed and highly erodible soil layer; and
- increased wave action due to boating, or change in the prevailing wind direction due to the removal or addition of nearby obstructions including trees or buildings.

17.5.4 Piping

a) Description

Erosion tunnels or pipes are formed where surface flows seep into the ground behind the bank and daylight at the bank face. The seepage flows dissolve and/or dislodge soil particles from the soil matrix and transport them to the face of the bank where they are removed by stream flow. On occasions, piping can be initialised by animal burrows or by decaying roots of dead trees, which leaves subsurface cavities.

b) Indicators/ Symptoms

- sink holes on the floodplain and especially when close to the stream bank;
- trenches or narrow line(s) of collapsed material extending laterally across the floodplain;
- concentrated seepage flows appearing on the bank; and
- burrows or tunnels on the bank.

c) Possible Causes

- poorly drained floodplain areas;

- animal burrows;
- decaying roots of dead trees close to the top of bank; and
- adversely orientated dispersive soil layer.

17.5.5 Bank Slumping

a) Description

Bank slumping (Figure 17.5) is the mass failure of the bank material due to either:

- deepening of the stream bed at the toe of the bank resulting in the bank becoming unstable and slumping into the stream under its own weight (or under some surcharge weight on the top of the bank); and
- high pore water pressure in the bank material not being balanced by adjacent hydrostatic pressures in the stream. The high pore water pressure weakens the structure of the bank material causing it to slump into the stream.

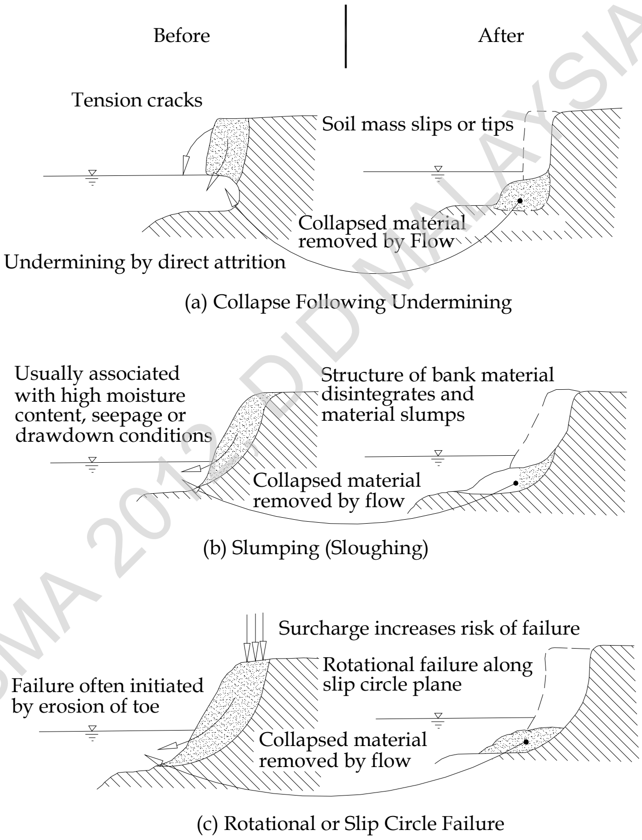


Figure 17.5: Mass Bank Failure Modes

b) Indicators/ Symptoms

- lateral movement or widening of the stream banks;
- tension cracks in overbank material running parallel to top of the bank;
- large dumps of vegetated bank slumped below the obvious original location of the vegetation;
- significant groundwater seepage from the face of the stream bank;
- near vertical, unvegetated banks; and
- the presence of mature vegetation types not normally associated with regular and prolonged inundation at the toe of the bank.

c) Possible Causes

- high velocity streamflow (often on the outside of stream bends) resulting in bed scour at the toe of the bank. This can be made worse by land clearing and/or urbanisation of the catchment;
- rapid drawdown of stream water level following a prolonged period of high flows, which have saturated the bank material. This is not common in urban streams and is more prevalent in regulated rural streams and irrigation channels;
- surcharge loading on the top of the bank; and
- lack of binding bank vegetation.

17.5.6 Bank Rotational Failure

a) Description

Bank rotational failure (also known as slip circle failure) is the failure of an embankment along a curved surface, which approximates to the plane of least resistance within the soil mass (Figure 17.5).

b) Indicators/ Symptoms

- block slippage of the bank exposing a curved failure surface on the bank. The slumped material may or may not be present depending on whether stream flows have removed the material from the base of the bank;
- lateral movement or widening of the stream banks;
- absence of bank vegetation; and
- the presence of mature vegetation types not normally associated with regular and prolonged inundation at the toe of the bank.

c) Possible Causes

- deepening of the stream bed at the toe of the bank resulting in the bank becoming unstable;
- surcharge weight on the top of the bank (vehicles, buildings);
- high pore water pressure in the bank material not being balanced by adjacent hydrostatic pressures in the stream; and
- removal of binding vegetation.

17.5.7 Lateral Bank Erosion

a) Description

Lateral bank erosion is most prevalent at locations where runoff from adjacent land is concentrated within culverts, roadways and drainage lines and depressions prior to entering the main stream. The erosion takes the form of an upstream progressing erosion head that propagates laterally from the main stream channel. Where the runoff reaches the top of bank as sheet flow the bank may display a columnar or vertical fold formation (Figure 17.4).

b) Indicators/ Symptoms

- ephemeral gullies or rills entering the river bank above the normal water level;
- columns or vertical erosion folds in the bank; and
- tension cracks in the bank and crumbling of the upper soil horizons.

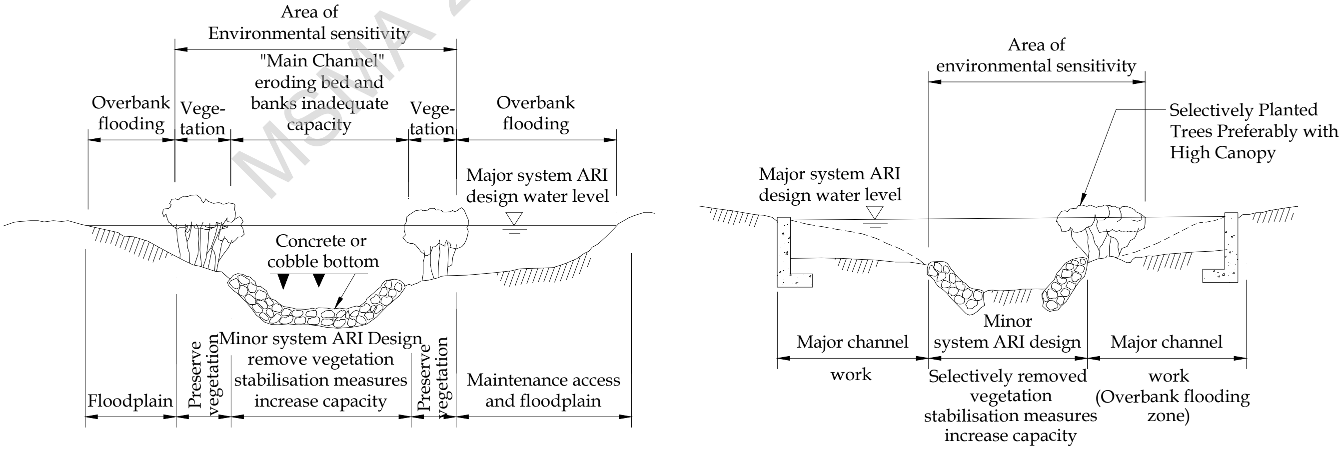
c) Possible Causes

- culverts, roadways, drainage swales etc. discharging concentrated stormwater runoffs into the stream channel without proper outlet protection measures;
- redirection of the stream channel due to urbanisation of the catchment and/or clearing of fringing bank vegetation (particularly where this occurs on the inside of a meander);
- erosion prone material at the top of the bank such as dispersive clays, soft silts, and unconsolidated sands and gravels; and
- lack of vegetation (especially ground cover) on the bank.

17.6 WATERCOURSE MANAGEMENT TECHNIQUES

The techniques listed in Table 17.3 are not an exhaustive list but represent the most commonly applied techniques suitable for urban streams and rivers. However not all techniques described would be suitable in an urban environment due to site specific constraints. Public safety, aesthetics, and cost will often determine the adoption of a technique.

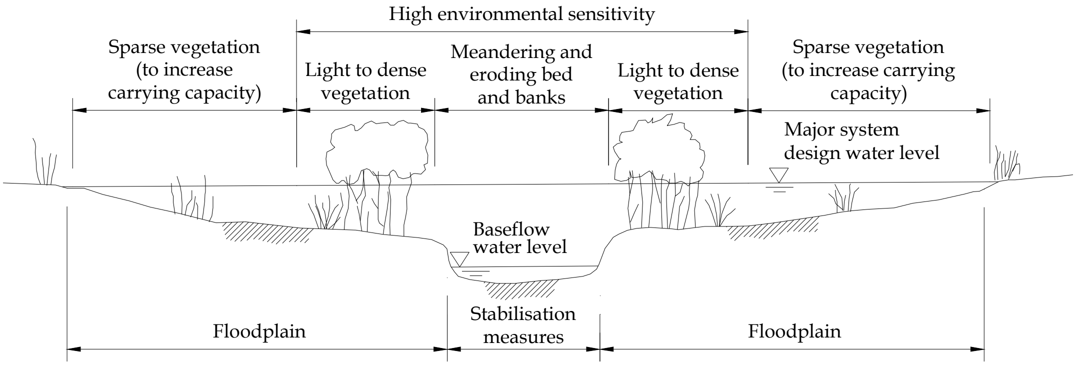
Figures 17.6 and 17.7 provide examples of slightly improved natural channels. Stabilisation measures in Figure 17.6(a) include check structures, riprap, minor grading, and short sections of retaining walls. In general, little or no channel capacity improvements are included. In Figure 17.6(b), channel capacity has been increased to lower or confine the design storm flow by excavating outside of the environmentally sensitive area and constructing retaining walls. Figure 17.7 shows possible drainage improvements for composite channels. Stabilisation measures in Figure 17.7(a) include check structures, riprap, grading, and retaining walls. Improvements to the main channel increase capacity for minor flood flows and may confine or reduce the depth of the design flood. In Figure 17.7(b), the main channel area has been left undisturbed (i.e. that area containing the base flow plus the immediate vegetation area) and the overbank conveyance capabilities improved by excavating the floodplain area. This 'improved' natural channel has increased capacity to safely convey the major system design flow. Provision should be provided for maintenance access to the channel. In stabilising the main channel and overbank, vegetation should be retained as much as possible to meet the objectives of enhancing stability and capacity. Multiple uses of the overbank flooding area should be encouraged, especially if the main channel capacity is substantial, i.e. overbank flooding is infrequent.



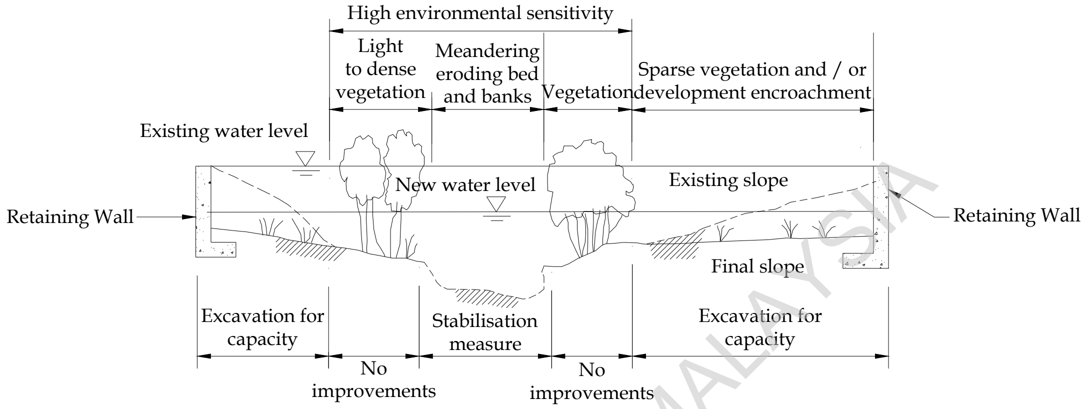
(a) With Stabilisation Measures

(b) With Increased Capacity

Figure 17.6: Typical Channel (ASCE, 1992)



(a) With Stabilisation Measures



(b) With Increased Capacity

Figure 17.7: Composite Channel (ASCE, 1992)

17.7 CHANNEL DESIGN CONSIDERATION

The design of 'natural channels' involves the creation of channels with the attributes of natural watercourses pertinent to the location within the watershed and should be based on a sound understanding of fluvial geomorphic principles. Guidelines on natural channel design methodology are provided by OMNR (1994). The suggested design steps are:

a) Define Design Objectives - Identify the objectives to be met for the design. Multiple objectives regarding conveying flood flows, aquatic habitat, recreation, aesthetics and maintenance may exist and frequently will appear to be in conflict.
b) Define Existing Conditions - The existing flow regime, sediment load, channel, valley and catchment conditions can be obtained or estimated.
c) Define the Expected Conditions - The expected flow, sediment loading and channel slope conditions can be estimated or calculated.
d) Identify Inconsistencies - Any inconsistencies between the existing and expected conditions should be identified and resolved.
e) Design Parameters - The design parameters for the channel for unconstrained design conditions should be developed to satisfy the objectives.
f) Identify Constraints - Constraints to the channel design are to be identified. Some of the more common constraints include funding, property boundaries, roads, services, flooding, and stakeholders or management disputes.
g) Identify Compromises - Compromises may be required to determine the optimum design conditions by considering all the site constraints.
h) Develop Design - The design of the channel system should emphasise on creating a channel in dynamic equilibrium with appropriate habitat features.
i) Evaluate Design - The resulting design should be compared to the optimum design and the extent of any discrepancies (there are usually some) are to be identified and assessed as to their importance in achieving the overall design objectives.

Table 17.3: Watercourse Management Techniques Suitable for Urban Streams and Rivers

Techniques	Application	Limitation
Bank armouring • Rock rip-rap	Direct protection against erosion	Requires a supply of hard sound rock.
Bank armouring • Articulated concrete block mattress	Provides protection and stability to eroding banks. Suitable for a range of bank conditions including fretting and direct attrition.	a) Toe apron requires anchoring. b) Where the stream is prone to bed scour an extensive toe apron is required.
Bank armouring • Rock filled wire baskets and mattresses	Provides protection and stability to eroding banks.	Usually fail by wire breakage due to high sediment loads carried by stream or by vandalism or by undermining.
Bank armouring • Brushing	Provides bank protection for a limited time to enable "permanent" vegetation to become established on the bank. The technique is effective against fretting and attrition erosion. It may also contribute to lowering the risk of mass failure by reducing the risk of material being removed from the toe of the bank.	Only provides short term protection.
Bank stabilisation • Battering	Reduce public safety hazard caused by steep banks, reduce erosion hazard caused by fast flowing overland flow, increase bank stability against rotational failure or mass failure, and create conducive environment for vegetation establishment.	Land take behind the bank line may be required if flat slopes are to be applied.
Bank stabilisation • Reinforced vegetation	Act as a separating layer between river flows and an eroding bank. Often combined with bank battering.	Limited effectiveness below normal river level.
Bank stabilisation • Retaining and training walls	Normally used as an alignment training technique but also provides protection and stability to eroding banks. Suitable for bank conditions involving: a) fretting; and b) direct attrition.	a) Where the stream is prone to bed scour the wall may be de-stabilised by undermining. b) Pile driving equipment may be required.
Bank stabilisation • Bio-reinforced embankments	Provides bank protection against undermining, piping, rotational, and slumping failure modes.	Requires a supply of suitable vegetative material. Toe scour may occur especially where the reinforced bank is terminated above the low water line.
Bank stabilisation • Reinforced earth proprietary products	Used to re-establish an eroded river bank or to reinforce an existing bank.	Requires a facing to limit the risk of continued scour.
Grade control structures • Check weirs	Used to reduce the effective hydraulic grade and control stream bed degradation (deepening) by promoting controlled sedimentation upstream of the weir. When the upstream ponding area is full of sediment the check weir behaves in the same manner as a Rock Chute.	Maybe subject to damage under certain depths of inundation. Disturbance of the bank is necessary to anchor the weir and prevent outflanking.
Land and water management • Fish refuges	Used to create variation in fish habitat that provides a variation in water temperature and velocity as well as protection from predators.	Stream must be sufficiently wide if jetties or boardwalks are contemplated. A good fishing location may be inadvertently created if there is a high level of people access along the stream.

17.7.1 Developing a Channel Design

Every watercourse is uniquely defined by the catchment hydrology, geology and soils, climate, vegetation, landuse, stream use, and its geological age (stream maturity). In designing an alluvial channel consideration should be given to the following criteria.

a) Planform

This refers to the shape or stream configuration when viewed on a plan. It covers characteristics such as stream sinuosity (a measure of meander shape, size and frequency), meander length and amplitude, channel pattern (straight, multi-channel, braided), and presence of ox-bow lakes, meander cutoffs, etc. An examination of a stream's planform can give an indication of whether the meanders are migrating, increasing or decreasing, or whether different reaches of the stream are aggrading (areas of deposition) or degrading (eroding).

b) Bedform

Bedforms can provide important clues to the stream processes that are taking place. The presence of recently formed or growing mid-stream bar(s) will generally indicate an area of deposition, which may have ramifications on the stability of the banks opposite the bars. The presence of pools and riffles will usually conform to a natural frequency of occurrence along a reach. In sand bed streams they are hardly noticeable, often only being defined by regular alternating deposits of coarser and finer bed deposits.

c) Flow resistance

This design characteristic influences the velocity profile both vertically and horizontally across a section. It is affected by bedform, bed and bank material, vegetation and natural or artificial obstructions in the channel or on the floodplain. The removal of bank vegetation will usually lower the surface resistance to flow thus increasing the near bank flow velocity. If the velocity increases above the scour threshold value for the bank material erosion will occur.

d) Stream slope

This is a measure of the longitudinal slope of the stream thalweg (line traced by the lowest point on successive cross sections). An examination of the stream slope can assist in identifying any sudden changes in slope, which may indicate the presence of stream instability due to bridge, dam or other in-stream structure. The causes of slope change can be natural or artificial (i.e. bridges, dams, or other in-stream structure or activity such as gravel or sand extraction). Changes in slope may be sudden (i.e. a waterfall or riffle) or gradual and only noticeable over a considerable distance.

e) Stream width and depth

When stream width is usually considered together with depth and the ratio of the two dimensions can provide further clues as to the dominant stream processes in the reach. In an alluvial stream a high width to depth ratio will often indicate that the stream is in a deposition stage where the load carrying capacity of the stream is greater than the sediment input at the top of the reach. Urban watercourses are often relatively deep and narrow which is a reflection on the increased water supply following urbanisation of the catchment.

f) Vegetation

When specifying the vegetation for a channel design an assessment of the height and extent of flooding for a range of flood frequencies and durations should be made. The plants should be selected according to the expected flooding regime, light conditions and soil types present. Unless the stream bank is particularly stable, care should be exercised when planting large trees close to the top of the bank where they maybe subject to undermining from stream flow or high winds which may cause them to fall into the stream and expose the bank. Natural vegetation is often unable to cope with the expected hydraulic loading and consideration can be given to reinforcing the vegetation with either temporary or permanent matting or other proprietary products.

17.7.2 Stable Channel Design

The stability of a channel or the suitability of various channel linings can be determined by first calculating both the mean velocity and tractive stress. Allowable tractive stresses for various types of soil, linings, ground covers, and stabilization measures including soil bioengineering treatments, are listed in Table 17.4. Additionally, product literatures from manufacturers can provide information on allowable tractive stresses or velocities for various types of erosion control products. Table 17.5 shows the factors influencing erosion. A general procedure for the application of information presented by Fischenich (2001) is outlined in the following paragraphs.

Step 1 - Estimate Mean Hydraulic Conditions

Flow of water in a channel is governed by the discharge, hydraulic gradient, channel geometry, and roughness coefficient. This functional relationship is most frequently evaluated using normal depth or backwater computations that take into account principles of linear momentum conservation. The latter is preferable because it accounts for variations in momentum slope, which is directly related to shear stress.

Step 2 - Estimate Local/Instantaneous Flow Conditions

The computed values for velocity and shear stress may be adjusted to account for local variability and instantaneous values higher than mean. A number of procedures are available to serve this purpose. Most commonly applied are empirical methods based upon channel form and irregularity. For straight channels, the local maximum shear stress can be calculated from the following simple equation:

$$\tau_{max} = 1.5\tau \tag{17.1}$$

For sinuous channels, the maximum shear stress should be determined as a function of the planform characteristics using Equation 17.2:

$$\tau_{max} = 2.65\tau \left(\frac{R_c}{W}\right)^{-0.5} \tag{17.2}$$

where,

R_c = Radius of curvature (m); and
W = Top width of the channel (m).

Equations 17.1 and 17.2 adjust for the spatial distribution of shear stress; however, temporal maximum in turbulent flows can be 10 to 20 percent higher, so an adjustment to account for instantaneous maximum should be added as well. A factor of 1.15 is usually applied.

Step 3 - Determine Existing Stability

Existing stability should be assessed by comparing estimates of local and instantaneous shear and velocity to values presented in Table 17.4. Both the underlying soil and the soil or vegetation condition should be assessed. If the existing conditions are deemed stable and are in consonance with other project objectives, then no further action is required. Otherwise, proceed to step 4.

Step 4 - Select Channel Lining Material

If existing conditions are unstable, or if a different material is needed along the channel perimeter to meet project objectives, a lining material or stabilization measure should be selected from Table 17.4, using the threshold values as a guideline in the selection. Only material with a threshold exceeding the predicted value should be selected. The other project objectives can also be used at this point to help select from among the available alternatives. Fischenich and Allen (2000) characterize attributes of various protection measures to help in the selection.

Table 17.4: Permissible Shear and Velocity for Selected Lining Materials (Fischenich, 2001)

Boundary Category	Boundary Type	Permissible Shear Stress (N/m^2)	Permissible Velocity (m/s)	Citation (s)
Soils	Fine colloidal sand	0.96 – 1.44	0.46	A
	Sandy loam (noncolloidal)	1.44 – 1.92	0.53	A
	Alluvial silt (noncolloidal)	2.15 – 2.39	0.61	A
	Silty loam (noncolloidal)	2.15 – 2.39	0.53 – 0.69	A
	Firm loam	3.59	0.76	A
	Fine gravels	3.59	0.76	A
	Stiff clay	12.45	0.91 – 1.37	A, F
	Alluvial silt (colloidal)	12.45	1.14	A
	Graded loam to cobbles	18.19	1.14	A
	Graded silts to cobbles	20.59	1.22	A
	Shales and hardpan	32.08	1.83	A
Gravel/Coble	25 mm	15.80	0.76 – 1.52	A
	50 mm	32.08	0.91 – 1.83	A
	150 mm	95.76	1.22 – 2.29	A
	300 mm	191.52	1.68 – 3.66	A
Vegetation	Class A turf	177.16	1.83 – 2.44	E, N
	Class B turf	100.55	1.22 – 2.13	E, N
	Class C turf	47.88	1.07	E, N
	Long native grasses	57.46 – 81.40	1.22 – 1.83	G, H, L, N
	Short native and bunch grass	33.52 – 45.49	0.91 – 1.22	G, H, L, N
	Reed plantings	4.79 – 28.73	N/A	E,N
	Hardwood tree plantings	19.63 – 119.70	N/A	E,N
Temporary Degradable RECPs	Jute net	21.55	0.30 – 0.76	E, H, M
	Straw with net	71.82 – 79.00	0.30 – 0.91	E, H, M
	Coconut fiber with net	107.73	0.91 – 1.22	E, M
	Fiberglass roving	95.76	0.76 – 2.13	E, H, M
Non-Degradable RECPs	Unvegetated	143.64	1.52 – 2.13	E, G, M
	Partially established	191.52 – 287.28	2.29 – 4.57	E, G, M
	Fully vegetated	383.04	2.44 – 6.40	F, L, M
Riprap	d_{50} = 150 mm	119.70	1.52 – 3.05	H
	d_{50} = 225 mm	181.94	2.13 – 3.35	H
	d_{50} = 300 mm	244.19	1.52 – 3.96	H
	d_{50} = 450 mm	363.89	1.68 – 4.88	H
	d_{50} = 600 mm	483.59	4.27 – 5.49	E
Soil Bioengineering	Wattles	9.58 – 47.88	0.91	C, I, J, M
	Reed fascine	28.73 – 59.85	1.52	E
	Coir roll	143.64 – 239.40	2.44	E, M, N
	Vegetated coir mat	191.52 – 383.04	2.90	E, M, N
	Live brush mattress (initial)	19.15 – 196.31	1.22	B, E, I
	Live brush mattress (grown)	186.73 – 392.62	3.66	B, C, E, I, N
	Brush layering (initial/grown)	19.15 – 299.25	3.66	E, I, N
	Live fascine	59.85 – 148.43	1.83 – 2.44	C, E, I, J
	Live willow stakes	100.55 – 148.43	0.91 – 3.05	E, N, O
Hard Surfacing	Gabions	478.80	4.27 – 5.79	D
	Concrete	598.50	> 5.49	H

[1] Ranges of values generally reflect multiple sources of data or different testing conditions.

A. Chang, H.H. (1988).
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E. Gray, D.H., and Sotir, R.B. (1996).
F. Julien, P.Y. (1995).
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K. Sprague, C.J. (1999).
L. Temple, D.M. (1980).
M. TXDOT (1999).
N. Data from Author (2001).
O. USACE (1997).

Table 17.5: Factors Influencing Erosion (Fischenich, 2001)

Factor	Relevant characteristics
Flow properties	Magnitude, frequency and variability of stream discharge; Magnitude and distribution of velocity and shear stress; Degree of turbulence
Sediment composition	Sediment size, gradation, cohesion and stratification
Climate	Rainfall amount, intensity and duration; Frequency and duration of freezing
Subsurface conditions	Seepage forces; Piping; Soil moisture levels
Channel geometry	Width and depth of channel; Height and angle of bank; Bend curvature
Biology	Vegetation type, density and root character; Burrows
Anthropogenic factors	Urbanization, flood control, boating, irrigation

Step 5 - Recompute Flow Values

Resistance values in the hydraulic computations should be adjusted to reflect the selected channel lining, and hydraulic condition should be recalculated for the channel. At this point, reach or section averaged hydraulic conditions should be adjusted to account for local and instantaneous extremes. Table 17.6 presents velocity limits for various channel boundaries conditions. This table is useful in screening alternatives, or as an alternative to the shear stress analysis presented in the preceding sections.

Step 6 - Confirm Lining Stability

The stability of the proposed lining should be assessed by comparing the threshold values in Table 17.4 to the newly computed hydraulic conditions. These values can be adjusted to account for flow duration using Figures 17.8 and 17.9 as a guide. If computed values exceed thresholds, Step 4 should be repeated. If the threshold is not exceeded, a factor of safety (FS) for the project should be determined from the following equations:

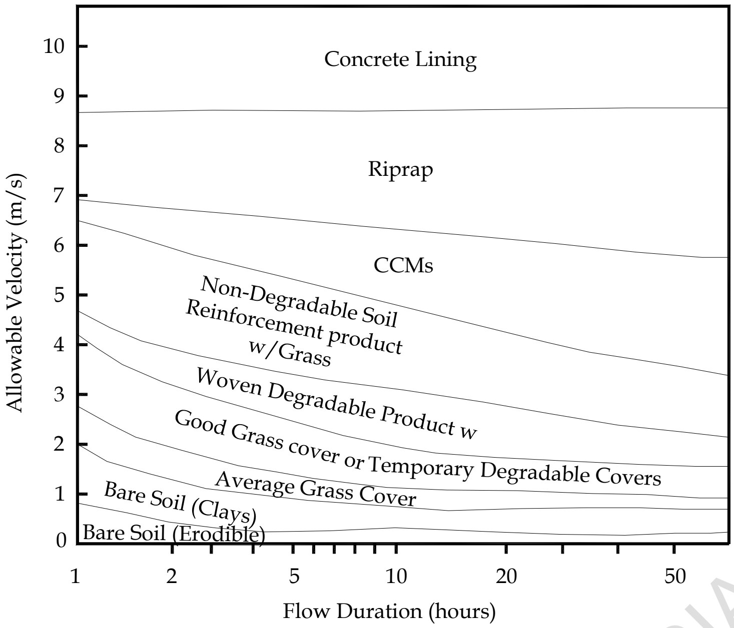
$$FS = \frac{\tau_{max}}{\tau_{computed}} \quad \text{or} \quad FS = \frac{V_{max}}{V_{computed}} \tag{17.3}$$

Table 17.6: Stability of Channel Linings for Given Velocity Ranges (Fischenich, 2001)

Lining	0 – 0.61 m/s	0.61 – 1.22 m/s	1.22 – 1.83 m/s	1.83 – 2.44 m/s	> 2.44 m/s
Sandy Soils	▒	□	□	□	□
Firm Loam	▒	▒	□	□	□
Mixed Gravel and	■	▒	▒	□	□
Cobbles	■	▒	▒	□	□
Average Turf	■	▒	▒	□	□
Degradable RECPs	■	▒	▒	▒	□
Stabilizing	■	▒	▒	▒	□
Bioengineering	■	▒	▒	▒	□
Good Turf	■	■	▒	▒	▒
Permanent RECPs	■	■	■	■	▒
Armoring	■	■	■	■	▒
Bioengineering	■	■	■	■	▒
CCMs & Gabions	■	■	■	■	▒
Riprap	■	■	■	■	■
Concrete	■	■	■	■	■

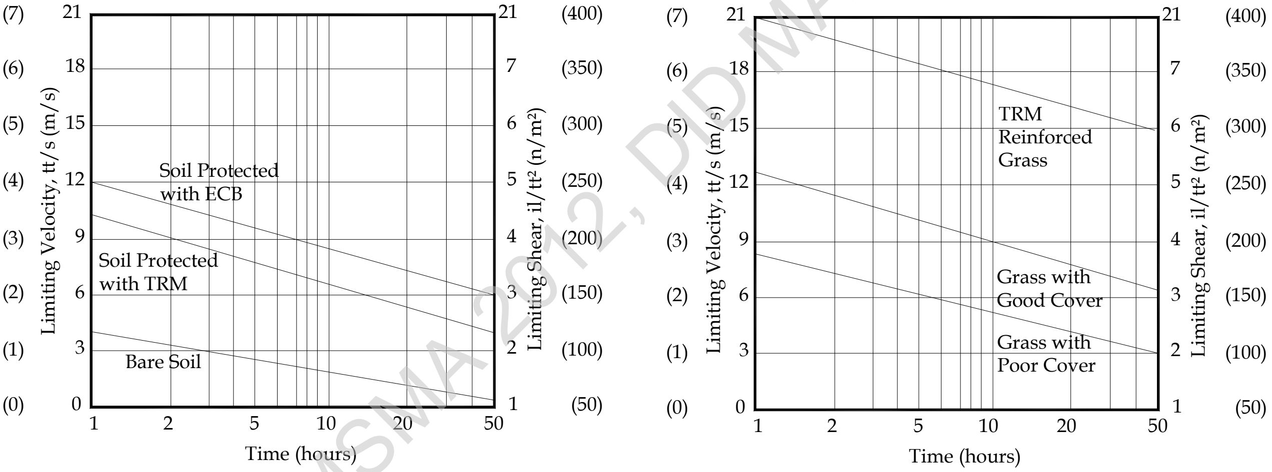
Key :

■	Appropriate
▒	Use Caution
□	Not Appropriate



* For slopes < 5%

Figure 17.8: Erosion Limits as a Function of Flow Duration (Fischenich and Allen, 2000)



a) Bare and TRM Protected Soils

b) Plain and TRM Reinforced Grass

Figure 17.9: Limiting Value for Velocity and Shear (Sprague, 1999)

17.7.3 Limitations

Techniques described in previous section are generally applicable to stream restoration projects that include revegetation of the riparian zone or bioengineering treatment. Detailed design criteria can be found in DID Manual, Volume 2-River Management (DID, 2009).

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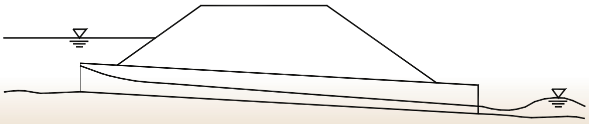


CULVERT 18

CHAPTER 18 CULVERT

18.1 INTRODUCTION

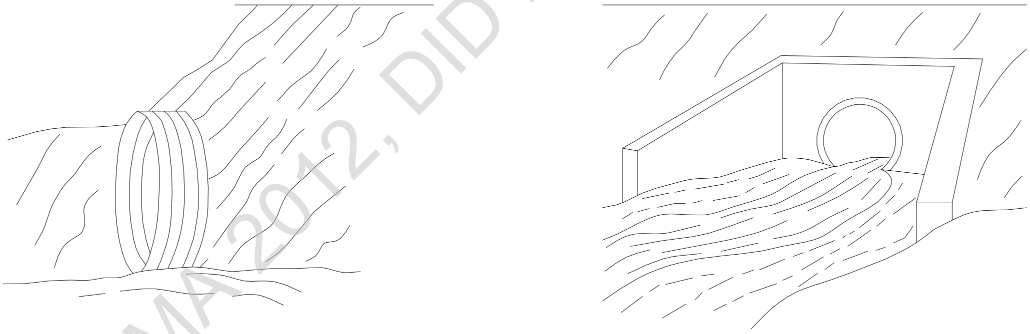
A culvert is a relatively short length of conduit used to transport stormwater through an embankment. A culvert acts as an enclosed channel that serves as a continuation for an open channel through the embankment. However, flow through culverts depends on entrance geometry and flow depth at the downstream end. Consequently, flow computations for culverts are more complex than open-channel flow analysis associated with pipes or drains. Culverts are typically designed to pass the design discharge without overtopping the embankment or causing extensive ponding at the upstream end.

This Chapter provides guidance and procedures for the hydraulic design of culverts which are based on "Hydraulic Design of Highway Culverts", Hydraulic Engineering Circular No 5 (FHWA, 1985).

18.1.1 Components

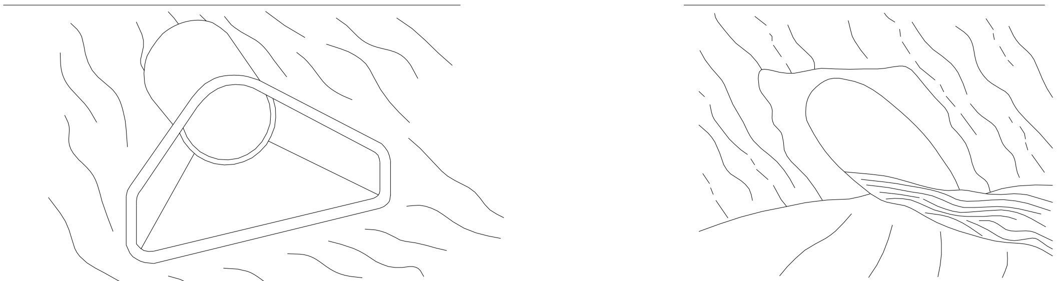
Major components of a culvert include the barrel, end treatment such as headwalls, endwalls and wingwalls, outlet protection, inlet improvements and debris control structures. Except for the barrel these components are used as the specific situation warrants.

End treatments such as headwalls and wingwalls protect the embankment from erosion, serve as retaining walls to stabilize the bank and add weight to counter any buoyancy effects. Ideally, the culvert's centreline should follow the line and grade of natural channel. In many cases this cannot be done and skewing headwalls and wingwalls helps accommodate the natural stream alignment to the culvert alignment. Figure 18.1 shows four types of inlet entrances.



a) Projecting Barrel

b) Cast In-place Concrete
Headwall & Wingwalls



c) Precast End Section

d) End Mitered to The Slope

Figure 18.1: Four Standard End Inlet Treatments

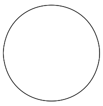
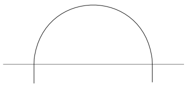
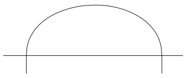
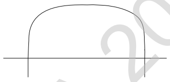
Debris barriers are sometimes constructed on the upstream end to prevent material from entering and clogging the culvert. The barriers are placed far enough away from the entrance so that accumulated debris does not clog the entrance.

At the inlet and outlet ends of the culvert endwalls and wingwalls serve as erosion protection for the embankment and inhibit piping along the culverts outside surface. Downstream wingwalls provide a smooth transition between the culvert and the natural stream banks.

18.1.2 Application

Barrels are available in various sizes, shapes and materials. Table 18.1 shows the typically used culvert shapes and their applications. Shape selection depends on construction limitations, embankment height, environment issues, hydraulic performance and cost. The most commonly used culvert materials are corrugated steel, corrugated aluminium and precast or cast-in-place concrete. Factors such as corrosion, abrasion and structural strength determine the selection of material. In cases where the culvert is located in a highly visible amenity area, selection of shape and material may be based on aesthetic as well as functional considerations.

Table 18.1: Typical Shapes and Uses

Shape		Uses
Round		Common uses
Arch		For low clearance large waterway opening and aesthetics
Low Profile Arch		Low-wide waterway enclosures and aesthetics
Box		Low-wide waterway enclosures

18.2 HYDRAULICS FUNDAMENTALS

18.2.1 Flow Conditions

A culvert barrel may flow full over all of its length or partly full. Full flow in a culvert barrel is rare. Generally, at least part of the barrel flows partly full. A water surface profile calculation is the only way to accurately determine how much of the barrel flows full.

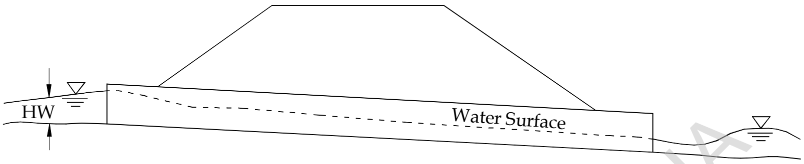
Full flow or pressure flow **-** One condition which can create pressure flow in a culvert is the back pressure caused by a high downstream water surface elevation. A high upstream water surface elevation may also produce full flow. It is therefore, regardless of the cause, the capacity of a culvert is affected by upstream and downstream conditions and by the hydraulic characteristics of the culvert.

Partly Full or Open channel flow - The appropriate open channel flow regimes, namely subcritical, critical, or supercritical must be determined and accomplished by evaluating the dimensionless Froude number Fr. Fr>1, the flow is supercritical and is characterised as swift. When Fr<1, the flow is subcritical and characterised as smooth and tranquil. If Fr=1, the flow is said to be critical. To analyse free surface flow conditions, a point of known depth and flow (control section) must first be identified. A definable relationship exists between critical depth and critical flow at the dam crest, making it a convenient control section.

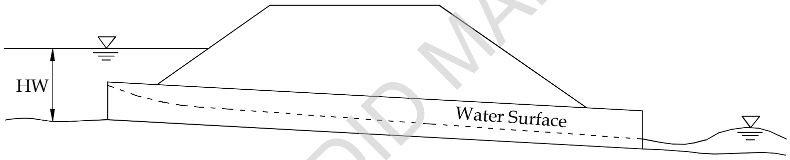
18.2.2 Type of Flow Control

Inlet and outlet control are the two basic types of flow control which define the control section. The characterisation of pressure, subcritical, and supercritical flow regimes play an important role in determining the location of the control section and thus the type of control.

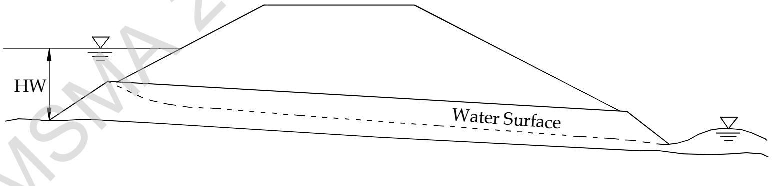
Inlet control - Occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. The upstream water surface elevation and the inlet geometry represent the major flow control. The inlet geometry includes the barrel shape, cross-sectional area, and the inlet edge. Figure 18.2 show types of inlet control.



A. Projecting End – Unsubmerged Inlet



B. Projecting End – Submerged Inlet



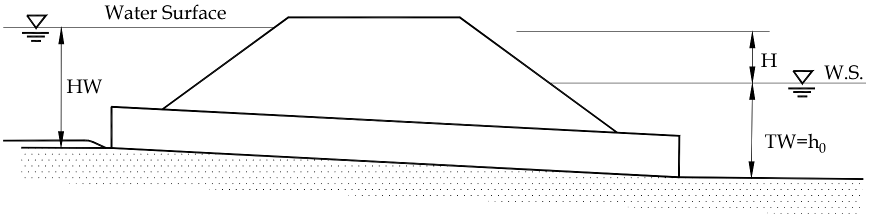
C. Mitred End – Submerged Inlet

Figure 18.2: Types of Inlet Control

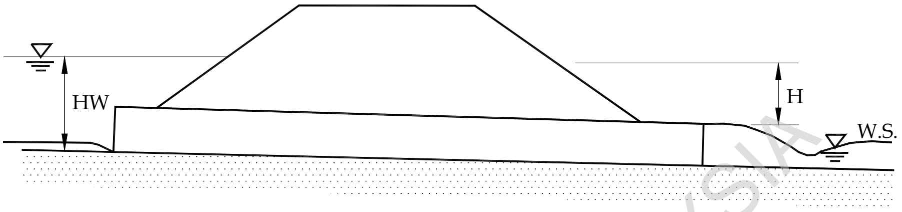
Outlet control - Occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under this conditions. All the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, water surface elevation at the outlet, and the slope, length, and hydraulic roughness of the culvert barrel. Figure 18.3 show types of outlet control.

(a) Determination of Energy Head (H)

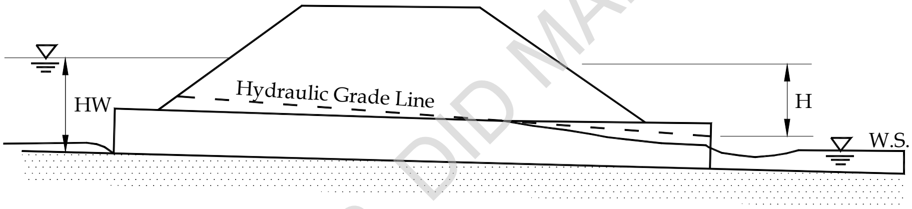
The head, *H* (Figure 18.3) or energy required to pass a given flow through a culvert operating under outlet control is made up of three major parts. These three parts are usually expressed in metres of water and include



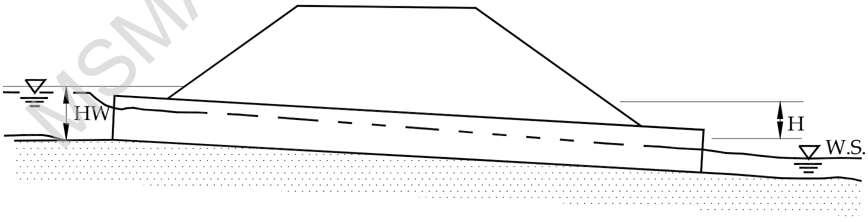
(a) Culvert Flowing Full, Submerged Outlet



(b) Culvert Flowing Full, Unsubmerged Outlet



(c) Culvert Flowing Full, for Part of Length



(d) Culvert not flowing full

Figure 18.3: Type of Outlet Control

a velocity head, H_v, an entrance loss, H_e and a friction loss, H_f. The energy head is expressed in equation form as:

$$H = H_v + H_e + H_f \tag{18.1}$$

The velocity head, H_v is given by,

$$H_v = \frac{V^2}{2g} \tag{18.2}$$

where V is the mean velocity in the culvert cell and g is the acceleration due to gravity. The mean velocity is the discharge, Q, divided by the cross-sectional area A of the cell.

The entrance loss is expressed as,

$$H_e = K_e \frac{V^2}{2g} \tag{18.3}$$

The entrance loss coefficient, K_e , depends on the inlet geometry primarily through the effect it has on contraction of the flow. Values of K_e determined from experiment, range from 0.2 for a well rounded entrance, through 0.5 for a square edged inlet in a vertical headwall to 0.9 for a sharp pipe (e.g. corrugated steel) projecting from an embankment. K_e coefficients are given on Design Chart 18.A1.

Since most engineers are familiar with Manning's n, the following expression is used to calculate the friction loss, H_f along the conduit:

$$H_f = \frac{2gn^2L}{R^{1.33}} x \frac{V^2}{2g} \tag{18.4}$$

where,

n = Manning's friction factor;

L = Length of culvert cell (m);

V = Mean velocity of flow in culvert cell (m/s);

g = Acceleration due to gravity (9.80 m/s^2);

R = Hydraulic radius (m) = A/W_p;

A = Area of flow for full cross-section (m^2); and

W_p = Wetted perimeter (m).

Substituting in Equation 18.1 and simplifying, we get for full flow:

$$H = \left[1 + K_e + \frac{2gn^2L}{R^{1.33}}\right] \frac{V^2}{2g} \tag{18.5}$$

Figure 18.4 shows the terms of Equation 18.5, the energy line, the hydraulic grade line and the headwater depth, *HW*. The energy line represents the total energy at any point along the culvert cell. The hydraulic grade line is defined as the pressure line to which water would rise in small vertical pipes attached to the culvert wall along its length. The difference in elevation between these two lines is the velocity head, $V^2/2g$.

By referring to Figure 18.4 and using the culvert invert at the outlet as datum, we get:

$$h_1 + \frac{V_1^2}{2g} + LS = h_2 + H_v + H_e + H_f \tag{18.6}$$

Then,

$$h_1 + \frac{V_1^2}{2g} + LS - h_2 = H_v + H_e + H_f \tag{18.7}$$

and,

$$H = h_1 + \frac{V_1^2}{2g} + LS - h_2 = H_v + H_e + H_f \tag{18.8}$$

From the development of this energy equation and Figure 18.4, *H* is the difference between the elevation of the hydraulic grade line at the outlet and the energy line at the inlet. Since the velocity head in the entrance pool is usually small under ponded conditions, the water surface of the headwater pool elevation can be assumed to equal the elevation of the energy line.

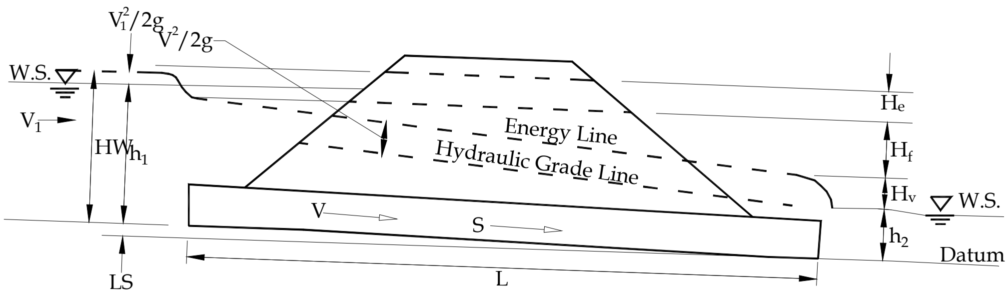


Figure 18.4: Hydraulics of Culvert Flowing Full under Outlet Control for High Tailwater.

Equation 18.5 can be readily solved for *H* by the use of the full flow nomographs in Appendix 18.A, Design Charts 18.A9 to 18.A11

(b) Determination of Headwater Depth (HW_o)

Headwater depth, HW_0 can be determined from an equation for outlet control:

$$HW_0 = H + h_0 - LS \tag{18.9}$$

where,

H = Head (m) determined from Design Charts 18.A9 to 18.A11 or from Equation 18.8;

h_0 = Greater of *TW* and $(h_c + D)/2$, in which $h \leq D$;

h_c = Critical depth (m) from the Design Charts 18.A7 and 18. A8 in Appendix 18.A;

D = Culvert height (m);

L = Length of culvert (m); and

S = Slope of cell (m/m).

(c) Determination of h_o

The determination of h_0 is an important factor in calculating both the headwater depth and the hydraulic capacity a culvert flowing under outlet control.

Tailwater depth, *TW* is the depth from the culvert invert at the outlet to the water surface in the outlet channel. Engineering judgement is required in evaluating possible tailwater depths. Tailwater is often controlled by a downstream obstruction or by water levels in another stream. A field inspection should be made to check on downstream conditions and flood levels. The Slope Area Method can be used to calculate flow depths, if downstream conditions do not provide an obvious control.

Fortunately, most natural streams are wide compared to the culvert and the depth of water in the natural channel is considerably less than critical depth in the culvert section. In such cases the natural tailwater does not govern.

Two tailwater conditions can occur with culverts operating under outlet control, (i) tailwater above the top of the opening and (ii) tailwater at or below top of opening:

(i) Tailwater above the top of opening – when the tailwater, *TW* in the outlet channel is above the top of the culvert outlet, Figure 18.3 (a);

$$h_0 = TW \qquad (18.10)$$

The relationship of h_0 to the other terms in Equation 18.9, for this situation, is illustrated on Figure 18.5.

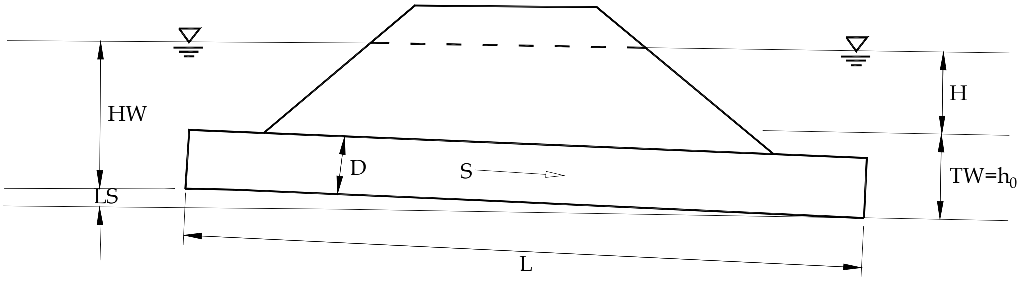


Figure 18.5: Determination of h$_o$ for High Tailwater.

(ii) Tailwater at or below top of opening – when the tailwater in the outlet channel is at or below the top of the culvert outlet, as on Figure 18.3 (b), 18.3 (c) and 18.3 (d), h_0 is more difficult to determine. Full flow depth at the outlet, Figure 18.3 (b), will occur only when the flow rate is sufficient to give critical depths equal or higher than the height of the culvert opening. For all such flows the hydraulic grade line will pass through the top of the culvert at the outlet and the head, *H* can be added to the level of the top of the culvert opening in calculating HW_0. When critical depth is less than the height of the culvert opening, the water surface drops as shown on Figures 18.3 (c) and 18.3 (d), depending on the flow. For the condition shown on Figure 18.3 (c), the culvert must flow full for of its length. Flow profile computations show that the hydraulic grade line, if extended as a straight line from the point where the water breaks away from the top of the culvert, will be at a height approximately halfway between critical depth and the top of the culvert, at the culvert outlet. i.e.:

$$h_0 = \frac{(h_c + D)}{2} \qquad (18.11)$$

This level should be used if it is greater than *TW*.

The head, *H* can be added to this level in calculating HW_0. The relationship of h_0 to the other terms in Equation 18.9 for this situation is illustrated on Figure 18.6. As the discharge decreases the situation approaches that of Figure 18.3 (d). For design purposes, this method is satisfactory for calculated headwater depths above 0.75*D*. For smaller values of headwater, more accurate result can be obtained by flow profile calculations or by the use of the capacity charts from Hydraulic Engineering Circular No 10 (US Federal Highway Administration, 1972).

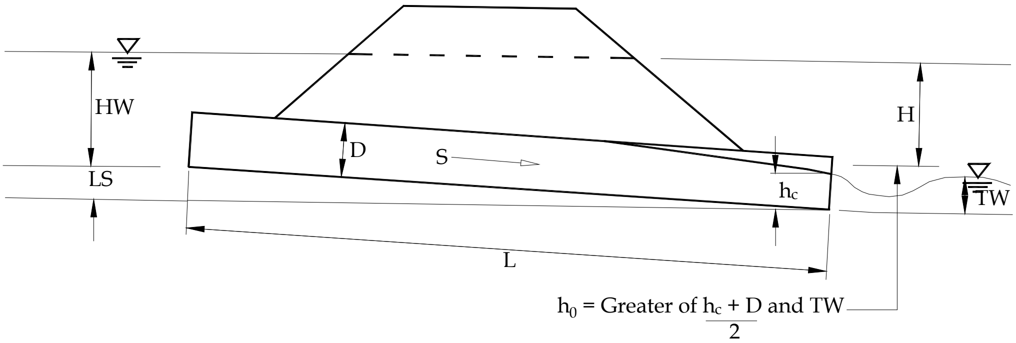


Figure 18.6: Determination of h$_o$- for Tailwater below Top of Opening.

18.3 DESIGN CONSIDERATION

Headwater - Headwater is the water surface in the upstream of culvert. The available headwater may be limited by the height of the surrounding ground or the elevation at which the road formation cuts through the HGL. The most economical culvert is one which utilize all of the available headwater to pass the design discharge. However, it is not always possible to utilize all of the available headwater due to constraints that limit the upstream water level. The following factors should be considered in the selection of the design headwater:

- Limits on backwater not to cause inundation of the properties upstream; and
- The outlet velocity and the potential scours.

Multiple Cells - The culvert shape selected will best fit the waterway of channel or stream. In narrow deep channel, a small number of large diameter pipes or box culverts are usually appropriate. In flat areas of no well defined waterway, the flood may be larger in volume, but shallow depth. A number of separate culverts spread over the width of the flooded area may be more appropriate. Special consideration should be given to multiple cell culverts where the approach flow is of high velocity, particularly if supercritical.

Culvert in Flat Terrain - In flat terrain, drainage channels are often not well defined. Multiple culverts can be considered to have a common headwater elevation. It is also necessary to construct levee banks to achieve the design headwater at the culvert location provide no danger of increased flooding of upstream properties. The approval of the local drainage authority must be obtained prior to construction of such levee bank.

18.4 DESIGN PROCEDURE

The approach is to analyse a culvert for various types of flow control and then design for the control which produces the minimum performance. Design for minimum performance ignores transient conditions which might result in periods of better performance. The benefit of designing for minimum performance are ease of design and assurance of adequate performance under the least favourable hydraulic conditions.

The design engineer should be familiar with all the equations in the previous Section before using these procedures. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structures. The procedures does not address the effect of storage. The design procedure is summarised on the Culvert Design Flowchart, Figure 18.7

The steps in culvert sizing are as follows:.

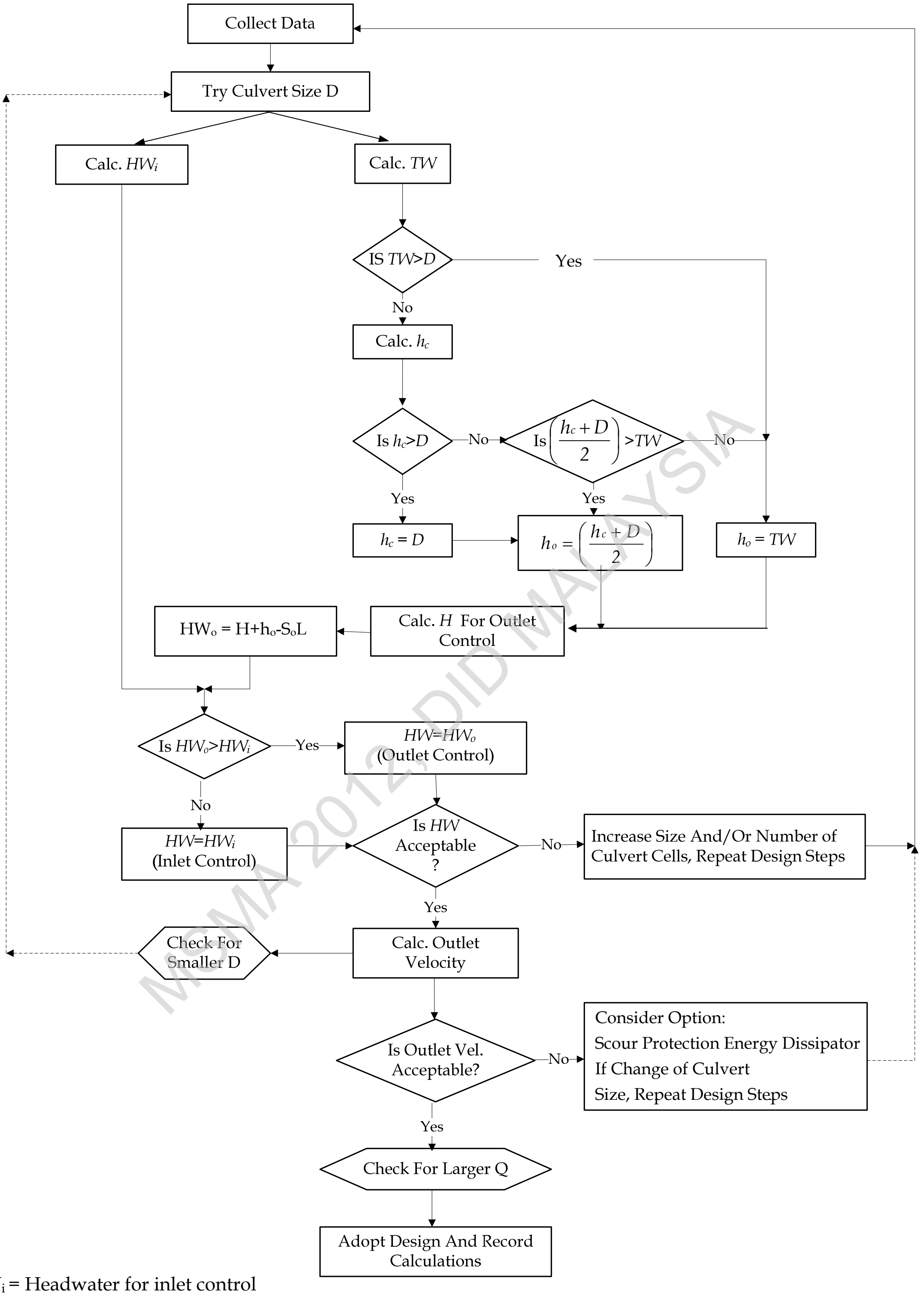
Step 1: Assemble Site Data

The following data are required for the design of culvert:

- Site survey and locality map;
- Embankment cross-section;
- Roadway profile;
- Photographs, aerial photographs;
- Details from field visit (sediment, debris and scour at existing structure);
- Design data for nearby structures;
- Studies by other authorities near the site, including small dams, canals, weirs, floodplains, storm drains; and
- Recorded and observed flood data.

Step 2: Determine Design Flood Discharge

Determine ARI of design flood and hence the design flood discharge Q, Please refer Chapter 2.



HW_i = Headwater for inlet control
HW_o = Headwater for outlet control

Figure 18.7: Design Flow Chart

Step 3: Commence Summarising Data on Design Form

The information obtained from step 1 and step 2 are put into Design form Chart 18.A1 in Appendix 18.A.

Step 4: Select Trial Culvert

- Choose culvert material, shape, size and entrance type.
- The initial trial size of culvert is determined, either by arbitrary selection or by assuming a velocity (say 2 m/s) and calculating a culvert area from $A = Q/V$

Step 5: Determine Inlet Control Headwater Depth, HW_i – Use inlet Control Design Charts 18.A2 to 18.A4.

The nomographs cover various culvert types and inlet configurations. Each nomographs has an example on it which is self-explanatory. Using the trial culvert size, the relevant nomograph can be used to calculate HW_i given a known Q. They can also be used in reverse to calculate Q given a known HW_i.

It should be noted that where the approach velocity is considerable, the approach velocity head can be calculated and deducted from the calculated HW_i to give the actual physical head required.

Step 6: Determine Depth, h_0 for Outlet control

Calculate both $(h_c + D)/2$ and the tailwater, TW from known flood levels, downstream controlling levels or from the Slope Area Method. If it is clear that the downstream tailwater conditions do not control, take $h_0 = (h_c + D)/2$. h_c can be calculated from Design Charts 18.A7. or 18.A8. If h_c exceeds D then take h_c as D.

h_0 is the larger of TW or $(h_c + D)/2$

Step 7: Determine Outlet Control Headwater Depth at Inlet, HW_0

- Determine entrance loss coefficient, K_e from Design Table 18.A1., Appendix 18.A.
- Calculate the losses through the culvert, H using the outlet control nomographs, Design Charts 18.A9 to 18.A11 (or Equation 18.5 if outside the range). As with the inlet control nomographs, these nomographs cover various culvert types and each nomograph has an self-explanatory example on it.
- If the Manning's n value of the culvert under consideration differs from the Manning n value shown on the nomograph, this can be allowed for by adjusting the culvert length as follows:

$$L_1 = L\left(\frac{n_1}{n}\right) \tag{18.12}$$

where,

L_1 = Adjusted culvert length;
L = Actual culvert length;
n_1 = Desired Manning n value; and
n = Manning n value given on the nomograph.

- Calculate $HW_0 = H + h_0 - LS$

As with inlet control, where the approach velocity is considerable, the approach velocity head can be calculated and deducted from the calculated HW_0 to give the actual physical head required.

- If HW_0 is less than 0.75D and the culvert is under outlet control, then the culvert may be flowing only part full and using $(h_c + D)/2$ to calculate h_0 may not be applicable. If required, more accurate

results can be obtained by flow profile calculations or the use of Hydraulic Engineering Circular No 10 (as discussed in Section 18.2.2 under (ii) tailwater depth at or below top of opening).

Step 8: Determine Controlling Headwater, HW_c

Compare HW_i and HW_0 and use the higher:

If $HW_i > HW_0$ the culvert is under inlet control and $HW_c = HW_i$; and

If $HW_0 > HW_i$ the culvert is under outlet control and $HW_c = HW_0$.

Step 9: Calculate Outlet Velocity, V_0

The average outlet velocity will be the discharge divided by the cross-sectional area of flow at the culvert outlet. The cross-sectional area of flow depends, in turn, on the flow depth at the outlet.

If inlet control is the controlling headwater, the flow depth can be approximated by calculating the normal depth, y_n, for the culvert cross-section using Manning's Equation. The flow area, A is calculated using y_n and the outlet velocity:

$$V_o = \frac{Q}{A} \tag{18.13}$$

The outlet velocity computed utilising the normal depth, y_n will usually be high, because the normal depth is seldom reached in the relatively short length of average culvert.

If outlet control is the controlling headwater, the flow depth can be either critical depth h_c , the tailwater depth TW (if below the top of the culvert), or the full depth D of the culvert depending on the following Use relationships:

- h_c, if $h_c > TW$;
- Use TW, if $h_c < TW < D$; *and*
- Use D, if $D < TW$.

Calculate flow area using appropriate flow depth and then outlet velocity using Equation 18.13.

Step 10: Review Results

Compare alternative design with the site constraints and assumptions. If any of the following conditions are not met, repeat steps 4 to 9:

- The culvert must have adequate cover;
- The final length of the culvert should be close to the approximate length assumed in design;
- The headwalls and wingwalls must fit the site;
- The allowable headwater should not be exceeded; and
- The allowable overtopping flood frequency should not be exceeded.

The performance of the culvert should also be considered, (i) with floods larger than the design flood to ensure such rarer floods do not pose unacceptable risks to life or potential for major damage and (ii) with smaller floods than the design flood to ensure that there will be no unacceptable problems of maintenance.

If outlet velocity is high, scour protection or an energy dissipater may be required.

Step 11: Improved Designs

Under certain conditions more economic designs may be achieved by consideration of the following:

- The use of an improved inlet for culverts operating under inlet control; and
- Allowing ponding to occur upstream to reduce the peak discharge, if a large upstream headwater pool exists.

18.5 FLOW VELOCITY

Culvert usually increase the flow velocity in the natural water course. When culverts flow full, the highest velocity occurs near the outlet and may cause erosion. Check on outlet velocity must be carried out in the culvert design discharging into unlined waterway.

Inlet Control - The outlet velocity for a pipe culvert flowing with inlet control can be obtained from the Colebrook-White equation, Design Chart 18.A12, Appendix 18.A for pipe roughness k=0.6. For other pipe material charts of appropriate k values should be used. Chart 18.A5 and 18.A6 for circular and box culvert respectively can be used to obtain velocity for part full flow. This approach assumes that the outlet flow depth corresponding to uniform flow. The depth of flow should be checked against critical depth as determined from Design Charts 18.A7 and 18.A8 for circular and box culverts respectively.

Outlet Control - For outlet control, the average outlet velocity will be equal to the discharge divided by the cross-sectional areas of flow at the outlet.

Erosion of Conduit - Very fast flow of higher than 7.5m/s in full flow pipe, and 12m/s in open conduit can cause cavitation and erosion to the conduit. Maximum recommended flow velocities for Precast concrete pipes and precast box culvert are 8.0m/s, while for insitu concrete and hard packed rock of 300mm minimum is 6.0m/s.

Scour at Inlets and Outlets - Scour can occur upstream of the culvert caused by high velocity and acceleration of flow as it leaves the natural channel and enters the culvert. Upstream wing walls, aprons, cut-off walls and embankment paving assist protecting the embankment and stream bed at the upstream end of a culvert. The flow of high velocity emerging from culvert can cause erosion and scour in the bed immediately downstream. Scour protection such as concrete apron, rock riprap, rock mattresses, or concrete filled mattresses may be considered.

Siltation - Flow velocity about 0.5m/s and below will cause settlement of fine to medium sand particles and siltation occurs. Higher velocity may be obtained by increase the slope and hence to be self-cleansing. Self cleansing may also be obtained by graded culverts to the average grade of the water course upstream and downstream of the culvert, and levels must represent the average stream levels before the culvert was built.

18.6 MINIMUM ENERGY CULVERTS

The tranquil flow occur in conduit laid on natural grade of low slope of a fraction of one per thousand as in most coastal areas. “The Minimum Energy Culvert” concept is to concentrate the flow in a narrow, deep cross section flowing with critical velocity under maximum design flow, taking advantage of the minimum specific energy under critical flow condition. This maximises the flow per unit length of waterway crossing. By keeping the flow outside the supercritical region, the designer avoids the energy loss in a hydraulic jump and the need for erosion protection, hence safe cost.

Here, the design requires knowledge of:

- Design discharge;
- Average natural slope of terrain;
- Flood Levels; and
- Survey details of flood plain adjacent to culvert.

Base on the above information and the following assumptions, a plan and longitudinal section of the culvert is drawn;

- The energy line parallels the natural fall of the terrain; and
- Energy losses at entry and exit of culvert are disregarded due to smooth transitions.

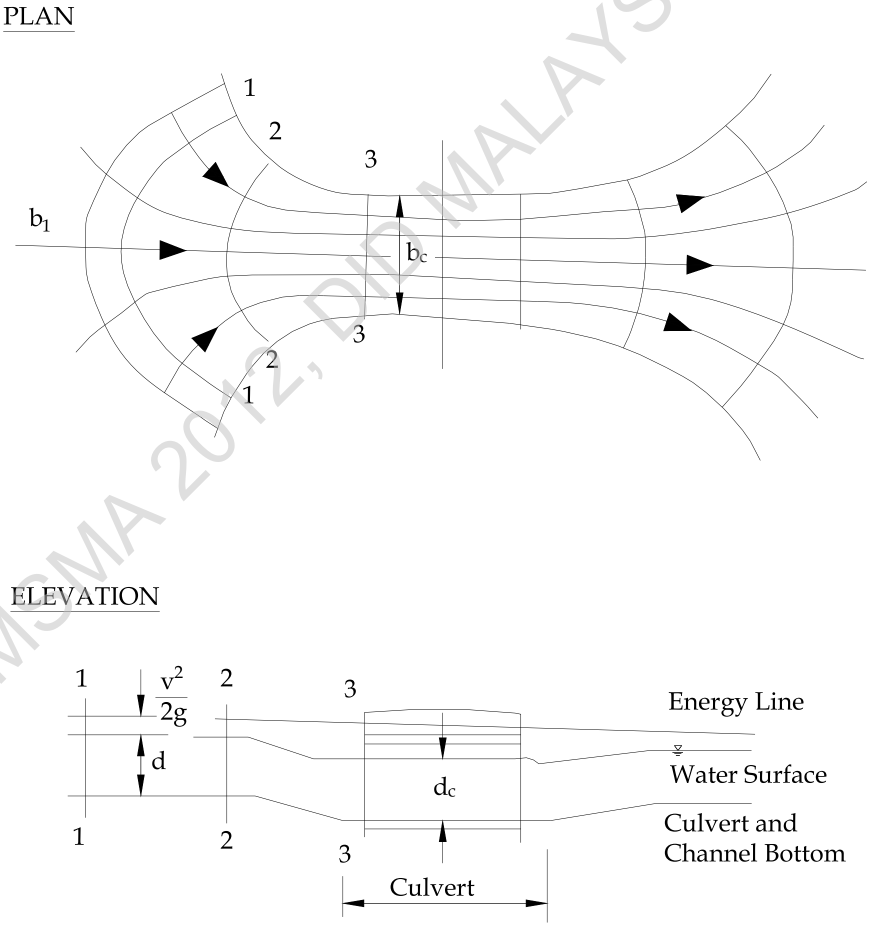
To avoid the formation of standing eddies, the expansion of exit stream bed should be smaller than the entry angle.

Using the following equations:

$$H_{s,c} = 1.5d_{c,} \quad (18.14a)$$

$$Q = bd_c\,(gd_c)^{0.5} \quad (18.14b)$$

Corresponding values of b, d_c and H_s can be tried and compared.


PLAN

ELEVATION

Figure 18.8: Characteristic Flow Line of Minimum Energy Culvert

REFERENCES

1. DID Malaysia (2000). *Urban Stormwater Management Manual for Malaysia*. Department of Irrigation and Drainage.

2. U.S. Federal Highway Administration (1972). *Capacity Chart for the Hydraulic Design of Highway Culverts,* Hydraulic Engineering Circular No.10, Washington DC.

3. U.S. Federal Highway Administration (1972). *Hydraulic Design of Highway Culverts,* Hydraulic Engineering Circular No.5, Washington DC.

APPENDIX 18.A DESIGN FORM, TABLE, CHARTS AND NOMOGRAPH

ENGINEER : ____________
DATE : ____________

PROJECT :

HYDROLOGICAL AND CHANNEL INFORMATION

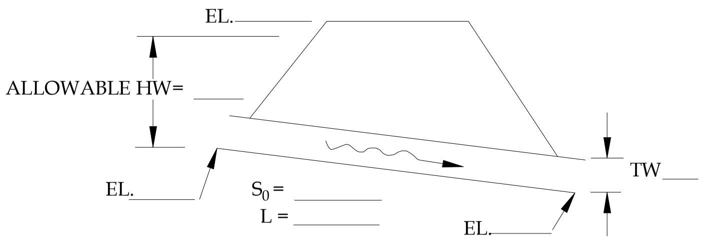
Q_1 = ____________
Q_2= ____________

TW_1 = ____________
TW_2 = ____________

(Q_1= Design Discharge)
(Q_1= Check Discharge)

SKETCH

STATION : ________



MEAN STREAM VELOCITY = ____________
MAX. STREAM VELOCITY = ____________

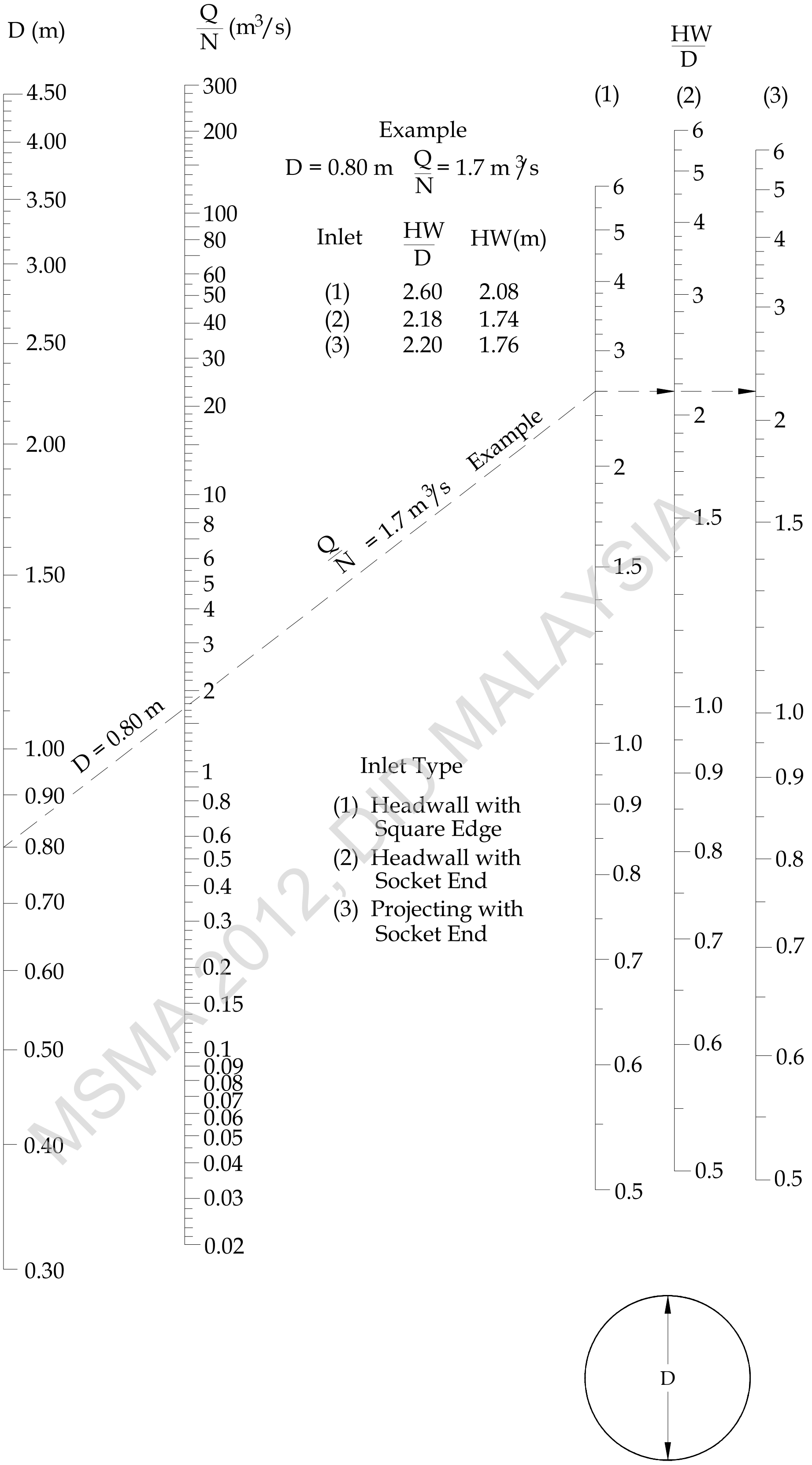
CULVERT DESCRIPTION (Entrance Type)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW_0 = H + h_0 - LS_0$											
			$\frac{HWi}{D}$	HW	K_e	H	h_c	$\frac{h_c + D}{2}$	TW	h_0	LS_0	HW_0				

SUMMARY AND RECOMMENDATIONS:

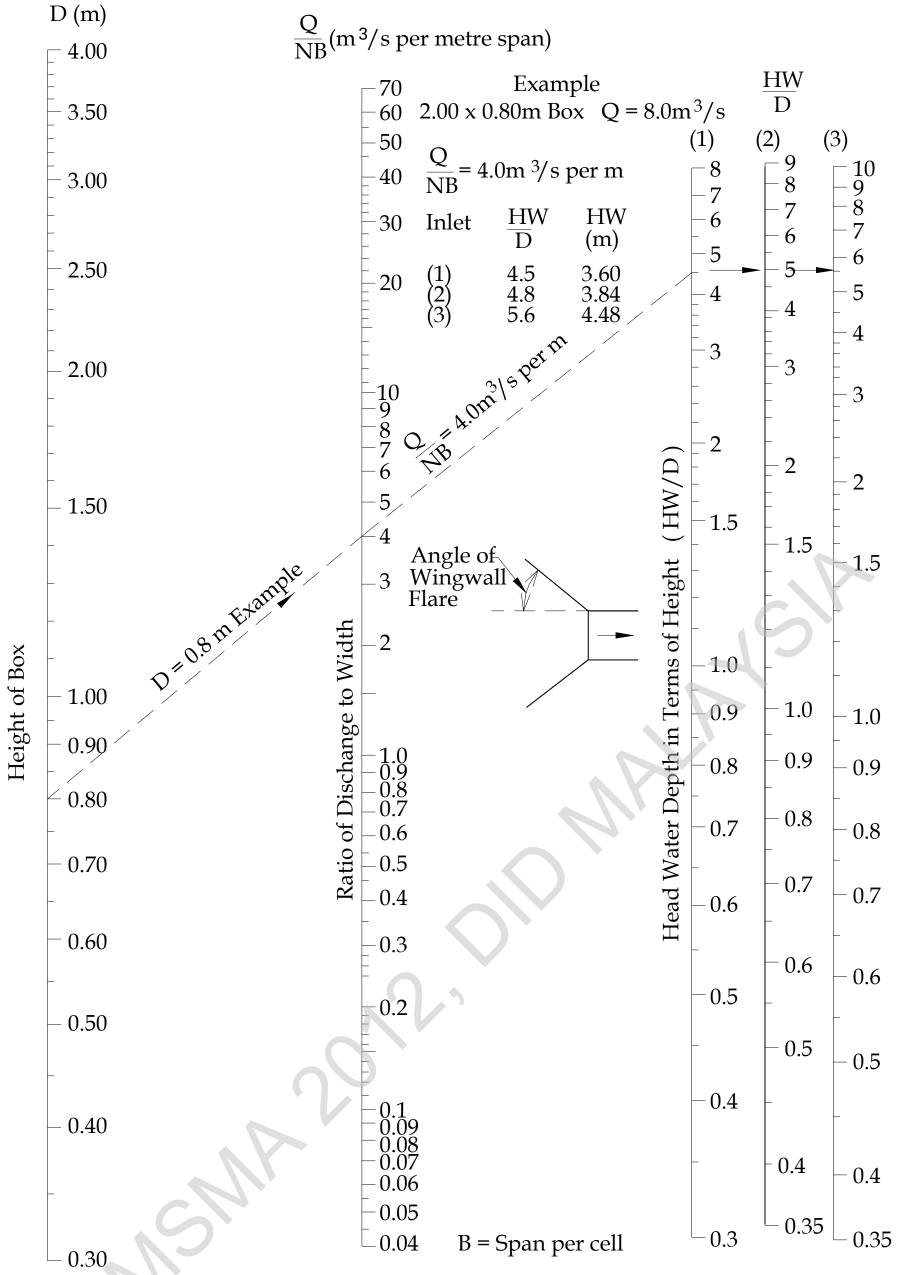
Design Chart 18.A1: Design Form for Culvert Calculation

Design Table 18.A1: Entrance Loss Coefficients

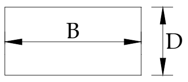
Type of Barrel And Inlet	Loss Coefficients
Pipe, Concrete	K_e
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitred to conform to fill slope	0.7
End-section conforming to fill slope (standard precast)	0.5
Bevelled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlets	0.2
Pipe, or Pipe-Arch, Corrugated Steel	
Projecting from fill	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitred to conform to fill slope	0.7
End-section conforming to fill slope (standard prefab)	0.5
Bevelled edges, 33.7° or 45° bevels	0.25
Side-tapered or slope-tapered inlets	0.2
Box, Reinforced Concrete	
Headwall	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, Or bevelled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension Or bevelled top edge	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2
Projecting	
Square-edged	0.7*
Bevelled edges, 33.7° or 45° bevels	0.2*
* Estimated	



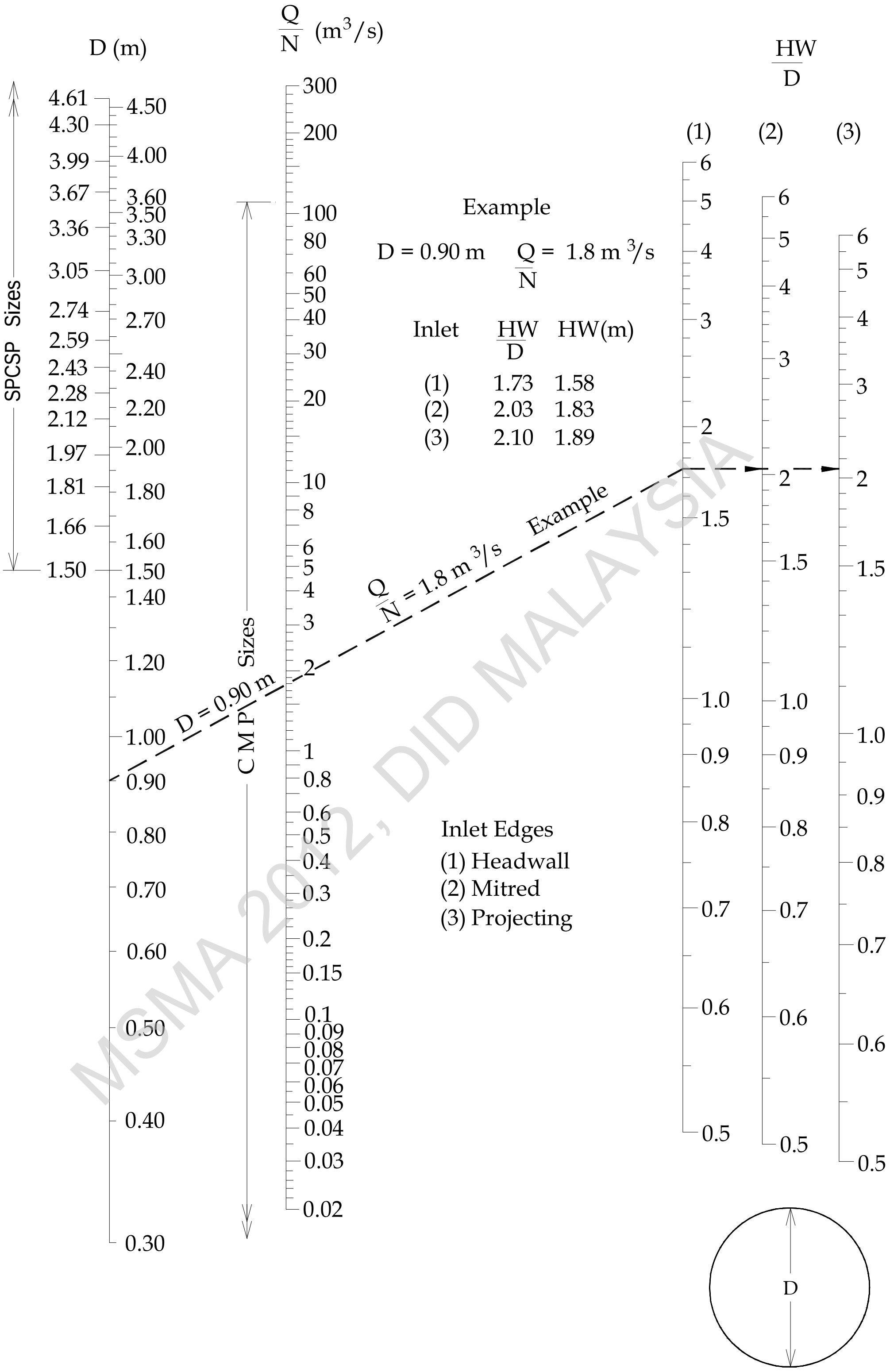
Design Chart 18.A2: Inlet Control Nomograph – Concrete Pipe Culvert



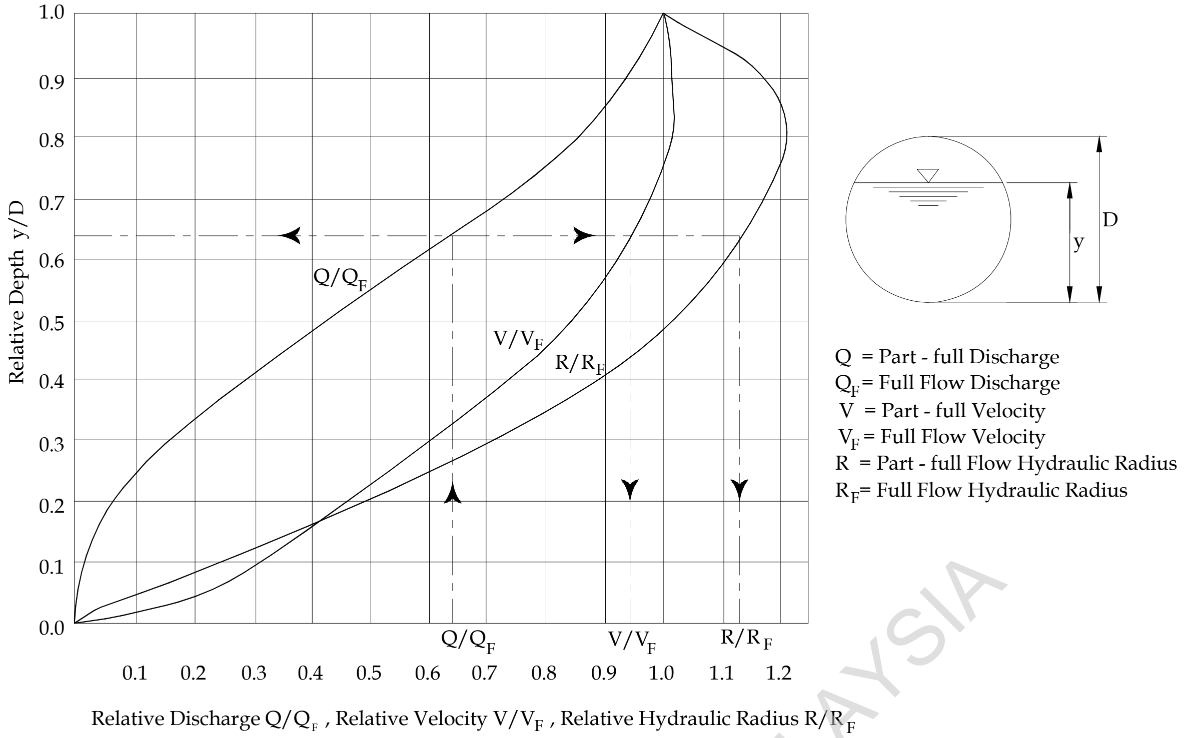
Wingwall Flare	HW/D Scale
30° - 75°	1
90° (headwall)	2
0° (parallel)	3



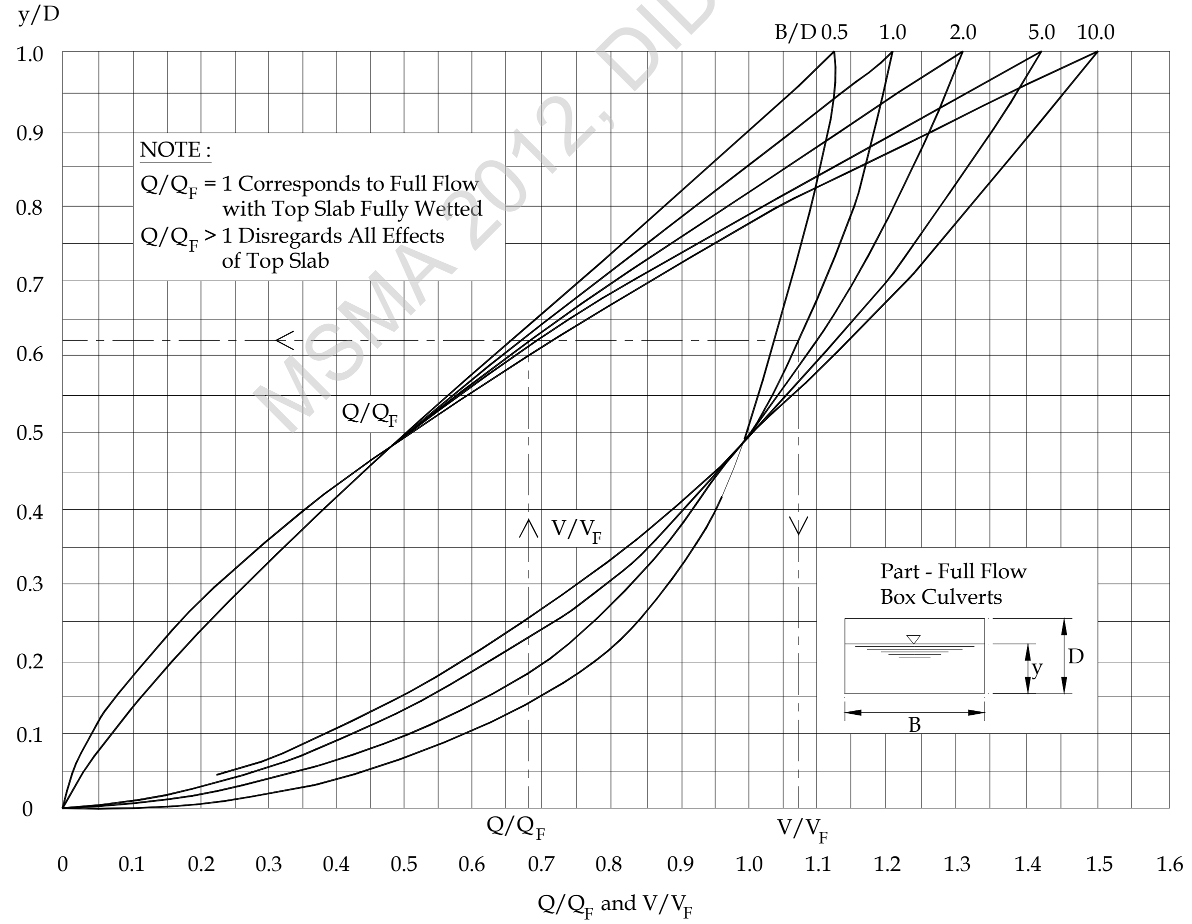
Design Chart 18.A3: Inlet Control Nomograph – Box Culvert



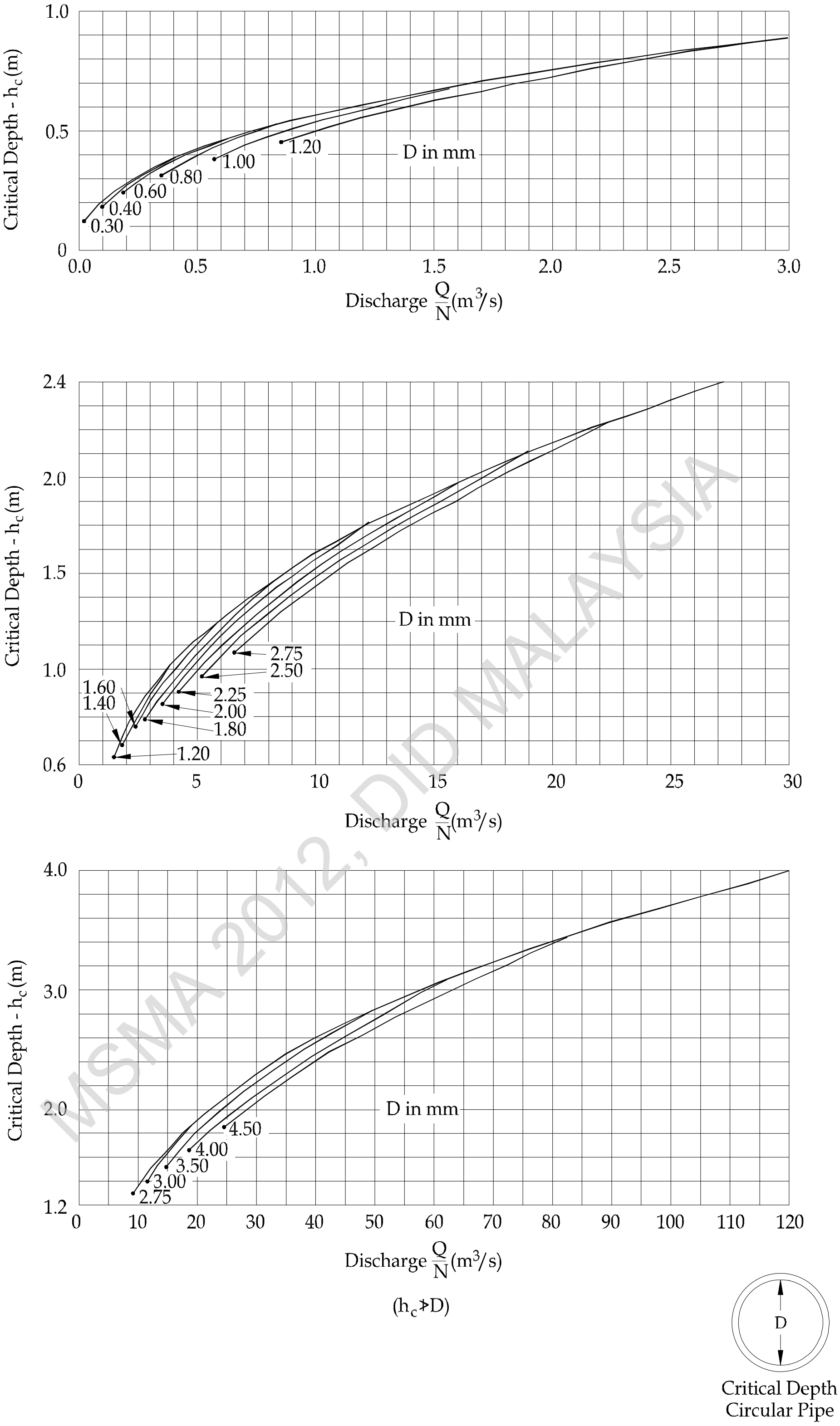
Design Chart 18.A4: Inlet Control Nomograph – Corrugated Metal Pipe (CMP) Culvert



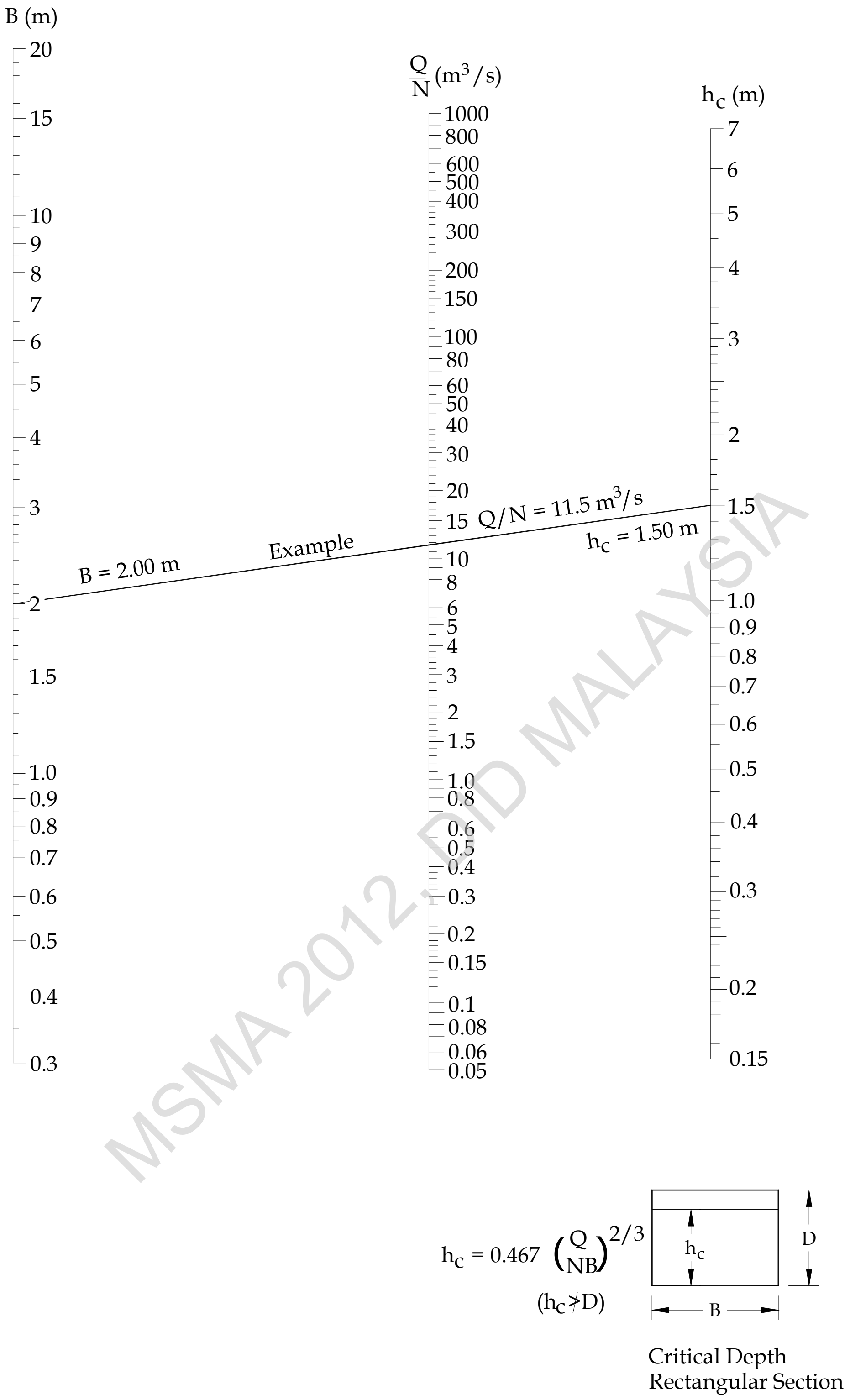
Design Chart 18.A5: Relative Discharge, Velocity and Hydraulic Radius in Part-full Pipe Flow.



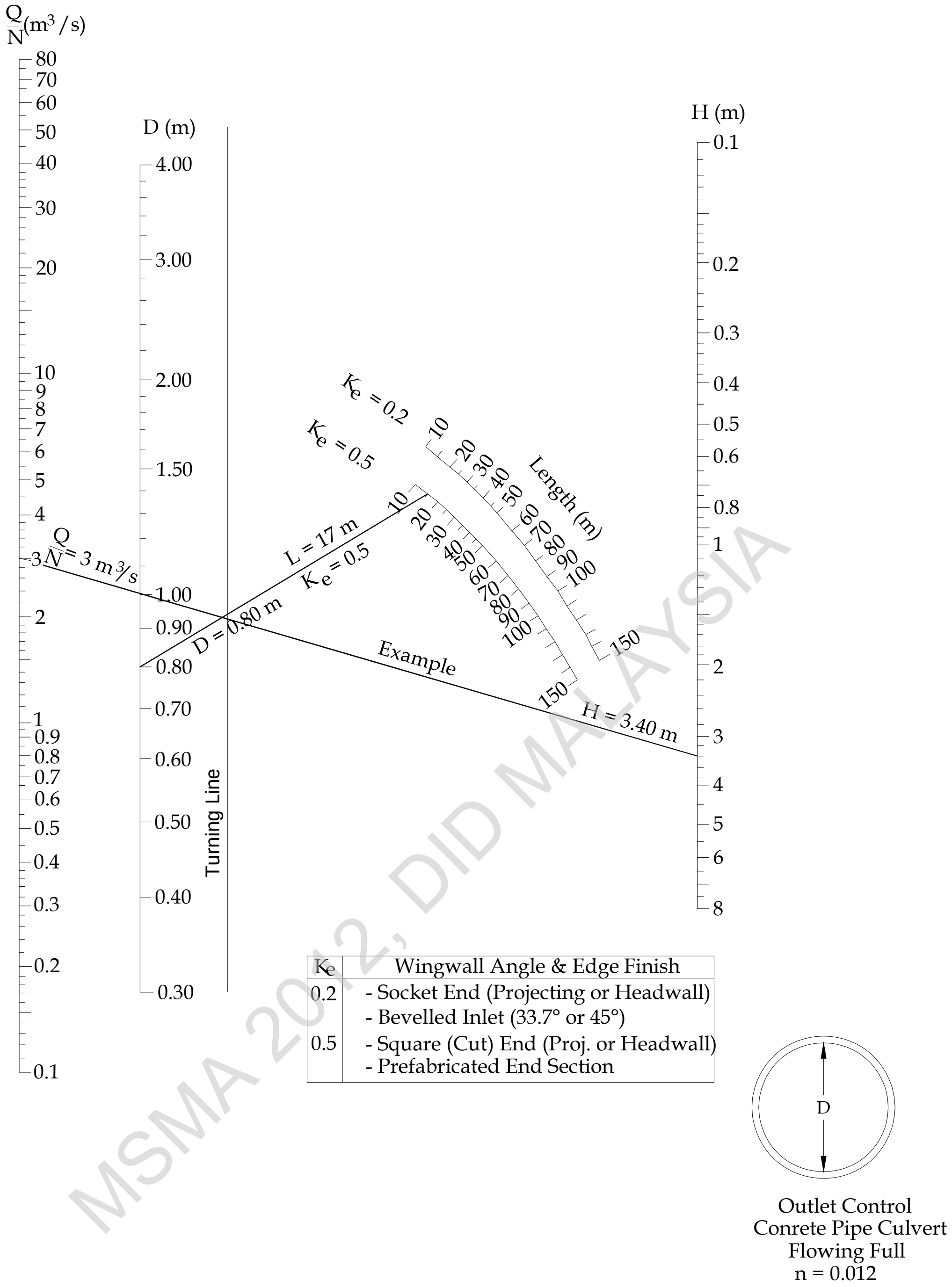
Design Chart 18.A6: Relative Discharge, Velocity and Hydraulic Radius in Part-full Box Culvert Flow



Design Chart 18.A7: Critical Depth in a Circular Pipe

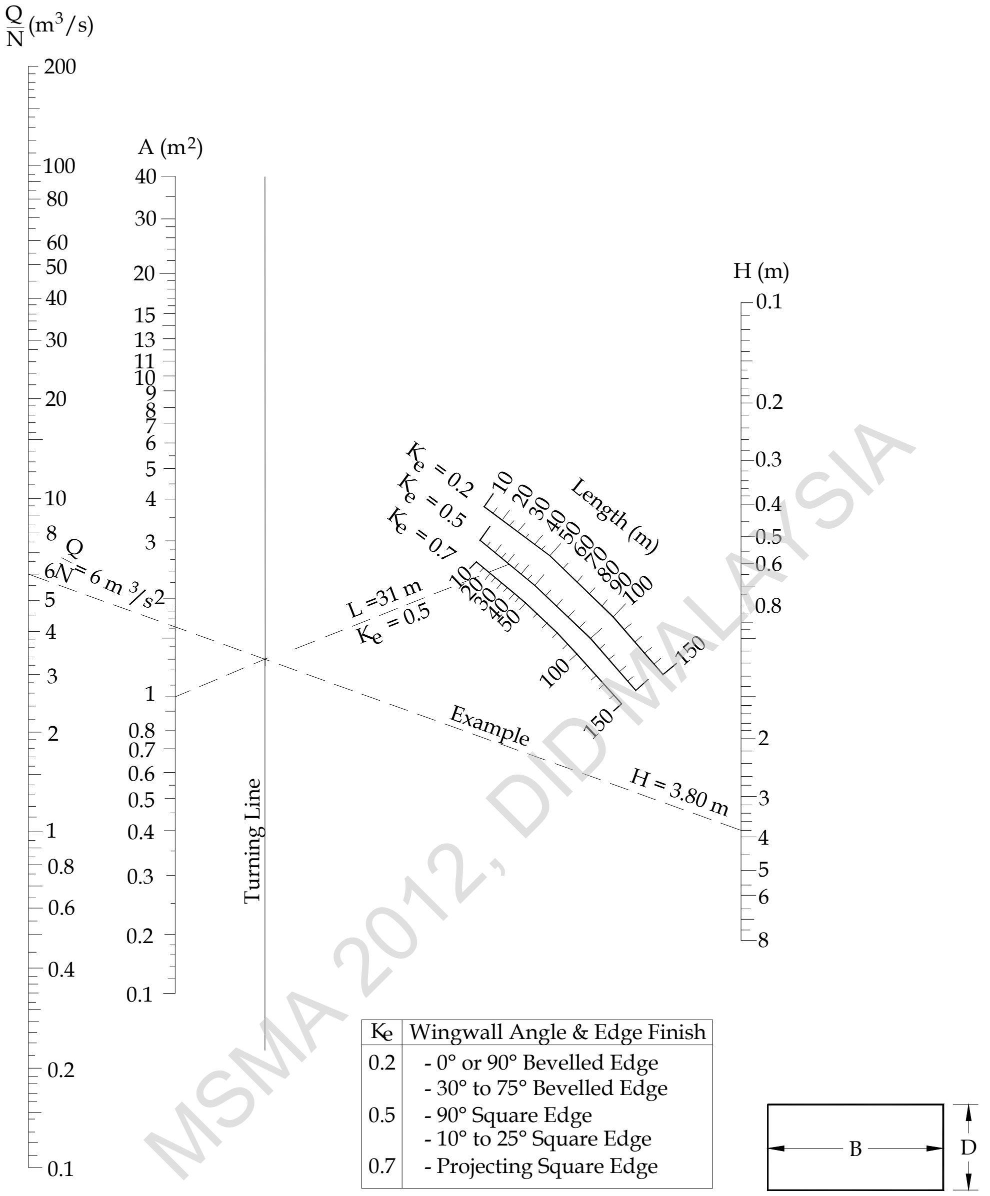


Design Chart 18.A8: Critical Depth in a Rectangular (Box) Section



Ke	Wingwall Angle & Edge Finish
0.2	- Socket End (Projecting or Headwall) - Bevelled Inlet (33.7° or 45°)
0.5	- Square (Cut) End (Proj. or Headwall) - Prefabricated End Section

Design Chart 18.A9: Outlet Control Nomograph – Concrete Pipe Culvert Flowing Full with n = 0.012



Ke	Wingwall Angle & Edge Finish
0.2	- 0° or 90° Bevelled Edge - 30° to 75° Bevelled Edge
0.5	- 90° Square Edge - 10° to 25° Square Edge
0.7	- Projecting Square Edge

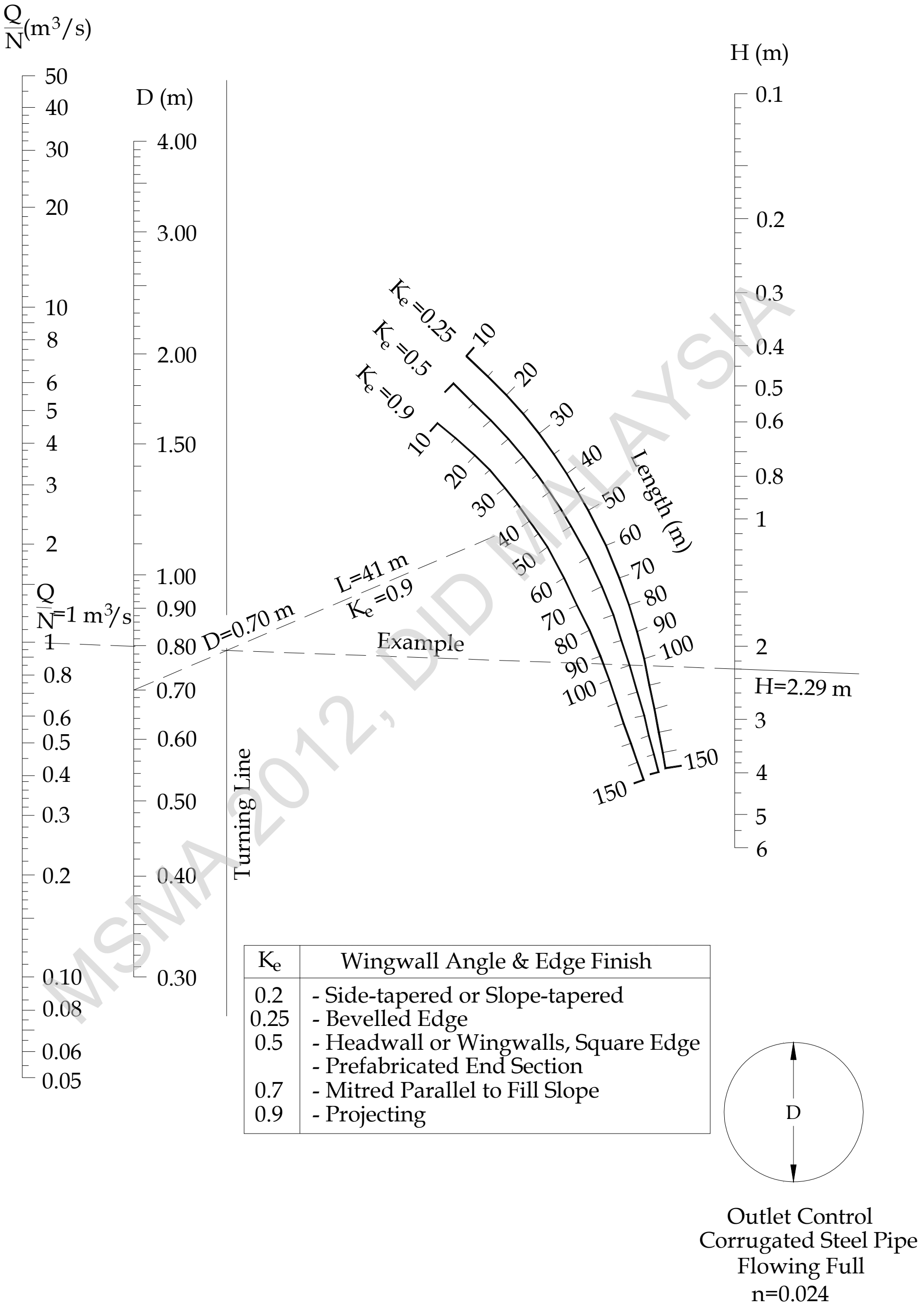
NOTE:

If B/D = 0.5 to 2.0
Calculate H from
E 7.5

A=BD

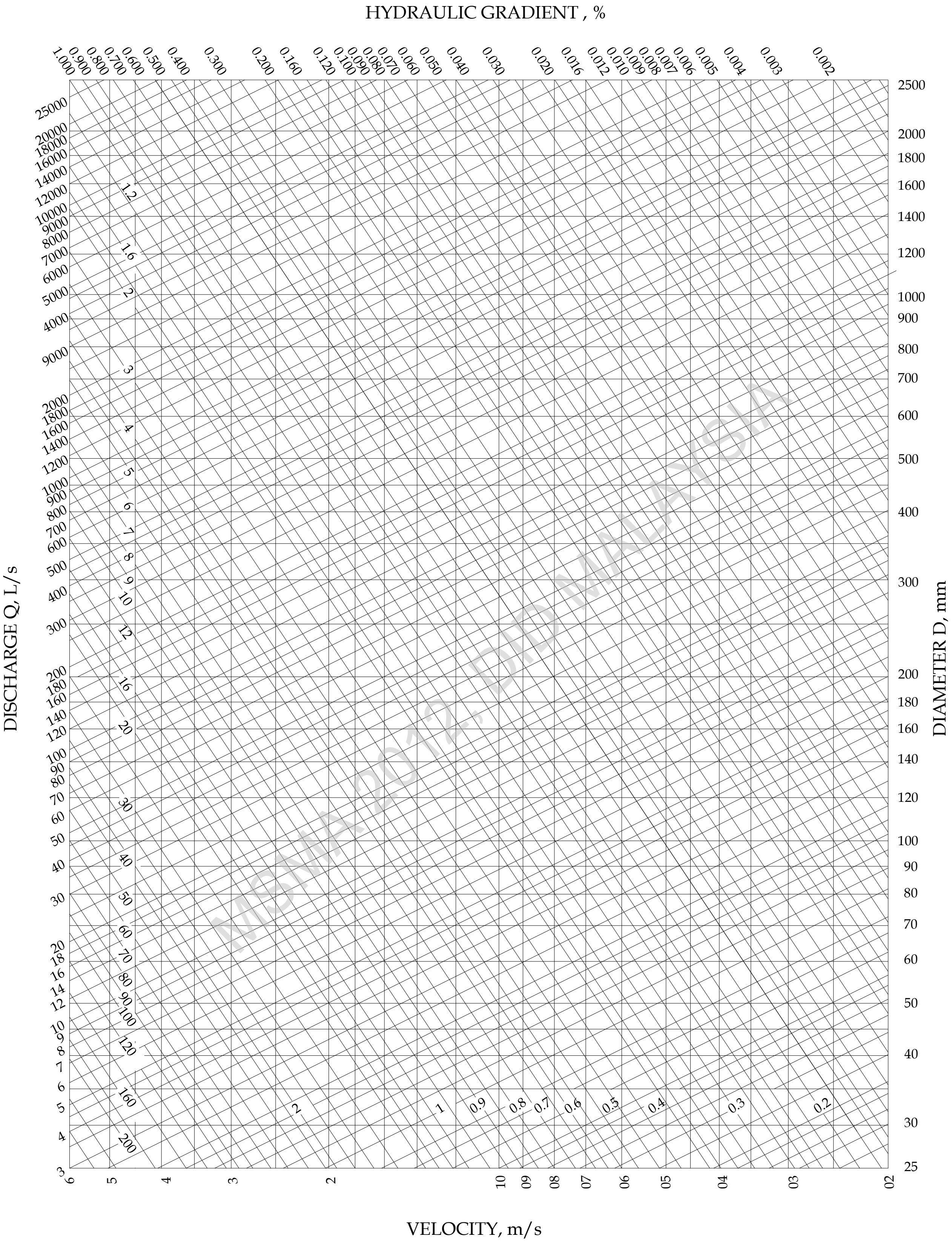
A = Cross-sectional Area per Cell

Design Chart 18.A10: Outlet Control Nomograph Concrete Box Culvert Flowing Full with n = 0.012



K_e	Wingwall Angle & Edge Finish
0.2	- Side-tapered or Slope-tapered
0.25	- Bevelled Edge
0.5	- Headwall or Wingwalls, Square Edge - Prefabricated End Section
0.7	- Mitred Parallel to Fill Slope
0.9	- Projecting

Design Chart 18.A11: Outlet Control Nomograph – Corrugated Metal Pipe (CMP) Flowing Full with n = 0.024



Design Chart 18.A12: Hydraulic Design of Pipes – Colebrook-White Formula – k = 0.60 mm

APPENDIX 18.B EXAMPLE – CULVERT DESIGN

18.B1 Pipe Culvert (Inlet Control)

Problem:

Figure 18.B1 shows a proposed culvert located near a road intersection to be sized to accommodate a given design flow of 4.8m^3/s. Road level as well as culvert inflow and outflow inverts are as given. Determine a suitable pipe culvert (k = 0.6mm) and calculate the velocity to check if erosion will be a problem.

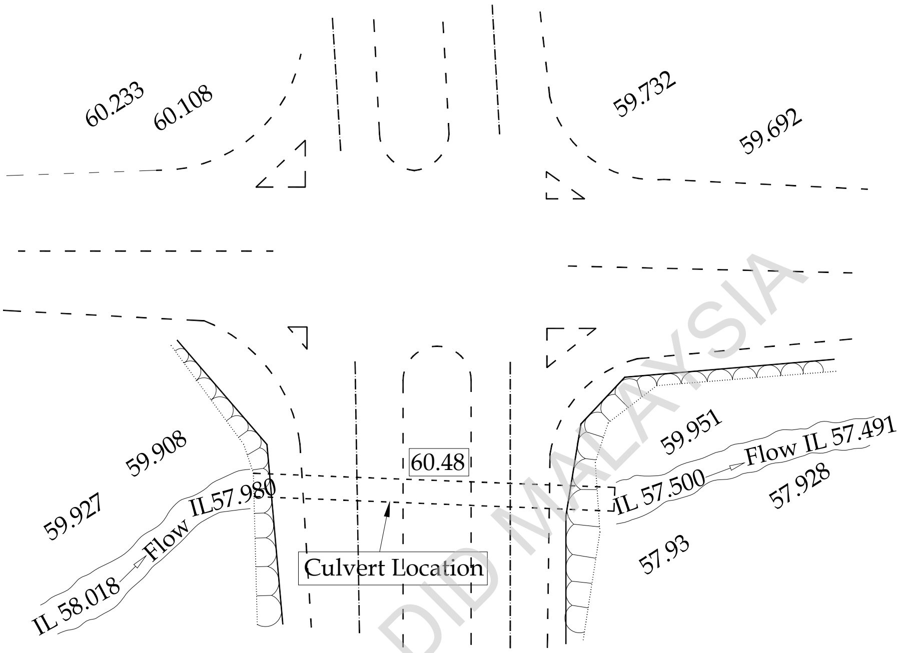
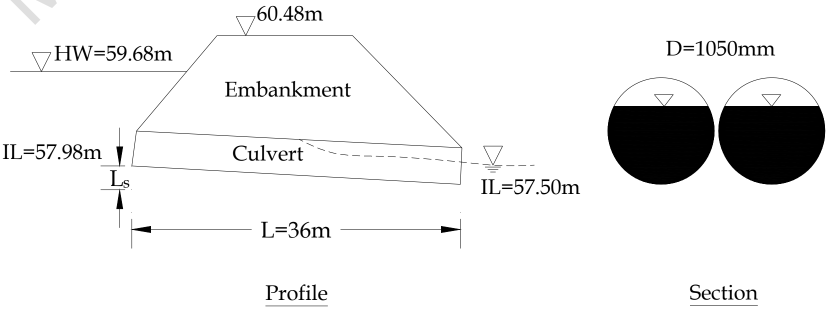


Figure 18.B1: Culvert Location and Levels

Solution:

Reference	Calculation	Output
	STEP 1: Data Flow Q = 4.8m^3/s Culvert length, L = 36m Natural waterway inverts level: Inlet = 57.98m Outlet = 57.50m Acceptable upstream flood level = 59.73m Proposed pavement level = 60.48m Minimum freeboard = 0.30m Estimated downstream tailwater level = 58.30m Maximum headwater height, HW_{max}, is the lesser of: i) 60.48 – 0.3 – 2.98 = 2.20m ii) 59.73 – 2.98 = 1.75m Therefore maximum headwater height, HW_{max} = 1.75 m	HW_{max} = 1.75m

	STEP 2: Assume Inlet Control Estimate required waterway flow area by assuming flow velocity, V = 2.0m/s. Estimated flow area, $A = Q/V = 2.40\text{m}^2$	
Use Design Chart 18.A2	i) Try 1650mm diameter reinforced concrete pipe (RCP), D = 1.65m Enter Design Chart 18.A2 with $Q/N = 4.8/1 = 4.8\text{m}^3/\text{s}$; Inlet Type (1): Obtain HW_i/D = 1.10m HW_i = 1.81m > 1.75m maximum. Not acceptable.	$HW_i > HW_{max}$, not acceptable
Use Design Chart 18.A2	ii) Try RCP 1800 mm diameter Enter Design Chart 18.A2 with $Q/N = 4.8/1 = 4.8\text{m}^3/\text{s}$; Inlet Type (1): Obtain HW_i/D = 0.94m HW_i = 1.69m < 1.75m But maximum culvert height available only 1.75m	
Use Design Chart 18.A2	iii) Try twin lines, 2/1050mm diameter Enter Design Chart 18.A2 with $Q/N = 4.8/2 = 2.40\text{m}^3/\text{s}$; Inlet Type (1): Obtain HW_i/D = 1.62m HW_i = 1.70m < 1.75m Use 2/1050mm diameter pipes	$HW_i < HW_{max}$, ok
	STEP 3: Check Outlet Control $TW = 58.30 – 57.50 = 0.80\text{m} < D$ (1.05m)	
Use Design Chart 18.A7	Enter Design Chart 18.A7 with D = 1.05m; $Q/N = 4.8/2 = 2.4\text{m}^3/\text{s}$ h_c = 0.80m< D (1.05m) $(h_c+D)/2 = (0.80 + 1.05)/2 = 0.93\text{m} > TW = 0.80\text{m}$	
Use Design Chart 18.A9	Calculate HW_o for outlet control: Enter Design Chart 18.A9 with D = 1.05 m; $Q/N = 4.8/2 = 2.4\text{ m}^3/\text{s}$; K_e = 0.5; and L = 36m Obtain H = 0.85m Fall of culvert invert, $L_S = 57.98 – 57.50 = 0.48\text{m}$ hence: $HW_o = (h_c+D)/2 + H - L_S$ $= 0.93 + 0.85 – 0.48$ $= 1.30\text{m}$	
	HW_i (Inlet control) = 1.70m greater than HW_o (Outlet control) = 1.30m Therefore inlet control governs.	Inlet Control $HW_i < HW_{max}$, ok

	STEP 4: Flow Velocity For 1050mm diameter pipes: $A = (\pi D^2)/4 \quad = 0.87\text{m}^2$ $S = 0.48/36 \quad = 0.0133$ (= 1.33%)	
Use Design Chart 18.A12	From Colebrook-White's Chart for k = 0.6mm (Chart 18.A12) $Q_f = 3.40\text{m}^3/\text{s}$ $V_f = 3.80\text{m/s}$	
Use Design Chart 18.A5	Because the culvert does not flow full it is necessary to use part-full flow relationships plotted in Design Chart 18.A5: $Q/Q_f \quad = 2.4/3.4 = 0.71$ and from Design Chart 18.A5, $V/V_f \quad = 0.96$ and $V \quad = 0.96 \times 3.80 \quad = 3.65\text{m/s}$ $y/D \quad = 0.68$ and $y = 0.68 \times 1.05$ $y \quad = 0.71\text{m} < h_c = 0.80\text{m}$	
	STEP 5: Summary Use 2/1050mm diameter concrete pipes with square edge entrance.	Use 2/1050mm mm diameter pipes with square edge entrance.
	Pipes will flow with inlet control with headwater height of 1.70m and headwater at RL 59.68m. Outlet velocity = 3.65m/s and the possibility of scour or the formation of a hydraulic jump at the outlet must be checked.  Profile Section	

18.B2 Box Culvert (Inlet Control)

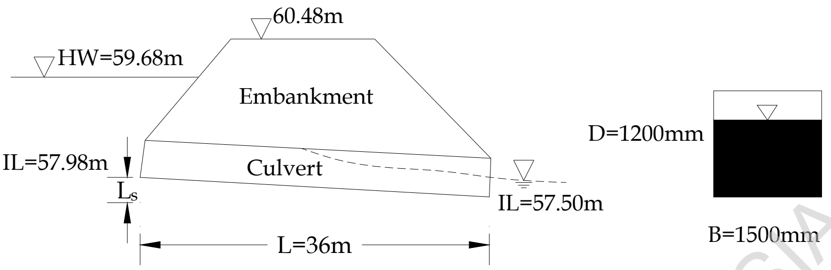
Problem:

Using the same data as provided for the previous pipe culvert (Example 18.B1), calculate a suitable box culvert size and check for the effects of the outlet velocity.

Solution:

Reference	Calculation	Output
	STEP 1: Data Flow Q = 4.8m^3/s Culvert length, L = 36m Natural waterway inverts level: Inlet = 57.98m Outlet = 57.50m Acceptable upstream flood level = 59.73m Proposed pavement level = 60.48m Minimum freeboard = 0.30m Estimated downstream tailwater level = 58.30m Maximum headwater height, HW_{max}, is the lesser of: i) 60.48 – 0.3 – 2.98 = 2.20m ii) 59.73 – 2.98 = 1.75m Therefore maximum headwater height, HW_{max} = 1.75 m	HW_{max} = 1.75m
	STEP 2: Assume Inlet Control Estimate required waterway flow area by assuming flow velocity, V = 2.0m/s. Estimated flow area, $A = Q/V$ = 2.40m^2 Try 1500mm (wide) and 1200mm (high) box culvert	
Use Design Chart 18.A3	Enter Design Chart 18.A3 with D = 1.2m; Q/NB = 4.8/(1x1.5) = 3.2m^3/s/m; Inlet Type (1): Obtain HW_i/D = 1.42m HW_i = 1.70m > 1.75m maximum, which is acceptable	HW_i<HW_{max}, ok
	STEP 3: Check Outlet Control TW = 58.30 – 57.50 = 0.80m< D (1.20m)	
Use Design Chart 18.A8	Enter Design Chart 18.A8 with B = 1.50m; Q/N = 4.8/1 = 4.80m^3/s h_c = 1.00m< D (1.20m) (h_c+D)/2 = (1.00 + 1.20)/2 = 1.10m > TW = 0.80m	

	Calculate headwater for outlet control (HW_o):	
Use Design Chart 18.A10	Enter Design Chart 18.A10 with A = 1.2 x 1.5 = 1.80 m^2; Q/N = 4.8/1 = 4.8 m^3/s; K_e = 0.5; and L = 36m	
	Obtain H = 0.75m	
	Fall of culvert invert, L_S = 57.98 – 57.50 = 0.48m hence: HW_o = $(h_c+D)/2 + H - L_S$ = 1.10 + 0.75 – 0.48 = 1.37m	
	HW_i (Inlet control) = 1.70m greater than HW_o (Outlet control) = 1.37m	Inlet Control $HW_i < HW_{max}$, ok
	Therefore inlet control governs.	
	STEP 4: Flow Velocity	
	Hydraulic radius R = Area/wetted perimeter	
	R = 1.8/(2(1.5+1.2)) = 0.33m^2 Equivalent D = 4 x 0.33 = 1.33m and S = 0.48/36 = 0.0133 (= 1.33%)	
Use Design Chart 18.A12	From Colebrook-White's Chart for k = 0.6mm (Chart 18.A12)	
	V_f = 4.50m/s Q_f = 1.8 x 4.50 = 8.10m^3/s	
Use Design Chart 18.A6	Because the culvert does not flow full it is necessary to use part-full flow relationships plotted in Design Chart 18.A6:	
	Q/Q_f = 4.8/8.1 = 0.59 and from Design Chart 18.A6 B/D = 1.25,	
	V/V_f = 1.05 and V = 1.05 x 4.50 = 4.73m/s	
	y/D = 0.57 and y = 0.57 x 1.20 y = 0.68 < h_c = 1.00m	

	STEP 5: Summary Use 1500mm (wide) by 1200mm (high) concrete box culvert with square edges. Culvert will flow with inlet control with headwater height of 1.70m and headwater at RL 59.68m. Outlet velocity = 4.73m/s and the possibility of scour or the formation of a hydraulic jump at the outlet must be checked.  Profile Section	Use 1500mm (wide) by 1200 (high) box culvert with square edges.

18.B3 Pipe Culvert (Outlet Control)

Problem:

Figure 18.B2 shows a different culvert crossing located at a road junction to be sized to accommodate a given design flow of 0.90m^3/s. Road level as well as culvert inflow and outflow inverts are as given. Determine a suitable pipe culvert (k = 0.6mm) and calculate the velocity to check if erosion will be a problem.

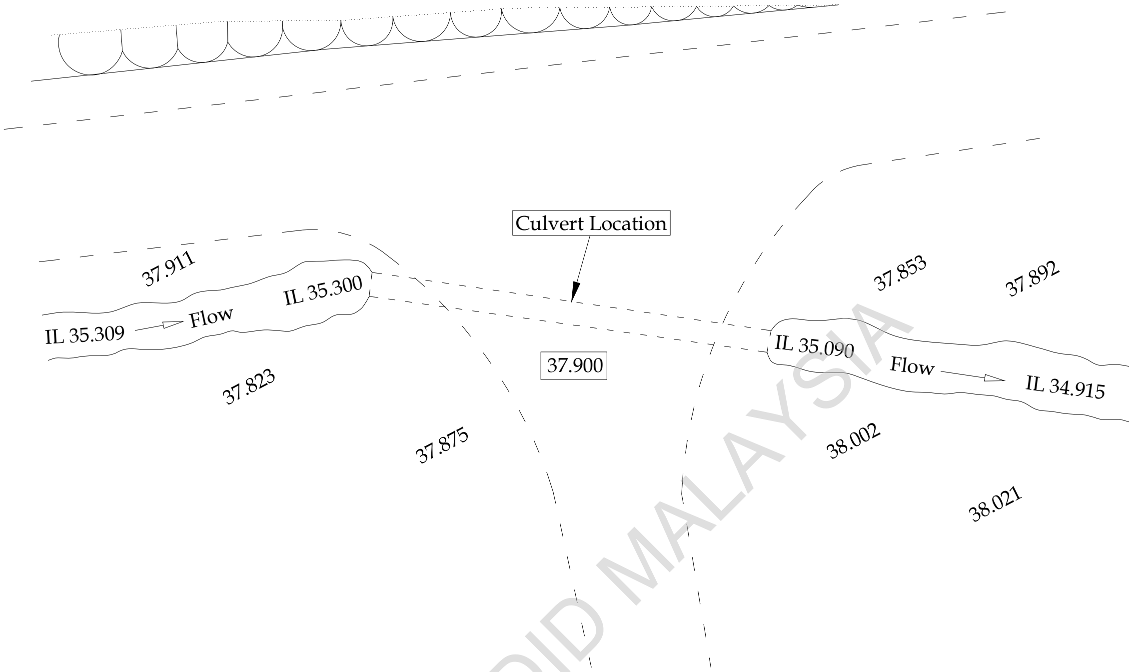


Figure 18.B2: Culvert Location and Levels

Solution:

Reference	Calculation	Output
	STEP 1: Data Flow Q = 0.90m^3/s Culvert length, L = 25m Natural waterway inverts level: Inlet = 35.30m Outlet = 35.09m Acceptable upstream flood level = 37.50m Proposed pavement level = 37.90m Minimum freeboard = 0.30m Estimated downstream tailwater level = 36.59m Maximum headwater height, HW_{max}, is the lesser of: i) 37.90 – 0.3 – 5.30 = 2.30m ii) 37.50 – 5.30 = 2.20m Therefore maximum headwater height, HW_{max} = 2.20 m	HW_{max} = 2.20m

	STEP 2: Assume Inlet Control Estimate required waterway flow area by assuming flow velocity, V = 2.0m/s. Estimated flow area, $A = Q/V$ = 0.45m^2 Try 600mm diameter concrete pipe, D = 0.60m	
Use Design Chart 18.A2	Enter Design Chart 18.A2 with D = 0.60m; Q/N = 0.90/1 = 0.90m^3/s; Inlet Type (1): Obtain HW_i/D = 2.80m HW_i = 1.68m for inlet control This depth is less than the limit of 2.20m	HW_i<HW_{max}, ok
	STEP 3: Check Outlet Control	
Use Design Chart 18.A9	TW = 36.59 – 35.09 = 1.50m> D (0.60m), the culvert is flowing full with a submerged outlet. Enter Design Chart 18.A9 with D = 0.60m; Q/N = 0.90/1 = 0.90m^3/s; K_e = 0.5; and L = 25m Obtain H = 1.25m	
	Fall of culvert invert, L_S = 35.30 – 35.09 = 0.21m hence: HW_o = TW + H - L_S = 1.50 + 1.25 – 0.21 = 2.54m	HW_o>HW_{max}, not acceptable
	Note that because 2.54m > 1.68m for inlet control, the culvert is under outlet control.	
	However the design is unacceptable because HW_{max}= 2.20m	
Use Design Chart 18.A2	Return to Step 2 using 750mm pipe diameter in Design Chart 18.A2 Obtain HW_i/D = 1.45m HW_i = 1.45 x 0.75 = 1.09m for inlet control	HW_i<HW_{max}, ok
Use Design Chart 18.A9	Now check for outlet control. Re-enter Design Chart 18.A9 with D = 0.75m and obtain H = 0.48m HW_o = TW + H - L_S = 1.50 + 0.48 – 0.21 = 1.77m	
	HW_o (Outlet control) = 1.77m greater than HW_i (Inlet control) = 1.09m	Outlet control HW_o<HW_{max}, ok
	Therefore outlet control governs.	

STEP 4: Flow Velocity

With HW and TW both well above the crown of the pipe and a moderate slope of 0.21/25 = 0.0084 the pipe will flow full hence:

$V = Q/A$

$= 4 \times 0.9/(3.142 \times 0.75^2) = 2.04 \text{m/s}$

The velocity must be checked against erosion at outlet.

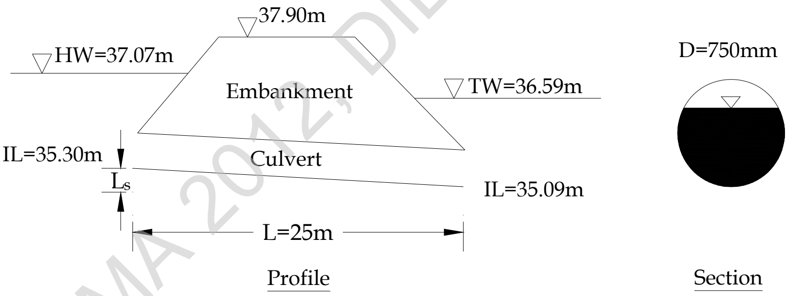
STEP 5: Summary

Use single line of 750mm diameter concrete pipe with square edge entrance.

Use concrete pipe 750mm diameter with square edge.

Pipe will flow full under outlet control with headwater height of 1.77m and headwater at RL 37.07m.

Outlet velocity = 2.04m/s and the possibility of scour or the formation of a hydraulic jump at the outlet must be checked.



Profile

Section

18.B4 Box Culvert (Outlet Control)

Problem:

Using the same data as provided for the previous pipe culvert (Example 18.B3), calculate a suitable box culvert size and check for the effects of the outlet velocity.

Solution:

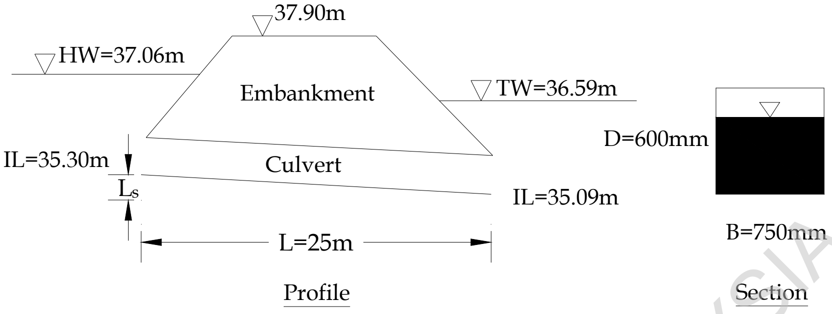
Reference	Calculation	Output
	STEP 1: Data	
	Flow Q = $0.90 m^3/s$	
	Culvert length, L = 25m	
	Natural waterway inverts level:	
	Inlet = 35.30m Outlet = 35.09m	
	Acceptable upstream flood level = 37.50m	
	Proposed pavement level = 37.90m	
	Minimum freeboard = 0.30m	
	Estimated downstream tailwater level = 36.59m	
	Maximum headwater height, HW_{max}, is the lesser of:	
	i) 37.90 – 0.3 – 5.30 = 2.30m	
	ii) 37.50 – 5.30 = 2.20m	
	Therefore maximum headwater height, HW_{max} = 2.20 m	HW_{max} = 2.20m
	STEP 2: Assume Inlet Control	
	Estimate required waterway flow area by assuming flow velocity, V = 2.0m/s.	
	Estimated flow area, $A = Q/V = 0.45m^2$	
	Try 750mm (wide) and 600mm (high) box culvert	
Use Design Chart 18.A3	Enter Design Chart 18.A3 with D = 0.60m and Q/NB = 0.90/(1x0.75) = $1.2m^3/s/m$; Inlet Type (1):	
	Obtain HW_i/D = 1.50m	
	HW_i = 1.5 x 0.60	
	= 0.9m < 2.20 maximum, which is acceptable	HW_i<HW_{max}, ok

Use Design Chart 18.A10	STEP 3: Check Outlet Control TW = 1.5m (see Example 18.B3) > 0.6m hence the culvert is flowing full with a submerged outlet. Calculate H from Design Chart 18.A10, noting that B/D=1.25, so the chart is applicable: Enter Design Chart 18.A10 with A = 0.75 x 0.60 = 0.45m²; Q/N = 0.90/1 = 0.90m³/s; K_e = 0.5; and L = 25m Obtain H = 0.47m Fall of culvert invert, L_S = 35.30 – 35.09 = 0.21m hence: HW_o = TW + H - L_S = 1.50 + 0.47 – 0.21 = 1.76m HW_o(Outlet control) = 1.76m greater than HW_i (Inlet control) = 0.9m Therefore outlet control governs. STEP 4: Flow Velocity As the culvert flows full, $V = Q/A$ = 0.9/(0.75 x 0.60) = 2.00m/s The velocity must be checked against erosion at outlet.	Outlet control HW_o<HW_{max}, ok

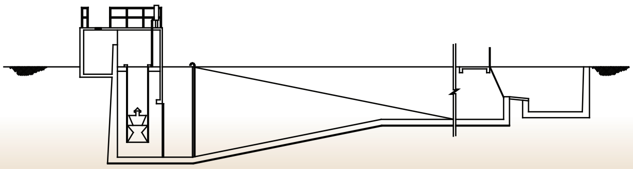
STEP 5: Summary

Use a single 750mm (wide) by 600mm (high) concrete box culvert with square edges entrance.

The culvert will flow with outlet control with headwater height of 1.76m and headwater at RL 37.06m and outlet velocity = 2.00m/s and the possibility of scour or the formation of a hydraulic jump at the outlet must be checked.



Use 750mm (wide) by 600mm (high) box culvert with square edges.



GATE AND PUMP

19

CHAPTER 19 GATE AND PUMP

19.1 INTRODUCTION

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

19.2 SYSTEM COMPONENTS

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

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

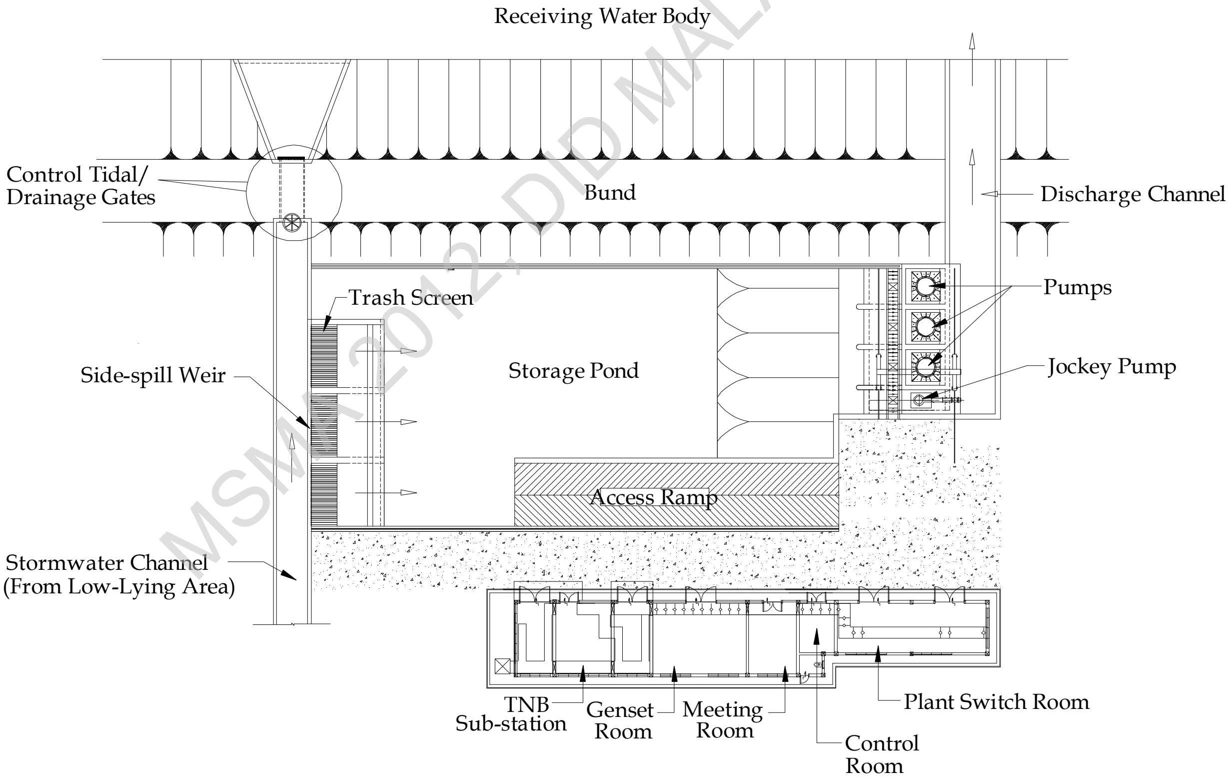


Figure 19.1: Typical Layout and System Components of Gates and Pump Station

Figure 19.2: Photograph of Pump Station

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

19.3 TIDAL AND DRAINAGE GATES

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

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

19.3.1 Types of Gates

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

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

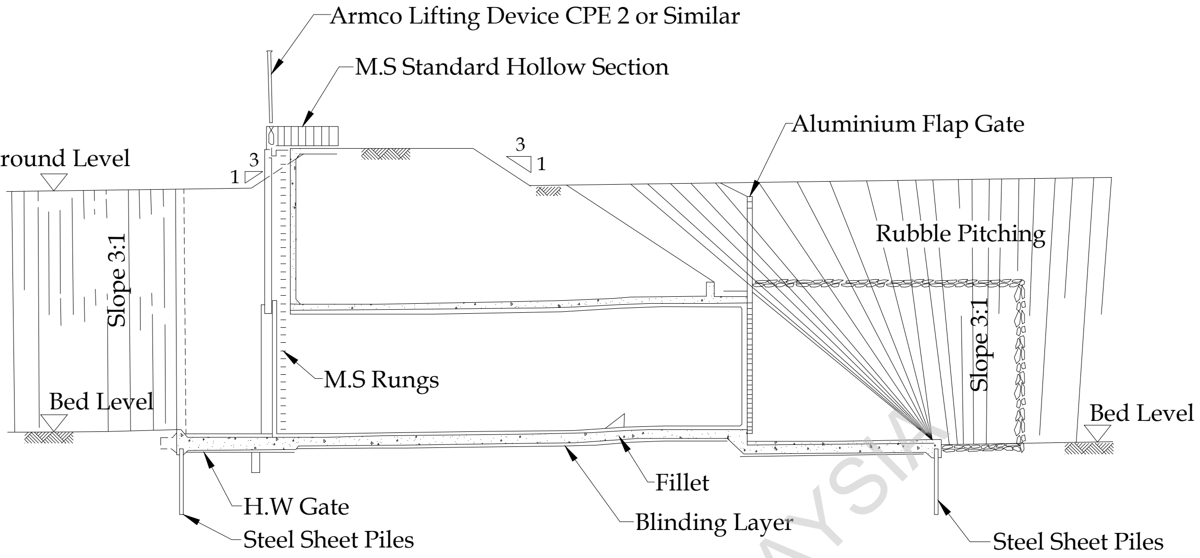


Figure 19.3: Typical Culvert Type Outlet Structure

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

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

19.3.2 Design Consideration for Tidal Gates

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

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

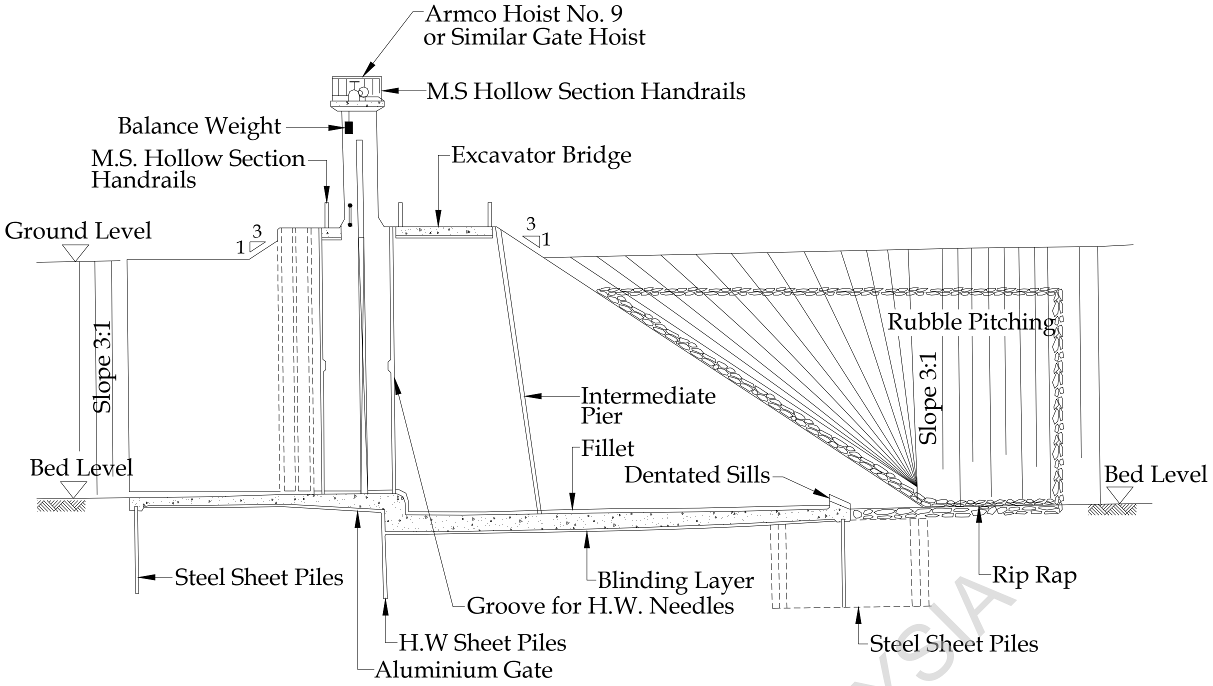
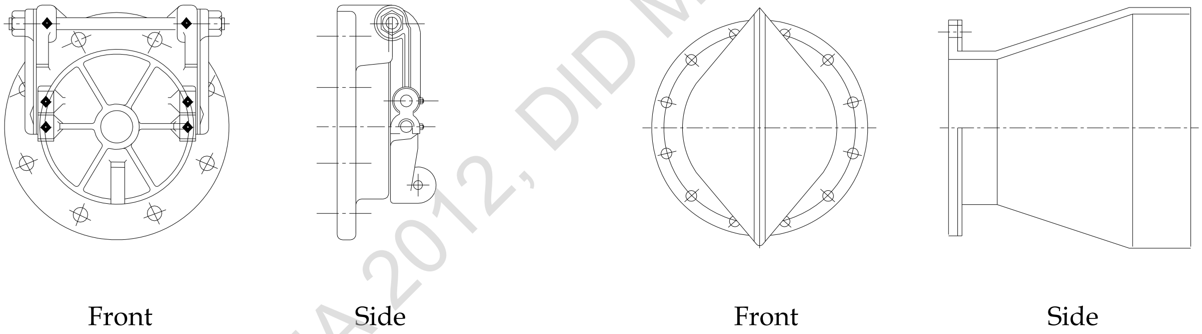


Figure 19.4: Typical Flume Type Outlet Structure



a) Flap Gate b) Duckbill Valve

Figure 19.5: Typical Flap Gate and Duckbill Check Valve

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

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

19.3.3 Design Consideration for Drainage Gates

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

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

19.4 PUMPING STATION

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

19.4.1 Planning Process

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

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

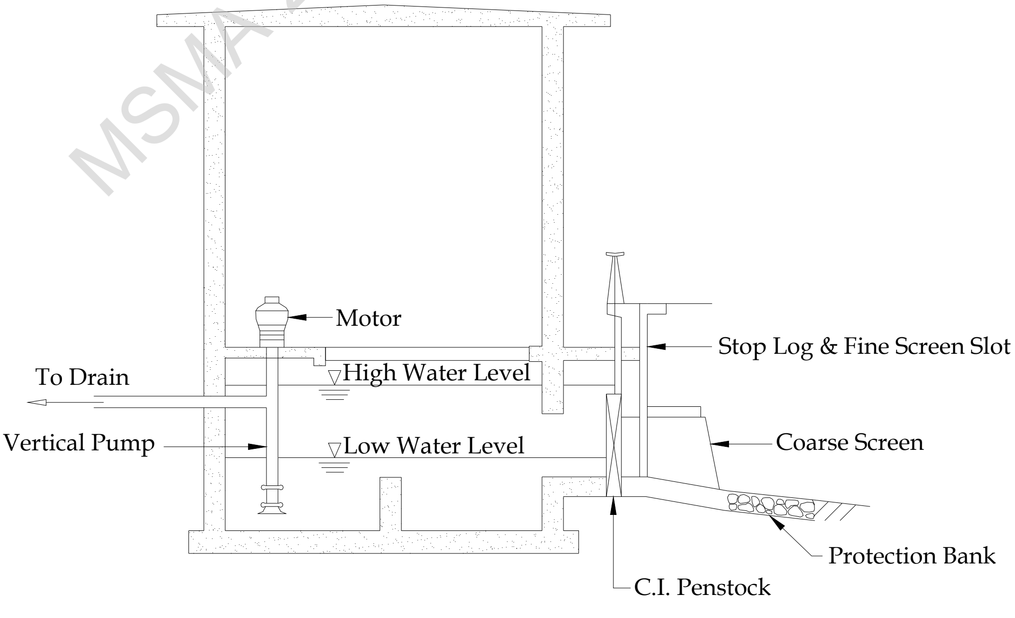


Figure 19.6: Typical Wet-pit Station

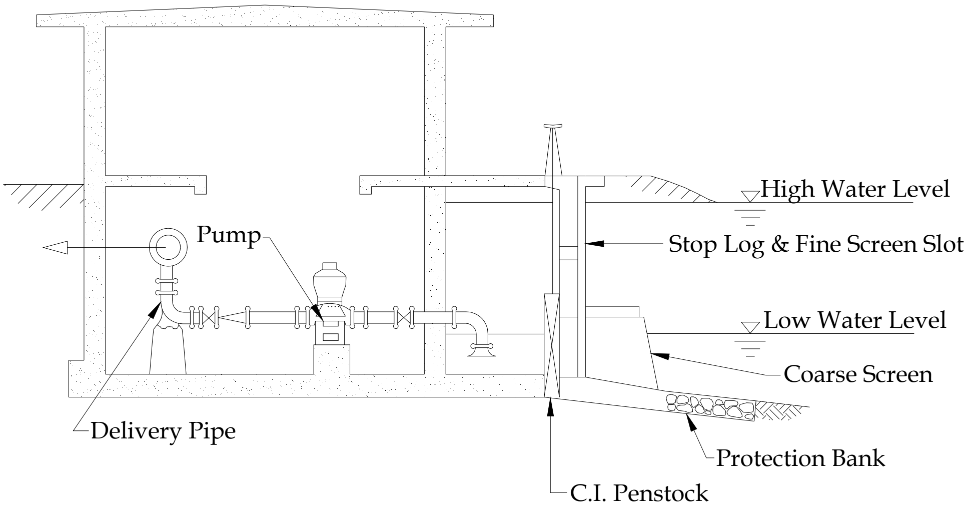


Figure 19.7: Typical Dry-pit Station

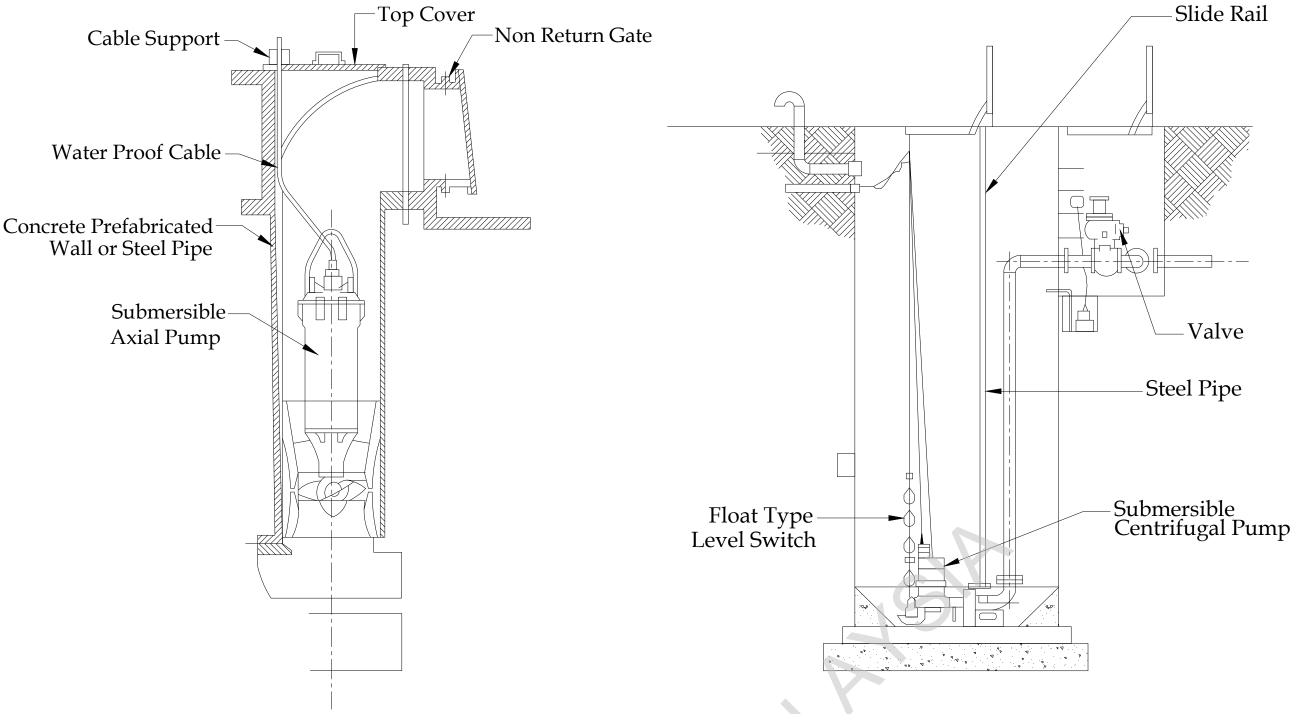
Pump Types - Various types of pumps are used and they are as listed out in Table 19.1 and shown in Figures 19.8 and 19.9. For centrifugal and axial pumps, submersible type frequently provides special advantages in simplifying the design, construction, maintenance and therefore the cost of the pumping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

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

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

Table 19.1: Pump Types for Stormwater Drainage

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



a) Axial b) Centrifugal

Figure 19.8: Typical Submersible Pumps

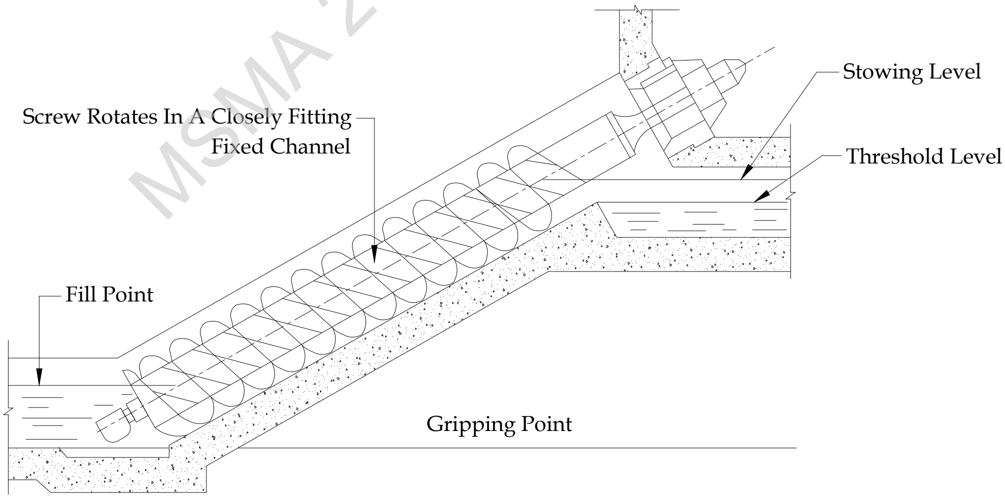
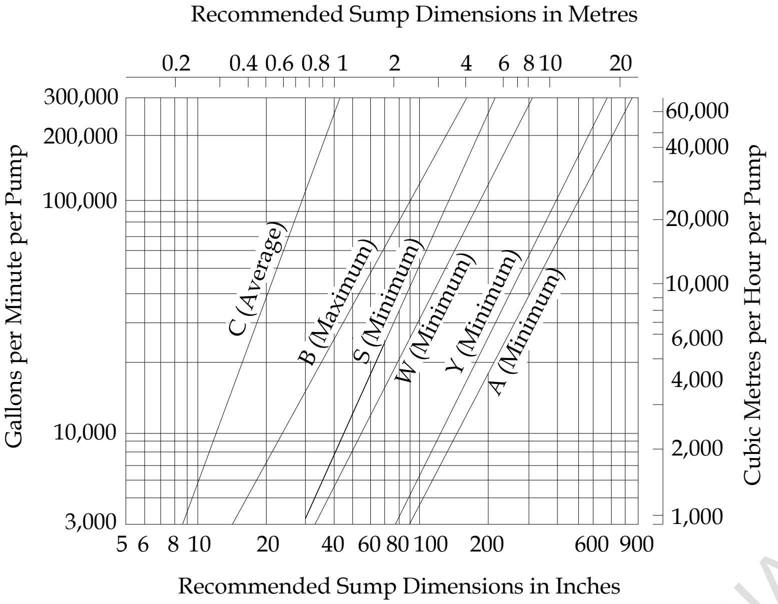
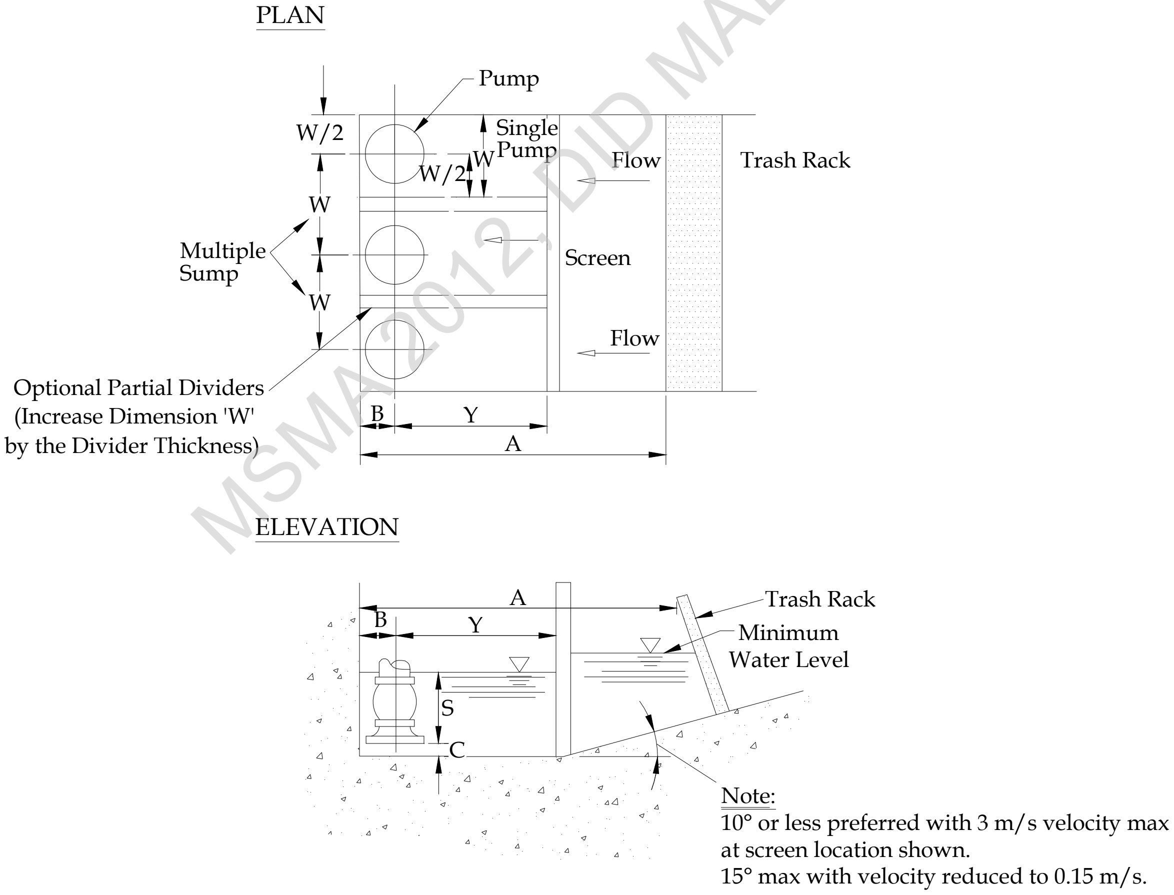


Figure 19.9: Typical Screw Pump



a) Sump Dimensions (see b for dimension location)



b) Plan and Elevation

Figure 19.10: Wet Pit Type Pump

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

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

19.4.2 Design Considerations for Pump and Storage Sizing

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

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

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

where

TDH = Total dynamic head (m);
H_s = Static head (m);
h_f = Friction head (m);
h_v = Velocity head (m); and
h_l = Losses through fittings, valves, etc. (m)

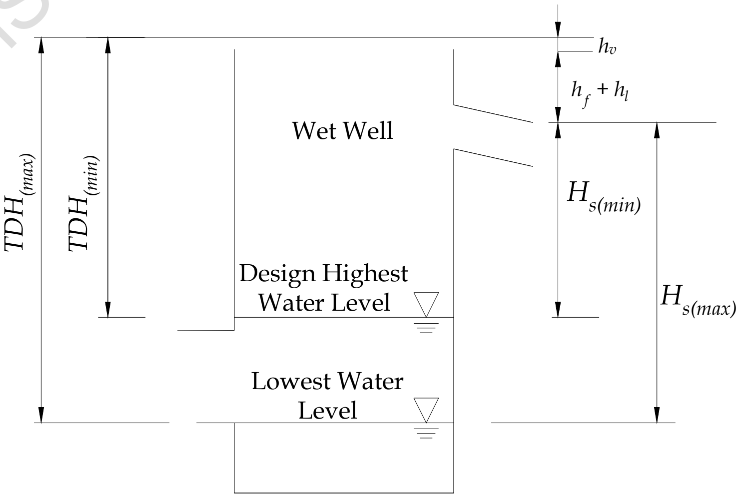
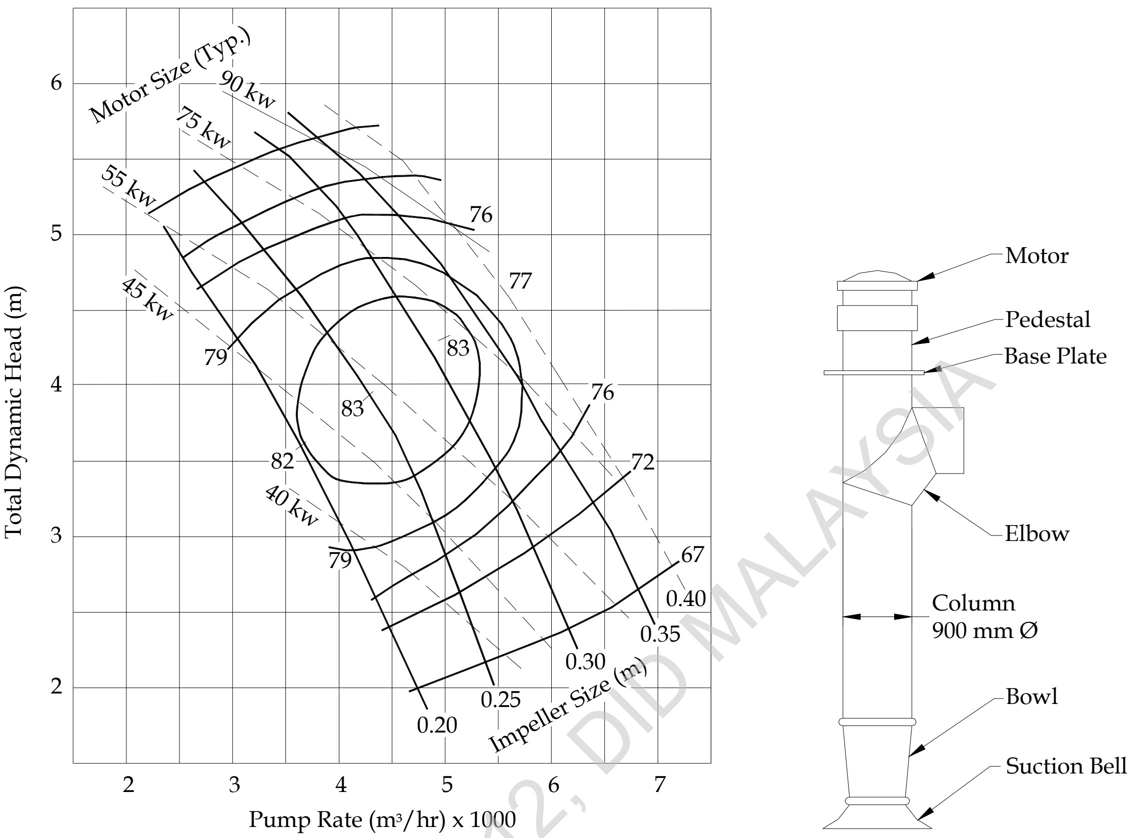


Figure 19.11: Components of Total Dynamic Head (TDH)

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



Typical Outline of 900 mm Propeller Pump

Figure 19.12: Performance Curve for 900mm Pump Rotating

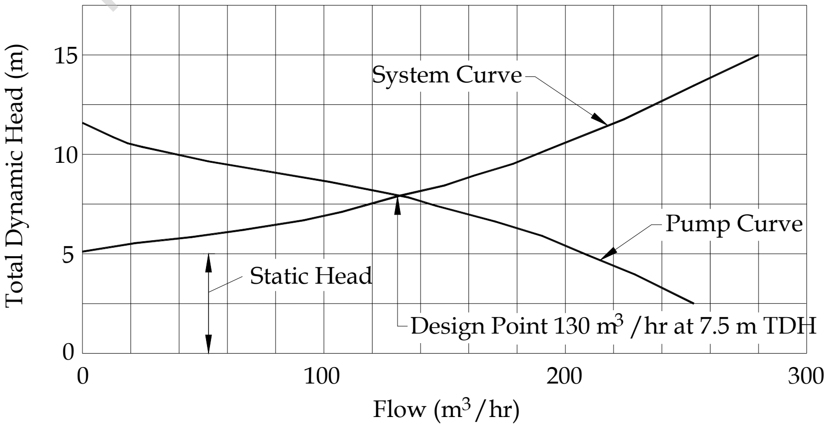


Figure 19.13: System Head Curve

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

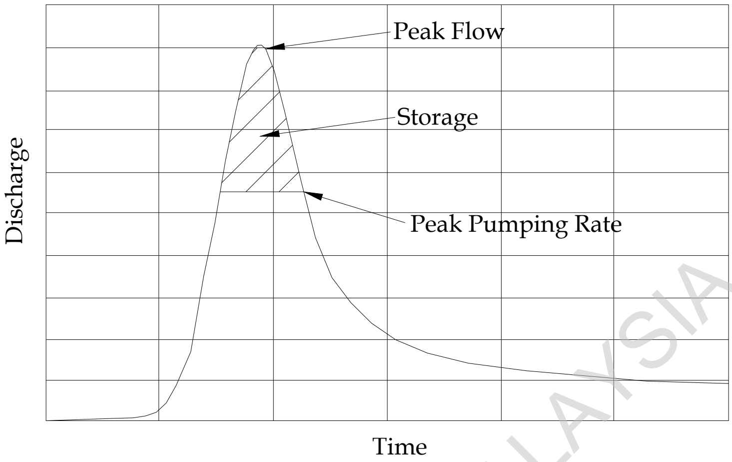


Figure 19.14: Estimation of Required Storage from Inflow Hydrograph.

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

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

19.4.3 Other Facilities and Requirements

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

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

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

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

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

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

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

19.5 DESIGN PROCEDURE FOR TIDAL GATES

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

19.6 DESIGN PROCEDURE FOR DRAINAGE GATE

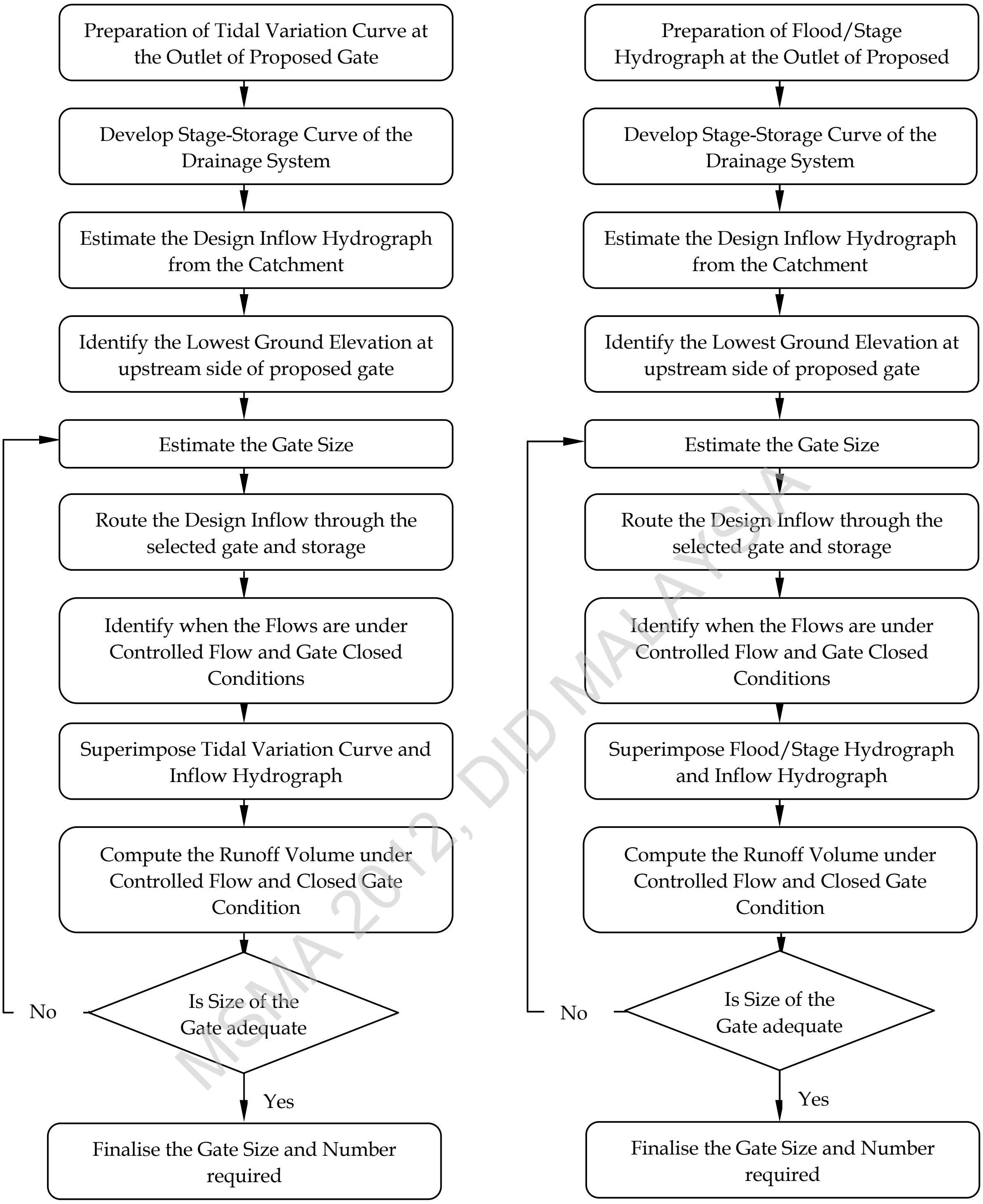
For the larger-size drainage gates, design is normally based on routing methods, which require a fairly detailed knowledge of the topography of the catchment area discharging through the gate in order to be able to assess accurately storage effects throughout the design flood hydrograph of the downstream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

19.7 DESIGN PROCEDURE FOR PUMPING STATION

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

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

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



a) Tidal Gate b) Drainage Gate

Figure 19.15: Flowcharts for Gate Design

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

Step 1: Inflow to Pump Station

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

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

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

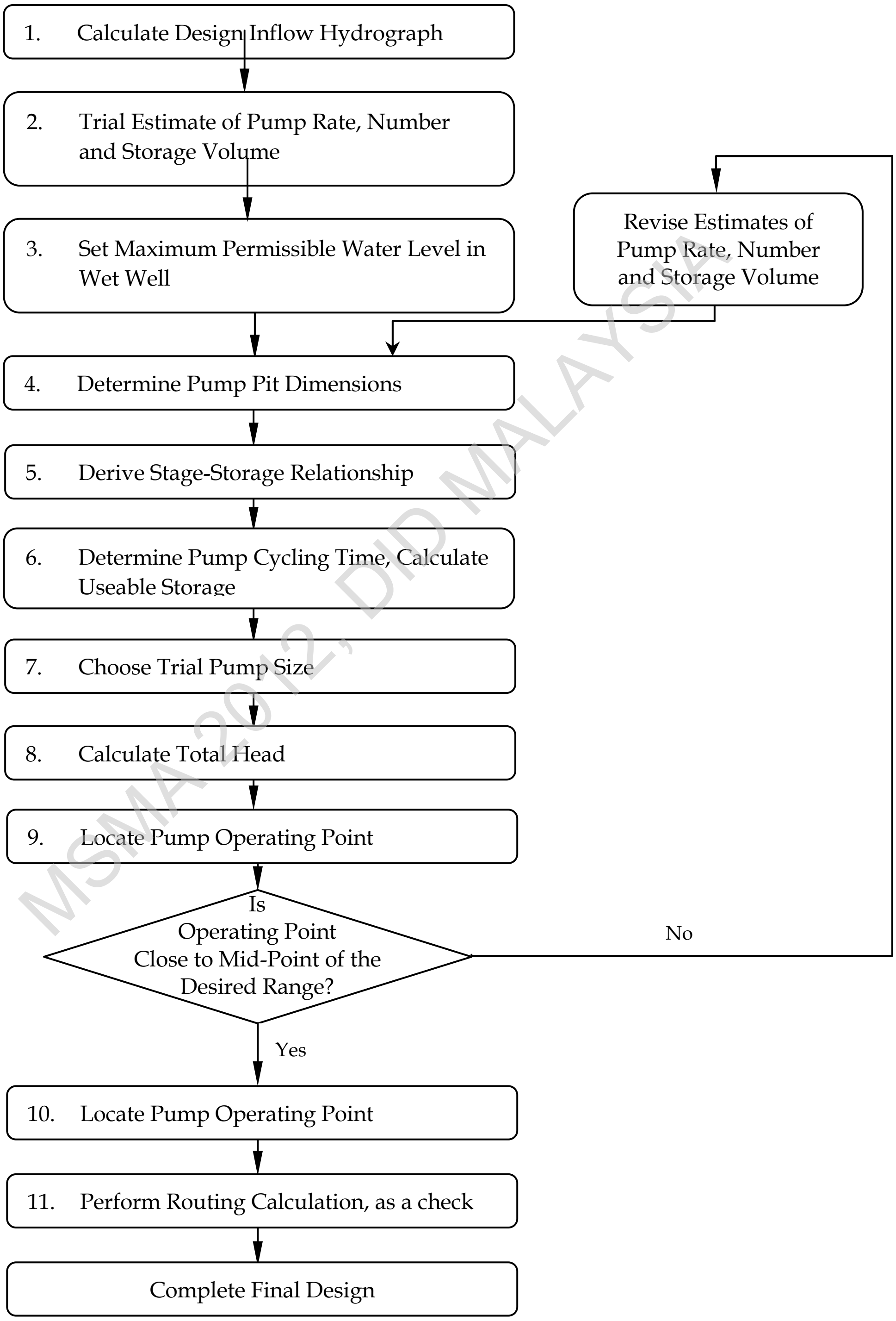


Figure 19.16: Flowchart for Pump Station Design

Step 3: Design High Water Level

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

Step 4: Determine Pump Pit Dimensions

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

Step 5: Stage – Storage Relationship

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

Step 6: Pump Cycling and Usable Storage

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

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

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

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

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

where;

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

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

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

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

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

Table 19.2: Pump Cycle Time Limits

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

Step 7: Trial Pumps and Pump Station Piping

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

Step 8: Total Dynamic Head

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

Step 9: Pump Design Operating Point

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

Step 10: Power Requirements

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

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

The pump power output can be determined as:

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

where;

P = Power output from the pump (kW);

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

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

H = Pump head (m).

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

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

where;

P_m = Pump motor rating (kW);

ε = Efficiency (%)

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

Step 11: Storage Routing

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

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APPENDIX 19.A STORAGE AND PUMP CAPACITY ESTIMATION

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

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

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

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

where,

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

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

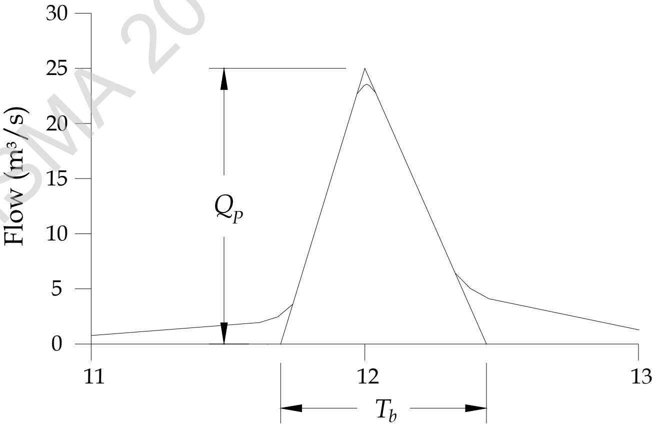


Figure 19.A1: Triangular Approximation of Inflow Hydrograph (Baumgardner, 1982)

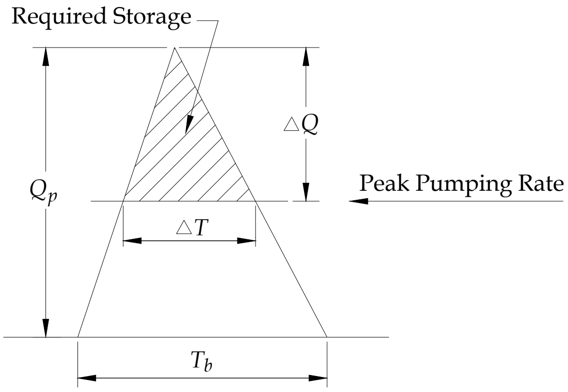


Figure 19.A2: Estimation of Required Storage Based on a Selected Peak Pumping Rate (Baumgardner, 1982)

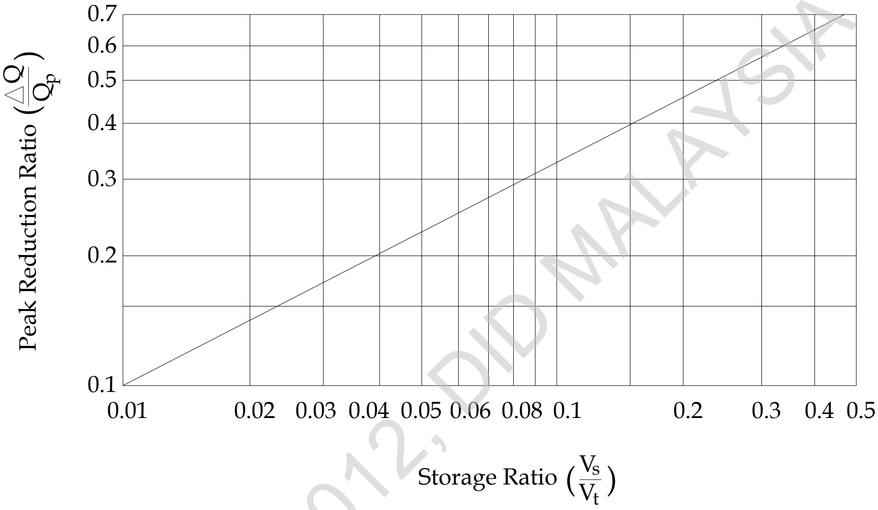


Figure 19.A3: Relationship between Peak Reduction Ratio and Storage Ratio : $Vs/V_T = (\Delta Q/Q_p)^2$ (Baumgardner, 1981)

APPENDIX 19.B EXAMPLE - PUMPING STATION DESIGN

Problem:

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

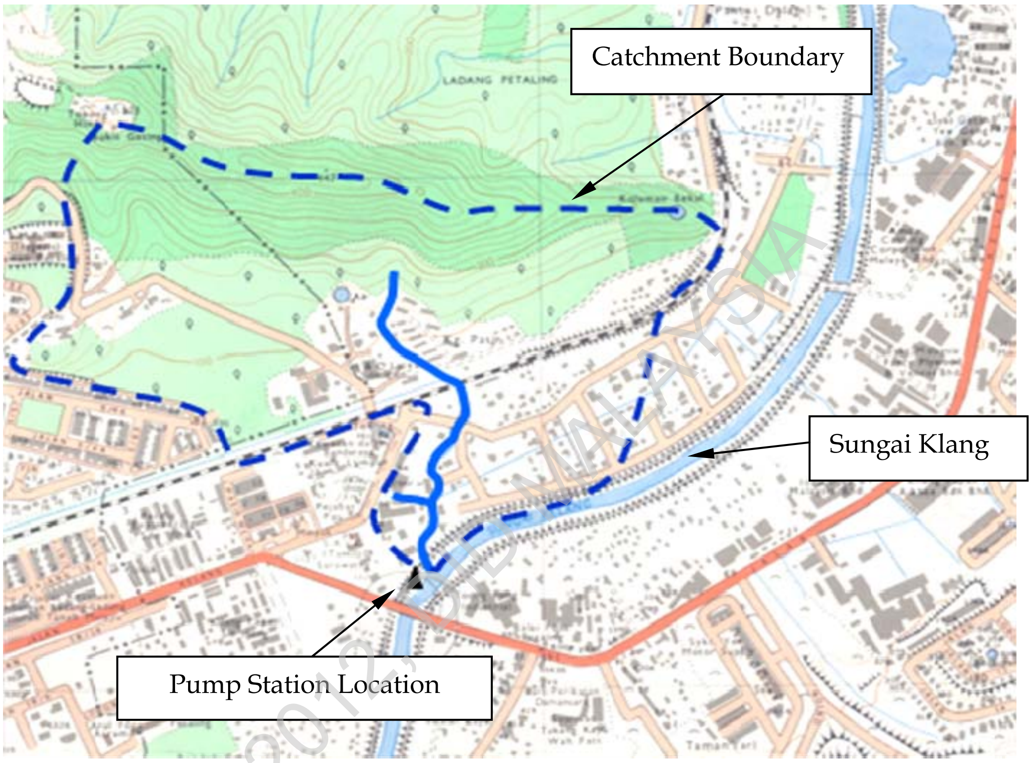


Figure 19.B1: Kampung Pasir Baru Pump Station

Solution:

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

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

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

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

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

Table 19.B1: Inflow Hydrographs using Rational Hydrograph Method

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

Table 19.B2: Stage – Storage Tabulation

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

Table 19.B3: Pump Controls and Operational Parameters

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

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

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

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

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

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

Time, t (mins)	I (m³/s)	$(I_1+I_2)/2$ (m³/s)	$S_1/\Delta t+Q_1/2$ (m³/s)	Q_1 (m³/s)	$S_2/\Delta t+Q_2/2$ (m³/s)	Q_2 (m³/s)	WL (mRL)	WL (mRL)	Discharge, Q (m³/s)	Storage, S (m³)	$S/\Delta t+Q/2$ (m³/s)
0	0.00				0.00	0.00	16.50	16.500	0.000	0.00	0.00
2	0.37	0.18	0.00	0.00	0.18	0.00	16.55	16.750	0.000	111.00	0.93
4	0.74	0.56	0.18	0.00	0.74	0.00	16.70	16.950	0.000	199.80	1.67
6	1.11	0.92	0.74	0.00	1.66	0.00	16.95	17.000	2.027	222.00	2.86
8	1.47	1.29	1.66	0.00	2.95	2.03	17.02	17.250	2.072	333.00	3.81
10	1.84	1.66	2.95	2.03	2.57	1.54	16.99	17.450	2.105	421.80	4.57
12	2.21	2.03	2.57	1.54	3.06	2.04	17.05	17.500	4.227	444.00	5.81
14	2.58	2.40	3.06	2.04	3.42	2.05	17.15	17.750	4.302	578.44	6.97
16	2.95	2.77	3.42	2.05	4.13	2.09	17.33	17.950	4.358	723.49	8.21
18	3.32	3.13	4.13	2.09	5.18	3.14	17.47	18.000	6.557	759.75	9.61
20	3.68	3.50	5.18	3.14	5.53	3.75	17.49	18.250	6.658	987.94	11.56
22	4.05	3.87	5.53	3.75	5.65	3.95	17.49	18.500	6.756	1263.00	13.90
24	4.42	4.24	5.65	3.95	5.94	4.23	17.53	18.750	6.855	1584.94	16.64
26	4.79	4.61	5.94	4.23	6.31	4.26	17.61	19.000	6.957	1953.75	19.76
28	5.16	4.98	6.31	4.26	7.02	4.30	17.76	19.250	7.066	2369.44	23.28
30	5.53	5.35	7.02	4.30	8.06	4.35	17.93	19.500	7.184	2832.00	27.19
32	5.90	5.71	8.06	4.35	9.42	6.26	17.99	19.750	7.313	3341.44	31.50
34	6.26	6.08	9.42	6.26	9.24	5.97	17.99	20.000	7.456	3897.75	36.21
36	6.63	6.45	9.24	5.97	9.71	6.56	18.01	For Rising Water Level			
38	7.00	6.82	9.71	6.56	9.97	6.58	18.05				
40	7.37	7.19	9.97	6.58	10.58	6.61	18.12				
42	7.74	7.56	10.58	6.61	11.52	6.66	18.24				
44	8.11	7.92	11.52	6.66	12.79	6.71	18.38				
46	8.47	8.29	12.79	6.71	14.37	6.77	18.54				
48	8.84	8.66	14.37	6.77	16.26	6.84	18.72				
50	8.47	8.66	16.26	6.84	18.07	6.90	18.87	16.450	0.000	0.00	0.00
52	8.11	8.29	18.07	6.90	19.46	6.95	18.98	16.500	1.920	0.00	0.96
54	7.74	7.92	19.46	6.95	20.44	6.98	19.05	16.750	1.977	111.00	1.91
56	7.37	7.55	20.44	6.98	21.01	7.00	19.09	16.950	2.018	199.80	2.67
58	7.00	7.18	21.01	7.00	21.20	7.00	19.10	17.000	4.054	222.00	3.88
60	6.63	6.82	21.20	7.00	21.01	7.00	19.09	17.250	4.145	333.00	4.85
62	6.26	6.45	21.01	7.00	20.47	6.98	19.05	17.450	4.211	421.80	5.62
64	5.90	6.08	20.47	6.98	19.57	6.95	18.98	17.500	6.340	444.00	6.87
66	5.53	5.71	19.57	6.95	18.33	6.91	18.89	17.750	6.452	578.44	8.05
68	5.16	5.34	18.33	6.91	16.76	6.86	18.76	18.000	6.557	759.75	9.61
70	4.79	4.97	16.76	6.86	14.88	6.79	18.59	18.250	6.658	987.94	11.56
72	4.42	4.61	14.88	6.79	12.69	6.70	18.37	18.500	6.756	1263.00	13.90
74	4.05	4.24	12.69	6.70	10.22	6.59	18.08	18.750	6.855	1584.94	16.64
76	3.68	3.87	10.22	6.59	7.50	6.40	17.63	19.000	6.957	1953.75	19.76
78	3.32	3.50	7.50	6.40	4.60	4.12	17.19	19.250	7.066	2369.44	23.28
80	2.95	3.13	4.60	4.12	3.61	3.61	16.99	19.500	7.184	2832.00	27.19
82	2.58	2.76	3.61	3.61	2.77	2.18	16.95	19.750	7.313	3341.44	31.50
84	2.21	2.39	2.77	2.18	2.98	2.54	16.96	20.000	7.456	3897.75	36.21
86	1.84	2.03	2.98	2.54	2.47	2.01	16.90	For Falling Water Level			
88	1.47	1.66	2.47	2.01	2.12	1.99	16.80				
90	1.11	1.29	2.12	1.99	1.42	1.95	16.62				
92	0.74	0.92	1.42	1.95	0.39	0.79	16.47				
94	0.37	0.55	0.39	0.79	0.16	0.32	16.46				
96	0.00	0.19	0.16	0.32	0.03	0.03	16.45				
98	0.00	0.00	0.03	0.03	0.00	0.00	16.45				
100	0.00	0.00	0.00	0.00	0.00	0.00	16.45				

Table 19.B7: Summary of the Routing Results

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

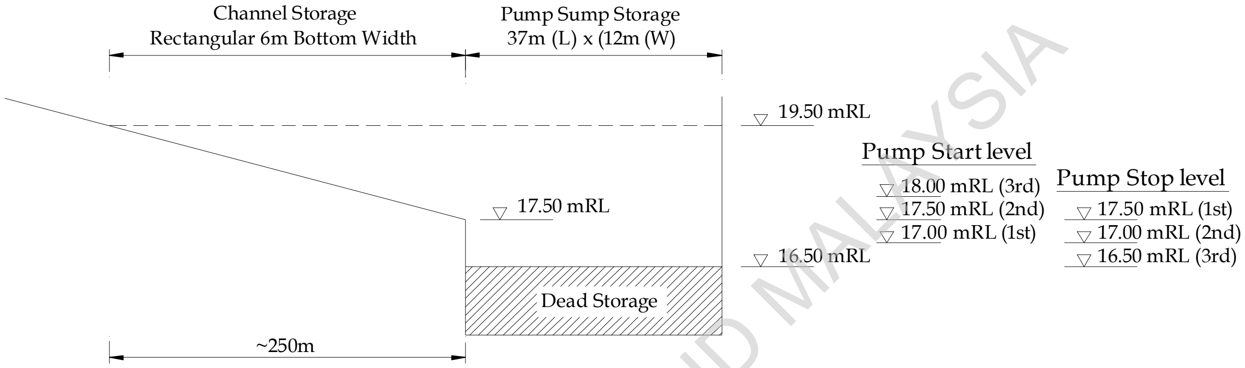


Figure 19.B2: Storage Sketch and Pump Start-Stop Levels

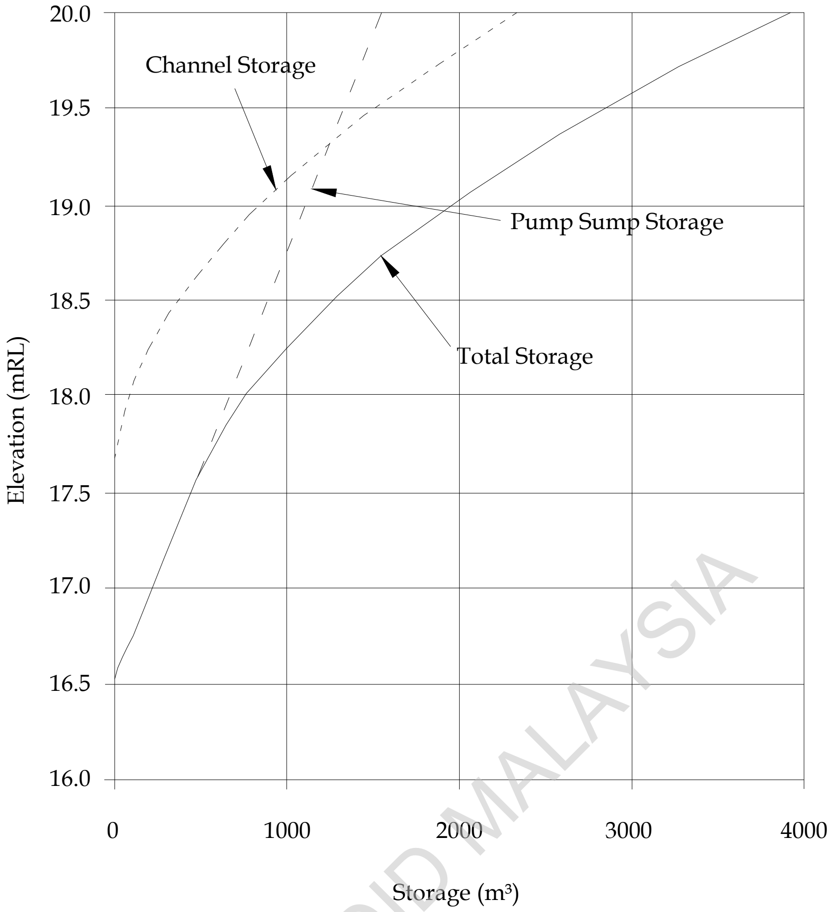


Figure 19.B3: Stage – Storage Curves

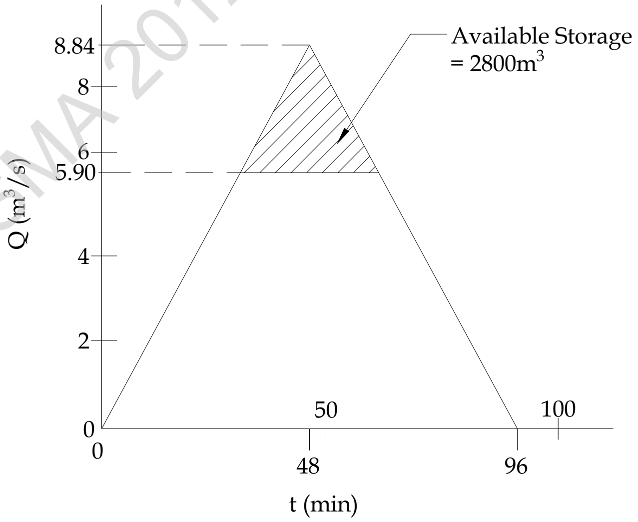
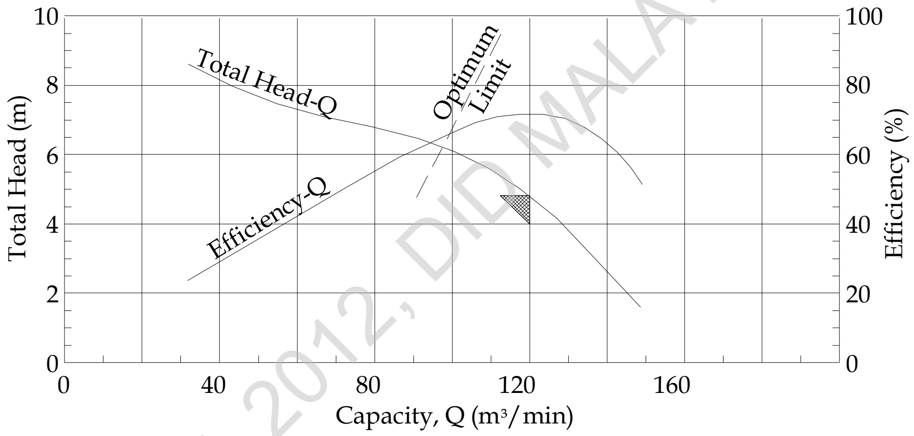


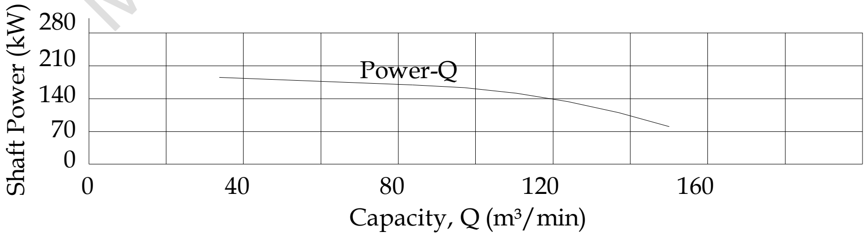
Figure 19.B4: Estimated Pumping Rate from Inflow Hydrograph and Available Storage

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

a) Motor and Pump Design Data



b) Head and Efficiency – Capacity Curves



c) Power – Capacity Curve

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

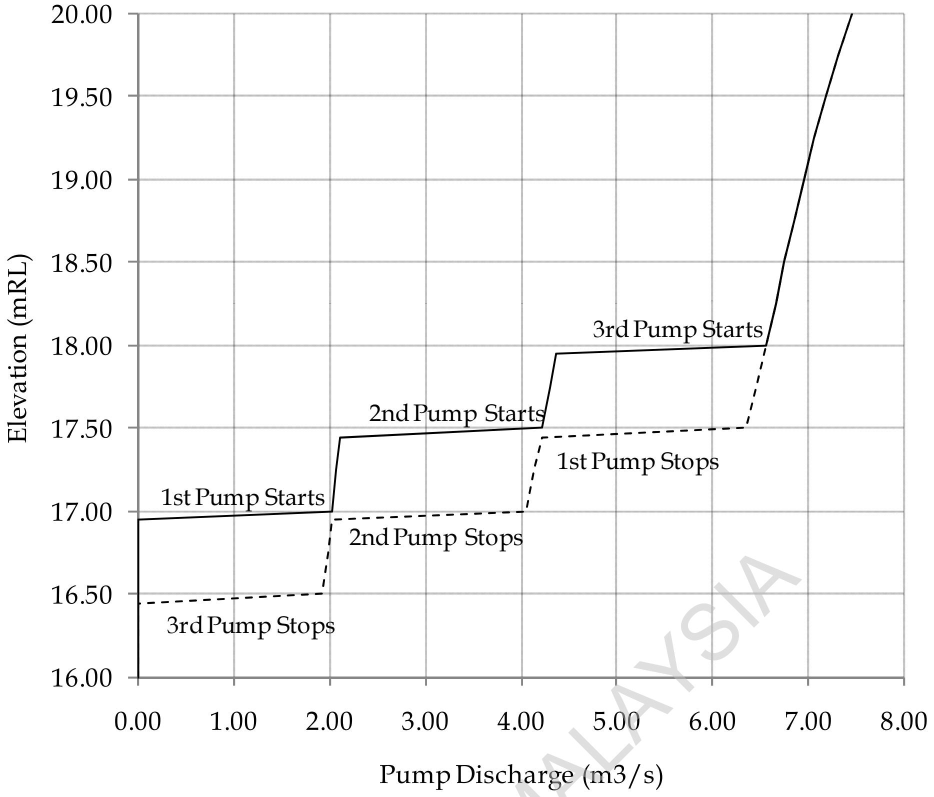


Figure 19.B6 Stage – Pump Discharge Curves

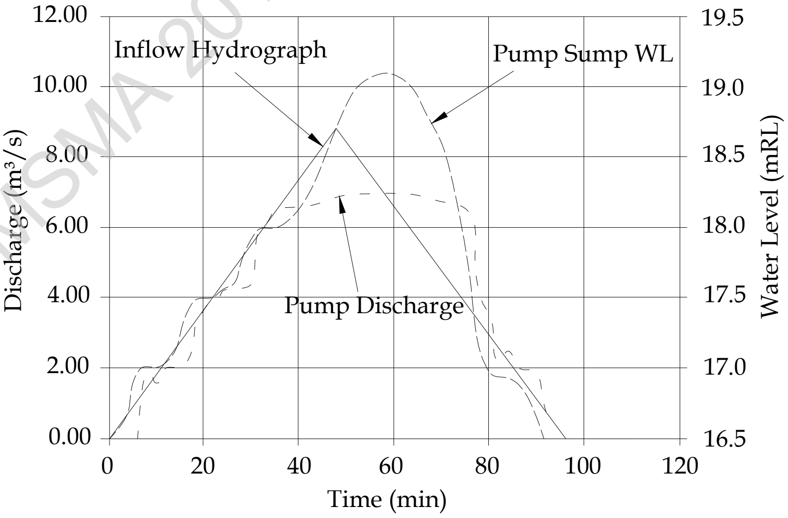


Figure 19.B7: Routing Results for the 5-year ARI 48 min Inflow Hydrograph

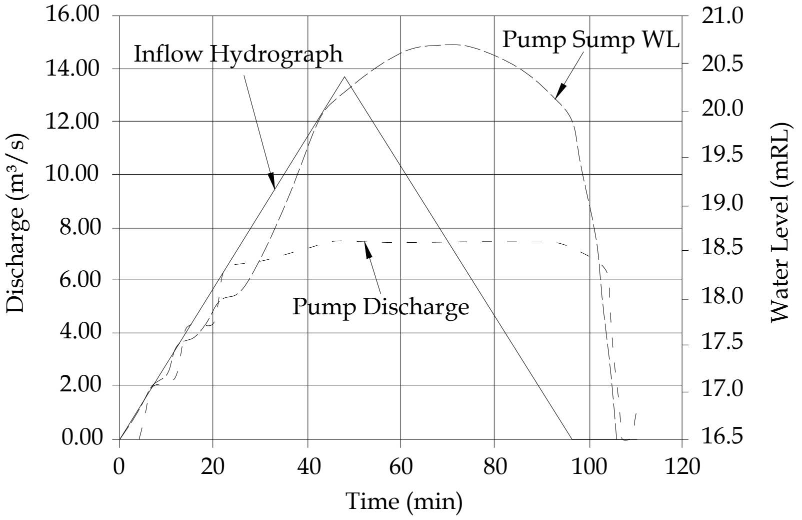
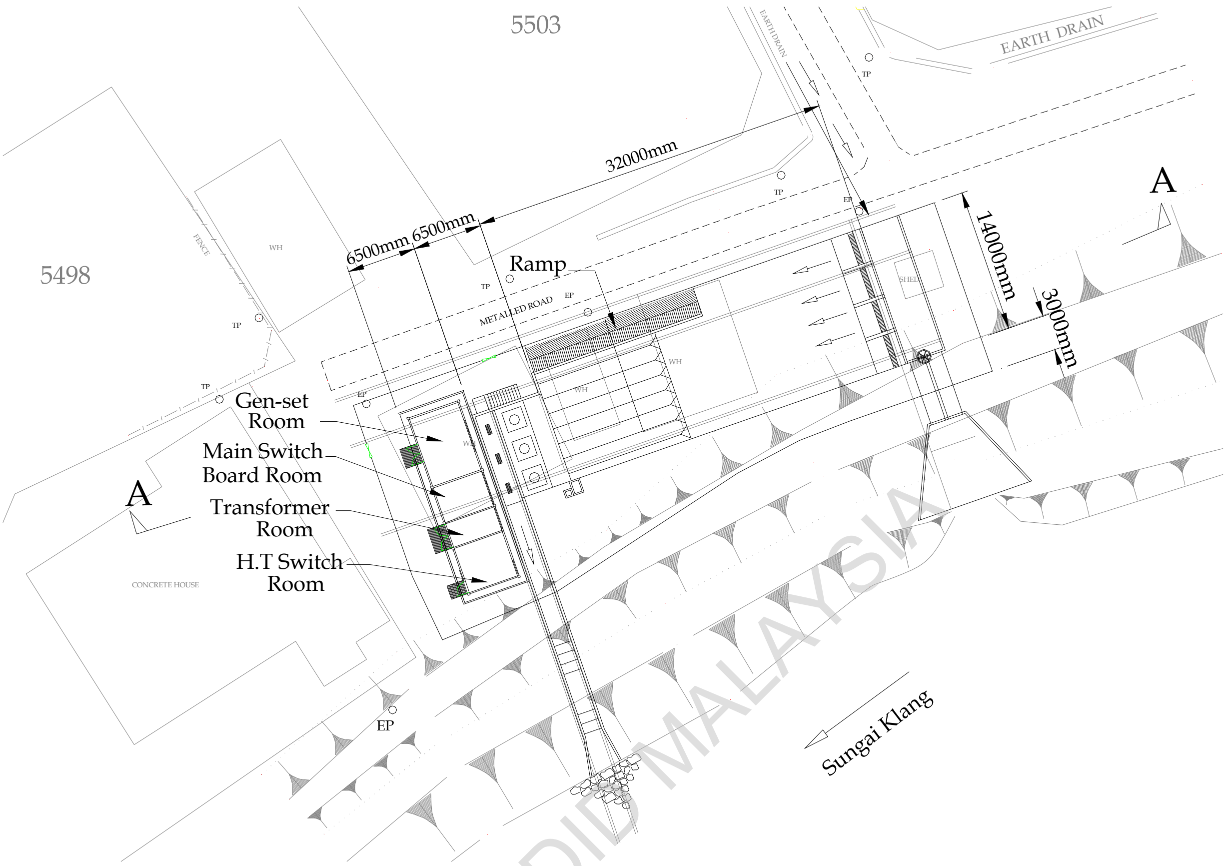
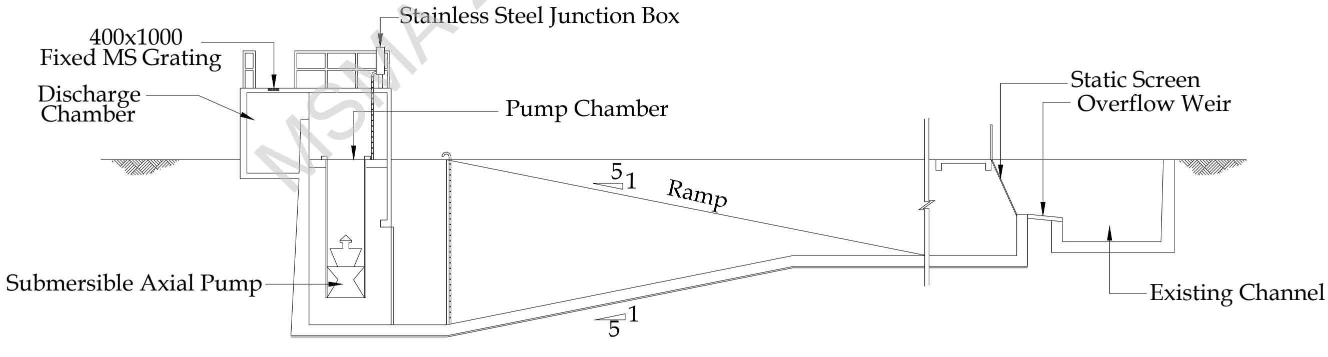


Figure 19.B8: Routing Results for the 100-year ARI 48 min Inflow Hydrograph



a) Plan



b) Longitudinal Profile (A-A)

Figure 19.B9: Layout of Pump Station

APPENDIX 19.C EXAMPLE - TIDAL GATE DESIGN

Problem:

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

Figure 19.C1: Location Plan of the Tidal Gate

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

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

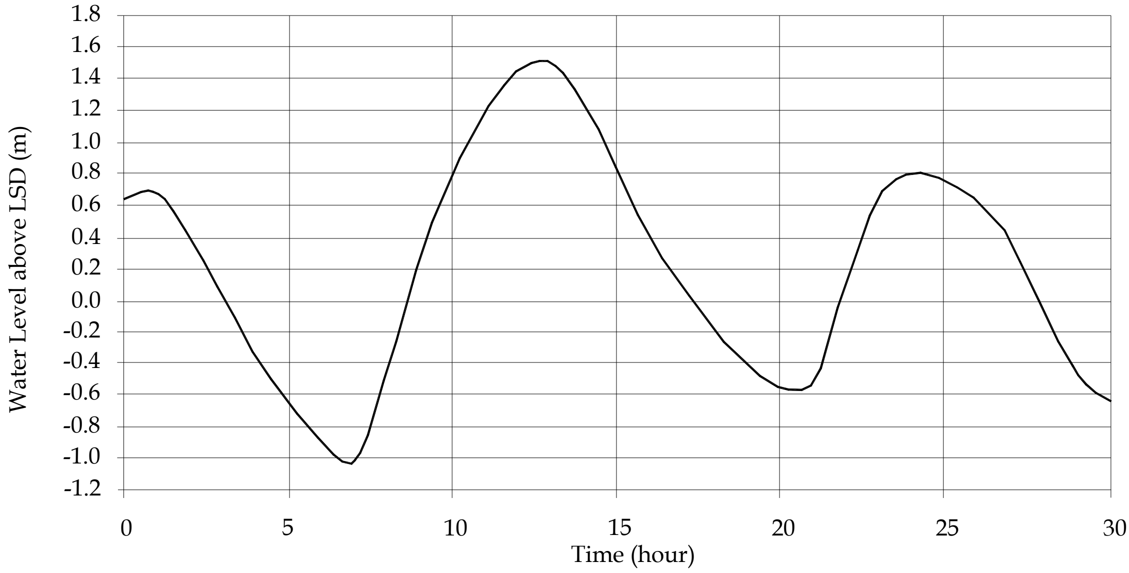


Figure 19.C2: Variation of Tide Levels at Proposed Outlet Gate

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

Solution:

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

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

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

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

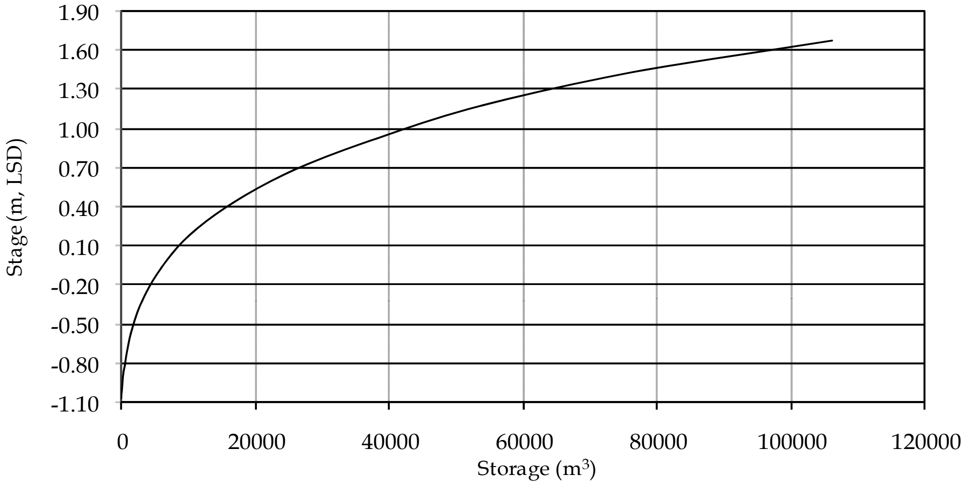


Figure19.C3: Stage-storage Relationship of the Existing Drainage System

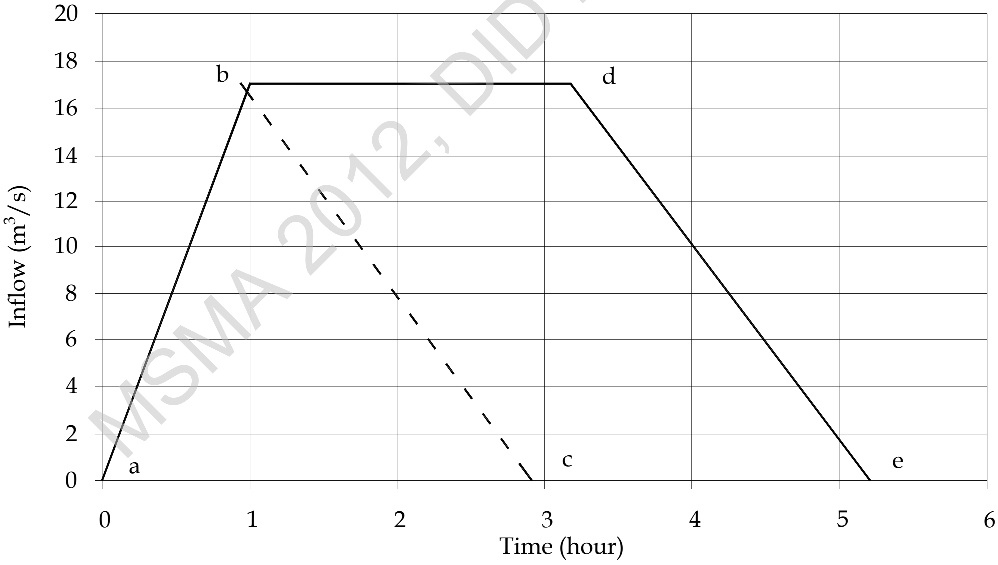


Figure 19.C4: Design Inflow Hydrograph at the Proposed Tidal gate

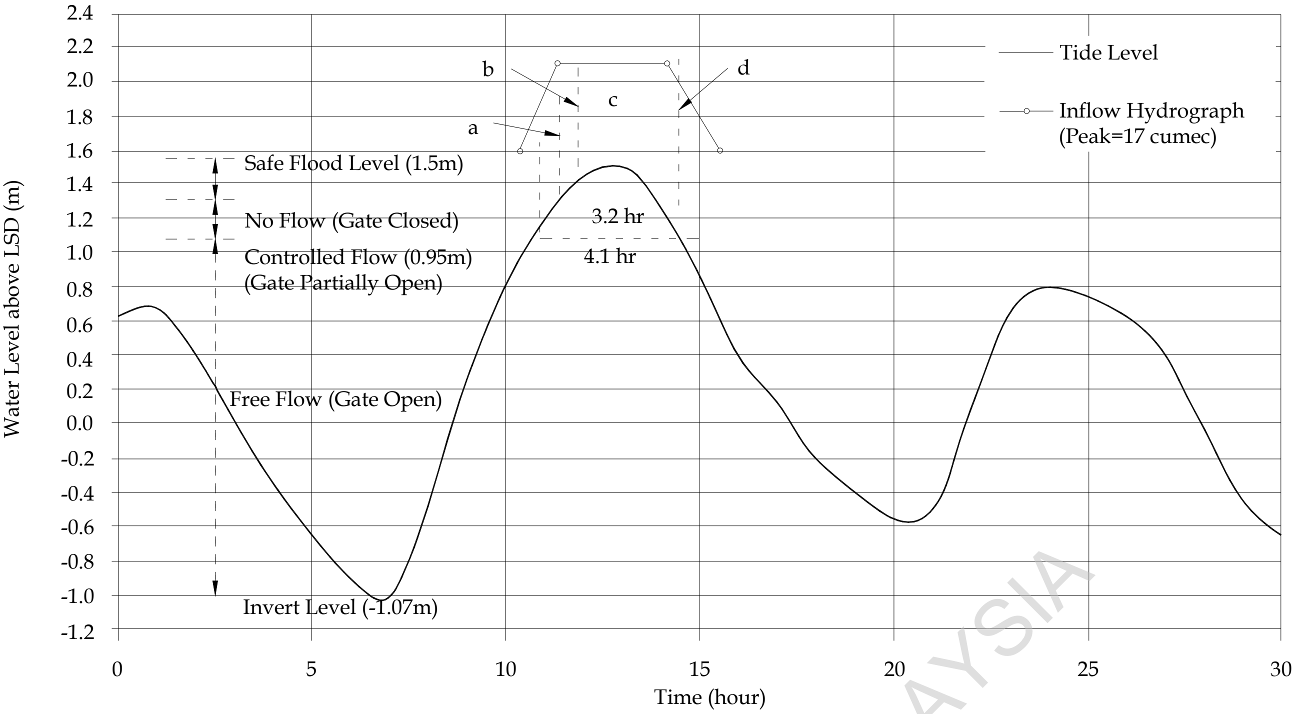


Figure 19.C5: Superimposed Inflow Hydrograph on the Tide Level Graph

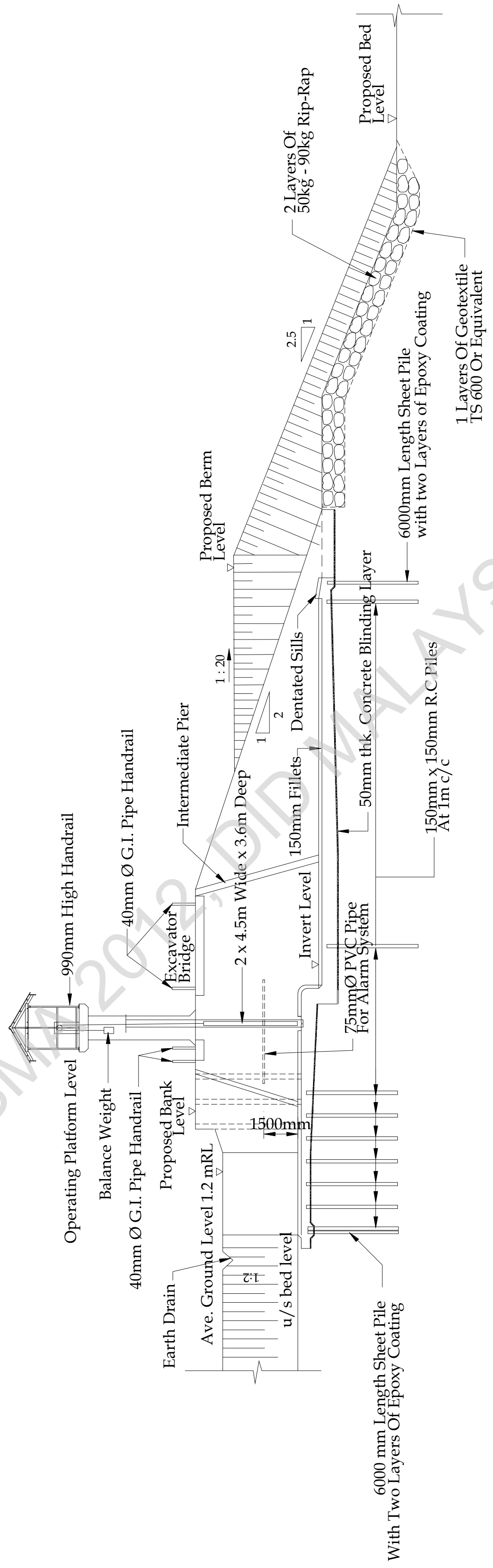


Figure 19.C6: Sectional Plan of the Tidal Gates

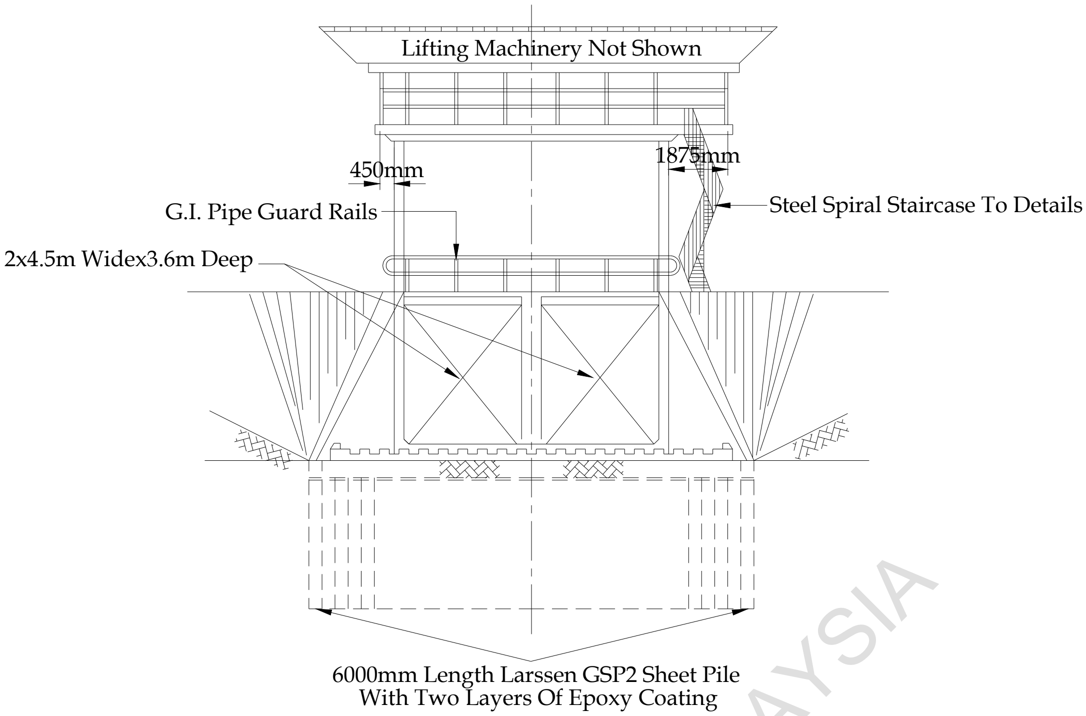
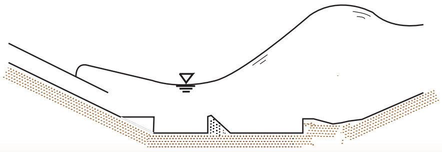


Figure 19.C7: Downstream Elevation View of the Tidal Gates



HYDRAULIC STRUCTURES 20

CHAPTER 20 HYDRAULIC STRUCTURES

20.1 INTRODUCTION

Hydraulic structures are used to positively control water flow velocities, directions and depths as well as to sustain the stream bed elevation and slope and the general configuration of a waterway characteristics.

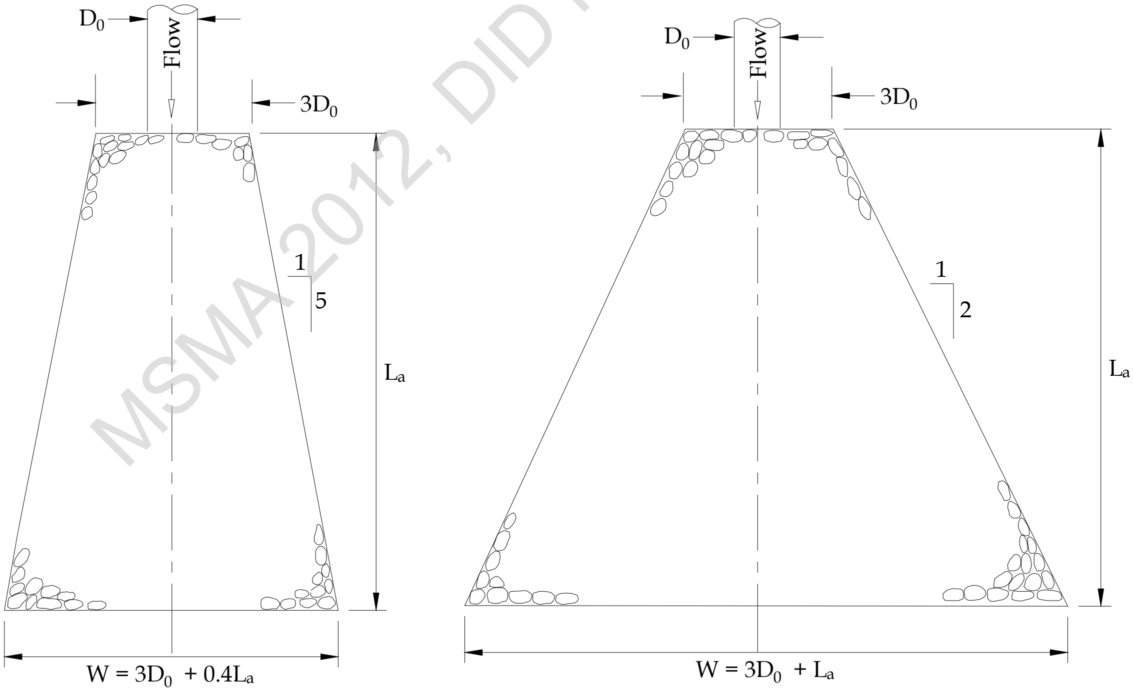
Many of these structures are special and costly, where their selection requires careful and thorough hydraulic engineering judgement. Proper application of hydraulic structures can reduce capital and maintenance costs by changing the characteristics of the flow and by reducing the size of related facilities to fit the needs of a particular project.

The shape, size, and other features of a hydraulic structure can vary widely for different projects, depending upon the functions to be accomplished. Hydraulic design procedures govern the final geometry/shape of all structures. This may include model testing when a proposed design requires a configuration that differs significantly from documented practices.

20.2 EROSION AND SCOUR PROTECTION

20.2.1 Description

When the flow velocity at a conduit outlet (Figure 20.1) exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required at the outlet and, possibly the inlet. This protection usually consists of an erosion resistant reach, such as riprap, located between the outlet and the downstream channel. The design of such protection is normally based on design ARIs from minor and major system application. Lower tailwater conditions during smaller events can create more stressful conditions at the outlet and need to be checked for. When protection is needed at the outlet, one option is to provide a horizontal (zero slope) apron.



a) Apron for tailwater ⩾ $0.5D_0$

b) Apron for tailwater < $0.5D_0$

Figure 20.1: Configuration of Conduit Outlet Protection (U.S. EPA, 1976)

20.2.2 Design Consideration

The length of an apron (L_a) is determined using the following empirical relationships (USEPA, 1976):-

$$L_a = \frac{3.26\ Q}{D_o^{3/2}} + 7D_o \qquad \text{for } TW < D_o/2 \tag{20.1}$$

and

$$L_a = \frac{5.44\ Q}{D_o^{3/2}} + 7D_o \qquad \text{for } TW > D_o/2 \tag{20.2}$$

where,

TW = Tailwater depth (m);
D_o = Maximum inside culvert diameter (m); and
Q = Pipe discharge (m^3/s);

Where there is no well defined channel downstream of the apron, the width, W, of the outlet and of the apron (as shown in Figure 20.1) should be as follows:-

$$W = 3D_o + 0.4L_a \qquad \text{for } TW \geq D_o/2 \tag{20.3}$$

and

$$W = 3D_o + L_a \qquad \text{for } TW < D_o/2 \tag{20.4}$$

The following criteria apply in apron design:-

- The width of the apron at the culvert outlet should be at least 3 times the culvert width.
- Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least 300 mm above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert;
- The side slopes should be 2:1 or flatter;
- The bottom slope should be level; and
- There should be an overfall at the end of the apron or culvert.

The apron material of median stone, diameter d_{50}, is determined from the following equation:

$$d_{50} = \frac{0.066(Q)^{4/3}}{TW(D_o)} \tag{20.5}$$

Existing or pre-shaped scour holes may be used where flat aprons are impractical. Figure 20.2 shows the general design of such a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.041(Q)^{4/3}}{TW(D_o)} \qquad \text{for } y = D_o/2 \tag{20.6}$$

also

$$d_{50} = \frac{0.027(Q)^{4/3}}{TW(D_o)} \qquad \text{for } y = D_o \tag{20.7}$$

where,

y = Depth of scour hole below culvert invert (m).

Aprons constructed of man-made materials are often a viable alternative. Designer should refer to the man-made materials for design consideration.

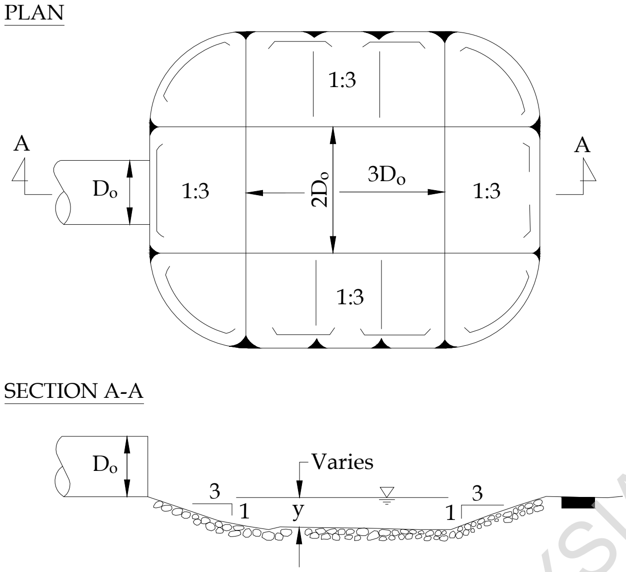


Figure 20.2: Preformed Scour Hole (ASCE, 1975)

20.3 ENERGY DISSIPATORS

20.3.1 Description

Energy dissipators are required in the immediate vicinity of hydraulic structures where high impact loads, erosive forces, and severe scour are expected. In other words, they are usually required where the flow regime changes from supercritical to subcritical, or where the flow is supercritical and the tractive forces or flow velocities are higher than the maximum allowable values. The basic hydraulic parameter that identifies the flow regime, and is used in connection with energy dissipators in general, and with hydraulic jump dissipators in particular, is the Froude number. The Froude number is a ratio of the flow velocity and wave celerity.

Energy dissipation structures act as transitions, which reduce high flow velocities that may exist under a range of flows. Energy dissipators localise hydraulic jumps and act as stilling basins. The use of energy dissipators is very common downstream of hydraulic structures where common channel protection cannot be used alone because of potential damage. If riprap or other protection is used for energy dissipation, it should be confined in a basin and secured in place with grout or mesh.

20.3.2 Riprap Basins

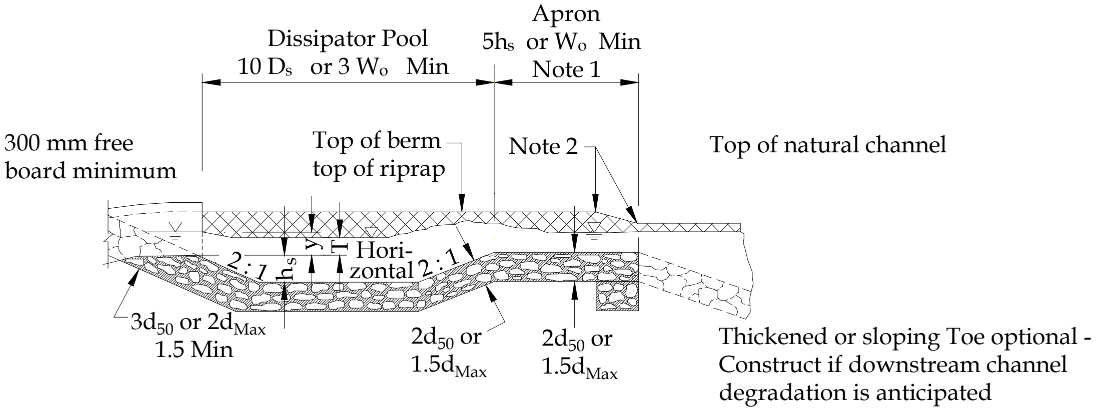
The most commonly used energy dissipators is riprap basins (Figure 20.3). The riprap placed in the basin must be inspected and repaired, if necessary, after major storms. The median stone diameter can be estimated based on the exit velocity of the pipe or culvert. The length of the basin is estimated based on the width or diameter of the conduit. The depth of the basin is based on the median stone diameter.

20.3.3 Stilling Basins

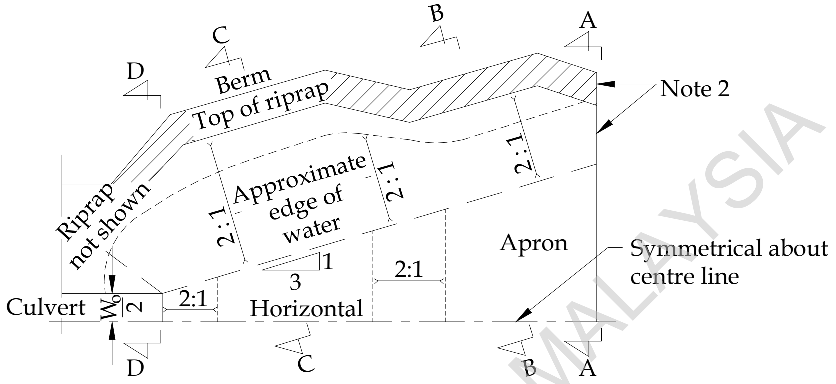
If a hydraulic jump is used for energy dissipation, it should be confined to a heavily-armoured channel reach, the bottom of which is protected by a solid surface such as concrete to resist scouring.

20.3.3.1 Design Considerations

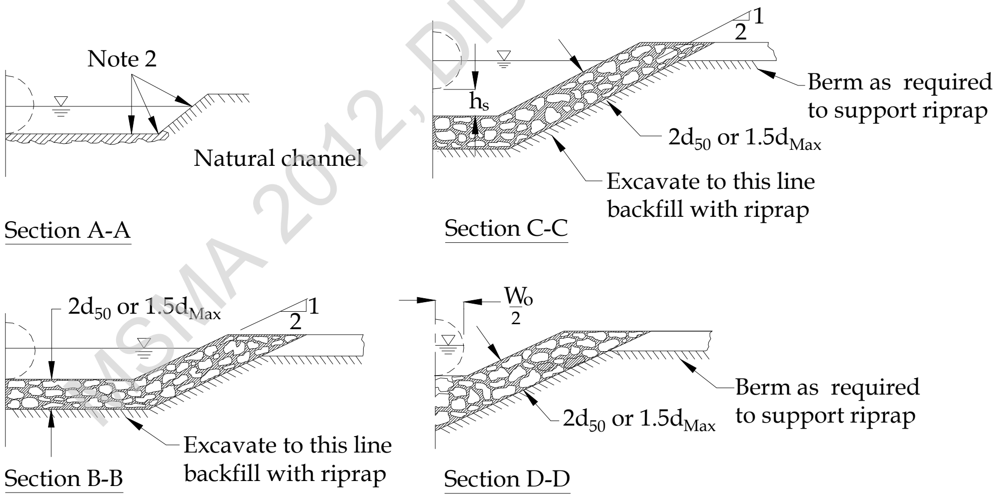
There are several considerations that should be included in designing hydraulic jumps and stilling basins (Chow, 1959; US DOT, 1983):



(a) Centreline Section



(b) Half-plan



(c) Sections

Notes:

1. If a maximum allowable exit velocity, V_e ,from the basin is specified, extend the basin as required to obtain sufficient cross-sectional area at Section A-A (i.e. $A_{A-A} = Q/V_e$) for the specified velocity;
2. Warp the basin to conform to the natural stream channel. The top of the riprap in the basin floor should be at the same elevation or lower than the natural channel bottom at Section A-A.

Figure 20.3: Typical Riprap Basin: (a) Centreline Section (b) Half-plan and (c) Sections (US FHWA, 1983).

- Jump Position: there are three positions or alternative patterns that allow a hydraulic jump to form downstream of the transition in the channel. These positions are controlled by tailwater.
- Tailwater Conditions: tailwater fluctuations due to changes in discharge complicate the design procedure. They should be taken into account by classification of tailwater conditions using tailwater and hydraulic jump rating curves; and
- Jump Types: various types of hydraulic jumps that may occur are summarised in Figure 20.4. Oscillating jumps in a Froude number range of 2.5 to 4.5 are best avoided unless specially designed wave suppressers are used to reduce wave impact.

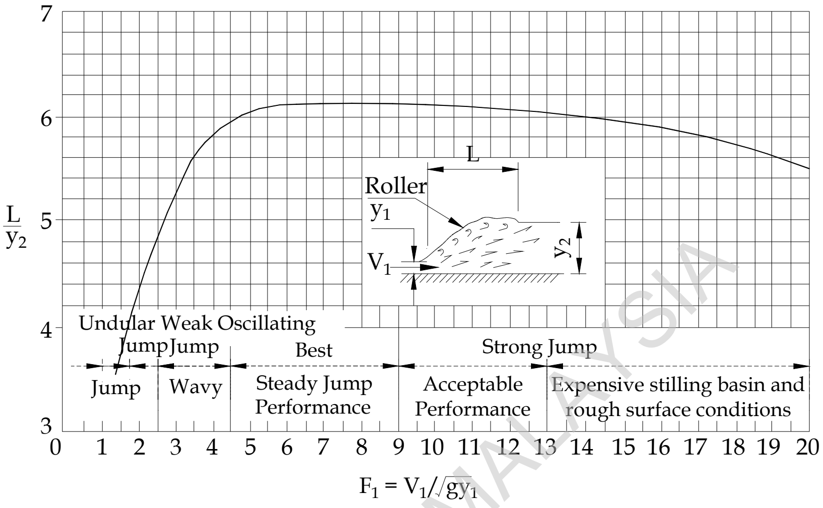


Figure 20.4: Lengths and Types of Hydraulic Jumps in Horizontal Channels (Bradley and Peterka, 1957; Chow, 1959)

As Froude number increases, the tailwater effect becomes more significant. Therefore, for a Froude number as low as 8, the tailwater depth should be greater than the sequent depth downstream of the jump so that the jump will stay on the apron. When the Froude number is greater than 10, the common stilling basin dissipator may not be as cost-effective as a special bucket type dissipator will be required (Peterka, 1958).

20.3.3.2 Controls of Jumps

Jumps can be controlled by several types of appurtenances such as sills, chute blocks and baffle piers. The purpose of a sill located at the end of a stilling basin is to induce jump formation and to control its position under most probable operating conditions. Sharp crested or broad crested weirs can be used to stabilise and control the jump. Chute blocks are used at the entrance to the stilling basin. Baffle piers are blocks placed in intermediate positions across the basin floor for dissipating energy mostly by direct impact action.

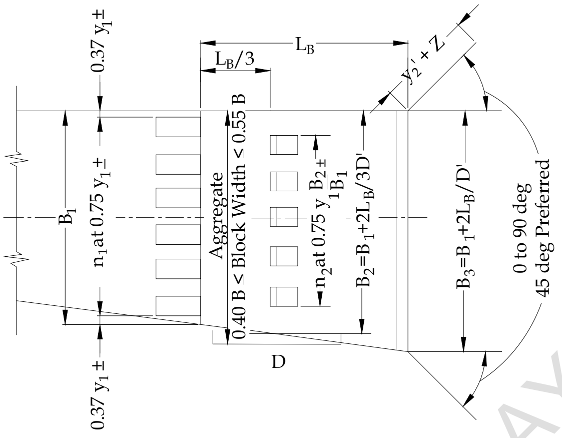
20.3.3.3 Stilling Basin Categories

The following three major categories of basins are used for a range of hydraulic conditions. Design details can be found in the AASHTO Drainage Handbook (1987), Chow (1959), and US DOT (1983).

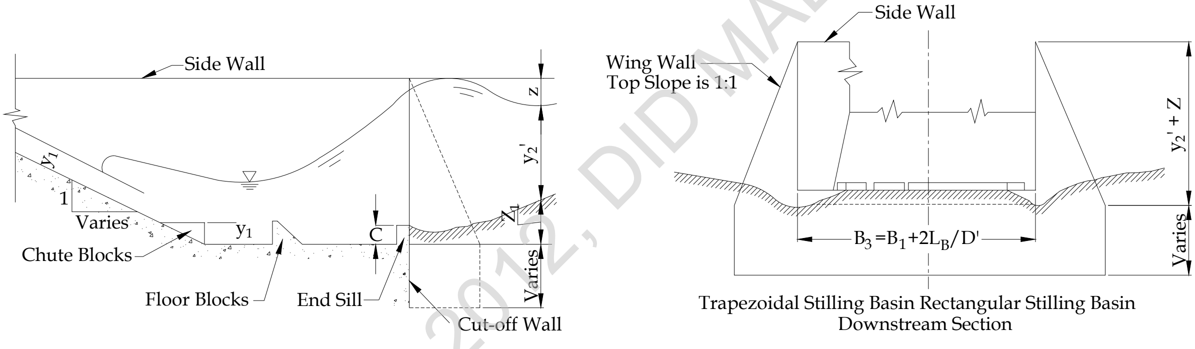
- The SAF ("St. Anthony Falls" Stilling Basin): This basin, shown in Figure 20.5, is recommended for use on small structures such as spillways and outlet works where the Froude number varies between 1.7 and 17. The appurtenances used for this dissipator can reduce the length of the basin by approximately 80%. This design has great potential in urban stormwater systems because of its applicability to small structures. Stilling Basin III developed by the US Bureau of Reclamation (UBSR) is similar to the SAF basin, but it has a higher factor of safety.
- The UBSR Stilling Basin II: This basin, shown in Figure 20.6, is recommended for controlling jumps with Froude numbers greater than 4.5 at large spillways and channels. This basin may reduce the length of the jump by a third and is used for high-dam and earth-dam spillways. Appurtenances used in this basin

include chute blocks at the upstream end of the basin and a dentated sill at the downstream end. No baffle piers are used in this basin because of the cavitation potential.

- The UBSR Stilling Basin IV: This basin, shown in Figure 20.7 is used where jumps are imperfect or where oscillating waves occur with Froude numbers between 2.5 to 4.5. This design reduces excessive waves by eliminating the wave at its source through deflection of directional jets using chute blocks. When a horizontal stilling basin is constructed without appurtenances, the length of the basin is made equal to the length of the jump.



(a) Rectangular Stilling Basin Half-plan



(b) Trapezoidal Stilling Basin Elevation

(c) Centreline Section

Figure 20.5: Proportions of the SAF Basin (Chow, 1959)

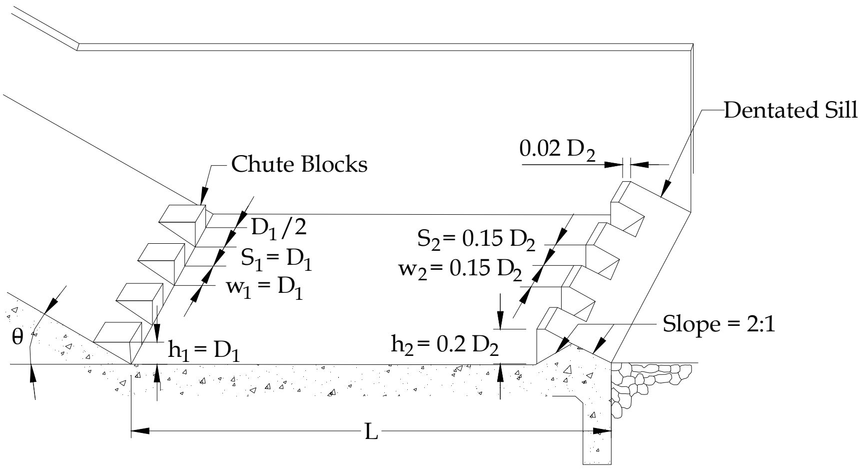


Figure 20.6: Proportions of the USBR Basin II (Chow, 1959)

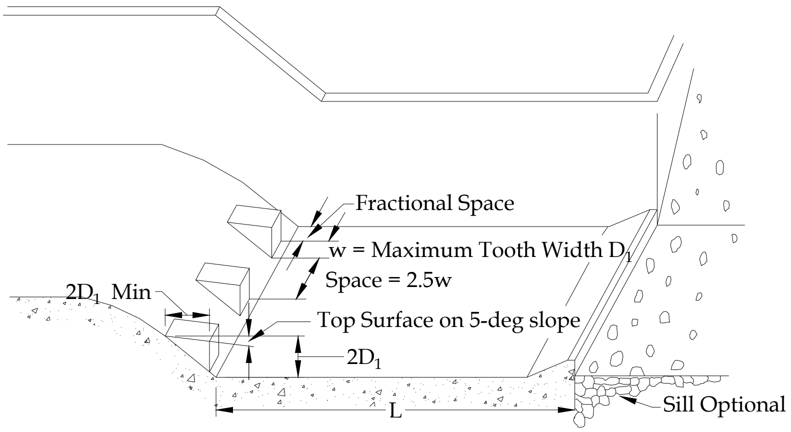
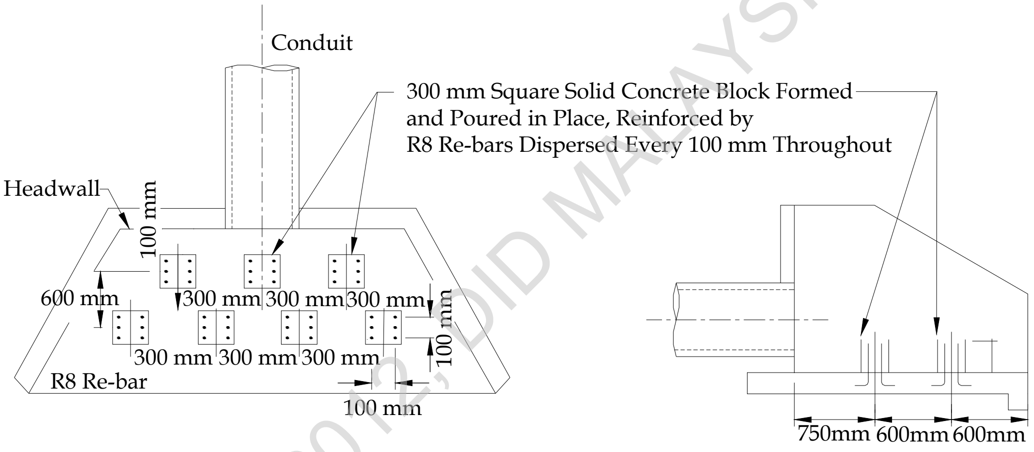


Figure 20.7: Proportions of the USBR Basin IV (Chow, 1959)

20.3.4 Energy-Dissipating Headwalls

Another simple type of energy dissipators that can be used at culvert outlets is an energy dissipating headwall. Three typical headwalls are shown in Figures 20.8 to 20.10.



(a) Plan

(b) Side Elevation

Figure 20.8: Standard Energy Dissipating Headwall, Type I (Chow, 1959)

NOTE:
Height of Energy Dissipators
should be 0.5 D

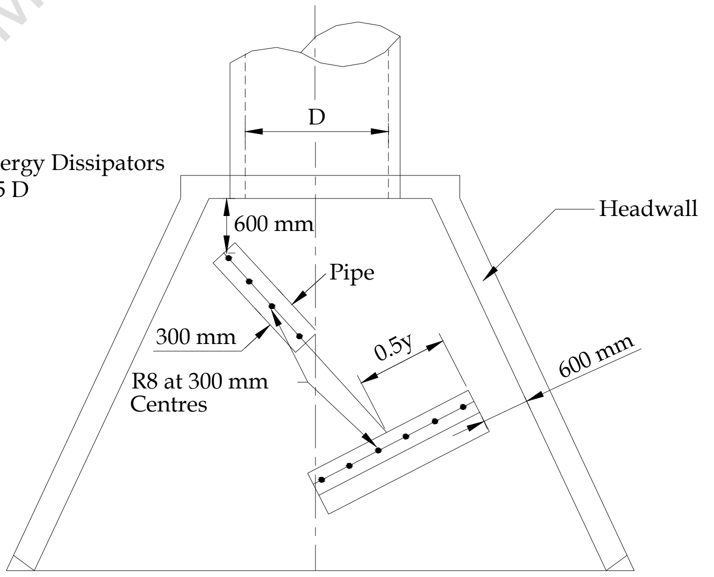
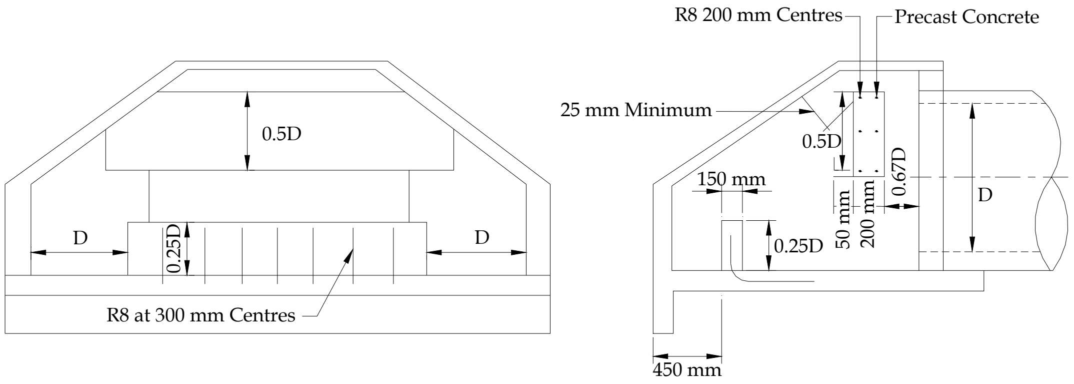


Figure 20.9: Standard Energy Dissipating Headwall, Type II (ASCE, 1992)



(a) Front Elevation

(b) Side Elevation

Figure 20.10: Standard Energy Dissipating Headwall, Type III (ASCE, 1992)

20.3.5 Design Criteria

The design criteria for energy dissipators are as follows:

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs;
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system;
- Energy dissipator designs will vary based on discharge characteristics and tailwater conditions. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence;
- Energy dissipators should be designed to return flows to non-erosive velocities to protect downstream channels; and
- Care must be taken during construction that design criteria are followed exactly. Each part of the criteria is important to the proper function.

Table 20.1 provides a summary of selected parameters, and may be used for preliminary identification of alternative types of energy dissipators.

Table 20.1: Dissipator Criteria (U.S. Department of Transportation, 1983)

Dissipator Type	Froude Number, Fr	Allowable Debris			Tailwater, TW	Special Considerations
		Silt and Stand	Boulders	Floating		
Free Hydraulic Jump	> 1	H	H	H	Required	
CSU Rigid Boundary	< 3	M	L	M	–	
Tumbling Flow	> 1	M	L	L	–	$4\% < S_o < 25\%$
Increased Resistance	–	M	L	L	–	Check Outlet Control HW
USBR Type II	4 to 14	M	L	M	Required	
USBR Type III	4.5 to 17	M	L	M	Required	
UBSR Type IV	2.5 to 4.5	M	L	M	Required	
SAF	1.7 to 17	M	L	M	Required	
Contra Cost	< 3	H	M	M	< 0.5D	
Hook	1.8 to 3	H	M	M	–	
USBR Type VI	–	M	L	L	Desirable	$Q < 11\ m^3/s$, $V < 15\ m/s$
Forest Service	–	M	L	L	Desirable	y < 900 mm
Drop Structure	< 1	H	L	M	Required	Drop < 5 m
Manifold	–	M	N	N	Desirable	
Corps Stilling Well	–	M	L	N	Desirable	
Riprap	< 3	H	H	H	–	

Note: N = None; L = Low; M = Moderate; H = Heavy

20.4 DROP STRUCTURES

20.4.1 Description

Vertical drop structures are controlled transitions for energy dissipation in steep channels where riprap or other energy dissipation structures are not as cost effective. Drop structures used for stormwater drainage can be categorised primarily as either open channel transitions (drop spillways) or transitions between open channels and closed conduits (drop shafts).

Drop structures should be constructed from concrete because of the forces involved; however, riprap or gabion stilling basins may be used where physical, economic, and other constrains arise.

Drop structures in open channels change the channel slope from steep to mild by combining a series of gentle slopes and vertical drops. Flow velocities are reduced to non-erosive velocities, while the kinetic energy or flow velocity gained by the water as it drops over the crest of each spillway is dissipated by an apron or stilling basin.

Open channel drop structures generally requires aerated nappe and subcritical flow conditions at both the upstream and downstream sections of the drop. The stilling basin can vary from a simple concrete apron to baffle blocks or sills. Figure 20.11 shows the flow geometry and important variables at a vertical (straight) drop structure. The flow geometry at such drops can be described by the drop number, D_N, which is defined (Chow, 1959) as:

$$D_N = \frac{q^2}{gh^3} \tag{20.8}$$

where,

q = Discharge per unit width of crest overfall (m³/s/m);
g = Acceleration due to gravity (9.81 m/s²); and
h = Height of drop (m);

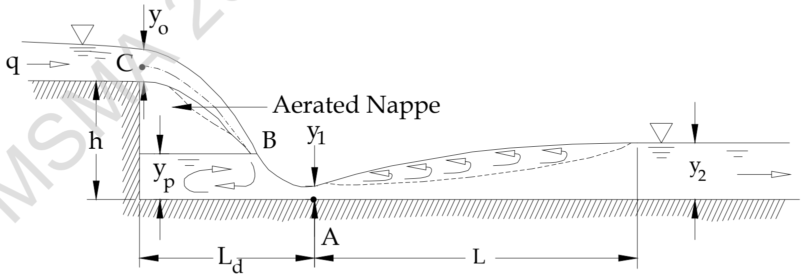


Figure 20.11: Flow Geometry of a Straight Drop Spillway (Chow, 1959)

The drop functions are:

$$\frac{L_d}{h} = 4.30{D_N}^{0.27} \tag{20.9a}$$

$$\frac{y_p}{h} = 1.0{D_N}^{0.22} \tag{20.9b}$$

$$\frac{y_1}{h} = 0.54{D_N}^{0.425} \tag{20.9c}$$

$$\frac{y_2}{h}=1.66D_N^{\ 0.425} \qquad (20.9d)$$

where,

L_d = drop length (m);
y_1 = the depth of the toe of nappe (m);
y_p = pool depth under the nappe (m); and
y_2 = tailwater depth sequent to y_1 (m).

For a given drop height, h, discharge, q, drop length, L_d, sequent depth, y_2 and can be estimated by Equation 20.9(a) and 20.9(d), respectively. The length of the jump can be estimated by techniques discussed in Section 20.3. If the tailwater is less than y_2, the hydraulic jump will recede downstream. Conversely, if the tailwater is greater than y_2, the jump will be submerged. If the tailwater is equal to y_2, no supercritical flow exists on the apron and the distance L_d is minimum.

When the tailwater depth is less than y_2, it is necessary (according to the US Department of Transportation, 1983) to provide either;

- An apron at the bed level and a sill or baffles; and
- An apron below the downstream bed level and an end sill.

The choice of drop structure type and dimensions depends on the unit discharge, q, drop height, h, and tailwater depth, TW. The design should take into consideration the geometry of the undisturbed flow. If the spillway (overflow crest) length is less than the width of the approach channel, the approach channel must be designed properly to reduce the effect of the end contractions to avoid scour. The two most common vertical open channel drops are the straight drop structure and the box inlet drop structure.

(a) Straight Drop Structure

Figure 20.12 shows the layout of a typical straight drop structure and hydraulic design criteria developed by US Soil Conservation Service. McLaughlin Water Engineers (1983) provides specific criteria and reviews design considerations related to the hydraulic, geotechnical, and structural design of drop structures.

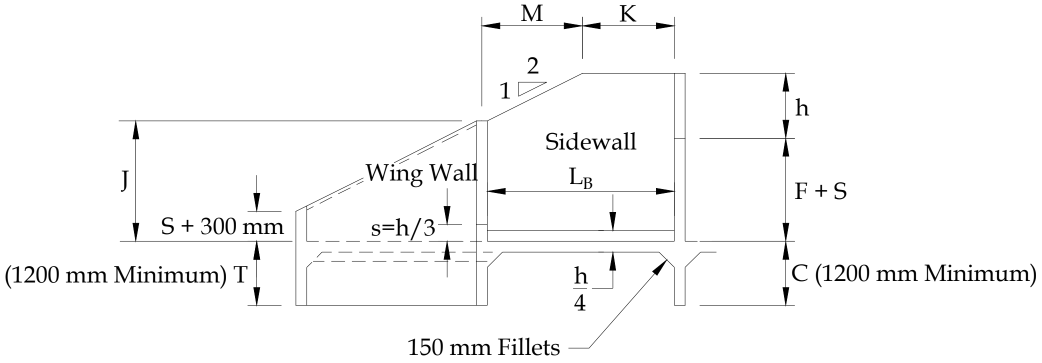
(b) Box Inlet Drop Structure

The box inlet drop structure is a rectangular box with openings at the top and downstream end as shown in Figure 20.13. Water is directed to the crest of the box inlet by earth dikes and a headwall. Flow enters over the upstream end and two sides. The long crest of the box inlet permits large flows to pass at relatively low heads. The width of the structure should not be greater than the downstream channel. Box inlet drop structures are applicable to drops from 0.6 to 3.6 m.

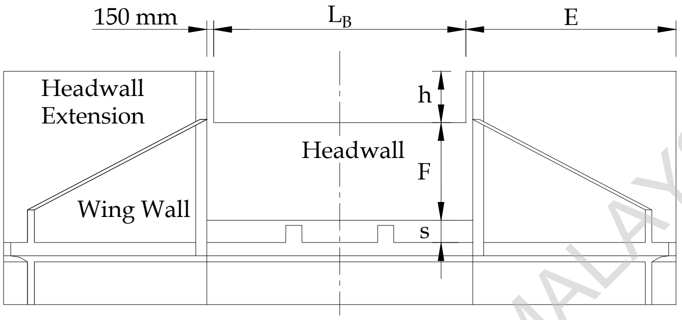
20.4.2 Design Criteria

Design criteria for these structures, based on US Soil Conservation Services and St. Anthony Falls Hydraulic Laboratory, are available in US Department of Transportation (1983) and Blaisdell and Donnely (1956). The parameters to be considered for the hydraulic design of the drops are,

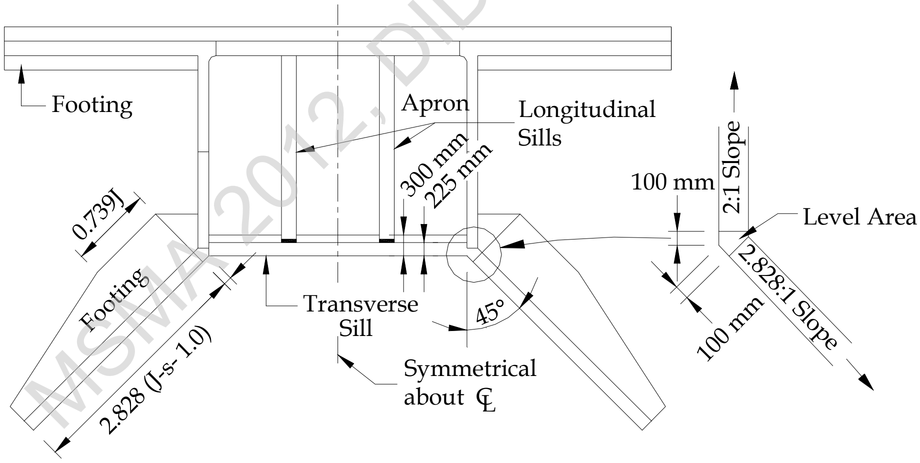
- Section (length) of the crest of the box inlet;
- Opening of the headwalls;
- Discharge, discharge coefficients, and flow regime changes;
- Box inlet length and depth; and
- Minimum length and width of stilling basin.



(a) Section on Centreline



(b) Downstream Elevation



(c) Plan

Where :

E = Minimum length of headwall extension = [3h+0.61] or [1.5F] whichever is greater

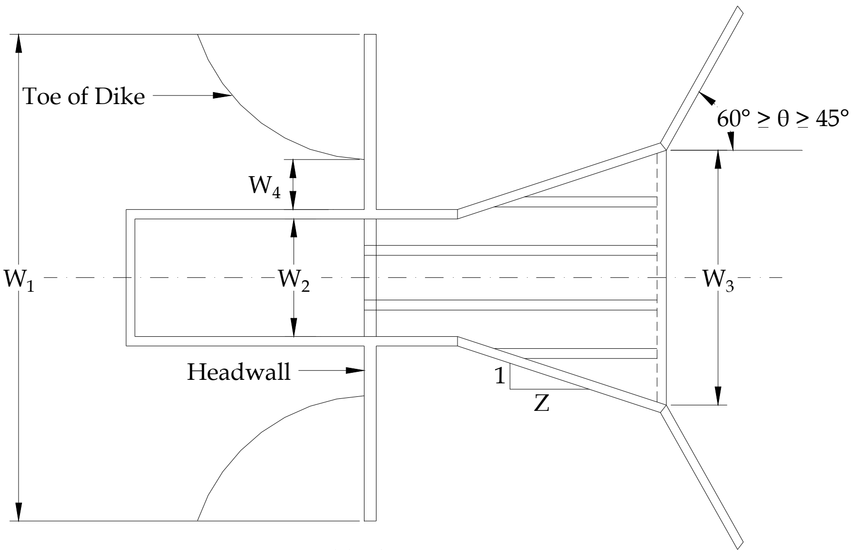
J = Height of wing wall and sidewall at junction = [2h] or $\left[F+h+s-\left(\frac{L_B+0.13}{2}\right)\right]$ or [t+1] whichever is greater

L_B= Length of basin =$\left[F\left(2.28\ \frac{h}{F}+0.52\right)\right]$

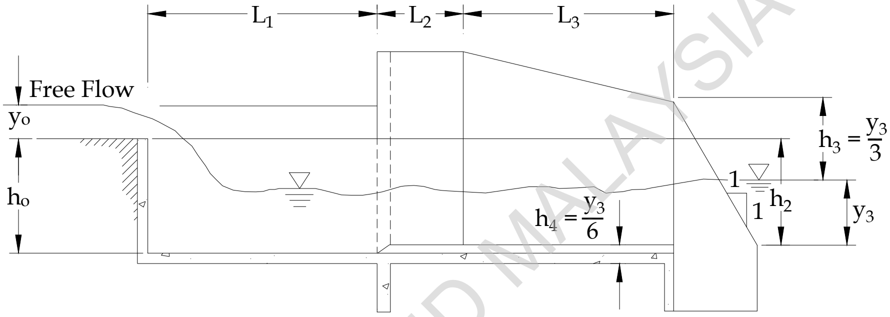
M= [2 (F+ 1.3 h-J)]

K = [(L_B + 0.13)- M]

Figure 20.12: Typical Drop Spillway and Some Hydraulic Design Criteria (US SCS, 1954)



(a) Plan



(b) Section on Centreline

Figure 20.13: Box Inlet Drop Structure (US Dept. of Transportation, 1983)

20.5 OUTFALLS

20.5.1 Description

All stormwater drains of a locality have an outlet where flow from the local drainage system is discharged. The discharge point, or outfall, can be either a natural river or stream, or a stormwater drain or channel. The procedure for calculating the hydraulic grade line through a storm drainage system begins at the outfall. Therefore, consideration of the outfall conditions is an important part of stormwater drainage design.

20.5.2 Design Criteria

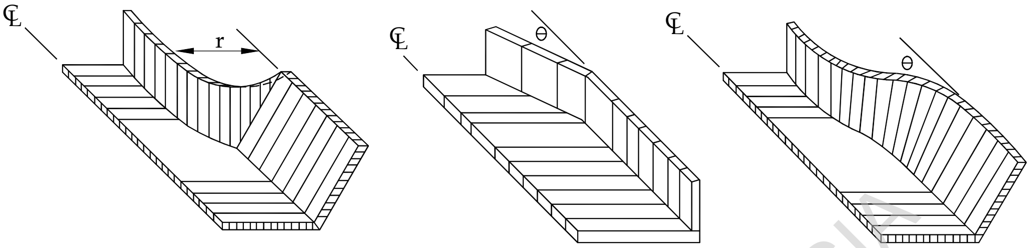
Most of the design criteria for stormwater drain outfalls are included:

- The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall;
- Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation;
- If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time;
- Energy dissipation is always needed to protect the storm drain outlet and the receiving natural or man-made channel. Riprap aprons or energy dissipators should be provided if high velocities are expected; and
- The outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction.

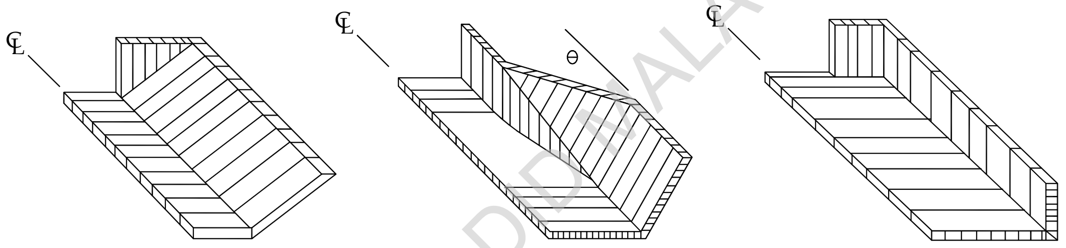
20.6 TRANSITION AND CONSTRICTIONS

20.6.1 Description

Channel transitions (Figure 20.14) are typically used to alter the cross-sectional geometry, to allow the waterway to fit within a more confined right-of-way, or to purposely accelerate the flow to be carried by a specialised high velocity conveyance. Constrictions can appreciably restrict and reduce the conveyance capacity in a manner which is either detrimental or beneficial. The purpose of this section is to briefly outline typical design procedures for transition and constriction structures that may be required for stormwater systems.



a) Cylindrical Quadrant b) Straight Line c) Warped



d) Abrupt e) Wedge f) Abrupt

Figure 20.14: Transition Types

20.6.2 Transitions Analysis

(a) Subcritical Transitions

Transitions for subcritical flow frequently involve localised or bank lining configurations which allow change in the cross section and produce a water surface profile based on gradually varied flow. The energy lost through a transition is a function of the friction, eddy currents, and turbulence. The intent is often to minimise friction losses and/or erosion tendencies.

Standard water surface profile analysis is applied, with the addition of an energy loss at the transition. The loss is expressed as a function of the change in velocity head occurring across the contraction or expansion transition (from upstream to downstream locations). Figure 20.14 illustrate some of these transitions.

Analysis of transitions requires careful water surface profile analysis including verification of effective channel hydraulic controls. It is not uncommon to have a transition which is first thought to be performing in a subcritical mode, but subsequently found to produce a supercritical profile with a hydraulic jump.

(b) Supercritical Transition

The configuration of a supercritical transition is entirely different from subcritical transitions. Improperly designed and configured supercritical transitions can produce shock waves which result in channel overtopping and other hydraulic and structural problems.

20.6.3 Constrictions Analysis

(a) Constriction with Upstream Subcritical Flow

There are a variety of structures that are considered as constrictions. They can include bridges, culverts, drop structures, and flow measurement devices. Constrictions of various types are used intentionally to control bed stability and upstream water surface profiles.

The hydraulic distinction of constrictions is that they can cause rapidly varied flow. Significant eddies can form upstream and downstream of the constriction depending upon the geometry. Flow separation will start at the upstream edge of the constriction, then the flow contracts to be narrower than the opening width. Typically, the width of contraction is 10% of the depth at the constriction for each side boundary. Chow (1959) presents guidelines developed by the USGS for constrictions where the Froude number in the contracted section should not exceed 0.8. These cases are generally mild constrictions.

Constrictions used for flow depth control or flow measurement devices require a high degree of accuracy. The design information available that can be used for ensuring a high degree of accuracy is limited. It is advisable to use models tested or proven prototype layouts.

(b) Constriction with Upstream Supercritical Flow

Possible shock waves or choked flows causing high upstream backwater or a hydraulic jump are major concerns. The situation is to be avoided in urban drainage because of inherent instabilities.

20.7 BEND AND CONFLUENCES

20.7.1 Description

Channel confluences are commonly encountered in design. Flow rates can vary disproportionately with time so that high flows from upstream channel can discharge into downstream channel when it is at high or low level. Depending on the geometry of the confluence, either condition can have important consequences, such as supercritical flow and hydraulic jump conditions, and result in the need for structures.

20.7.2 Bends

(a) Subcritical Bends

Chow (1959), Rouse (1949) and others illustrate flow patterns, superelevation, and backwater or flow resistance characteristics of bends in detail. Superelevation refers to the rise in the water surface on the outer side of the bend. Effectively, the bend can behave like a contraction, causing backwater upstream and in accelerated velocity zones, with high possibility of erosion on the outside of the bend and other locations. Significant eddy currents, scour, sedimentation, and loss of effective conveyance can occur on the inside of the bend.

Concrete lined channels can be significantly affected by superelevation of the water surface. The designer should always add superelevation to the design freeboard of the channel. The equation for the amount of superelevation of the water surface, Δy, that takes place is given as :

$$\Delta y = C\left[\frac{V^2T}{gr}\right] \tag{20.10}$$

where,

C = Coefficient, generally 0.5 for subcritical flow;
V = Mean channel velocity (m/s);
T = Width of water surface in channel (m) ;
g = Acceleration of gravity (9.81 m/s); and
r = Channel centreline radius (m).

(b) *Supercritical Bends*

Supercritical channels are generally not desirable in urban drainage. However, special situations may occur where supercritical flows enters a curved channel, for example:

- At confluences where one channel is largely empty, and the entering flow expands and becomes supercritical;
- At a sharp bend in a conduit where slope inherently leads to supercritical conditions; and
- At a channel drop that unavoidably ends up on a curve.

The key phenomenon to be aware of is shock waves, of which there are two types, positive and negative. On the outside of an angular bend, a positive shock wave will occur, resulting in a rise in the water surface. The wave is stationary and crosses to the inside of the channel, and then can continue to reflect back and forth. Where the flow passes the inside of an angular bend, a separation will occur, resulting in a negative shock wave or drop in the water surface. This stationary negative shock wave will cross to the outside of the channel. Both shock waves will continue to reflect off the walls, resulting in a very disturbed flow pattern.

A basic control technique is to set up bend geometry to allow the positive shock wave to intersect the negative wave the point where the later is propagated. A bend usually requires two deflections on the outside and one bend on the inside of the bend. A beneficial aspect of the shock wave is that it turns the flow in a predictable pattern, and thus the channel walls have no more force imposed on them other than that caused by the increased (or decreased) depths.

Other control techniques include very gradual bends, super elevated floors, and control sills, but these methods are generally less efficient.

20.7.3 Confluences

One of the most difficult problems to deal with is confluences where the difference in flow characteristics may be great. When entering the combined channel, the flow can diverge and drop in level if the flow capacity is suddenly increased. This can result in high velocities or unstable supercritical flow conditions with high erosion potentials. When significant sediment flows exist, aggradation can occur at the confluence, resulting in the loss of capacity in one or both upstream channels.

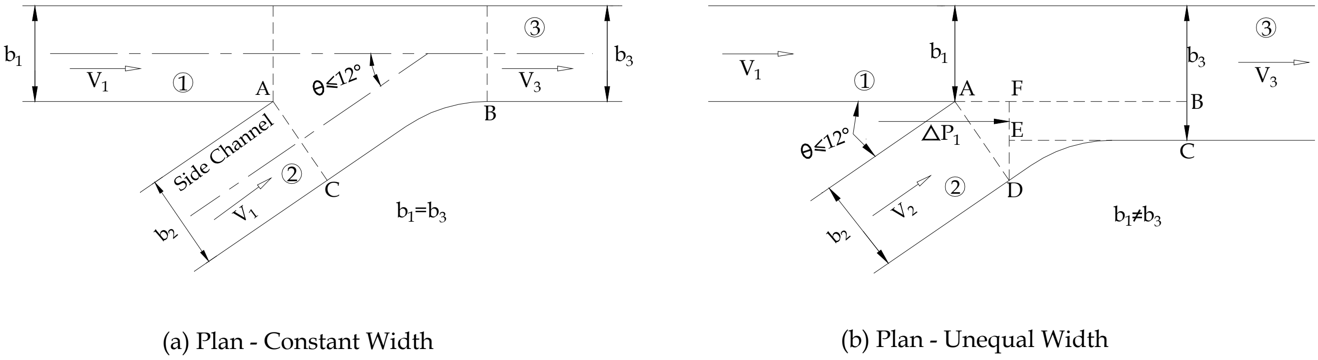
(a) *Subcritical Flow Confluence Design*

The design of channel junctions is complicated by many variables such as the angle of intersection, shape and width of the channels, flow rates, and type of flow. The design of large complex junctions should be verified by model tests.

Figure 20.15 illustrates two types of junctions. The following assumptions are made for combining subcritical flows:

- The side channel cross-section is the same shape as the main channel cross-section;
- The bottom slopes are equal for the main channel and side channel;
- Flows are parallel to the channel walls immediately above and below the junction;
- The depths are equal immediately above the junction in both the side and main channel; and
- The velocity is uniform over the cross-sections immediately above and below the junction.

The assumption ‘flows are parallel to the channel walls immediately above and below the junction’ implies that hydrostatic pressure distributions can be predicted, and assumption ‘the velocity is uniform over the cross-sections immediately above and below the junction’ suggests that the momentum correction factors are equal at the reference sections.



(a) Plan - Constant Width

(b) Plan - Unequal Width

Figure 20.15: Channel Junction Definition Sketches

(b) Supercritical Flow in Confluences

In contrast with subcritical flows at junctions, supercritical flows with changes in boundary alignments is generally complicated (Ippen, 1951, Rouse, 1949). In subcritical flow, backwater effects are propagated upstream, thereby tending to equalise the flow depths in the main and side channels. However, backwater cannot be propagated upstream in supercritical flow and flow depths in the main and side channels cannot generally be expected to be equal. Junctions for rapid flows and very small junction angles are designed assuming equal water surface elevations in the side and main channels.

Standing waves (Ippen, 1951) in supercritical flow at open channel junctions complicate flow conditions. These waves may necessitate increased wall heights in the vicinity of the junction. Wave conditions that may be produced by rapid flow at the downstream of a typical junction are shown in Figure 20.16. One area of maximum wave height can occur on the side channel wall opposite the junction point and another on the main channel right wall downstream from the junction. Supercritical flow may unavoidably occur in certain confluences.

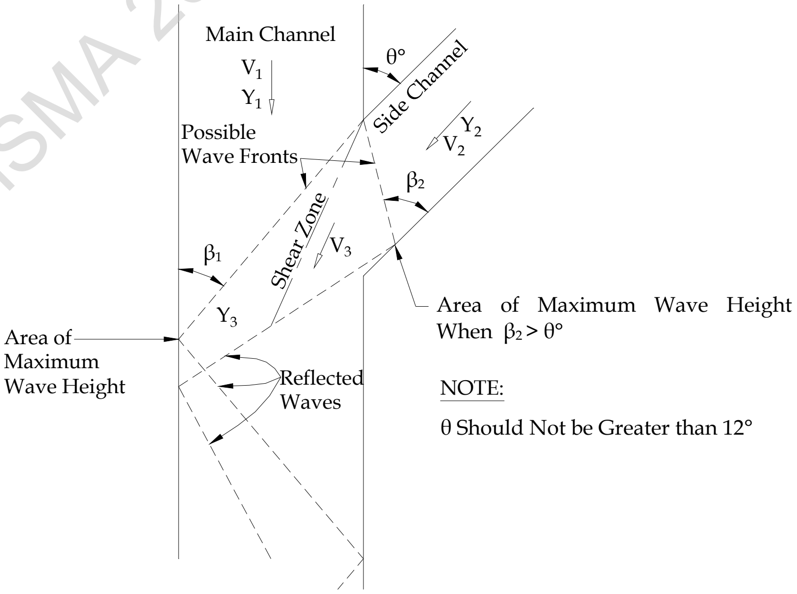


Figure 20.16: Open Channel Confluence, Standing Waves - Supercritical Flow

20.8 SIDE-OVERFLOW WEIRS

20.8.1 Description

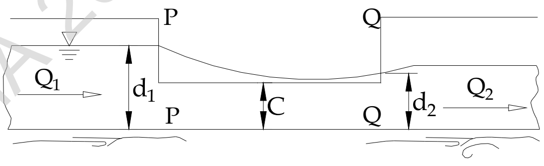
Side-overflow weirs facilitate overflow and diversion of stormwater by directing the discharge away from the original channel. Such structures are commonly used to direct channel discharges above predetermined levels into off-line stormwater detention facilities. Flow diversions occur only during storms.

20.8.2 Design Considerations

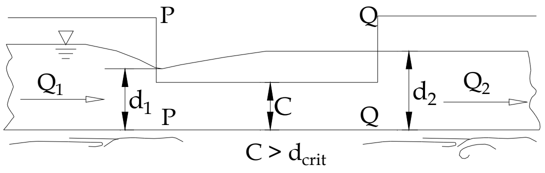
The design of side-overflow weirs is based on empirical equations which quantify the relationship between the discharge over the weir and geometric parameters at the weir, including the length of the weir and head (Hager, 1987). Figure 20.17 (Metcalf and Eddy, 1972) shows three head or water surface profile conditions that can prevail at a side-overflow weir:

- Condition 1 - the channel bed slopes steeply, producing supercritical flow. Under this condition, the weir has no effect upstream and along the weir there is a gradual reduction in depth. The flow depth in the original channel increases at the downstream of the weir before tending asymptotically to the normal depth corresponding to the remaining discharge;
- Condition 2 - The channel bed slopes mildly. Under this condition, subcritical flow prevails and the weir impact is noticed upstream of the weir only. The water surface profile downstream of the weir corresponds to the normal depth of the remaining discharge. Along the weir there is a gradual increase in depth and upstream of the weir the flow depth tends asymptotically to the normal depth for the initial discharge; and
- Condition 3 - The channel bed slopes mildly, but the weir crest is below the critical depth corresponding to the initial flow, and the flow at the weir is supercritical. Frazer (1957) indicates that conditions 1 and 3 may result in the development of a hydraulic jump at the weir.

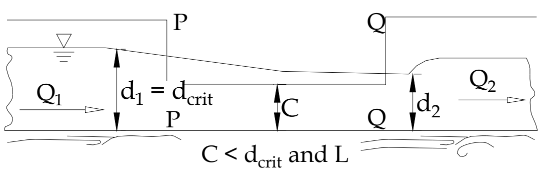
The most common condition that a designer will encounter is Condition 3, where the weir elevation is below the critical depth. When only a relatively small amount of the flow is diverted, a rising water surface profile occurs. According to Metcalf and Eddy Inc. (1972), a falling water profile will occur if the ratio of the height of the weir, c, to the channel specific energy, E_w referenced to the top of the weir, is less than 0.6.



(a) Condition 1



(b) Condition 2



(c) Condition 3

Figure 20.17: Possible Types of Water Surface Profiles at a Side-overflow Weir (Metcalf & Eddy, 1972)

20.8.3 Design Criteria

The design criteria refers to Figure 20.18.

(a) Falling Water Surface

The equations for computing weir length for the falling water surface profile combine Bernoulli's theorem with a weir discharge formula.

$$L=2.03\ B\ \left(5.28-2.63\frac{C}{E_w}\right) \qquad (20.11)$$

where,

L = Length of weir (m);
C = Height of weir (m);
B = Channel width (m); and
E_w = Channel specific energy (m).

and,

$$E_w=\alpha\frac{V^2}{2g}+\alpha'(y_n-C) \qquad (20.12)$$

where,

α = Velocity coefficient;
V = Normal velocity in the approach channel (m/s);
α' = Pressure-head correction;
C = Height of the weir above the channel bottom (m);
g = Acceleration due to gravity (m/s^2); and
y_n = Normal depth of flow in approach channel (m).

Values for α and α' of 1.2 and 1.0 respectively can be used in the approach channel, while at the lower end of the weir values of 1.4 and 0.95 can be used for α and α' respectively.

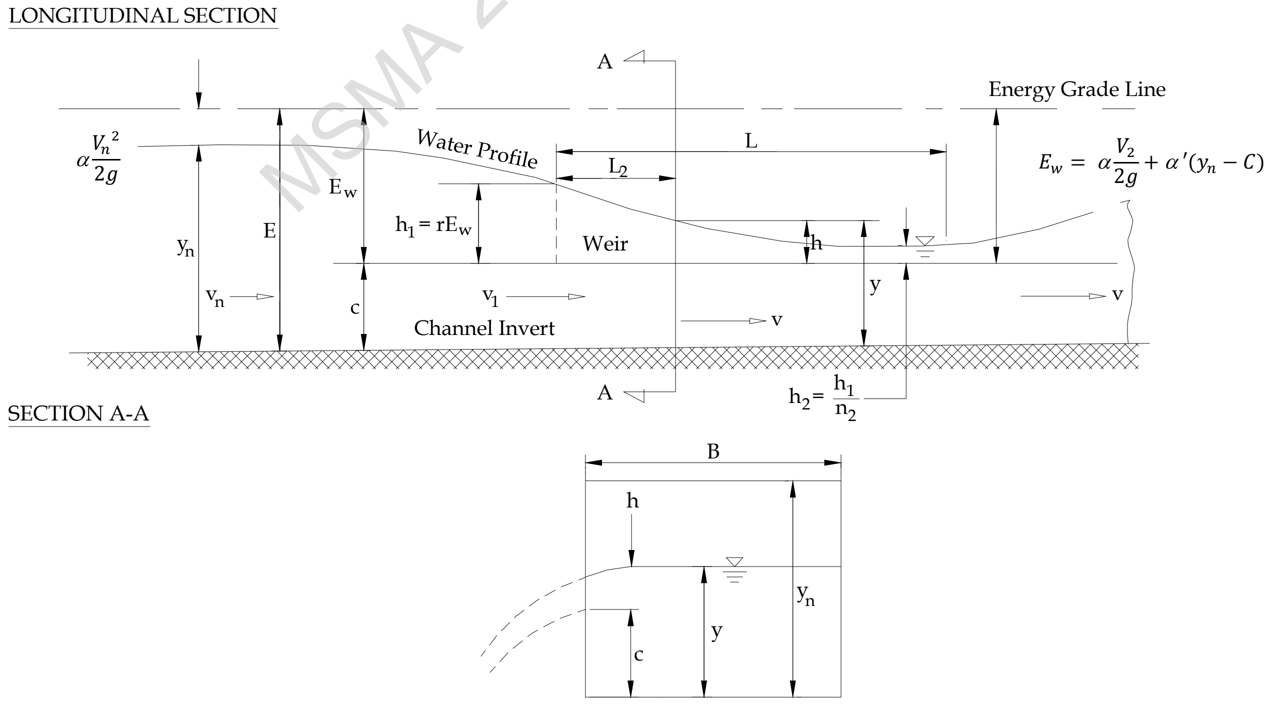


Figure 20.18: Typical Cross-sectional Hydraulics at a Side-Overflow Weir (Metcalf & Eddy, 1972)

(b) Rising Water Surface

The analysis for estimating the weir length for the rising water surface profile is based on Equations 20.13 below:

$$L=\frac{B}{C}\left[\emptyset\left(\frac{y_2}{E}\right)-\emptyset\left(\frac{y_1}{E}\right)\right] \tag{20.13}$$

where,

L = Length of weir (m);
C = Constant (0.35 for a free nappe);
B = Channel width (m);
E = Specific energy (m);
y_1, y_2 = Depth in channel (m); and
$\left[\emptyset\left(\frac{y}{E}\right)\right]$ = Varied flow function (Collinge, 1957).

Equation 20.13 is recommended for use only in the case of a rising water surface profile. Metcalf and Eddy Inc. (1972) indicates that this equation works best when the Froude number is between 0.3 and 0.92.

20.9 FLOW SPLITTER

20.9.1 Description

A flow splitter is a special structure designed to divide a single flow and divert the parts into two or more downstream channels. A flow splitter can serve three functions:

- Reduction in Water Surface Elevation - by dividing the flow from a large pipe into multiple conduits, the height of flow measured from the flow line to the water surface (or for pipes flowing full, the inside diameter can be reduced. This may be necessary to route flows under immovable obstructions.
- Dividing Flows - examples of this include division of existing large special-design conduits, such as arches or horseshoes, into less expensive multiple-pipe continuations and division of flow between low and high-flow conduits at the intake of an inverted siphon.
- Restriction of Flows to Water Quality Treatment Facilities - to restrict flows to water quality treatment facilities and bypass the remaining higher flows around the facilities (off-line). This can be accomplished by splitting flows in excess of the water quality design flow upstream of the facility and diverting higher flows to a bypass pipe or channel. The bypass typically enters a detention pond or the downstream receiving drainage system. A crucial factor in designing flow splitters is to ensure that low flow is delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows are diverted to the bypass system with minimal increase in head at the flow splitting structure to avoid surcharging the water quality facility under high flow conditions.

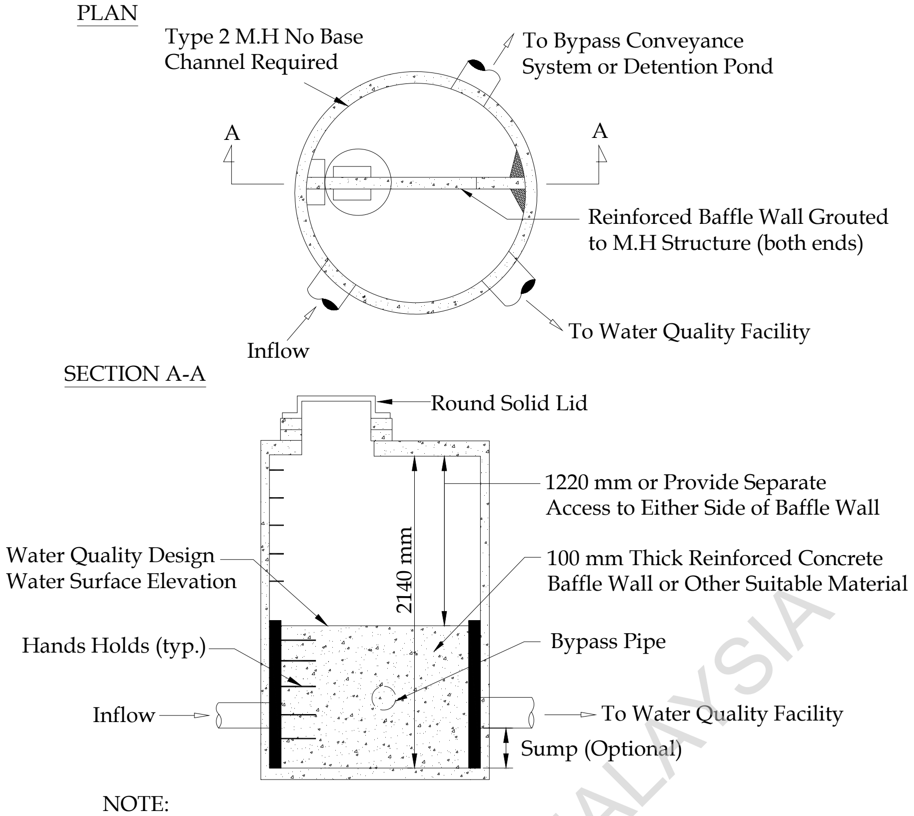
Figure 20.19 shows a typical flow splitter made of manholes with concrete baffles. Figure 20.20 shows a typical diversion/isolation structure.

20.9.2 Design Considerations

Two major considerations for the design of flow-splitting devices are as follows:

(a) Head Loss - Hydraulic disturbances at the point of flow division result in unavoidable head losses. These losses, however, may be reduced by the inclusion of proper flow deflectors in the design of the structure. Deflectors minimise flow separation by providing a gradual transition for the flow, rather than by forcing abrupt changes in flow direction.

(b) Debris - In all transitions from larger to smaller pipes, debris accumulation is a potential problem. Tree limbs and other debris that flow freely in the larger pipe may not fit in the smaller pipe(s) and may restrict flow. In addition, flow splitters cause major flow disturbances resulting in a region of decreased velocity.



NOTE:
The water quality discharge Pipe may require an orifice plate be installed on the Outlet to control the height of the design water surface (weir height). The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0 on the outlet Pipe.

Figure 20.19: Typical Flow Splitter Devices

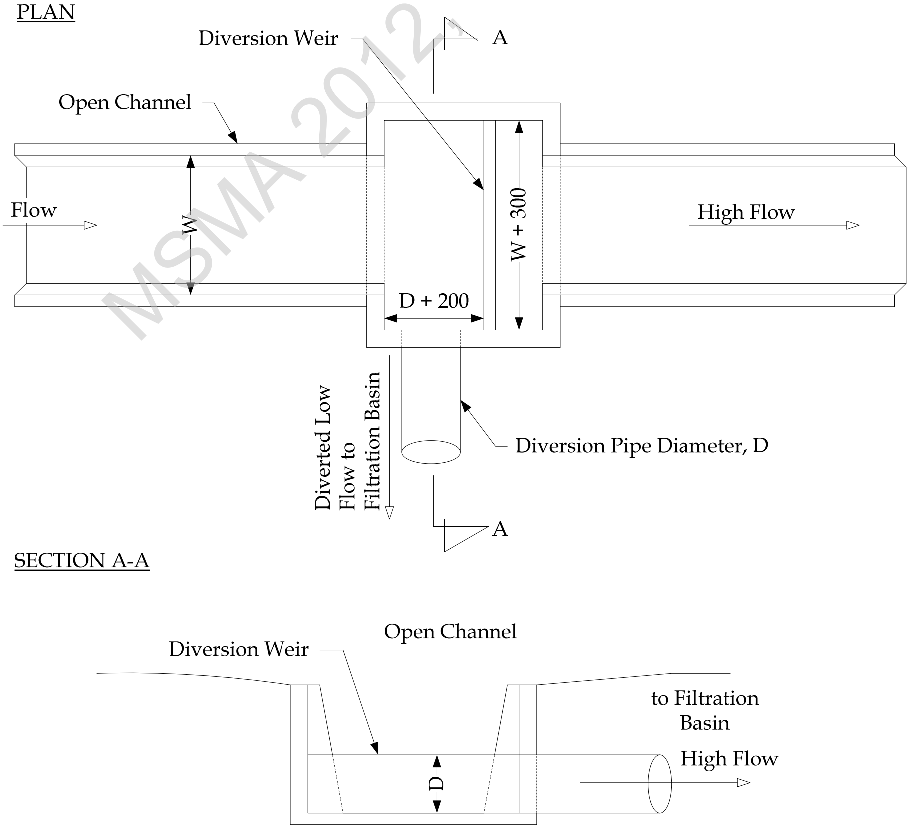


Figure 20.20: Typical Isolation/Diversion Structure

This reduction causes suspended materials in stormwater flow to settle in the splitter box. Although the deflector design should minimise velocity reduction as much as possible, total elimination of the problem is unlikely. Therefore, positive maintenance access must be provided. Because flow splitting devices are maintenance-intensive, their use should be judiciously controlled by the engineer.

20.9.3 Design Criteria

The design criteria for flow splitter are as follows (MassHighway, 2004):

- The top of the weir shall be located at the water surface for the 40mm rainfall depth for water quality design storm;
- The maximum head over the weir shall be minimised for flow in excess of the water quality design flows;
- Outlets must discharge to stable areas;
- Splitter structures must be designed to sustain anticipated dead and live loads;
- Construct splitters in accessible locations; and

20.10 FLOW SPREADER

20.10.1 Description

Flow spreaders are used to uniformly spread flows across the inflow portion of water quality facility (e.g. sand filter, biofiltration swale, or filter strip). Options A through C can be used for spreading flows that have already concentrated. Option D is only for flows that are already unconcentrated and enter a filter strip or biofiltration swale.

20.10.2 General Design Criteria

- Where flow enters the spreader through a pipe, it is recommended that the pipe be submerged to practically dissipate energy; and
- Rock protection is required at outfalls.

20.10.3 Design Criteria for Flow Spreading Options

The following presents the design criteria for each of the flow spreading options:

(a) Anchored Plate (Option A)

Figure 20.21 shows the details of the spreader.

- The spreader shall be preceded by a sump having a minimum depth of 200 mm and minimum width of 600 mm. The sump area shall be lined with steps to reduce erosion and to provide energy dissipation;
- The top of the flow spreader plate shall be level, projecting a minimum of 50 mm above the final grade of the invert of the water quality facility;
- The plate shall extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope;
- The plate shall be securely fixed in place; and
- The flow spreader plate may be either wood, metal, fibreglass reinforced plastic, or other durable material.

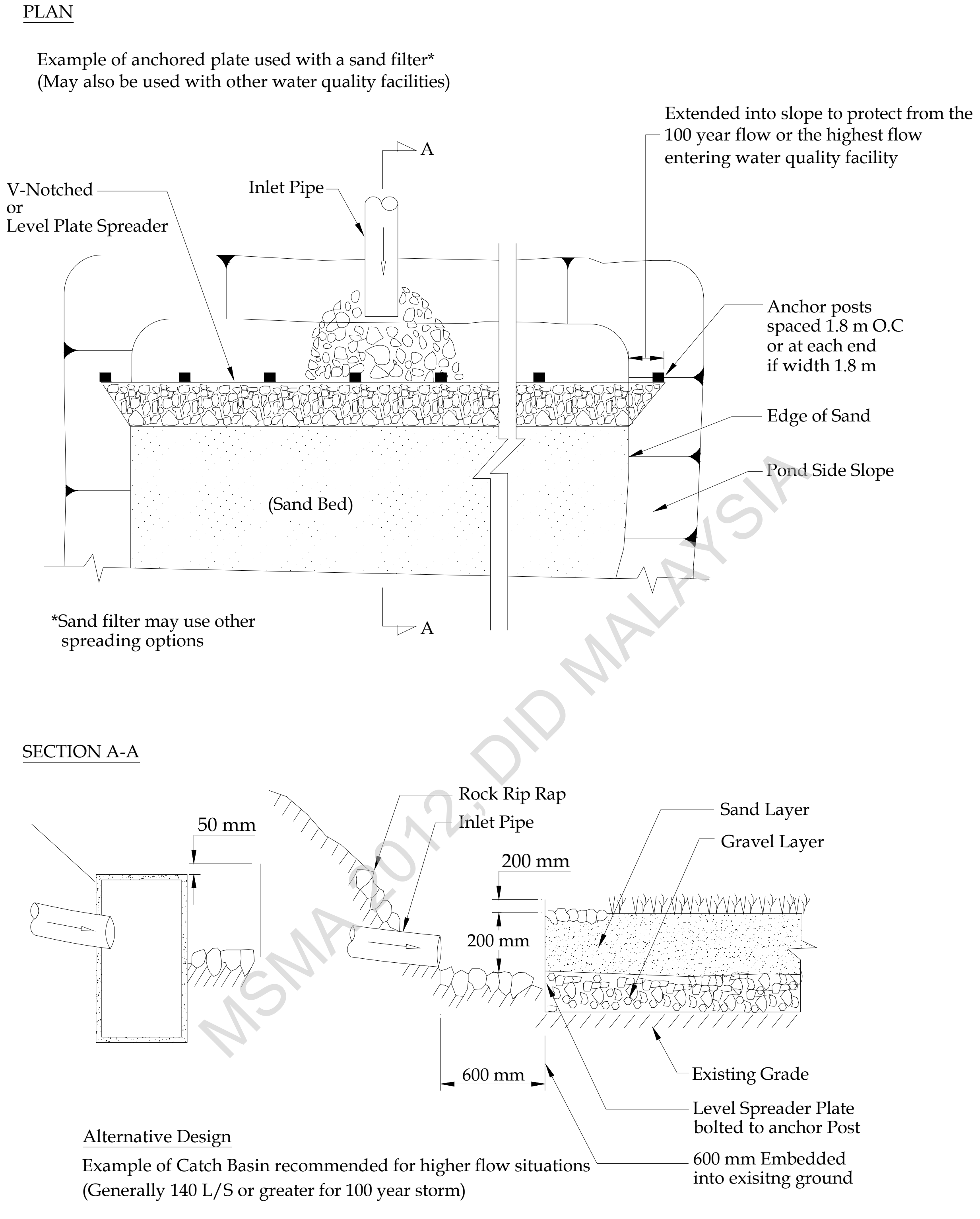


Figure 20.21: Flow Spreader (Option A)

(b) Concrete Sump Box (Option B)

This alternative uses a rectangular concrete sump (see Figure 20.22 for details).

- The wall of the downstream side of the concrete sump shall extend a minimum 50 mm above the invert of the treatment bed; and
- The downstream wall of the box shall have “returns” at both ends. Side walls and returns shall be slightly higher than the weir so that erosion of the side slope is minimised.

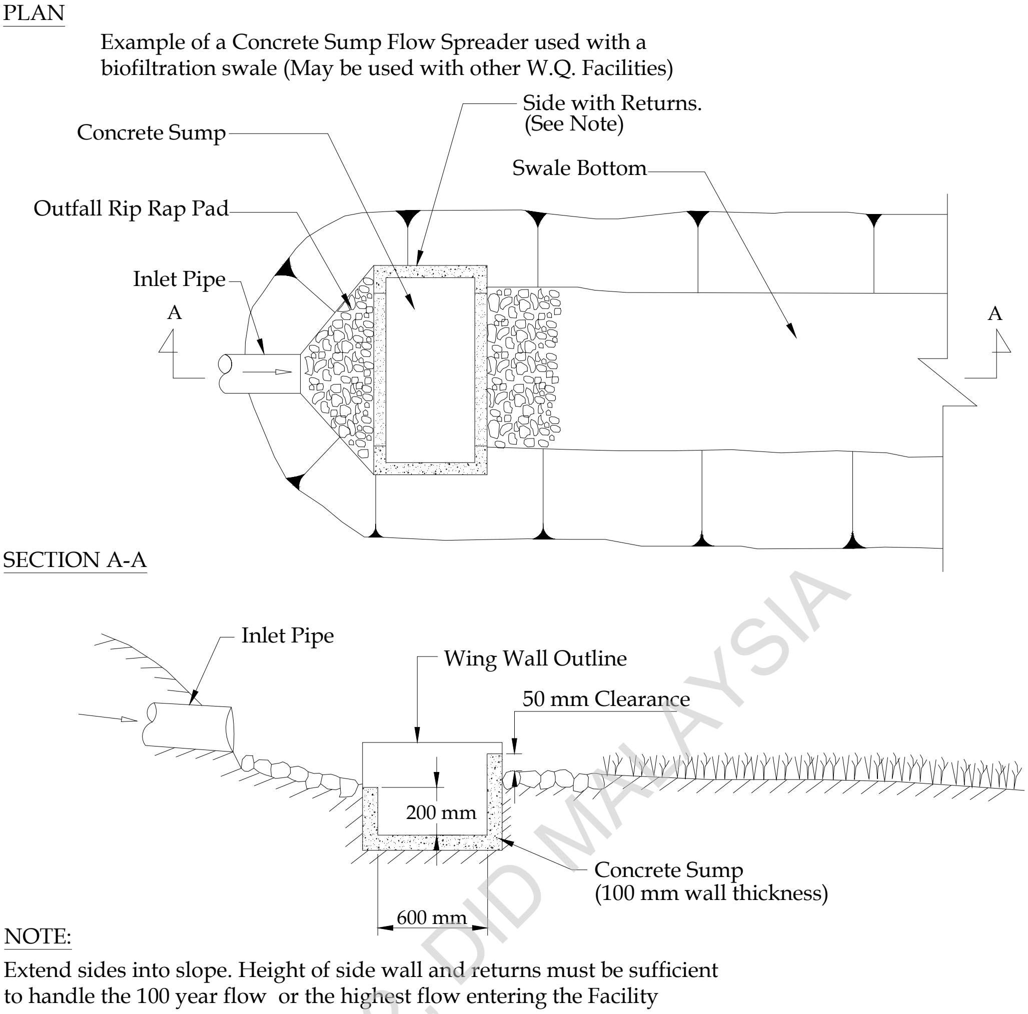


Figure 20.22: Flow Spreader (Option B)

(c) Flat-topped Notched Curb Spreader (Option C)

An example of flat-topped notched curb spreader used with a grassed swale is shown in Figure 20.23. The spreader sections are made of extruded concrete laid side by side and level. Typically four “teeth” per 1.25 m section provide good spacing. The space between adjacent “teeth” forms a v-notch.

(d) Through-Curb Ports (Option D)

Details of the spreader are shown in Figure 20.24. Unconcentrated flows from paved areas entering filter strips or continuous flow biofiltration swales can use curb ports to allow flows to enter the strip or swale. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to convey water to the water quality facility. Openings in the curbing shall be at regular intervals of 2 m (minimum). The width of each curb port opening shall be 275 mm minimum. Approximately 15 percent or more of the curb section length shall be in open ports, and no port should discharge more than about 10 percent of flow.

20.11 GATES

20.11.1 Description

The main operational requirements for gates are the control of floods, watertightness, minimum hoist capacity, convenience of installation and maintenance and above all failure free performance and avoidance of safety hazards to the operating staff and the public. Despite robust design and precautions, faults can occur and the works must be capable of tolerating these faults without unacceptable consequences (Novak et al., 2007). Table 20.2 shows the summary of types of gates.

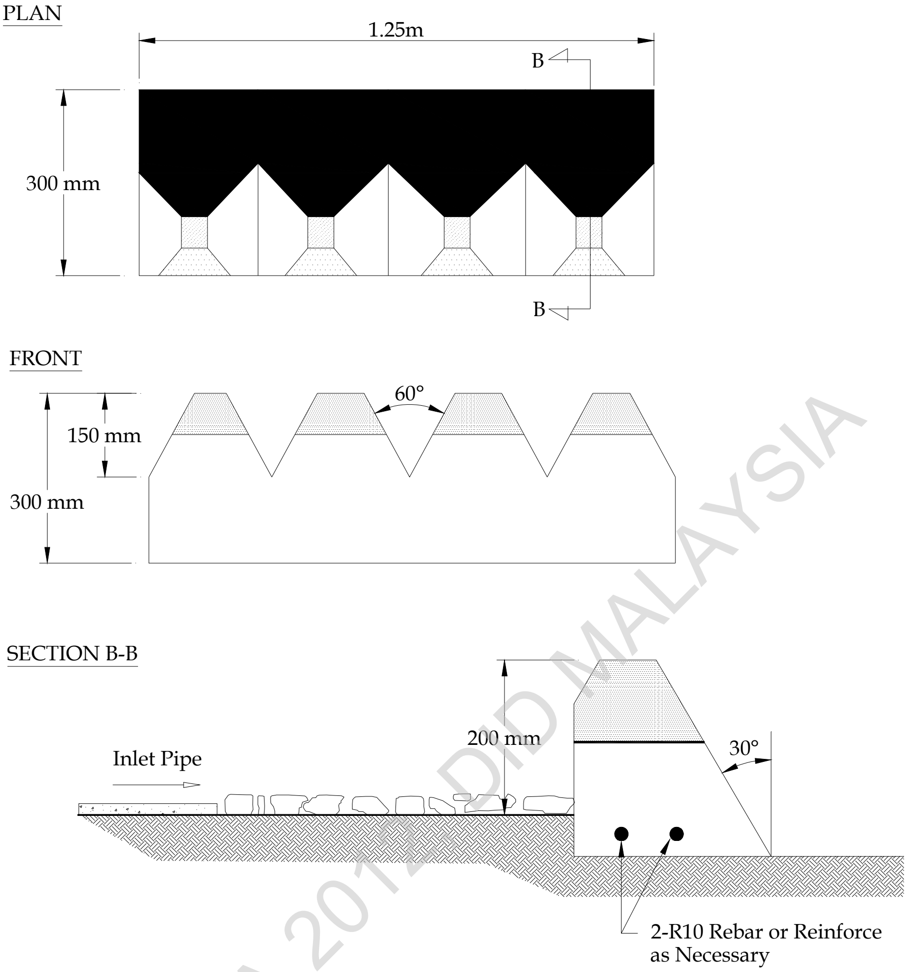


Figure 20.23: V-notch Flow Spreader (Option C)

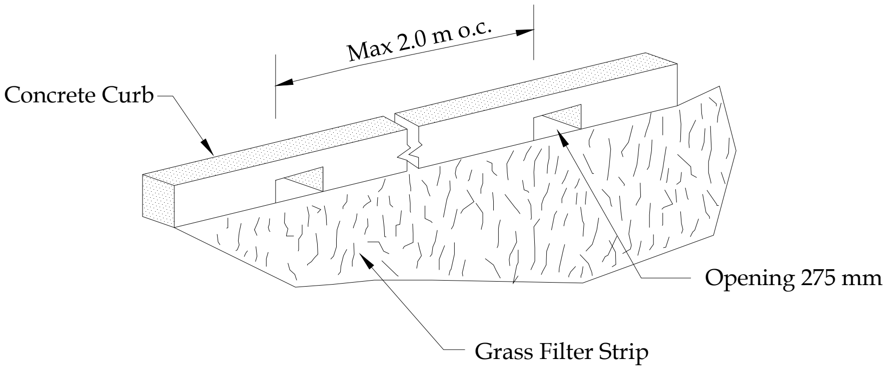


Figure 20.24: Through-curb Port (Option D)

20.11.2 Type of Gates

(a) Radial Gates

Radial gates are usually constructed as portals with cross-bars and arms (straight, radial, or inclined), but could also be cantilevered over the arms. Their support hinges are usually downstream but (for low heads) could also be upstream, resulting in shorter piers. The gate is usually hoisted by cables fixed to each end to prevent it from twisting and jamming. If the cables are connected to the bottom of the gate its top can be raised above the level of the hoist itself, if the layout of the machinery allows it (Novak et al., 2007).

(b) Drum and Sector Gates

Drum and sector gates are circular sectors in cross-section. Drum gates are hinged upstream and sector gates downstream. Gates on dam crests are usually of the upstream hinge type. Drum gates float on the lower face of the drum, whereas sector gates are usually enclosed only on the upstream and downstream surfaces (Novak et al., 2007).

(c) Flap Gates

Flap gates are one of the simplest and most frequently used types of regulating gates used mainly on weirs and barrages (rarely on dam crests), either on their own or in conjunction with plain lift gates. They were developed as a replacement for wooden flash-boards, originally as a steel-edged girder flap, which was later replaced by a torsion-rigid pipe; further development was achieved by placing the pipe along the axis of the flap bearing, with the skin plate transmitting the water pressure to cantilevered ribs fixed to the pipe. Next in use were torsion-rigid gate bodies with curved downstream sides (fish-belly gates), with torsion-rigid structures using prism-shaped sections being the latest development in flap gates (Brouwer, 1988).

(d) Top Hinged Flap Gates

Top-hinged flap gates are used in tidal structures to prevent flooding of an inland region by sea waters during rising tides or flood surges and to permit inland waters to drain off into the sea during ebb tide. They are also used in culverts and pumped drainage outfalls to rivers. They do not require an outside source of power and operate automatically. The construction of the gates is simple and little maintenance is required. The gates will not entirely exclude ingress of saline water if the downstream water level rises above the sill during discharge under the gate, when a lens of saline water can penetrate upstream against the flow.

They control water in one direction only and perform like a non-return valve. They cannot control upstream level. In stormwater discharge this facility is not required. Top-hinged flap gates can be operated under clear discharge conditions or drowned. When designing gates of this type a gravity bias is required in the closed position so that the gates close immediately before reversal of flow occurs (Lewin, 2001).

(e) Side Hinged Flap Gates

Side hinged flap gates are used for the conversion of wetland environments into agricultural land. These are very efficient in draining upstream lands, preventing the intrusion of saline waters and back flooding during high tides and some minor floods. These lead to the development of freshwater systems, where they were previously saline / brackish. Only suitable for tidal gating of channels set back from main rivers and when complete tidal isolation is required. Side hinged flap gate only allow flow in one direction (downstream) for drainage purposes (NSW Government of Industry and Investment, 2009).

(f) Roller Gates

A roller gate consists of a hollow steel cylinder, usually of a diameter somewhat smaller than the damming height; the difference is covered by a steel attachment, most frequently located at the bottom of the cylinder (in the closed position). The gate is operated by rolling it on an inclined track. Because of the great stiffness of the gate, large spans may be used, but roller gates require substantial piers with large recesses. New roller gate installations are not being used nowadays partly because large units of this type are very vulnerable to single point failure (Novak et al., 2007).

(g) Fabric Gates

Inflatable rubber or fabric gates can be pressurized by air, water or both. They usually have an inner shell and an outer casing, and can be used to close very large spans (Novak et al., 2007).

(h) Vertical Lift Gates

Vertical lift gates designed as a lattice, box girder, a grid of horizontal and vertical beams and stiffeners, or a single slab steel plate, may consist of single or double section (or even more parts can be involved in the closure of very high openings); in the case of flow over the top of the gate it may be provided with an additional flap gate. The gates can have slide or wheeled support. In the latter case fixed wheels (most frequent type), caterpillar or a roller train (Stoney gate) may be used; for fixed wheels their spacing is reduced near the bottom. The gate seals are of specially formed rubber (Novak et al., 2007).

Table 20.2: Summary of Type of Gates

Types	Advantages	Pictures
Radial Gates	• Smaller hoist; • Higher stiffness; • Lower (but longer) piers; • Absence of gate slots; and • Easier automation;	
Drum and Sector Gates	• Ease of automation and absence of lifting gear; • Fast movement; • Accuracy of regulation; and • Low piers.	
Top Hinged Flap Gates	• Provide fine level regulation; • Ease flushing of debris; • Cost effective and often environmentally more acceptable; • Naturally automated; • Simplistic construction and installation; • Requires little maintenance; • Long lifespan; and • Gates manufactured from non-metal materials have no scrap value and will have no attraction to scrap metal thieves.	

Table 20.2: Summary of Type of Gates (Continued)

Types	Advantages	Pictures
Side Hinged Flap Gates	• Naturally automated; • Requires small amount of energy (water pressure) to open gates. As a result, more robust materials can be used for the gate without compromising the efficiency of the structure; • Once open, the gate will remain open until the tide recedes; • Gates open wider than top hinged flap gates and for a greater period; • Full depth of the water column is available for fish passage; • Simplistic and more difficult to vandalise; • Requires little maintenance; and • Long lifespan.	
Roller Gates	• Very reliable; • Capable of operating at differential heads; and • Can be manufactured very wide.	
Fabric Gates	• Low cost; • Low weight; • Absence of lifting mechanism; • Little need for maintenance; • Acceptance of side slope (at river banks); and • Ease of installation.	
Vertical Lift Gates	• Can be fitted with overflow sections; • Short piers; • Wide span gates can be engineered to provide good navigation openings; and • Up and over gates can reduce height of supporting structure.	

20.12 VALVES

20.12.1 Description

Many types of valves have been invented by man to control the flow of fluids. Of those which have survived the test of time, each has at least several features which are unique or important. One offers tight shut-off, another low cost, others effective control of the fluid flow, still others, perhaps combinations of these, and on and on. To date, however, no valve inventor has discovered the ideal valve which combines all these features into one package and experience teaches us that it is unlikely anyone will. Thus, valve designers have created the globe, plug, ball, gate and numerous other valve types, all of which are in extensive use throughout the world's vast and complex industrial processes. The gate valve is among the most common because it offers several advantages of function and cost effectiveness over other types (Norden, 1975). Table 20.3 presented the summary of type of valves.

20.12.2 Type of Valves

(a) Check Valve

The check valve can be used to provide flow in one direction only through a culvert for floodplain drainage and flood and saline intrusion mitigation purposes. The check valve is constructed from an elastomer with a vertical slot that is flexible yet quite stiff and closed in its relaxed position. This elastomer material is resistant to the corrosive effects of marine and highly acidic waters, a significant issue associated with most metallic structures. The check valve can be mounted flush on a flat or curved headwall, or be clamped to culverts of varying shape, size or material. During backflow, the check valve seals shut (NSW Government of Industry and Investment, 2009).

(b) Cone Dispersion Valve

The cone dispersion valve is probably the most frequently used type of regulating valve installed at the end of outlets discharging into the atmosphere. It consists of a fixed 90° cone disperser, upstream of which is the opening covered by a sliding cylindrical sleeve The fine spray associated with the operation of the valve may be undesirable, particularly in cold weather; sometimes, therefore, a fixed large hollow cylinder is placed at the end of the valve downstream of the cone, resulting in a ring jet valve (Novak et al., 2007).

(c) Needle Valve

The needle valve, (and its variation the tube valve), has a bulb-shaped fixed steel jacket, with the valve closing against the casing in the downstream direction. When open, the valves produce solid circular jets and can also be used in submerged conditions. The valves may suffer from cavitation damage and produce unstable jets at small openings, and are expensive as they have to withstand full reservoir pressures (Novak et al., 2007).

(d) Hollow-Jet Valve

Most of the disadvantages of valves are overcome in the hollow-jet valve, which closes in the upstream direction (when closed the valve body is at atmospheric pressure); because of this the valve is, of course, not suitable for use in submerged conditions (Novak et al., 2007).

Table 20.3: Summary of Type of Valve

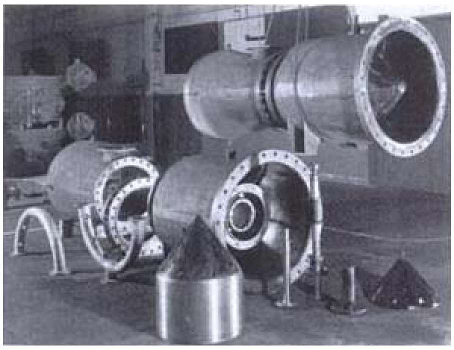
Types	Advantages	Pictures
Check Valve	• Self automated; • Can be mounted to a variety of culvert configurations - bolted to headwalls or clamped to pipes; • Can be installed at any angle including vertical; • Smaller valves can usually be installed by one person with simple hand tools; • Corrosion resistant and outer wrapping resistant to ozone; • Not as susceptible to jamming and will seal around entrapped floating debris during backflow. Can still operate when partially buried in sediment; • Flexible nature reduces risk of damage from floating debris; • No clearance is required for operation; and • Has no moving mechanical parts subject to wear and can last up to 50 years.	
Cone Dispersion Valve	• Very efficient energy dissipation valve; • Simple construction; • Relatively low cost; • Can be operated electro-mechanically or by oil hydraulics; • Good discharge coefficient; • Available in large sizes; and • Least flow obstruction of any terminal discharge valve.	
Needle Valve	• Dissipates energy; • Can be used as an in-line pressure-reducing valve; and • Best valve for pressure reduction when cavitation of downstream conduit is of concern.	

Table 20.3: Summary of Type of Valve (Continued)

Types	Advantages	Pictures
Hollow-jet Valve	• Dissipates energy; and • Can be arranged to discharge into a stilling basin at an angle.	

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ECOLOGICAL PLANTS

1
ANNEX

ANNEX 1 ECOLOGICAL PLANTS

AX1.1 INTRODUCTION

Planting has specific functions such as prevents erosion of soil surfaces, traps silt and prevents re-suspension, filters and treats pollution, provides wildlife habitat and promotes attractive and natural surroundings. Table AX1.1 which indicates the effectiveness of different vegetation in meeting specific objectives within a riparian buffer zone can be used for the mix design.

Table AX1.1: Relative Effectiveness of Different Vegetation Types for Providing Specific Benefits

Benefit	Vegetation		
	Grass	Shrub	Trees
Stabilizes bank erosion	Low to Medium	High	Medium to High
Traps sediment	High	Low to Medium	Low
Filters nutrients, pesticides, microbes			
- Sediment bound	High	Low	Low
- Soluble	Medium	Low	Medium
Provides aquatic habitat	Low	Medium	High
Provides wildlife habitat			
- Range/pasture wildlife	High	Medium	Low
- Forest wildlife	Low	Medium	High
Provides economic products	Medium	Low	
Provides visual diversity	Low to Medium	Medium	High
Prevents bank failures	Low	Medium	High
Provides flood conveyance	High	Low	Low

AX1.1.1 Plant Characteristics

Plant characteristics must be considered to determine how the plant provides interest and whether the plant will fit with the present and future landscapes. Some of these characteristics are colour, texture, and interest, i.e. flowers, fruit, leaves, stems or bark and growth rate. In urban or suburban settings, the landscape treatment of the stormwater facility shall be appealing and interesting. Careful consideration during designing and vegetation planting of a facility can result in greater public acceptance and increased property value.

AX1.1.2 Environmental Influences on Plants

General environmental factors and threats to investigate during site analysis are shows in Table AX1.2.

Table AX1.2: General Site Condition to Investigate (Shaw and Schmidt, 2003)

Environmental factors	Environmental threats
• Texture, organic content and pH of the soil • Anticipated water levels or soil moisture • Adjacent plant communities • Slopes • Surrounding weedy vegetation • Amount of sun or shade • Aspect (north, south, east or west facing slope)	• Flood depth and duration • Nutrients • Low water levels • Salt • Flood frequency • Turbidity • Wave energy • Erosion • Sediment loads • Invasive plants • Pollutants and toxins • Herbivores

AX1.1.3 Prohibited and Poisonous Plants

There are also plant species that are prohibited to be imported or grown in Malaysia under the Plant Quarantine Act (1976). If convicted, the offender(s) may be fined up RM10,000. Designers should refer to the quarantine and poisonous list of plant species provided by the Department of Agriculture in National Landscape Department Guideline for any landscape design.

AX1.2 SPECIFIC PLANTING CRITERIA

AX1.2.1 Ponds and Wetlands

a) Plant Selection

Basically ponds and wetlands should consist of vegetation with the following attributes:

- adaptation to the local climate and soils (native species);
- tolerance to pollutants in stormwater runoffs;
- high biomass production;
- perennial species;
- rapid growth but to avoid usage of noxious species; non-weedy, aesthetic habit;
- valuable as wildlife habitat; and
- broadest possible feasible mixture of plant species to maximise plant diversity and enhance stability of the pond or wetland.

b) Planting Zones

Planting zones are categorised into the 6 different zones, which is shown in Figure AX1.1. The criteria and recommended plant species for each zone are shown in following section.

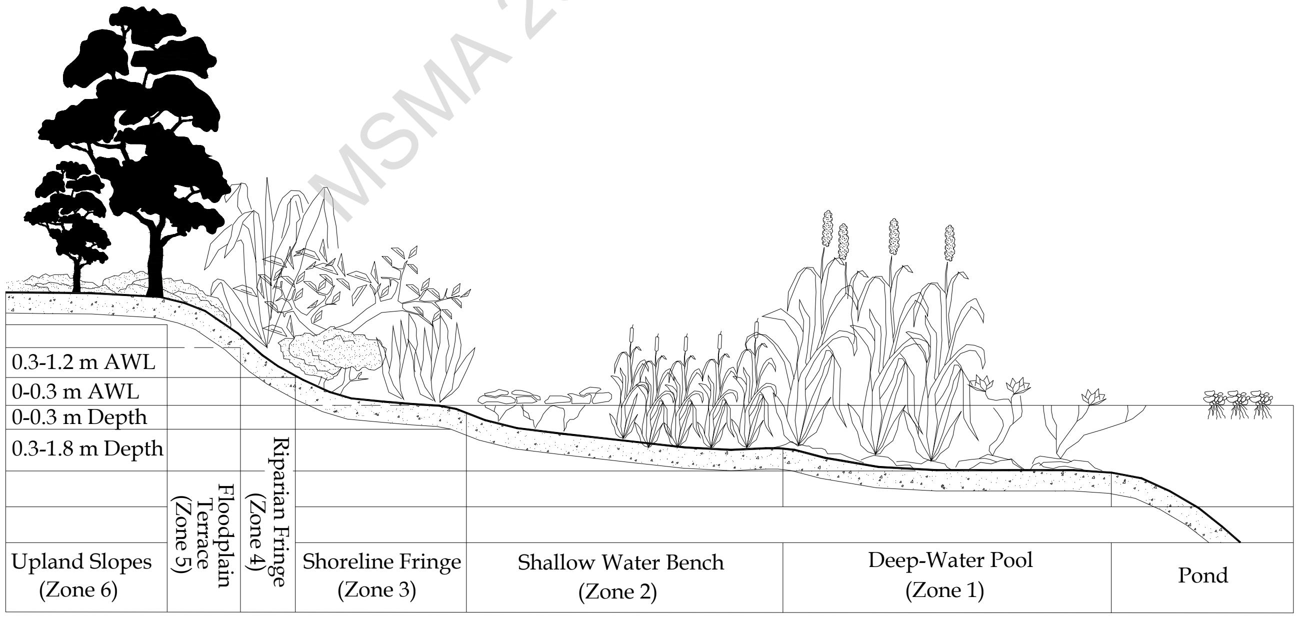

Figure AX1.1: Longitudinal Section of Typical Pond or Wetland *i) Zone 1: Deep-water Pool (0.3 – 1.8 m BWL)*

Zone 1 should contain submerged aquatic plants that help to increase diversity and create habitat. (Table AX1.3). The functions of this zone is to reduce re-sedimentation and improve oxidation.

Table AX1.3: Recommended Plant Species for Zone 1 (Deep-water Pool)

Botanical Name	Common Name	P	B	E	A	I
Cyperus compactus	Swamp mariscus; Para-para	•				
Cyperus digitatus	Finger flatsedge; Rumput bunga satuan, Rumput musang	•				
Cyperus halpan	Sheathed flatsedge; Rumput sumbu, Bilis jantan, Para air	•		•		
Lepironia articulata	Tube sedge, Grey sedge, Blue rush, Twigrush; Purun, Kercut	•		•		
Nasturtium sp.	Nasturtium	•		•		
Nelumbo nucifera	Sacred lotus, Indian lotus, Bean of India; Telipok, Seroja India			•	•	
Nymphae lotus dentata	Tiger lotus, White lotus, Egyptian white water lily			•	•	
Nymphae nouchali	Star lotus, Red and blue water lily, Blue star water lily; Teratai putih hutan, Tanjung putih			•	•	
Nymphae rubra	India red water lily; Teratai merah			•	•	
Nymphae tashkent	Purple Water lily; Teratai ungu			•	•	
Phragmites karka	Common reed, Tall reed, Tropical reed; Rumput gedabong	•		•		
Phylidrum lanuginosum	Wooly water lily, Frogmouth, Fan grass; Rumput kipas	•		•		
Rynchospora corymbosa	Golden beak sedge; Rumput sendayan	•		•		
Scirpus grossus	Greater club rush; Rumput menderong, Rumput kumbar	•		•		
Scirpus juncoides	Upright club-rush; Rumput bulat	•				
Scleria sumatrensis	Sumatran scleria; Rumput kumba	•				
Typha latifolia	Bulrush, Broadleaf cattail; Banat	•		•	•	
Victoria sp.	Victoria water lily, Giant water lily, Royal water lily				•	

P = Pollution control B = Bank/slope protection

E = Ecological A = Aesthetic I = Indigenous

ii) Zone 2: Shallow Water Bench (0 to 0.3 m BWL)

Primary area for the emergent plants (Table AX1.4) may be located at the edge of the pond. When planted, Zone 2 can be an important habitat for many aquatic and non-aquatic animals creating a diverse food chain.

Table AX1.4: Recommended Plant Species for Zone 2 (Shallow Water Bench)

Botanical Name	Common Name	P	B	E	A	I
Cleome spinosa	Spider flower, Spider legs, Spiny spiderflower; Maman		•	•		
Eleocharis variegata	Spike rush; Ubi purun, Puron	•		•		
Eriocaulon longifolium	Asiatic pipewort, Longleaf pipewort; Rumput butang	•		•		
Fimbristylis globulosa	Globular fimbristylis, Globe fimbry; Rumput sadang	•		•		
Fuirena umbellata	Hairy blue sedge, Yefen; Rumput kelulut	•		•		
Hanguana malayana	Common hanguana, Common susum; Bakong, Bakong rimba		•		•	
Ludwigia adscendens	Floating Malayan willow, Creeping water primrose; Tinggir bangau		•		•	•
Ludwigia octovalis	Shrubby water primrose; Tinggir pasir		•	•		
Monochoria hastata	Monocharia, Arrowleaf pondweed; Keladi agas	•		•		
Pandanus immersus	Swamp/riverine pandanus; Pandan rasau		•			•
Pandanus sp.	Screw pine, Screw palm; Pandan pantai		•		•	•
Rynchospora corymbosa	Golden beak sedge; Rumput sendayan	•				
Sagittaria sagitaefolia	Arrowhead, Verigated lesser arrowhead; Bunga sagitaria kuning	•				
Scleria sumatrensis	Sumatran scleria; Rumput kumba	•				
Stachytapheta jamaicensis	Spotted basil, Blue porterweed; Selasih dandi, Pokok kecut kuda		•	•		
Vanda hookeriana	Kinta weed, Pencil Orchid; Anggrek pensil				•	•
Zingiberacae sp.	Wild ginger; Halia hutan			•	•	

P = Pollution control B = Bank/slope protection

E = Ecological A = Aesthetic I = Indigenous *iii) Zone 3: Shoreline Fringe (0 to 0.3 m AWL)*

This zone can be found in a wet pond or shallow marsh. Many of the emergent plants in Zone 2 can also thrive in Zone 3 (Table AX1.5). If shading is needed along the shoreline, tree species are also recommended.

Table AX1.5: Recommended Plant Species for Zone 3 (Shoreline Fringe)

Botanical Name	Common Name	P	B	E	A	I
Alstonia spathulata	Marsh pulai, Siamese balsa, Hard milkwood; Pulai paya				•	
Artocarpus altilis	Breadfruit; Sukun			•		•
Cyrtostachys lakka	Red sealing-wax palm, Dwarf lipstick palm; Pinang merah, Pinang raja				•	
Dillenia suffruticosa	Shrubby simpoh, Shrubby dillenia; Simpoh air		•	•	•	•
Melaleuca leucadendron	Cajaput tree, Paper-bark tree, Weeping teat tree; Gelam			•		•
Pometia pinnata	Fijian longan, Island lychee; Kasai		•	•	•	•
Saraca thaipingensis	Yellow ashoka, Yellow saraca; Saraka kuning, Pokok gapis			•	•	•
Shorea longifolia	Meranti hitam paya				•	•
Shorea platycarpa	Light red meranti; Meranti paya				•	•
Sindora coriaceae	Sepetir licin, Sepetir minyak				•	•

P = Pollution control
B = Bank/slope protection
E = Ecological
A = Aesthetic
I = Indigenous

iv) Zone 4: Riparian Fringe (0.3 – 1.2 m AWL)

Zone 4 extends from 0.3 m to 1.2 m in elevation above the normal pool. Plants in this zone are subject to periodic inundation after storms, and may experience saturated or partly saturated soil condition. Recommended plant species for Zone 4 are shows in Table AX1.6.

Table AX1.6: Recommended Plant Species for Zone 4 (Riparian Fringe)

Botanical Name	Common Name	P	B	E	A	I
Arachis pintoi	Yellow peanut plant, Pinto peanut; Kekacang, Kacang hias		•		•	
Asystasia gangetica	Creeping foxglove; Rumput itik			•	•	
Bambusa vulgaris	Common bamboo, Giant yellow clumping bamboo, Feathery Bamboo; Buluh minyak, Buluh gading, Buluh aur		•			
Caryota no	Giant fishtail palm; Tukas			•	•	
Cocoloba uvifera	Sea grape, Hopwood, Horsewood			•		
Cratoxylon arborescens	Mabberley; Geronggang, Seronggang		•			
Dillenia suffruticosa	Shrubby simpoh, Shrubby dillenia; Simpoh air		•	•	•	
Elaeocarpus nitidus	Walnut oil fruit; Pinang punai				•	
Ficus benjamina	Weeping fig, Benjamin fig; Ara beringin, Ara waringin	•	•	•		•
Ficus globosa	Bling fig; Ara kelalawar, Ara paya		•	•	•	
Johannesteijmannia altifron	Johanna palm, Diamond Joey, Joey palm		•	•	•	•
Koompassia malaccensis	Kempas tree; Kempas			•	•	•
Licuala spinosa	Mangrove fan palm, Spiny licuala, Good luck palm; Palas duri		•	•	•	•
Melia excelsa	Marrango tree, Philipine neem tree; Sentang				•	
Nephrolepis sp.	Sword fern; Paku		•	•	•	

P = Pollution control
B = Bank/slope protection
E = Ecological
A = Aesthetic
I = Indigenous

v) Zone 5: Floodplain Terrace (Infrequently Inundated)

Zone 5 is periodically inundated by floodwaters that quickly recede in a day or less. Key landscaping objectives for Zone 5 are to stabilise the steep slope of this zone and establish low maintenance natural vegetation. Recommended plant species for Zone 5 are shows in Table AX1.7.

Table AX1.7: Recommended Plant Species for Zone 5 (Floodplain Terrace)

Botanical Name	Common Name	P	B	E	A	I
Alstonia angustiloba	Common pulai; Pulai		•	•	•	•
Archontophoenix alexandrae	Alexandra palm, Alexander palm, King palm; Palma Iskandar		•		•	
Costus speciosus	Malay ginger, Crape ginger, Spiral flag; Setawar tawar			•	•	•
Dendrocalamus giganteus	Giant bamboo; Buluh betong		•	•		•
Dyera costulata	Jelutong tree; Jelutong, Jelutong burit, Jelutong paya		•		•	•
Fagraea fragrans	Tembusu tree; Tembusu		•	•	•	•
Heliconia psittacorum	Parrot's beak, Parakeet flower, Parrot's flower			•	•	
Lagerstroemia flos-reginae	Queens crape myrtle, Pride of India, Rose of India; Bungor		•		•	•
Melastoma malabathricum	Malabar melastome, Straits Rhododendron; Senduduk, Keduduk, Senggani,		•	•		•
Messua ferrea	Ceylon ironwood, Indian rose chestnut; Penaga lilin		•	•	•	•
Mussaenda erythrophylla	Ashanti blood, Red flag bush, Tropical dogwood; Janda kaya			•	•	
Oncosperma horridum	Thorny palm, Mountain nibung palm; Bayas		•			•
Oncosperma tigillarium	Nibung palm; Nibung		•	•		•
Pandanus pigmeus	Small screwpine; Pandan kuning		•		•	
Pisonia alba	Lettuce tree, Cabbage tree, Moonlight tree; Menkudu siam		•		•	
Tacca chantrieri	Bat head lily, Bat Flower, Devil Flower; Misai baung			•	•	•

P = Pollution control
B = Bank/slope protection
E = Ecological
A = Aesthetic
I = Indigenous

vi) Zone 6: Upland Slopes (Seldom or Never Inundated)

The last zone extends above the maximum 100-year water surface elevation, and often includes the outer buffer of a pond or wetland. Care should be taken to locate plants so they will not overgrow these routes or create hiding places that might make the area unsafe. Recommended plant species for Zone 6 are shows in Table AX1.8.

Table AX1.8: Recommended Plant Species for Zone 6 (Upland Slopes)

Botanical Name	Common Name	P	B	A	E	I
Bauhina blakeana	Hong Kong ochid tree; Tapak kuda			•		•
Cananga odorata	Cananga tree, Dwarf Ylang Ylang; Kenanga			•	•	•
Canarium vulgaris	Kanari nut tree; Kenari			•	•	
Cassia fistula	Golden shower tree, Indian laburnum; Senong, Dulang				•	
Cicca accida	Tree bears; Cermai			•	•	•
Cinnamomum iners	Wild cinnamonhindi; Kayu manis			•		•
Dryobalanops aromatica	Sumatra camphor; Kapur barus			•		•
Eucalyptus deglupta	Mindanao gum; Kayu putih			•		
Flacourtia inermis	Batoko plum; Rokam			•	•	•
Hibiscus mutabilis	Confederate rose, Cotton rosemallow; Baru landak, Bebaru			•		
Livistona rotundifolia	Footstool palm; Serdang			•	•	
Melia excelsa	Marrango tree, Philipine neem tree; Sentang			•	•	•
Milletia atropurpurea	Purple milletia; Tulang daing			•	•	•
Peltophorum pterocarpum	Yellow flame; Batai laut		•	•	•	•
Pritchardia pacifica	Fiji fan palm; Palma kipas Fiji			•		
Raphis excelsa	Broadleaf lady palm, Bamboo palm; Rafis, Pinang rotan			•		
Roystonea regia	Royal palm; Palma diraja			•		
Tectona grandis	Teak; Jati			•	•	•
Zizyphus mauritania	Indian Jujube; Bidara			•		

P = Pollution control
B = Bank/slope protection
E = Ecological
A = Aesthetic
I = Indigenous

AX1.2.2 Infiltration Systems

Suitable plant species for these systems are given in Table AX1.9. They, however, are subjected to the following design constraints:

- Planting a vegetated filter strip of at least 5.5 m width will cause sediments to settle out before reaching the facility, thereby reducing the possibility of clogging;
- Determine areas that will be saturated with water and water table depth so that appropriate plants may be selected (hydrology will be similar to bioretention facilities);
- Plants known to send down deep taproots should be avoided in system where filter fabric is used as part of facility design;
- Test soil condition to determine if soil amendments are necessary;
- Plants shall be located so that access is possible for structure maintenance;
- Stabilise heavy flow areas with erosion control mats or sod; and
- Temporarily divert flows from seeded areas until vegetation is established.

Table AX1.9: Recommended Shrubs and Grass Species for the System

Botanical Name	Common Name
Alocasia sp.	Alocasia, Taro; Keladi
Alpinia sanderae	Variegated Ginger; Halia hiasan
Calathea sp.	Peacock plant; Lerek
Canna generalis	Canna lily; Bunga tasbih
Cassia alata	Wild senna, Ringworm bush; Gelenggang
Cleome speciosa	Spiny spiderflower; Maman
Gesneriaceae sp.	Cloudforest flower; Letup-letup
Ipomea involucrata	Keledek nyiru
Ixora javanica	Jungle flame, Jungle geranium; Siantan
Turnera ulmiflora	Yellow buttercups, Yellow alder, Sage Rose, Cuban buttercup; Turnera
Zoysia matrella	Manila grass, Manila temple grass, Korean grass; Rumput siglap

AX1.2.3 Bioretention Systems

a) Soil Bed Characteristic

Soil bed characteristics for bioretention systems are perhaps as important as the facility, location, size, and treatment volume. The soil must be permeable enough to allow runoff to infiltrate through the media, while having characteristics suitable to promote and sustain a robust vegetative cover crop. Therefore, the soils must have balance soil chemistry and physical properties to support biotic communities above and below ground.

b) Planting Plan Design Consideration

- Native plant species should be specified over exotic or foreign species.
- Appropriate vegetation should be selected based on the zone of hydraulic tolerance.
- Species layout should generally be random and natural.
- A canopy should be established with an under storey of shrubs and herbaceous materials.
- Woody vegetation should not be specified in the vicinity of inflow location.
- Trees should be planted primarily along the perimeter of the bioretention area.
- Urban stressors (e.g. wind, sun, exposure, insect and disease).

- Infestation and drought should be considered when laying out the planting plan.
- Noxious weeds should not be specified.
- Aesthetics and visual characteristics should be a prime consideration.
- Traffic and safety issues must be considered.
- Existing and proposed utilities must be identified and considered

Plants selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an upland-species ecosystem. The community should be dominated by trees, but have a distinct community of under storey trees, shrubs and herbaceous materials (see Table AX1.10).

Table AX1.10: Suggested Plant Species for Bioretention Areas

Botanical Name	Common Name
Ground Cover/Shrubs/Palms	
Arundina graminifolia	Tapah weed, Bamboo orchid, Bird orchid; Anggerik buluh, Anggerik tanah
Cyclosorus aridus	Dry wood-fern; Paku paya
Ipomoea cairica	Railway creeper, Ivy-leaved Morning Glory; Seri pagi jalar
Ishaemum muticum	Seashore centipede grass, Drought grass; Rumput tembaga jantan, Rumput Kemarau
Trees	
Alstonia spathulata	Marsh pulai, Siamese balsa, Hard milkwood; Pulai paya
Ploiarium alternifolium	Cicada tree; Riang-riang
Saraca thaipingensis	Yellow ashoka, Yellow saraca; Saraka kuning, Pokok gapis

There are essentially three zones within the bioretention system as show in Figure AX1.2.

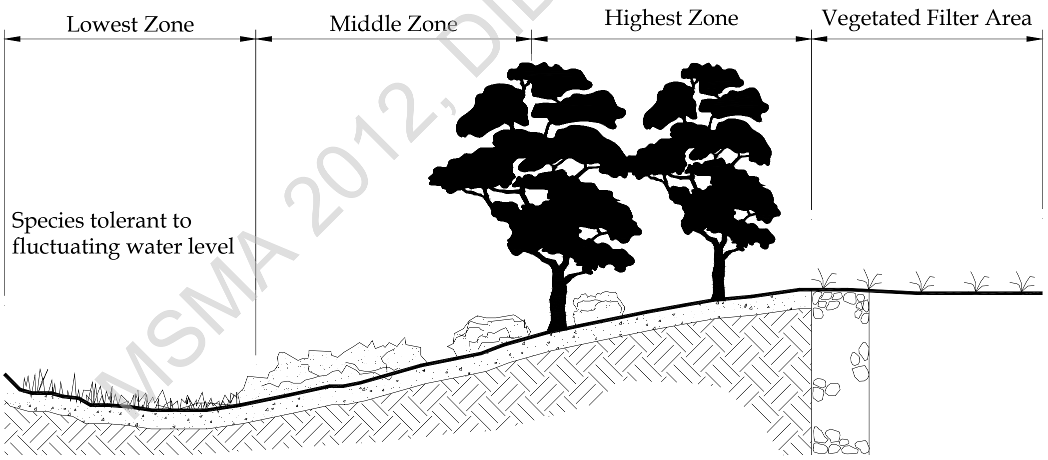

Figure AX1.2: Planting Zones for Bioretention System

Table AX1.11: Recommended Ground Cover Species for Grassed Channel, Vegetated Filter Area and Swale

	Botanical Name	Common Name
Grassed Channel, Vegetated Filter Area	*Ageratum conyzoides*	Chick weed, Goatweed; Rumput tahi ayam
	Arachis pintoi	Yellow peanut plant, Pinto peanut; Kekacang, Kacang hias
	Asystasia gangetica	Creeping foxglove; Rumput itik
	Chloris barbata	Swollen finger grass; Rumput mekar
	Clidemia hirta	Soupbush; Senduduk bulu
	Commelina nudiflora	Common spiderwort; Rumput aur
	Croton hirtus	Kroton berbulu, Cenderai gajah
	Digitaria fuscescens	Yellow crab grass; Rumput jejari halus
	Echinochloa colonum	Junglerice; Rumput kekusa kecil
	Elephantopus scaber	Prickly-leaved elephant's foot; Rumput tutup bumi
	Eupatorium odoratum	Common floss flower; Rumput kapal terbang
	Gomphrena globosa	Globe amaranthus; Bunga butang
	Merremia umbellata	Yellow wood rose; Akar senduduk belanga
	Paspalum conjugatum	Buffalo grass; Rumput lembu
	Phanera audax	Akar merak
	Phanera integrifolia	Akar kuning raja
	Phaseolus pubescens	Kacang faseolus bulu
	Pueraria phaseoloides	Puero; Kekacang
	Stachytapheta jamaicensis	Spotted basil, Blue porterweed; Selasih dandi, Pokok kecut kuda
Swale	*Axonophus compressus*	Cow grass; Rumput pahit
	Brachiaria sp.	Tanner grass; Rumput malela
	Cynodon dactylon	Bermudagrass; Rumput bermuda
	Panicum virgatum	Switch grass, Tall panic grass, Water panicum, Thatchgrass
	Vetiveria zizaniodes	Vertiver grass; Rumput wangi

Table AX1.12: Recommended Main Ground Cover for Channel Slope Erosion/Treatment

Botanical Name/Synthetic Material	Common Name
Axonopus compressus 'mutiara'	Pearl grass; Rumput mutiara
Axonopus affinis	Narrowleaf carpet grass; Rumput karpet
Brachiaria sp.	Tanner grass; Rumput malela
Cynodon dactylon	Bermuda grass; Rumput bermuda
Digitaria didactylia	Serangoon grass
Panicum virgatum	Switch grass, Tall panic grass, Water panicum, Thatchgrass
Stenotaphrum secundatum	St. Augustine
Stenotaphrum secundatum variegatum	Variegated St. Augustine grass
Vetiveria zizaniodes	Vertiver grass; Rumput wangi
Zoysia sp.	Emerald Grass

AX1.2.4 Swales, Grassed Channel and Vegetated Filter Areas

Flows are reduced by roughness of grasses and water quality is further improved. These grasses are sod farming and withstand frequent inundation, and are thus ideal for the grassed channel, vegetated filter area, and swale environments (Tables AX1.11 and AX1.12).

a) Topsoil

Topsoil is important in preserving and protecting the ground surface from erosion and is able to absorb stormwater runoff more efficiently. Removal of topsoil will deplete the land fertility for planting and also cause erosion and siltation of the channels.

Compacted soils will need to be tilled before grass seeding or planting. At least 100 mm of the following recommended topsoil mix is required: 50-80% sandy loam, 10-20% clay and 10-20% composite organic matter.

b) Seeding Criteria

The planting criteria for swale, grassed channel, and vegetated filter area are as follows:

- Ground cover should be tolerant to frequent inundation and erosion. Where possible one or more of the grasses should be in the seed mixes;
- grass should be able to survive flood, drought, grazing animals and other forces of nature;
- Cheap and easy to establish and maintain;
- Has deep penetration root system, which can grow up to 3 metres in length. Long roots are very useful in improving stability of earth slopes as they provide reinforcement by holding the soil particles together and more importantly, remove subsoil mixture, which is detrimental to slope stability; and
- Able to survive on many soil types almost regardless of fertility, alkalinity or salinity.

c) Planting Plan

The quality of the grass seed used is important. Fresh and recleaned grass seeds of the latest crop available shall be used. General guidelines for establishing an effective grass lining are as follows:

- Prepare a good, firm seed bed;
- Use a crop residue or a mulch to protect the grass during establishment;
- Allow 3 months for grass to show an adequate stand;
- Select a simple grass mixture that best fits the conditions of the swale;
- Use good quality seed from grass origins and strains known to be adaptable to the site;
- Plant at the best date for the selected grass species;
- Use planting equipment and methods that give uniform distributions and proper placement of seed;
- Water grass as required to supplement rainfall until it is established;
- Fertilise according to the needs of the grass and the soils as shown by soil tests;
- Overseed or repair bare spots with sod chunks or mulch as necessary;
- Avoid driving vehicles on the swale or damaging the sod with tillage; and
- Mow when grass can make good regrowth.

Table AX1.13: Recommended Palm and Shrub Species for River Corridor

Botanical Name	Common name	Planting Zones					Plant Tolerances 1= tolerant 2= tolerance some 3= intolerant				
		Margin	Lower Bank	Upper Bank	Terrace Face	Upper Terrace	Sun	Shade	Wet	Dry	Wind
Palms											
Archontophoenix alexandrae	Alexandra palm, Alexander palm, King palm; Palma Iskandar				•	•	2	1	1	2	1
Arenga pinnata	Sugar palm; Kabung				•	•	2	2	1	2	1
Calamus sp.	Rattan; Rotan				•	•	3	1	2	2	3
Carpentaria acuminata	Carpentaria palm				•	•	1	1	1	3	2
Cyrtostachys renda	Malaya sealing wax palm; Pinang merah				•	•	2	1	1	2	1
Dendrocalamus giganteus	Giant bamboo; Buluh betong			•	•	•	1	2	1	2	1
Eugeissona tristis	Wild bornean sago; Bertam				•	•	3	1	2	2	2
Licuala spinosa	Mangrove fan palm, Spiny licuala, Good luck palm; Palas duri				•	•	1	1	1	3	2
Metroxylon sagu	Sago palm; Sagu				•	•	2	2	1	3	2
Oncosperma horridum	Thorny palm, Mountain nibung palm; Bayas				•	•	2	2	1	2	1
Oncosperma tigillarium	Nibung palm; Nibung				•	•	2	2	1	2	1
Phyllostachys sulphurea	Sulphur bamboo; Buluh kuning			•	•	•	1	2	1	2	1
Pinanga malaiana	Malaya sealing wax palm; Legong/Pinang hutan				•	•	3	1	1	3	3
Ptychosperma macarthurii	Macarthur palm; Cluster palm, Hurricane palm; Palma Macarthur				•	•	2	2	1	2	2
Shrubs											
Alpinia purpurata	Red ginger; Alpinia merah				•	•	2	1	2	3	2
Ardisisa crenata	Hen's eyes; Mata ayam			•	•	•	2	2	2	2	2
Asplenium nidus	Bird's nest fern; Paku langsuir, Daun semun		•	•	•	•	3	1	1	3	2
Cassia alata	Wild senna, Ringworm bush; Gelenggang				•	•	2	2	1	2	3
Cyperus sp.	Nutsedge; Rusiga	•	•	•	•	•	1	2	1	2	1
Gleichenia linearis	Tangle fern; Paku resam		•	•	•	•	2	1	1	2	1
Heliconia rostrata	Hanging lobster claw; Heliconia sepit ketam				•	•	3	1	2	3	3
Monochoria hastata	Monocharia, Arrowleaf pondweed; Keladi agas	•					1	2	1	3	2
Nephrolepis exaltata	Sword fern; Paku		•	•	•	•	3	1	1	3	2
Pandanus malayanus	Screw pine; Pandan		•	•	•	•	2	1	1	2	2
Phyllagathis rotundifolia	Solomon's sole; Akar serau malam, Tapak Sulaiman, Tapak gajah, Seri bulan			•	•	•	3	1	1	2	2
Platycerium coronarium	Stagshot, Stag's horn fern, Tanduk rusa		•	•	•	•	3	1	1	3	3
Sagittaria sagitaefolia	Arrowhead, Verigated lesser arrowhead; Bunga sagitaria kuning	•					2	2	1	3	2
Syngonium podophyllum	White butterfly, Singonium		•	•	•	•	3	1	1	3	2
Tacca chintrieri	Bat head lily, Bat Flower, Devil Flower; Misai baung		•	•	•	•	3	1	1	3	3
Typha latifolia	Bulrush, Broadleaf cattail; Banat	•					1	2	1	3	2

Table AX1.14: Recommended Tree Species for River Corridor

		Planting Zones					Plant Tolerances				
							1= tolerant 2= tolerate some 3 =intolerant				
Botanical Name	Common Name	Margin	Lower Bank	Upper Bank	Terrace Face	Upper Terrace	Sun	Shade	Wet	Dry	Wind
Trees											
Alstonia spathulata	Marsh pulai, Siamese balsa, Hard milkwood; Pulai paya		•	•	•	•	1	2	1	2	2
Artocarpus peduncularis	Terap tree; Terap		•	•	•	•	1	1	1	2	1
Calophyllum sp.	Punna; Bintangor			•	•	•	1	2	1	2	1
Cananga odorata	Cananga tree, Dwarf Ylang Ylang; Kenanga			•	•	•	1	1	1	2	1
Daemonorops angustiloba	Water rattan; Rotan getah		•	•	•	•	3	1	1	3	3
Derris heptaphylla	Tuba			•	•	•	1	2	1	2	1
Eugenia densiflora	Kelat jambu air			•	•	•	1	2	1	2	3
Eugenia spicata	Firefly bush, Spicate eugenia; Kelat nenasi		•	•	•	•	1	2	1	2	3
Ficus benjamina	Weeping fig, Benjamin fig; Ara beringin, Ara waringin		•	•	•	•	1	1	1	1	1
Ficus globosa	Bling fig; Ara kelalawar, Ara paya		•	•	•	•	1	1	1	1	1
Ficus hispida	Hairy fig; Ara kelumpang		•	•	•	•	2	2	1	2	2
Fragrae fragrans	Ironwood; Tembusu		•	•	•	•	1	1	1	2	1
Gluta velutina	Water rengas; Rengas air			•	•	•	1	1	1	2	1
Intsia palembanica	Marabaw Tree of Malacca, Malacca teak; Merbau		•	•	•	•	1	1	1	2	1
Koompasia malaccensis	Kempas tree; Kempas				•	•	1	1	1	2	1
Lagerstroemia flos-reginae	Queens crape myrtle, Pride of India, Rose of India; Bungor			•	•	•	1	2	1	2	1
Licuala spinosa	Mangrove fan palm, Spiny licuala, Good luck palm; Palas duri			•	•	•	3	1	1	3	3
Macaranga sp.	Mahang tree; Mahang				•	•	1	1	1	2	2
Mallotus sp.	Balik angin				•	•	1	1	1	1	1
Melaleuca leucadendron	Cajaput tree, Paper-bark tree, Weeping teat tree; Gelam				•	•	1	2	1	2	1
Milletia hemsleyana	Stem bark; Jada		•		•	•	1	1	2	2	1
Parkia javanica	Sataw; Petai kerayung		•		•	•	1	1	1	2	1
Polyalthia sclerophylla	Mast tree; Mempisang, Jangkang			•	•	•	1	1	1	2	2
Pometia pinnata	Fijian longan, Island lychee; Kasai		•	•	•	•	1	1	1	1	1
Pterocarpus indicus	Malay paduak, New Guinea rosewood; Sena, Anggsana			•	•	•	1	1	1	1	2
Pterolobium javanicum	Bullock's eye; Mata lembu		•		•	•	1	1	1	2	2
Saraca thaipingensis	Yellow ashoka, Yellow saraca; Saraka kuning, Pokok gapis			•	•	•	3	1	1	3	2
Sonneratia caseolaris	Apple mangrove; Perepat			•	•	•	3	1	1	3	2

AX1.2.5 Natural Channel, River Corridor and Riparian Zone

Riverside or riparian vegetation helps to protect the riverbank, provide breeding ground for aquatic life, temporarily holding overflow, as well as trap sediments and some pollutants (Tables AX1.13 and AX1.14). Identification of suitable plant species shall be based on the hydrologic zones or sections of the channel.

a) Planting Guide

- Determine the profile of the river to identify the different characteristics or vegetation zones;
- Prepare a planting plan with composition of the plant species for the zones;
- Space plants according to the zone they belong in, and their mature size. An approximation of one plant per square metre will be generally sufficient. Rushes, small sedges and ferns can be planted up to three plants per square metre;
- Select indigenous and hardy species that are adaptable and tolerant to site and soil conditions of floodplains and riparian zones;
- Order plants well in advance of planting. Select a nursery specialising in native plants;
- Plant appropriate species right down to the water's edge or margin;
- Prepare the site well in advance of planting;
- Remove invasive weeds such as *Imprata cylindrica*, *Euchornia sp*, *Mimusa pudica* and *Mimusa* i*ndica* (Semalu);
- Clear all vegetation for about 1 metre diameter around each planting position;
- Set out plants in their correct zones. Plants should be spaced out according to how large they will eventually grow into;
- Before planting, prune off entangled roots. Set the plants into a bed of soft, worked soil at the bottom of the hole, and repack crumbed soil around the root mass tightly to prevent air gaps;
- Ensure plants within the channel are well planted and compacted around the base;
- On wet sites, plant in a shallower hole so that the top of the root mass and associated soil are at ground level or even slightly mounded above it in permanently saturated condition;
- For poor soil, slow-release fertiliser should be applied to each plant. Short-term fertiliser should be applied to the ground after planting and before mulching; and
- Mulch should not be applied on wet sites or anywhere near the water flow, as mulch is likely to be washed away and may caused stream blockages.

b) Selecting Plant Species

Due to the different conditions for establishment and growth of plants with soggy and inundated soil, riverside plants can be categorised into different vegetated zones. These zones are based on slope condition and distance from the water edge. The species commonly found along the rivers are recommended for planting in restoring the river and its corridor into its natural forms and function creating the riverine landscape and parkland.

AX1.3 OTHER CONSIDERATIONS IN PLANTING

AX1.3.1 Wild Collection

Wild plants are important as they are more adapted to the local environmental conditions (Table AX1.15). Wild plants have acclimated to local soils, typical hydrologic region and weather. Wild plants will initiate new growth more quickly and develop more robust growth habits at earlier stage than plants secured from nurseries as seed or potted plants.

Table AX1.15: Recommended Wild Plant Species

Botanical Name	Common Name
Alstonia spathulata	Marsh pulai, Siamese balsa, Hard milkwood; Pulai paya
Bambusa vulgaris	Common bamboo, Giant yellow clumping bamboo, Feathery Bamboo; Buluh minyak, Buluh gading, Buluh aur
Lepironia articulata	Tube sedge, Grey sedge, Blue rush, Twigrush; Purun, Kercut
Litsea teysmanni	Medang kelor
Ludwigia adscendens	Floating Malayan willow herb; Tinggir bangau
Monchoria hastata	Hastate-leafed pondweed; Keladi agas
Pandanus immersus	Swam/riverrine pandanus; Pandan rasau
Phragmites karka	Common reed, Tall reed, Tropical reed; Rumput gedabong
Phylidrum lanuginosum	Fan grass; Rumput kipas
Pometia pinnata	Fijian longan, Island lychee; Kasai
Oncosperma tigilarium	Nibung palm; Nibung
Scirpus grassus	Greater club rush; Rumput menderong
Turnera ulmifolia	Holy rose, Yellow buttercup, Cuban buttercup; Turnera, Lidah kucing, Bunga pukul delapan
Typha angustifolia	Narrow cattail, Lesser bulrush, Lesser reedmace; Banat

AX1.3.2 Habitat Creation

Riparian vegetation performs a long list of important functions in the creation and maintenance of fish and wildlife habitat. Those functions can be summarised as follows:

- Riparian vegetation moderates water temperature, making the river habitable for fish and other aquatic life;
- Tree roots, shrub species and other growth bind the stream bank soil and provide resistance to erosive forces of the water (Tables AX1.16 and AX1.17). This produces deeper channels with banks that are undercut but held together with exposed root systems. These undercut banks complete with overhang vegetation, provide important escape cover for fish;
- Most of the river/stream's biological energy and the base of the food chain for stream life come from the leaves, fruits, seeds, cones and other parts of the plants; and
- Woody debris that falls into the river forms pools for fish, creates habitat by causing backwater pools and provides storage areas for sediment that otherwise might be released into spawning areas.

Planting for ponds, wetlands and large channels such as river shall incorporate opportunities for creation of wildlife habitat (Figure AX1.3).

Malayan Box Turtle Small Clawed Otter Monkey Common Myna Little Egret

Figure AX1.3: Local Wildlife Attracted to the River Ecosystem (Wildlife at USM Wetland)

Table AX1.16: Recommended Trees and Palm Species for Wildlife Habitat

Botanical Name	Common Name
Artocarpus altilis	Breadfruit; Sukun
Ceiba pentandra	Kapok tree, Silk cotton, Java cotton; Kekabu, Kabu-kabu, Kapuk randu
Cyrtostachys lakka	Red sealing wax palm, Dwarf lipstick palm; Pinang merah, Pinang raja
Cordia sebestana 'aurea'	Orange geiger tree
Dillenia indica	Elephant apple; Simpoh India
Diospyros discolor	Butter fruit; Mentega
Eugenia polyantha	Indonesian bay leaf; Kelat
Ficus benjamina	Weeping fig, Benjamin fig; Ara beringin, Ara waringin
Intsia palembanica	Marabaw tree of Malacca, Malacca teak; Sepetir
Livistona chinensis	Fountain palm; Serdang
Melia excelsa	Marrango tree, Philipine neem tree; Sentang
Mimusop elengi	Bullet-wood Tree; Bunga tanjung
Muntigia calabura	Cherry tree, Strawberry tree, Cotton candy berry; Kerukup Siam, Ceri kampung
Musa sp.	Wild banana; Pisang hutan
Pitcellobium dulce	Madras thorn, Manila tamarind, Monkeypod; Asam Belanda
Pometia pinnata	Fijian longan, Island lychee; Kasai
Ptychosperma macarthurii	Macarthur palm; Cluster palm, Hurricane palm; Palma Macarthur
Samanea saman	Rain tree, Cow tamarind; Hujan-hujan, Pukul lima jari
Sapium indicum	Tallow tree; Gurah
Sterculia foetida	Hazel sterculia, Great sterculia Skunk flower; Kelumpang
Sterculia nobilis	Chinese chestnut
Terminalia catappa	Tropical almond, Sea almond; Ketapang

Table AX1.17: Recommended Shrub Species for Wildlife Habitat

Botanical Name	Common Name
Ardisia crispa	Hen's eyes, Coral berry; Mata ayam, Mata Pelanduk, Akar bebuluh
Asplenium nidus	Bird's nest fern; Paku langsuir, Daun semun
Asystasia gangetica	Creeping foxglove; Rumput itik
Carissa grandiflora	Common carissa, Natal palm, Boxwood beauty
Cassia alata	Wild senna, Ringworm bush; Gelenggang
Gesneriaceae sp.	Cloudforest flower; Letup-letup
Graminae sp.	Darnel; Rumput tebu
Hanguana malayana	Common hanguana, Common susum; Bakong, Bakong rimba
Ixora javanica	Jungle flame, Jungle geranium; Siantan
Melastoma malabathricum	Malabar melastome, Straits Rhododendron; Senduduk, Keduduk, Senggani
Nymphae sp.	Water lily; Teratai
Phragmites karka	Common reed, Tall reed, Tropical reed; Rumput gedabong
Placerium coronarium	Stagshot, Stag's horn fern; Tanduk rusa
Premna obtusifolia	Premna; Bebuta
Stachytapheta jamaicensis	Spotted basil, Blue porterweed; Selasih dandi, Pokok kecut kuda
Tacca chantrieri	Bat head lily; Janggut baung
Turnera ulmifolia	Holy rose, Yellow buttercup, Cuban buttercup; Turnera, Lidah kucing, Bunga pukul delapan
Typha latifolia	Bulrush, Broadleaf cattail; Banat

AX1.4 GENERAL PLANTING AND CONSTRUCTION METHOD

General planting methods apply to all plants discussed earlier to be used in various stormwater facilities such as ponds, wetlands, swales, engineered channels and river corridors.

- Trees or shrubs known to have long taproots should not be within the vicinity of earth dam, weir or subsurface drainage facilities;
- Trees or shrubs shall be away from the maintenance width requirements and in accordance with reserve width as specify in design criteria;
- Tree and shrubs should be at least 5 m away from perforated pipes;
- Trees and shrubs should be at least 7.5 m away from a riser structure;
- Provide 4.5 m clearance from a non-clogging, low flow orifice;
- Herbaceous embankment plantings should be limited to 30 cm in height. This is to allow visibility for the inspector who is looking for burrowing rodents that may compromise the integrity of the embankment;
- Provide slope stabilisation methods for slopes steeper than 2:1 such as erosion control mats. Also, use seed mixes with quick germination rates in this area;
- Augment temporary seeding measures with container crowns or root mats for more permanent plants;
- Use erosion control mats and fabrics in channels that are subject to frequent washouts;
- Stabilise all emergency spillways with plants that can withstand strong flows;
- Select plants with fibrous root system and not taproot root system to avoid damage to underground components of certain stormwater facilities such as underdrains;
- Sod channel areas that are not stabilised by erosion control mats;
- Divert flows temporarily from seeded areas until stabilised;
- Check water tolerances of existing plant materials prior to inundation of area;
- Stabilise aquatic and safety benches with emergent wetland plants and wet seed mixes;
- Do not block maintenance access to structures with trees or shrubs;
- Avoid plantings that will require routine or intensive chemical application (i.e. turf area) ;
- Have soil tested to determine if there is a need for amendments;
- Decrease the areas where turf is use. Use low maintenance ground cover to absorb run-off;
- Plant stream and water buffers with trees, shrubs, ornamental grasses and herbaceous materials where possible, to stabilise banks and provide shade;
- Maintain and frame desirable views. Be careful not to block views at entrances, exits, or difficult road curves. Use plants to screen off unattractive views of the site or facility. Aesthetics and visual characteristics should be a prime consideration;
- Use plants to prohibit pedestrian access to pools or slopes that may be unsafe;
- The designer should carefully consider the long-term vegetation management strategy for the BMP, keeping in mind the 'maintenance legacy for the future owners. Keep maintenance areas and access free of vegetation to allow vehicle clearance. Provide a planting surface that can withstand the compaction of vehicles using maintenance access roads. Make sure the facility maintenance agreement includes requirements for landscaping or vegetation maintenance;
- If a BMP is likely to receive excessive amounts of deicing salt, salt tolerant plants should be used;
- Provide signage at areas of stormwater facilities to help educate the public;

- Avoid the overuse of any plant species; and
- Preserve existing natural vegetation when possible.

It is necessary to test the soil to be used as planting medium in order to determine the following:

- pH, whether acid, neutral or alkaline;
- Major soil nutrients; nitrogen, phosphorus, potassium; and
- Minerals; such as chelated iron, lime.

AX1.5 POST PLANTING MANAGEMENT

Post planting management covers proper horticultural practices and maintenance to encourage the establishment of newly planted trees.

- Newly installed plant requires water in order to recover from the shock of being transplanted. Some source of water is to be provided especially during dry periods. This will reduce plant loss and provide the new plant with chance to establish root growth;
- Weeding around plants is essential to avoid competition and stress. This should be carried out after 2 months of planting or on a monthly basis as required;
- At the water margin, careful weed control is needed on an on-going basis until the area is self-maintaining, or until the plantings have overtopped the grass;
- Clearing of weeds and pruning of trees after 4 and 12 months of planting are required;
- After 6 months of planting, pruning and trimming of unwanted shoots should be carried out. This will encourage growth and development of quality plants in term of height. Weeding shall be required too;
- Familiarity with the common problems and indications of post planting stress could aid in recognising stress early and minimising the potential damage;
- Stressed plants are at higher risks to attract pests and diseases;
- Stress can be minimised and eliminated by judicious watering;
- Excess watering especially from irrigation systems causes anaerobic (low oxygen) soils, killing the small absorbing roots. With unhealthy roots the symptom can be similar to drought stress, with dull or drooping leaves and branch tips, scorched leaves margins, and eventual dieback;
- Regular check on the plant's health for several years, normally up to 4 years after establishment;
- Insect and disease control may periodically be required; and
- Monitor the growth of the riverine vegetation and enjoy the sight as they thrive and attract wildlife and become self maintaining.

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APPENDIX AX1.A Recommended Plants Species for Zone 1 (Deep Water Pool)

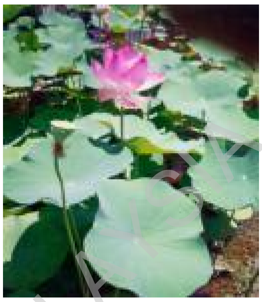
Cyperus compactus

Cyperus digitatus

Cyperus halpan

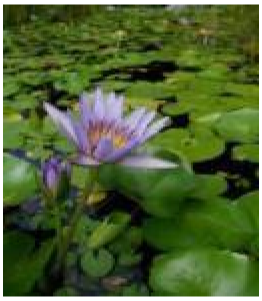
Lepironia articulata

Nasturtium sp.



Nelumbo nucifera

Nymphaea lotus dentata



Nymphaea nouchali

Nymphaea rubra

Nymphaea tashkent

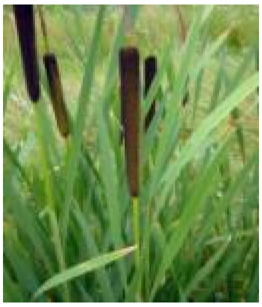
Phyragmites karka

Phylidrum lanuginosum

Rynchospora corymbosa

Scirpus grassus

Scirpus juncoides



Typha latifolia

Scleria sumatrensis

Victoria sp.

APPENDIX AX1.B Recommended Plants Species for Zone 2 (The Shallow Water Bench)


Cleome spinosa


Eleocharis vaiegata


Eriocalon longifolium


Fimbristylis glabulosa


Fuirena umbellata


Hanguana malayana


Ludwigia adscendens


Ludwigia actovalvis


Monocharia hastata


Pandanus immersus


Pandanus sp.


Rynchospora corymbosa


Sagittaria sagietafolia


Scleria sumatrensis


Stachhytapheta jamaicensis


Vanda hookeriana


Zingiberaceae sp.

APPENDIX AX1.C Recommended Plants Species for Zone 3 (Shoreline Fringe)

Alstonia spathulata

Artocarpus altilis

Barringtonia asiatica

Caryota mitis

Cystostachys lakka

Dillenia suffructicosa

Melaleuca leucadendron

Pometia pinnata

Saraca thaipingensis

Shorea platycarpa

Sindora coriaceae

Spathodea campanulata

APPENDIX AX1.D Recommended Plants Species for Zone 4 (Riparian Fringe, Periodically Inundated)


Arachis pintoi


Asystasia gangentica


Bambusa vulgaris


Caryoto no


Cocoloba uvifera


Cratoxylon arborescens


Dillenia suffruticosa


Elaeocarpus


Ficus benjamina


Ficus globosa


Johannesteijsmannia altifrons

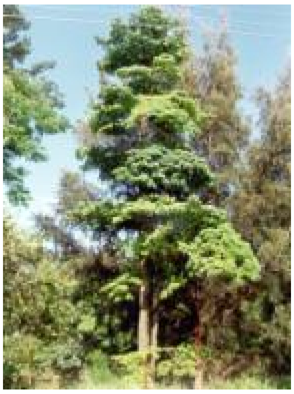

Koompassia malaccensis


Licuala spinosa


Melia excelsa


Nephrolepis sp.

APPENDIX AX1.E Recommended Plants Species for Zone 5 (Floodplain Terrace, Infrequently Inundated)

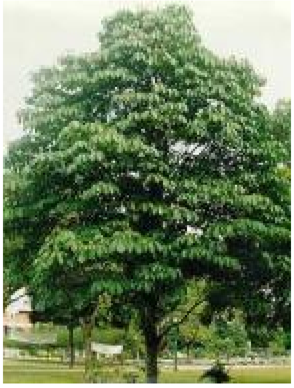


Alstonia angustiloba

Archontophoenix alexandrae

Costus speciosus

Dendrocalamus giganteus



Dyera costulata

Fragrae fragrans

Heliconia psitacorrum 'yellow'

Legerstroemia flos reginea

Melastoma malabathricum

Messua ferrea

Mussaenda erythrophylla

Oncosperma horridum

Oncosperma tigillarium

Pandanus pigneus

Pisonia alba

Tacca chantrieri

APPENDIX AX1.F Recommended Plants Species for Zone 6 (Upland Slopes, Seldom or Never Inundated)


Bauhinia blakeana


Cananga odorata


Canarium vulgaris


Cassia fitsula


Cicca acida


Cinnamomum iners


Dryobalanops aromatic


Eucalyptus deglupta


Flacourtia inermis


Hibiscus mutabilis


Livistona rotundifolia


Melia excelsa

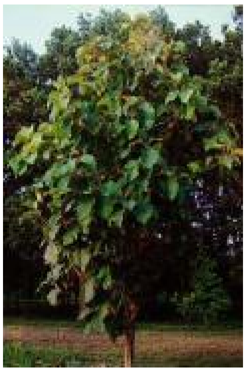

Milletia atropurpurea


Peltophorum pterocarpum


Pritchardia pacifica


Rhapis excelsa


Roystonea regia


Tectona grandis


Zizyphus mauritinia

APPENDIX AX1.G Recommended Grass Species for Grassed Channel, Vegetated Filter Strips and Swales


Axonopus compressus


Axonopus affinis


Digitaria didactylia


Panicum virgatum


Brachiaria sp.


Cynodon dactylon


Stenotaphrum secundatum


Stenotaphrum secundatum variegatum


Zoysia sp.


Vetiveria zizaniodes


Croton hirtus


Asystasia gangetica


Arachis pintoi


Chloris barbata


Eupatorium odoratum

APPENDIX AX1.H Recommended Plants Species for River Margin


Scirpus grossus


Scirpus juncoides


Cyperus compactus


Cyperus digitatus


Pandanus sp.

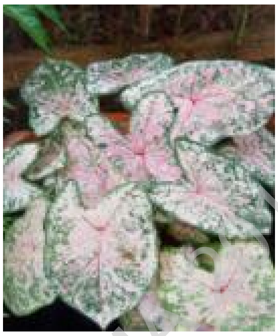

Zingiberaceace sp.


Typha latifolia


Fimbristylis glabulosa


Phragmites karka


Vanda hookeriana


Caladium hortulanum


Dieffenbachia maculata


Cleome spinosa


Ludwigia octovalvis


Stachytarpheta jamaicensis


Monochoria hastata


Limnocharis flava


Ipomea aquatica


Ludwigia adscendens


Hanguana malayana

APPENDIX AX1.I Recommended Plants Species for Stream Lower Bank


Oncosperma horridum


Cyrtostachys lakka


Pometia pinnata


Ficus rocemosa


Anglaia odoratissima


Sacara thaipingensis


Bambusa vulgaris


Milletia hemsleyana


Spondias pinnata


Alstonia spathulata


Pandanus immersus


Pentaspadon velutinum


Licuala spinosa


Heliconia sp.


Baekia fructescens

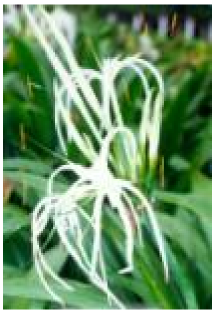

Cassia alata


Pandanus sp.


Alocasia acrorrhiza


Piper sarmentesum


Ludwigia adscendens


Hymenocallis specisa


Nephrolepis sp.


Quisqalis indica


Arundina graminifolia


Hydrocutycle asitica

APPENDIX AX1.J Recommended Plants Species for Stream Upper Bank


Sterculia foetida


Gironniera rarvifolia


Pellacalyx axillaris


Cocoloba uvifera


Sterculia sp.

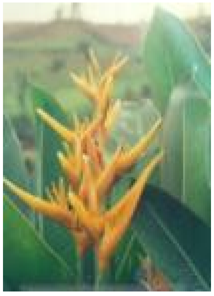

Lagerstroemia speciosa


Bauhinia integrifolia


Bambusa sp.


Ptychosperma macarthurii


Musa velatino


Heliconia psittacorum


Piper sarmentosum


Crinum asiaticum


Gesneriaceae sp.


Sindora coriacea


Parkia javanica


Eugenia densiflora


Dillenia sp.


Croton sp.


Asystasia gangetica

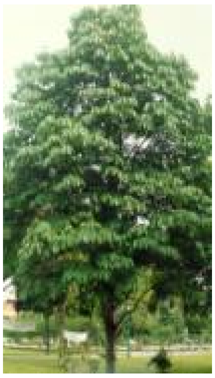

Mussaenda erythophylla


Costus speciosus


Vinca rosea

APPENDIX AX1.K Recommended Plants Species for Stream Terrace Face


Pterocymbium javanicum


Dyera costulata


Hopea odorata


Cassia spectabilis


Ficus benjamina


Lagerstroemia flos-reginae


Spondias pinnata


Swietenia mahagoni


Artocarpus altilis


Pisonia alba


Eugenia polyantha


Heliconia indiva


Gigantochloa wrayi


Dendrocalamus giganteus


Phyllostachys aures


Arenga pinnata


Ardisia crispa


Hemigraphis colorata


Ervatamia corymbosa


Nicolaia clatior


Pandanus sp. 'green dwarf'


Daemonocarpus angustifolia


Turnera ulmifolia

APPENDIX AX1.L Recommended Plants Species for Stream Upper Terrace


Agathis dammara


Tabebuia rosea


Pisonia alba


Cinnamomum iners


Filicium decipiens


Amherstia nobilis

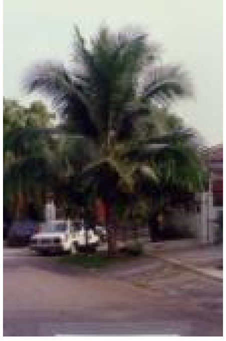

Mimusops elengi


Cassia fitsula


Erythrina glauca


Delonix regia


Flacourtia inermis

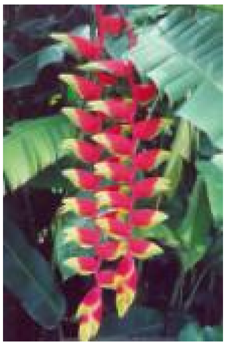

Cocos nucifera


Livistona


Pritchardia


Archontophoenix


Samanea saman


Heliconia rostrata


Canna generallis


Zephyranthes sp.


Furaraea giganteca


Muntingia calabura


Nerium oleander


Heliconia sp.


Croton sp.

APPENDIX AX1.M Recommended Plants Species for Ecological Plants/Fruits Bearing Trees


Artocarpus altilis

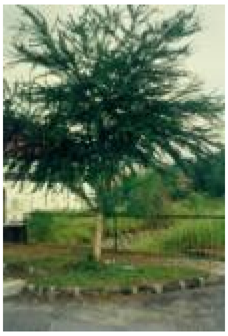

Terminalia catappa


Ficus benjamina


Pometia pinnata


Diospyros discolor


Mimosop elengi


Pitchelobium dulce


Spondias pinnata


Sterculia foetida

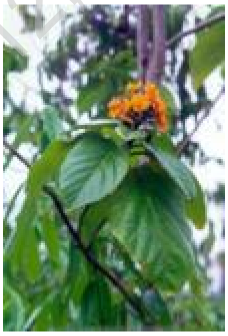

Melia excelsa


Ficus sp.


Eugenia polyantha


Sterculia nobilis


Sapium indicum


Cordia sebestena aurea


Musa sumatrana


Premna obtusifolia


Carissa grandiflora


Ptychosperma macarthurii

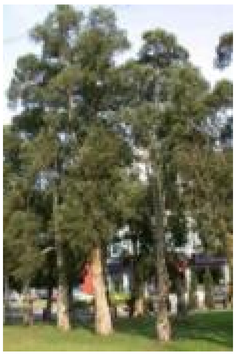

Cyrtostachys lakka


Livistona chinensis


Ardisia crispa


Muntingia calabura

APPENDIX AX1.N Recommended Plants Species for Wild Life Attraction/Breeding Habitat Creation


Melaleuca leucadendron


Cassia alata

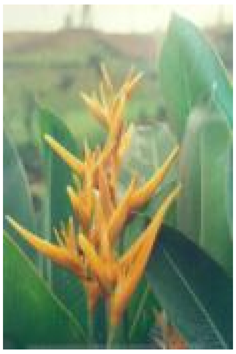

Samanea saman


Intsia palembanica


Stachytarpheta jamaicensis


Gesneriaceae sp.


Graminae sp.


Heliconia psittacorum
'hybrid'


Nymphaea sp.

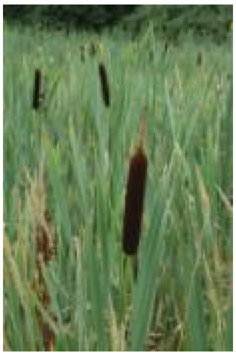

Setaria pumila


Tacca chantrieri


Asystasia gangetica


Hanguana malayana


Fuirena umbellata


Typha latifolia


Ixora javanica


Asplenium nidus


Phragmites karka

MAINTENANCE

2 ANNEX

ANNEX 2 MAINTENANCE

AX2.1 INTRODUCTION

This Section, Annex 2, provides a general maintenance procedures/requirements for each of constructed stormwater infrastructure covered in this design workbook; quantity control, quality treatment BMPs and conveyance facilities. The asset owner of a stormwater drainage system is responsible for the lifecycle maintenance of the built system.

A maintenance activity should be prepared and included in the detailed design submission. It is a set of operating instructions for future property owners and/or occupiers. It should be clearly and simply set out and include the necessary information.

AX2.2 ON-SITE DETENTION

AX2.2.1 General

On-site detention (OSD) systems are intended to regulate flows over the entire life of the facilities. This cannot be achieved without some regular periodical maintenance to ensure they are kept in good working order and operate as designed. The task is to optimise the frequency of maintenance and make the job as simple as possible.

The majority of OSD systems, particularly those where a large proportion of the storage is located above-ground, will be able to be maintained by property owners, residents, or handymen. Larger below-ground systems, particularly those with limited access and/or substantial depth, may require the owner to engage commercial cleaning companies with specialised equipment.

The type of routine maintenance necessary to keep OSD facilities in good working order must be clearly and simply specified. Some of the issues that need to be addressed are:

- where the storages are located
- which parts of the system need to be accessed for cleaning and how access is obtained
- a description of any equipment needed (such as key and lifting devices) and where they can be obtained
- the location of screens and how they can be removed for cleaning

AX2.2.2 Maintenance Scheduling

The owner should be provided with advice on how frequently the system needs to be inspected and approximately how often it will require cleaning. The frequencies of both inspections and maintenance will be highly dependent on the nature of the development, location of the storage, and the occurrence of major storms. Suggested frequencies are provided in Table AX2.1.When inspecting OSD facilities, if any of following items are noticed, cleaning and/or repair should be undertaken:

- clogged outlet and obstructed inlets
- excessive deposits
- corrosion of metal parts
- deterioration of concrete
- any other damage or visible problems

Table AX2.1: Suggested Frequencies for Inspection and Maintenance

Premise	Frequency
Residential	Inspect system every 3 months and after heavy rainfall. Clean system as required, generally at least every 6 months
Commercial and Industrial	Inspect system every 2 months and after heavy rainfall Clean system as required, generally at least every 4 months

AX2.3 RAINWATER HARVESTING

Maintenance is generally involving regular inspection and cleaning of tanks, gutters and downpipes. Maintenance typically consists of the removal of dirt, leaves and other accumulated material. Cleaning should take place bi annually or before the start of the major rainfall season. Filters in the inlet should be inspected every about three (3) months. Cracks in storage tanks can create major problems and should be repaired immediately. Below are other maintenances in rainwater harvesting system.

AX2.3.1 Gutters

Every month, check for rusting or leaks in gutters. Use sandpaper and aluminum gloss paint to fix the rusting areas, and use silicon glue to repair leaks in the gutters. Make sure the gutters are secured to the wooden flashings and that the wood has not begun to rot (Figure AX2.1).

Figure AX2.1: Gutter Maintenance

If gutter screens installed, make sure the gutter screens are not damaged or clogged with leaves or dirt (Figure AX2.2). If the gutter screens are clogged, water will not flow into the gutter. Sweep the screens to clean them out. If the screens have been damaged, replace them. The screens are made of wire fly mesh that is cut into pieces and are long enough to cover the length of the gutter.

Figure AX2.2: Partially Clogged Gutter Screen

AX2.3.2 First Flush

After every rainstorm, the first flush must be cleaned out. Unscrew the end cap slowly and drain water from the pipes. Remove any debris that has collected at the bottom of the pipe. Screw the end cap back on, but not all the way so as to allow for the pipes to drain slowly during a storm. The slow drain prevents a build-up of pressure in the pipes.

AX2.3.3 Downpipe

Downpipe and outlets located in landscape areas should be inspected every six months to ensure that splash pad placement is correct and that there is positive drainage away from the outlet and adjacent buildings maintain a minimum of 2 percent slope for the first 1.5 m. Check for animal inhabitations and clogs. Check landscape growth every two weeks during growing periods to protect from overgrowth, which could obstruct positive drainage. Flush gutters and downpipe once all debris is removed to wash away any remaining dirt or materials. Downpipe shall be checked if any leaking then fix it (Figure AX2.3).

Figure AX2.3: Leaking Downpipe

AX2.3.4 Tank

Every six (6) month, clean and disinfect tank to prevent slime, algae, bacterial growth, and the build-up of sediments. This will allow the tank to last longer and for the water to remain clean. When cleaning, do not enter the tank. To clean the tank, first drain all the water from the tank and close the tap. It is preferable to wait until the tank is almost empty (at the end of the dry season), to clean the tank. Wash and remove dirt from inside surfaces of the tank with water. Drain the wash water and sediment from the bottom of the tank by opening the spigot.

Use chlorination to disinfect the inside surfaces of the tank. Once the water inside the tank is chlorinated, let the chlorine solution sit in the tank for 3-5 hours, and then drain the tank completely. Now, fresh water can be added to the tank. Run the water from the spigot until there is no smell of chlorine, and then continue normal usage of the tank.

AX2.4 DETENTION POND

A detention pond, in common with other methods of stormwater quantity control, will prove effective only if it is maintained regularly. This requires a sound understanding of operational and maintenance requirements.

AX2.4.1 Consultation

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

AX2.4.2 Planned Maintenance and Inspection

The design storm for most detention ponds is a relatively rare occurrence and the pond and its outlet structures must be kept in good working order in the intervening period so that it performs satisfactorily when required. It is essential, therefore, that all detention ponds are subject to regular inspection and maintenance. In some circumstances, failure to carry out routine maintenance could result in blockage of the primary outlets and premature filling of the pond under normal flow conditions, leaving no storage available for flood control. It is essential that the responsibility for future maintenance should be clearly established and formal arrangements should be drawn up for inspection and maintenance. In addition to basic engineering requirements, the arrangements must cover the amenity of the pond wherever recreational usage of the pond is provided. Inspection and maintenance must be carried out on a regular basis in order to minimise risks. The frequency and requirements for routine inspection will depend on the type and size of the pond, the local circumstances, and the type and complexity of the primary outlets. The frequency of inspections and maintenance visits may vary widely and should be reviewed continually in the light of any problems experienced on site and any long-term changes in maintenance requirements. A formal inspection and maintenance programme should be drawn up, staff allocated, and the duties and responsibilities confirmed in writing.

AX2.4.3 Grass Maintenance

Where embankments and/or spillways are subject to scour caused by high velocities of flow, regular mowing (at once a month) is required to keep the grass sward in good condition and discourage woody growth (Figure AX2.4). The depth of the grass for dry pond should be maintained 50 mm to keep the pond in better condition. Similar treatment is necessary in areas used for formal recreation.

Maintaining turf quality, where hydraulic protection is to be provided, requires a good supply of nutrients, which may require the use of fertilisers. The frequency of application depends on the quality of the soil. Normal soils may only require fertilising in the first year of growth while poor ones may demand annual treatment for a number of years. Some weeds can break up the turf cover and have serious effects on its ability to withstand erosive forces and must be controlled. Such weeds can be eradicated by using selective weed-killers.

Grass-cutting costs can be kept to a minimum (in areas used for formal recreation or where grass is used for scour protection) by keeping the slopes of embankments and other areas gentle enough for machine mowing. Equipment can operate on slopes of up to 4(H):1(V) but slopes of 6(H):1(V) or flatter are preferred.

Figure AX2.4: Maintaining Grass on Embankment

Engineered waterways upstream and downstream of a detention pond will require regular attention, particularly in urban districts. Banks below flood level should be mown where necessary to promote good grass growth, thereby providing protection against scour.

AX2.4.4 Thrash and Sediment Removal

All screens on primary outlets should be inspected and cleaned on a regular basis, particularly following a storm event. Particular problems can occur in urban areas where rubbish is often deposited in watercourses by residents. Any rubbish, debris, and silt should be removed in order to prevent the possible blockage of screens and primary outlet structures (Figure AX2.5).

Figure AX2.5: Primary Outlets Clogging

Regular removal of any accumulated silt and sediment from a detention pond is essential, particularly where the pond floor is used for recreational purposes. Removal of accumulated debris, trash, paper, etc. should take place every 6 months or so and vegetation growing within the pond should not grow taller than 0.5 m. No standing water should be allowed in the pond beyond a period of 72 hrs after a storm event. If such conditions occur, corrective maintenance should be undertaken (Figure AX2.6).

Figure AX2.6: Sediment Removal Work

AX2.4.5 Structural Repairs and Replacement

Inlet and outlet devices and riser structures have been known to deteriorate with time, and may have to be repaired and replaced (Figure AX2.7). The actual life of a structural component will depend on site specific criteria, such as soil conditions, type of construction, and frequency of operation.

Figure AX2.7: Structure Failure

AX2.5 INFILTRATION FACILITIES

Infiltration facilities, as with all BMPs, must have routine inspection and maintenance designed into the life performance of the facility. The principal maintenance objectives are to prevent clogging and groundwater contamination. Maintenance and inspection plans should be identified prior to establishment. Infiltration facilities and any pre-treatment BMPs should be inspected after large storm events to remove any accumulated debris or material. A more thorough inspection should be conducted annually. A summary of inspection and maintenance activities is provided in Table AX2.2

Depending on the specific system implemented, maintenance should include at least the followings:

- inspect and clean pre-treatment devices biannually (i.e. before and after the wet season) and ideally after major storm events;
- once the infiltration system is operational, inspections should occur after every major storm for the initial few months to ensure proper stabilisation and function. Attention should be paid to how long water remains standing after a storm; standing water within the system for more than 72 hours after a storm is an indication that soil permeability has been over-estimated;
- after the first wet season, infiltration systems should be inspected at least biannually (i.e. before and after the wet season). Important items to check and clean or repair if required include: accumulated sediment, leaves and debris in the pre-treatment device, signs of erosion, clogging of inlet and outlet pipes and surface ponding;
- when ponding occurs, corrective maintenance is required immediately.

Table AX2.2: Typical Maintenance Activities

Activity	Frequency
Ensure the contributing drainage area, facility and inlets are clear of debris Ensure that the contributing area is stabilized Remove sediment and oil/grease from pre-treatment devices, as well as overflow structures Mow grass filter strips as necessary. Remove grass clippings	Monthly
Check observation wells following three days of dry weather. Failure to percolate within this time period indicates clogging Inspect pre-treatment devices and diversion structures for sediment buildup and structural damage Remove trees that start to grow in the vicinity of the trench	Semi-annual
Replace peat gravel/topsoil and top surface filter fabric (when clogged)	As needed
Perform total rehabilitation of the trench to maintain design storage capacity Excavate trench walls to expose clean soil	Upon Failure

Source : US EPA, 1999

In infiltration facilities, clogging occurs most frequently on the surface. Ponded water lasting more than 24 hours usually indicates that the facility is clogged. Grass clippings, leaves and accumulated sediment should be removed routinely from the surface. If clogging appears to be only at the surface, it may be necessary to remove and replace the first layer of filter media and the geotextile filter.

The presence of ponded water inside the trench after an extended period indicates clogging at the base of the trench. Remediation includes removing all of the filter media and geotextile envelope, stripping accumulated sediment from the trench base, scarifying to promote infiltration and replacing new filter media and geotexile. Vegetation can assist in prevention of clogging as the root network breaks up the soil and thereby promotes infiltration.

In the case of infiltration basins, sediment should be removed when it is sufficiently dry so that the sedimentation layer can be readily separated from the basin floor.

Maintenance responsibility for an infiltration facility should be assigned to a responsible jurisdiction or authority through a legally binding and enforceable maintenance agreement completed as a condition of the site plan approval.

AX2.6 BIORETENTION SYSTEM

Routine inspection and attention to maintenance needs are required if bioretention basins are to continue to function correctly. High maintenance levels are required for new systems, but once established and correctly operating maintenance requirements are expected to decline. The property's normal landscaping contractor, when provided with appropriate training, can be expected to successfully maintain an established bioretention basin.

AX2.6.1 Checklist

This checklist notes the key features to be inspected and maintained during operation of a bioretention basin.

- Sediment accumulation at inflow points
- Litter within basin
- Erosion at inlet or other key structures
- Traffic damage
- Dumping (e.g. building waste)
- Vegetation condition (density, weeds etc.)
- Watering of vegetation
- Replanting
- Mowing/ slashing
- Clogging of drainage points (sediment or debris)
- Evidence of ponding
- Damage/ vandalism to structures
- Surface clogging
- Drainage system
- Resetting of system

AX2.6.2 Frequency

Inspections should occur every 1 - 6 months, depending on the size and complexity of the system. The followings are suggested maintenance frequencies:

- Project completion: Water plants daily for at least two weeks

- As Needed
  - Re-mulch void areas
  - Mow turf areas
  - Treat plant diseases
  - Water plants throughout periods of persistent drought.
  - Removal of top two to three inches of discolored planting medium and its replacement with fresh mix, when ponding of water lasts for more than 48 hours.

- Monthly
  - Inspect basin to evaluate condition and problems needing maintenance attention.
  - Remove litter and plant debris.
  - Repair eroded areas.

- Twice per year: Remove and replace dead and diseased plants.

- Once per year
  - Add new mulch.
  - Replace tree stakes and wires if needed.

AX2.7 GROSS POLLUTANT TRAPS

AX2.7.1 General Maintenance

Appropriate maintenance is essential to ensure the long-term pollutant trapping efficiency of all GPTs. It is important in planning a catchment wide strategy for installing GPTs pollution control devices to make adequate provision for maintenance. A written maintenance plan should be prepared.

(a) "Soft" Trash Racks/ Litter Collection Devices (LCDs)

The "soft" trash racks/ LCDs are cleaned by removing each sock in turn, undoing the tie at the base of the sock and dumping the collected material into a truck. The base of the sock is then re-tied and it is slotted back into place. Due to the effectiveness of the socks it has been found that during periods of rainfall the LCDs may need to be cleaned every two to three days.

(b) Modified Trap Gullies

Modified trap gullies are suited to cleaning using eduction. While modified trap gullies can be maintained as part of a regular maintenance program particular attention should be given to assessing the need to clean trap gullies after storm events to ensure that trapped material is not flushed from the trap gully during a subsequent storm event. Experience to date suggests that trap gullies should be maintained on average monthly in urban areas and or more frequently in commercial areas.

(c) 'SBTR' Gross Pollutant Traps

The SBTR-type GPTs can be cleaned out using front-end loaders, backhoes and standard tip trucks. Eductor shall be carried out trucks, if available, can also be used to clean SBTR Type GPTs. A review of the maintenance issues including maintenance equipment, de-watering, access for maintenance equipment and cleaning, inspection program and cleanout frequency, costs and safety.

(d) Proprietary Traps

The appropriate cleaning frequency for proprietary traps should be discussed with the trap suppliers and where possible the experiences of operators should be reviewed to gain an understanding of the plant, manpower requirements and the likely frequency of cleaning required.

AX2.7.2 Maintenance Provisions

Maintenance provisions should be considered at the design phase of the GPTs as follows:

(a) Frequency

GPTs should be inspected monthly, as well as after every major rainfall event, to ascertain whether clean-out is required.

Cleaning frequencies depend on the sediment and litter loading generated in the catchment, generally for SBTR traps are cleaned twice per year, on average. Suggested cleaning frequencies for other types of GPTs are to be determined from operational experience under Malaysian conditions.

More regular cleaning may be required to facilitate ease of removal (i.e. if trapped material becomes compacted and hard to remove; or if specialised equipment is not available), or if litter loads are excessive.

(b) Need for Special Equipment

Designs should be based on cleaning operations being undertaken with plant and equipment including;

- eductor truck;
- backhoe or front-end- loader;

- truck;
- pump and generator; and
- truck mounted crane.

Some designs require more specialised equipment, such as eductor trucks. Such equipment may be introduced into Malaysia during the life of this Manual, subject to discussions and approval by the regulatory authority to suit local conditions and contractor's expertise.

(c) De-watering

GPTs will need to be de-watered from time to time either as part of their general operation or for maintenance purpose. Usually this is done with portable pumps. Water released to stormwater drains or directly to receiving waters should not threaten environmental values and should therefore be consistent with locally applicable water quality objectives.

Prior to pumping out the supernatant water, the SBTR may be dosed with a non-toxic flocculating agent to promote settling of colloidal particles.

The following methods are alternatives that can be used for the disposal of poor quality supernatant water that is retained within the trap:

- *Via Infiltration or Filtration On-site* - The trap may be designed to allow supernatant water to be pumped to a de-watering area on site. The water could either be infiltrated on a grassed area, or filtered through geo-fabric and allowed to drain back to the waterway. An infiltration trench may be included to enhance water polishing and/or permit groundwater recharge.

  Such design shall;

  - have a suitable de-watering and sludge handling or drying area;
  - have stabilised banks to prevent erosion; and
  - not constitute a health hazard.

- *Direct to Sewer* - The SBTR trap may be designed, if necessary, to allow de-watering by pumping supernatant water to a nearby sewer (with the approval of the local sewerage agency). Where there is a sewer line within 200 metres of the facility, the sewer should be extended to provide a manhole with a bolt-down lid adjacent to the SBTR. This will enable the decanted supernatant to be pumped to the manhole and thence to the sewer.

- *Via Tanker* - Where there is no sewer available, provision shall be made for the decanted supernatant to be pumped to tanker for treatment and disposal by a licensed waste management operator.

AX2.8 WATER QUALITY POND AND WETLANDS

AX2.8.1 General Maintenance

As with any constructed facility, ponds require regular ongoing operation and maintenance. General maintenance including lawn mowing, rubbish removal, and inspection should be carried out at regular intervals not exceeding once every two weeks.

Structures such as GPTs, embankments, inlets, outlets, spillways and culverts must be routinely inspected for serviceability, safety, and cleaning and removal of trapped rubbish and sediment. Safety measures such as fences, booms and warning notices must be routinely inspected to ensure that they are in good working order. General guidelines for operation and maintenance of detention ponds are given in earlier section of this Annexure.

AX2.8.2 Aquatic Vegetation

Maintenance during the plant establishment phase is critical because it is during this phase that plants are most vulnerable to damage. Low water level, weed invasion, and damage by animals are possible causes of problems. Plants should be inspected at least weekly during the initial phase in order to detect any damage and allow corrective action. Aquatic plants should be inspected periodically to control pest species and to promote the desired mix of plants for conservation and landscape purposes. Occasional replanting may be necessary to maintain the desired mix of species.

It is not appropriate to regularly harvest macrophytes. The disturbance created by the harvesting process introduces the risk of remobilising sediments and nutrients, and introducing weed species.

AX2.8.3 Eutrophication and Other Problems

Under certain climatic conditions, nutrient enrichment of pond water can cause abundant plant and algal growth. The resulting algal blooms are unsightly and damaging to public health and can cause fish kills and episodes of poor water quality. The following conditions are most likely to encourage eutrophication:

- excessive nutrient loadings in inflows;
- high average temperatures and abundant sunshine;
- still water; and
- clear water (low turbidity).

Pond designers should try to avoid these conditions. For example, it may be inappropriate to locate a pond downstream of an oxidation pond discharge, which is rich in nutrients. In many parts of Malaysia the high turbidity of surface waters helps to prevent eutrophication by preventing sunlight penetration.

However, high turbidity promotes another problem, which is water column stratification. Heated surface waters become lighter than the bottom waters, effectively preventing any mixing. The resulting physical barrier prevents oxygen transfer to the bottom layers, which typically become deoxygenated. Deep ponds may be prone to stratification. There is a rapidly increasing body of scientific knowledge of both of these problems and there are methods, such as mechanical mixing, to overcome them. If any ponds are found to be subject to these problems specialised technical advice should be sought.

AX2.9 CONSTRUCTION EROSION AND SEDIMENT CONTROL BMP$_S$

Sediment basin, the most important BMPs facility in construction stormwater quality control, requires routine maintenance to remain effectiveness as sediment traps. When sediment reaches the maximum level assumed in the design, (usually one-third to one-half the basin volume) it must be removed. Excavated sediment must be placed in a location where it will not easily be eroded again. In addition to sediment cleanout, sediment basin should be inspected after storms to determine whether the embankment or spillways sustained any damaged that requires repair. If the outlet becomes clogged with sediments, it should be cleaned to restore its flow capacity.

The structure should be inspected after significant runoff events to check for the damage or operational problems. Once the contributing drainage area has been stabilized, the structure can be removed or if possible, modified to become part of permanent control features.

Below are other BMPs need to be checked and maintained to make sure the sediment control is working effectively:

- Stabilisation
  - Inspect monthly and after significant rainfall;
  - Re-anchor loosened matting and replace missing matting and staples as required;
  - Inspect periodically and after every significant rainfall; and
  - Repair as necessary.

- Water crossing
  - Inspect weekly and after each significant rainfall, including assessment of foundations;
  - Periodically remove silt from crossings; and
  - Replace lost aggregate from inlets and outlets of culverts.

- Road
  - Periodically apply additional aggregate on gravel roads;
  - Dirt construction roads that are in constant use are commonly watered three or more times per day during the dry season;
  - Inspect weekly, and after rainfall; and
  - Repair any eroded areas immediately.

- Diversion Channel
  - Inspect weekly and after each rainfall;
  - Repair any erosion immediately; and
  - Remove sediment, which builds up in the channel and restricts its flow capacity.

- Slope Drain
  - Structure must be inspected regularly and after rain;
  - Inlet must be free of undercutting and water should not circumvent the entry;
  - Outlets should not produce erosion; velocity dissipators must be maintained; and
  - Pipe anchors must be checked to ensure that the pipe remains anchored to the slope.

AX2.10 PAVEMENT DRAINAGE

Inlets shall be checked and cleaned regularly, to prevent an accumulation of litter and debris, which may cause blockage. Sag locations are particularly susceptible to blockage. Curb and inlets should be cleaned at least twice a year to ensure their proper function. If only one cleaning is possible it should occur prior to the rainy season. Water can pond on the outside edge of the travel way surface when debris, particularly aggregate and soil on turf shoulders, builds up. As debris accumulates on the shoulder, it raises the level of the edge, and eventually hinders run-off from flowing into side ditches or inlets.

Water ponding on the edge of the pavement contributes to the deterioration of the pavement edge and the rutting of stabilized soil supporting the pavement edge, which can result in additional safety hazards (Figure AX2.8). Edge drop-offs and shoulder scour are often caused when water is trapped at the pavement edge by the build-up of debris and vegetation growth.

Figure AX2.8: Deterioration of Pavement Edge

Drainage features that fail to remove run-off because they are too small or are clogged and pond water on the roadway can cause hydroplaning or force drivers to leave their lane (Figure AX2.9). Additionally, other drainage features which do not have anything to do with causing a crash can significantly contribute to the severity of the crash, such as an errant vehicle striking a culvert headwall.

Figure AX2.9: Clogging Inlet

AX2.11 DRAIN AND SWALES

a) Lined Drain

Lined open drains will require periodical maintenance to remove weed growth, sediment deposits, and debris and litter accumulation to maintain the designed hydraulic capacity of the drain. Damaged linings or displaced joints or strut beams should be repaired as soon as practical to prevent further deterioration or failure of sections of the drain.

b) Swale

Periodical maintenance will be required to maintain the hydraulic capacity of a swale. Grass should be regularly mown and sediment, litter, and debris deposits removed, particularly at flow restrictions such as vehicular crossing points. The suitable depth of grass to remove pollutants is about 150mm.

Bare patches and scoured areas must be repaired by removing dead grass, filling scour holes, and reseeding with a recommended permanent grass seed mix.

AX2.12 PIPED DRAIN

A well-maintained pipe drain system will be ready to convey the runoff from the next storm with minimal damage to the storm drainage facilities. A poorly maintained drainage system may not be able to function at its design conveyance and could be damaged by the runoff.

The owner of the facilities should establish a routine maintenance inspection program once the facility has been completed and placed in service. The inspections should be conducted on an annual or semi-annual basis, as well as following major storms. The inspections may be accomplished by visual means or by using a television camera, where applicable.

The inspection should be documented. Items to be recorded should include size and type of facility, date of inspection, location of facility, minor deficiencies, major deficiencies and areas of possible future problems. The documentation should be kept current and when any repair work has been accomplished, it should be recorded. Right-of-way constraints frequently dictate use of a piped drainage system, which in turn create particular maintenance constraints.

The maintenance shall include the followings:

- Debris control – Trash racks should be cleaned regularly to keep accumulations from forming. In-pipe debris should be removed if it is large enough to create a flow obstruction.

- Overflow channel maintenance – If the pipe system was designed with a surcharge or overflow channel it deserves occasional attention. It must be kept clear of excessive vegetation. In general, it should be maintained as an open channel to be ready to function when called upon.
- Inspection – A regular in-pipe inspection of piped drainage systems will detail long term changes and will point out needed maintenance work such as debris removal or joint patching. Special attention is necessary to insure the safety of the inspection team if the pipe is long. Small pipes that carry continuous flow can be viewed with automated equipment. Inspections should be done following major runoff events. Inlet grates should be checked for clogging and catch basins and pipes for sediment/waste blockage.

Typical problem areas then can signal the need for repair of a piped system include:

- Inlet and outlet structures – Local erosion to high velocities, lack of protection, or transition turbulence.
- Trench backfill – Subsidence of the trench, which can result from poor initial compaction or from pipe or joint failure. Earth settlement around manholes is a frequent indicator of compaction problems.
- Pipe joints – The first sign of problems in the system shows at the pipe joints. Spalled concrete, cracks, distorted pipe geometry, backfill movement and water inflow occur at the joints and are precursors of greater problems to come.

AX2.13 ENGINEERED CHANNEL

The owner of an engineered waterway should establish a routine maintenance inspection program once the facility has been completed and placed in service. The inspections should be conducted on an annual or semi-annual basis, as well as following major storms.

The following guidelines are general requirements for the maintenance of all engineered waterways:

- *Mowing* - Grassed waterways should be mowed often enough to maintain appearance and to control weeds.
- *Debris control* - Debris blockage at drainage structures often contributes to flooding problems. Structures such as inlet pits, headwalls, trash racks, and debris traps should be regularly cleaned. Debris should also be regularly removed along the length of the waterway. This should include trimming and thinning of trees if they encroach on the waterway main channel or if they have become overgrown.
- *Sediment and silt removal* - Some silt accumulation in energy dissipation structures and around waterway obstructions is inevitable and is harmless in limited amounts. Silt should be removed if it is severe enough to alter the water surface or affect the function of structures such as drops and culvert inlets. Silt accumulations can also cause trouble by supporting undesirable or obstructive vegetation.

  Sediment traps will need regular removal of trapped material to protect downstream facilities.
- *Access road and footpath repair* - Damaged access road and footpath sections should be repaired to ensure continued maintenance access and better pedestrian use.
- *Vandalism* - Drainage facilities can be attractive nuisances and can be damaged by those who use the area. Preventive measures may be necessary, to keep graffiti off walls, to keep rock riprap from being relocated, or to keep gabion baskets from being cut open.

AX2.14 BIOENGINEERED STREAM

Bioengineering is a stream bank stabilization technique that uses natural materials such as grasses, shrubs, trees, roots, and logs to divert water away from eroding banks, and stabilize the bank. Bioengineering is the preferred method of stream bank stabilization, and is permitted without notification where no work is done in stream with mechanized equipment; and where the work is done in accordance with an approved bioengineering plan.

In order to maintain bioengineered stream the following action must be taken into consideration:

- Check banks after every high-water event.
- Fixing gaps in the vegetative cover at once with structural materials or new plants.

- Mulching if necessary.
- Fresh cuttings from other plants may be used for repairs.

AX2.15 PUMP

Pump stations are vulnerable to a wide range of operational problems from malfunction of equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimise such failures and their consequences. Telemetry and SCADA system are options that should be considered for monitoring critical pump station. Operating functions may be telemeter from station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorised entry, explosive fumes, and high water levels can be monitored effectively in this manner. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

Since major storm events are infrequent, a comprehensive, preventive maintenance program should be developed for maintaining and testing the equipment so that it will function properly when needed. Instruments such as hours-run meters and number-of starts meters should be used on each pump to help schedule maintenance. Regular inspections as well as disassembly inspections should be conducted in conjunction with regular inspections to prevent any accidents from occurring. The following list will explain the basic maintenance procedures as well as detailed explanations regarding inspections and troubleshooting :

- A maintenance reference guide with details regarding the appropriate frequency and time intervals between inspections and part replacements should be created to assist with maintenance activities and smooth accident free operation;
- Create daily and monthly logs, and enter information regarding maintenance inspection etc. Always make note of condition of the machinery;
- Classify each unit and create a log to record detailed information on inspections, maintenance activities, repairs and other problems for future reference; and
- Organise and store special tools used for repairs and maintenance, spare parts and consumables so they will be readily available if needed.

Most of the pump suppliers will provide together with the pump the checklist, maintenance and service manuals for reference during the operation period.

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering. Pump stations should be classified as a confined space. In this case, access requirements along with any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorised personnel.

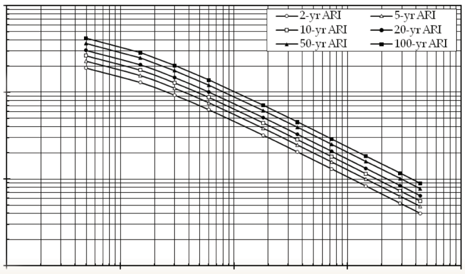
AX2.16 CULVERT

Poorly working culverts can cause flooding that significantly damages roads and bridges. A crushed or plugged culvert allows water to back up in roadside ditches, even during normal wet weather. This contributes to road deterioration because standing water prevents drainage from the road base and subgrade. Culverts should be inspected at least once a year. The elements for culvert inspection as listed below:

- Check the accumulation of debris, siltation or other flow impedimentsat inlets and outlets;
- Inspect the culvert barrel, if possible, for tree or other vegetation roots, mineral deposits, trash or silt accumulations and other foreign objects obstructing flow paths;
- Examine inlet and outlet areas for evidence of soil erosion, which generally leads to scour, undermining and caving of adjacent soil supporting the culvert. Soil erosion quickly leads to reduced structural and hydraulic performance;

- Inspect all visible structures such as sumps, headwalls, wing walls, culverts and aprons for sign of wear or breakage;
- Check upstream for evidence of backup or prolonged surface water presence that indicated reduced inflow. Check downstream for evidence of foreign material that indicate reduce filtration of soil or structural degradation of drainage system itself;

Figure AX2.10: Maintenance Work for Culvert



IDF CURVES

3
ANNEX

ANNEX 3 IDF CURVES FOR 5 MINUTES TO 72 HOURS STORM DURATION

AX3.1 STATE OF JOHOR

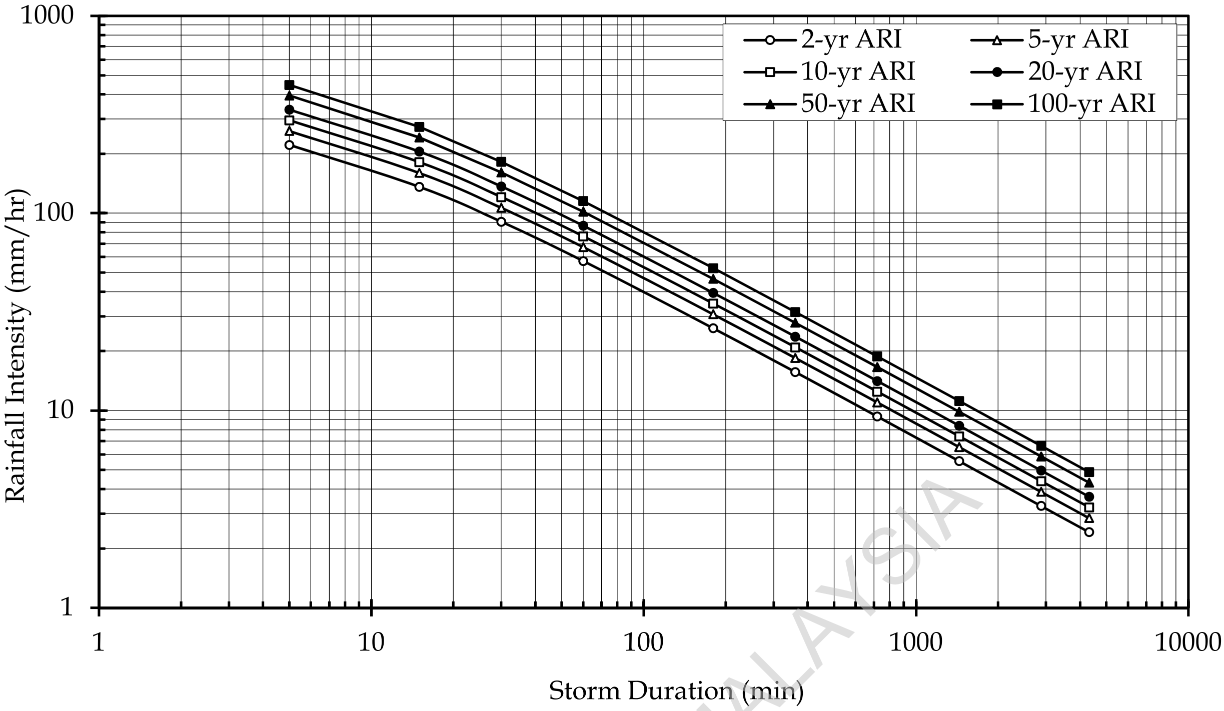


Figure AX3.1.1: Rainfall Station at Stor JPS Johor Baharu -1437116

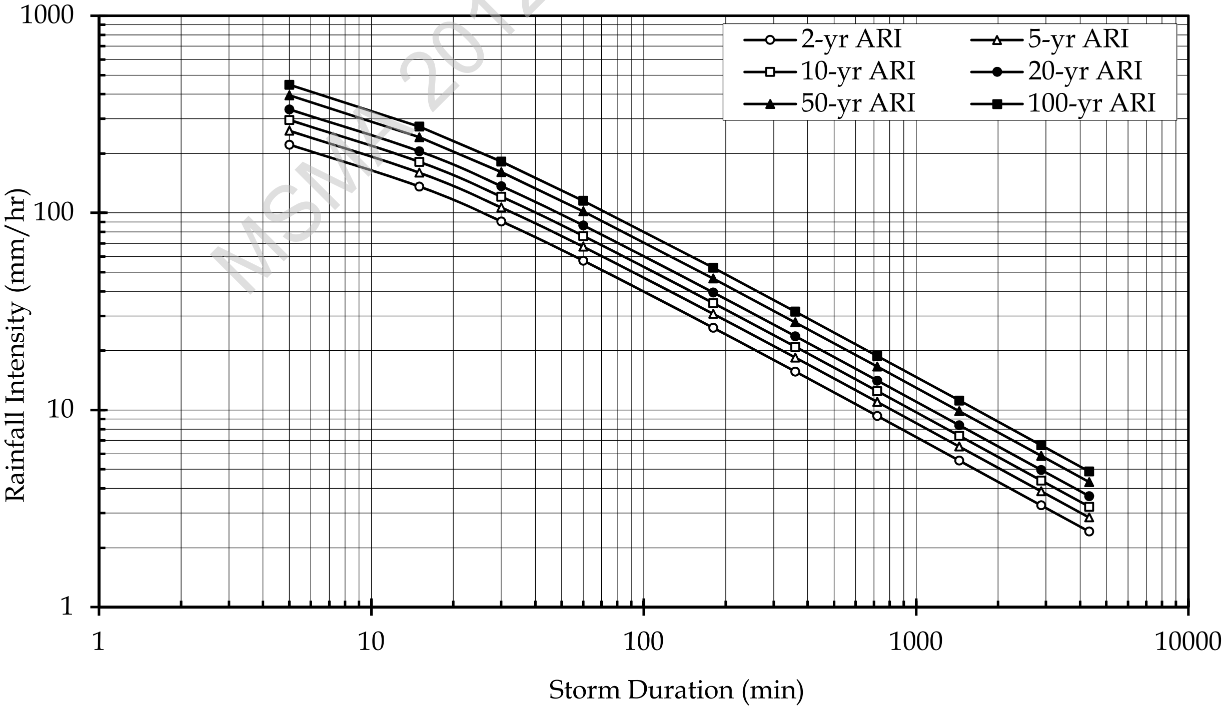


Figure AX3.1.2: Rainfall Station at Pusat Kem. Pekan Nenas -1534002

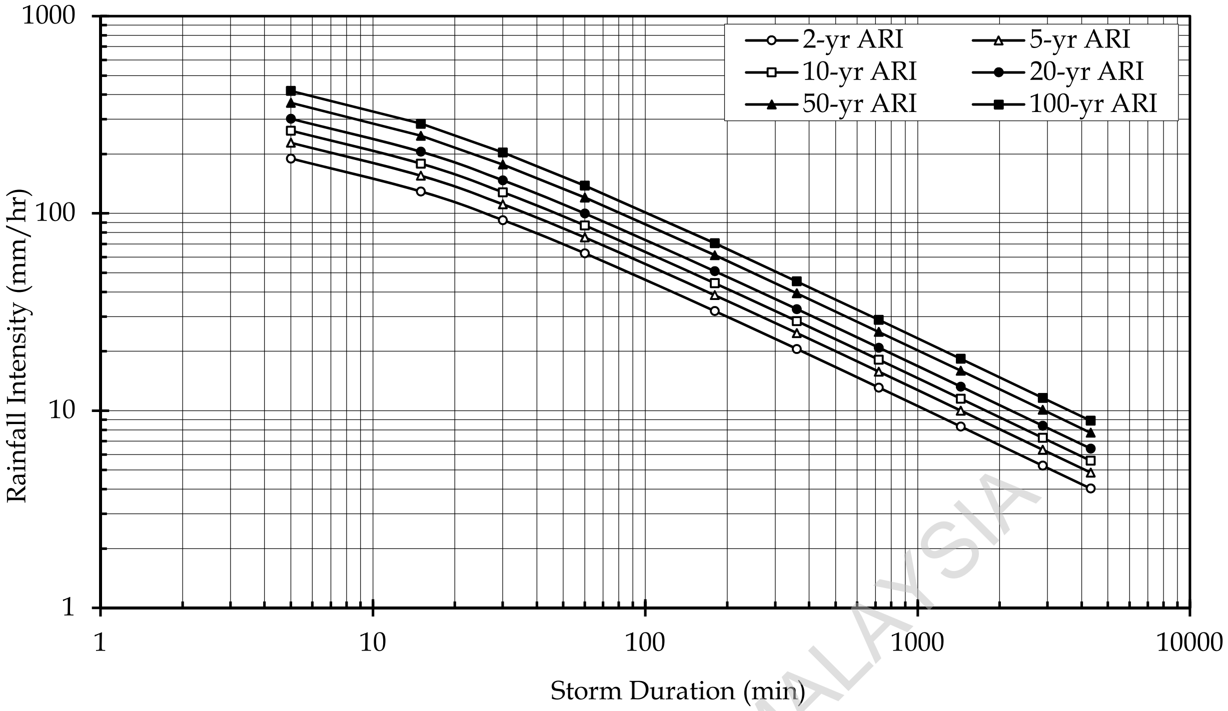


Figure AX3.1.3: Rainfall Station at Johor Silica-1541139

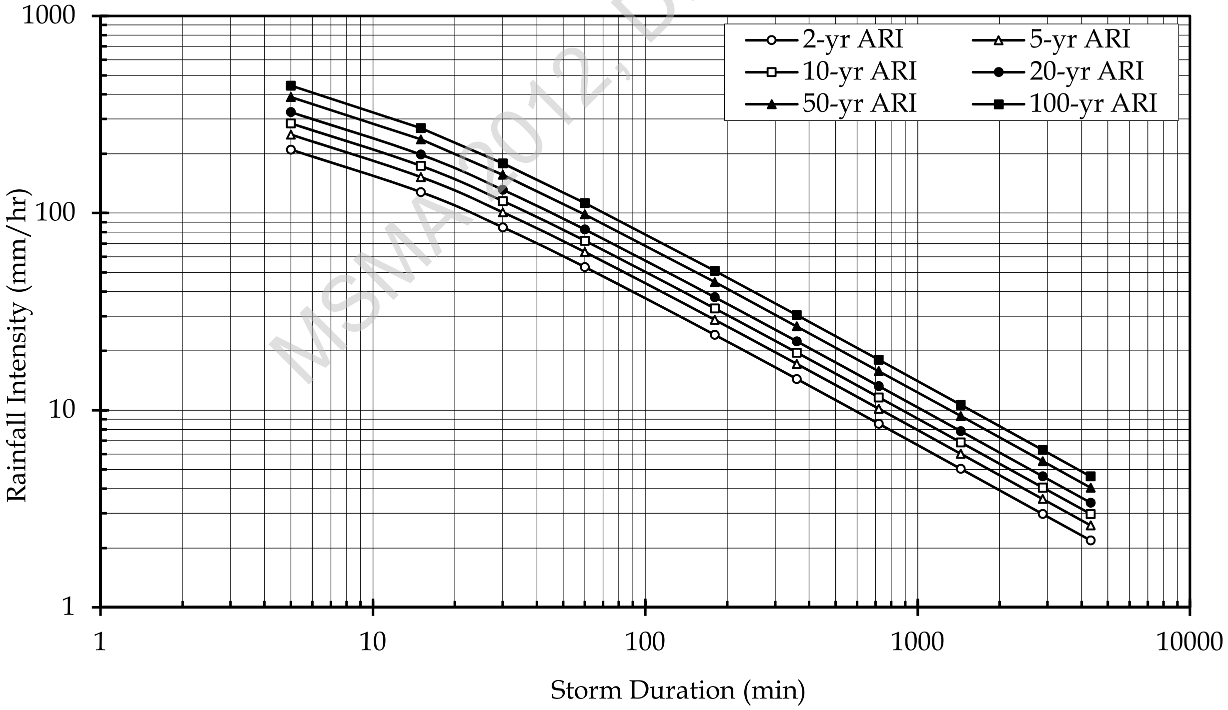


Figure AX3.1.4: Rainfall Station at Balai Polis Kg Seelong-1636001

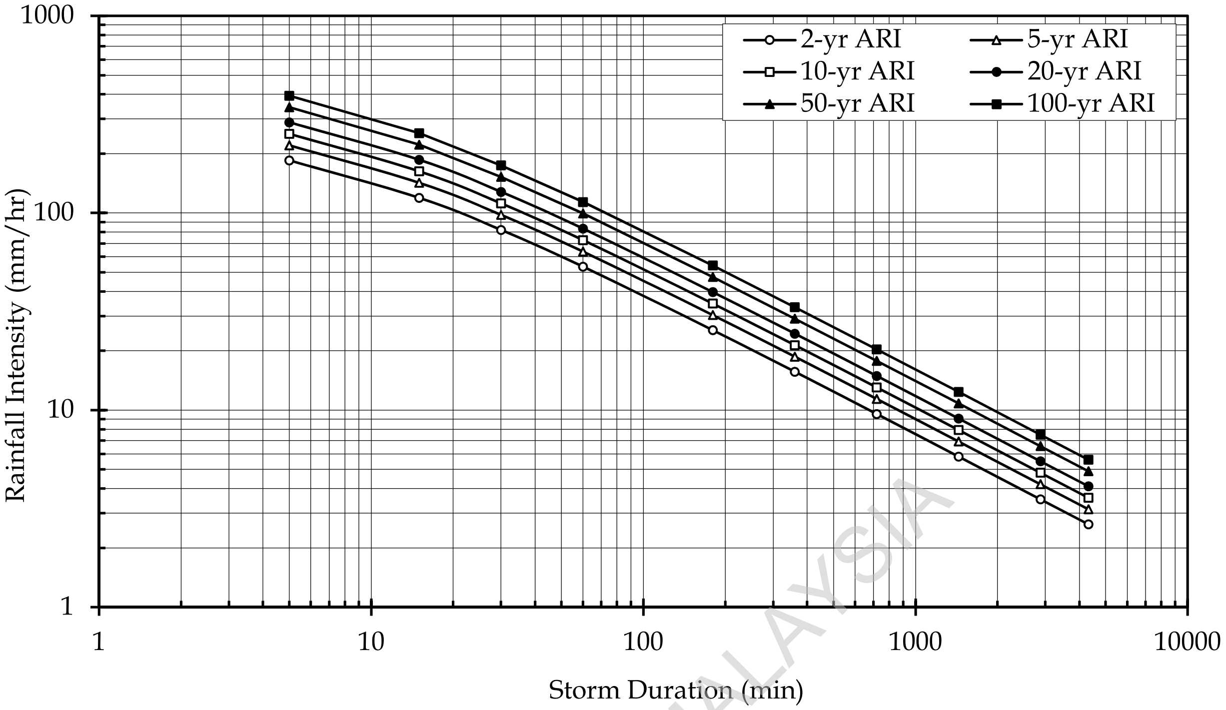


Figure AX3.1.5: Rainfall Station at SM Bukit Besar -1737001

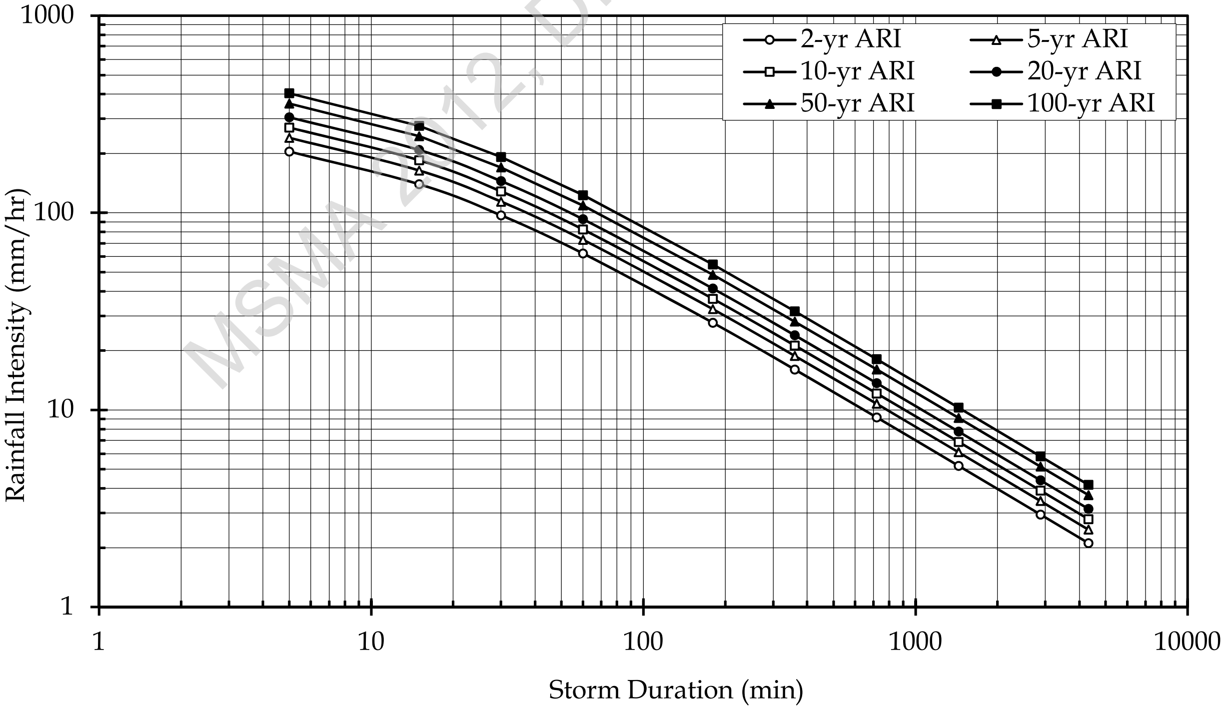


Figure AX3.1.6: Rainfall Station at Setor JPS B Pahat-1829002

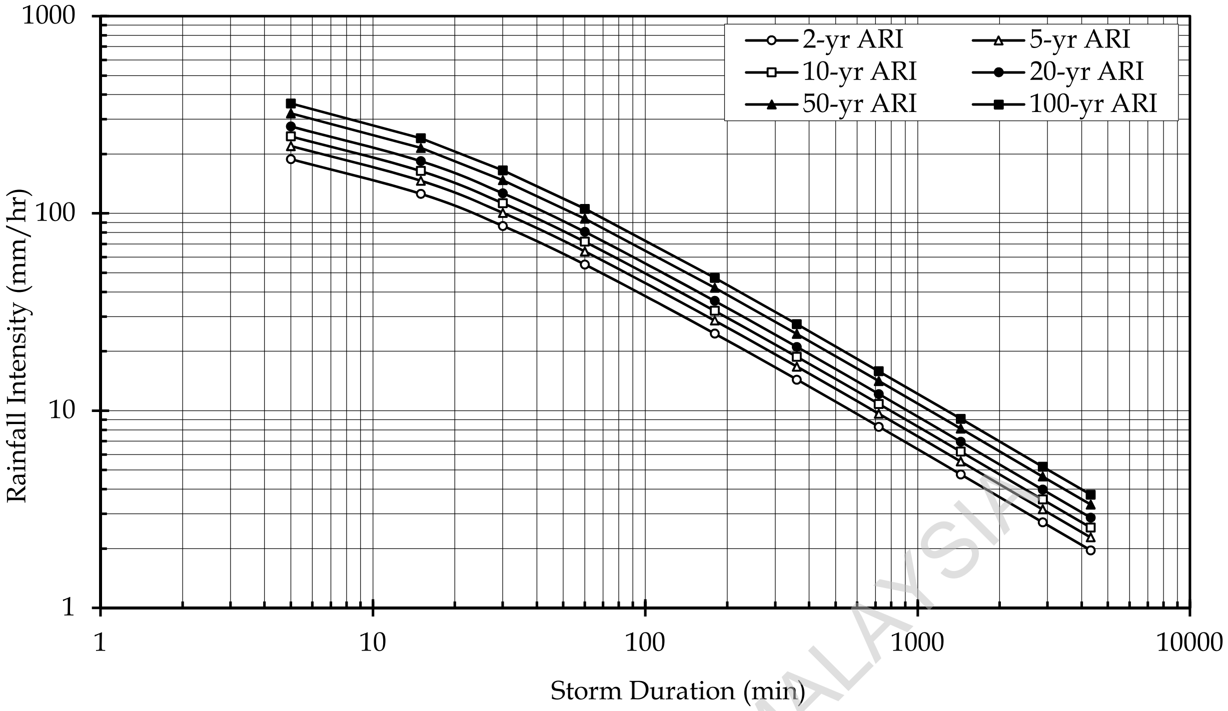


Figure AX3.1.7: Rainfall Station at Ladang Ulu Remis -1834124

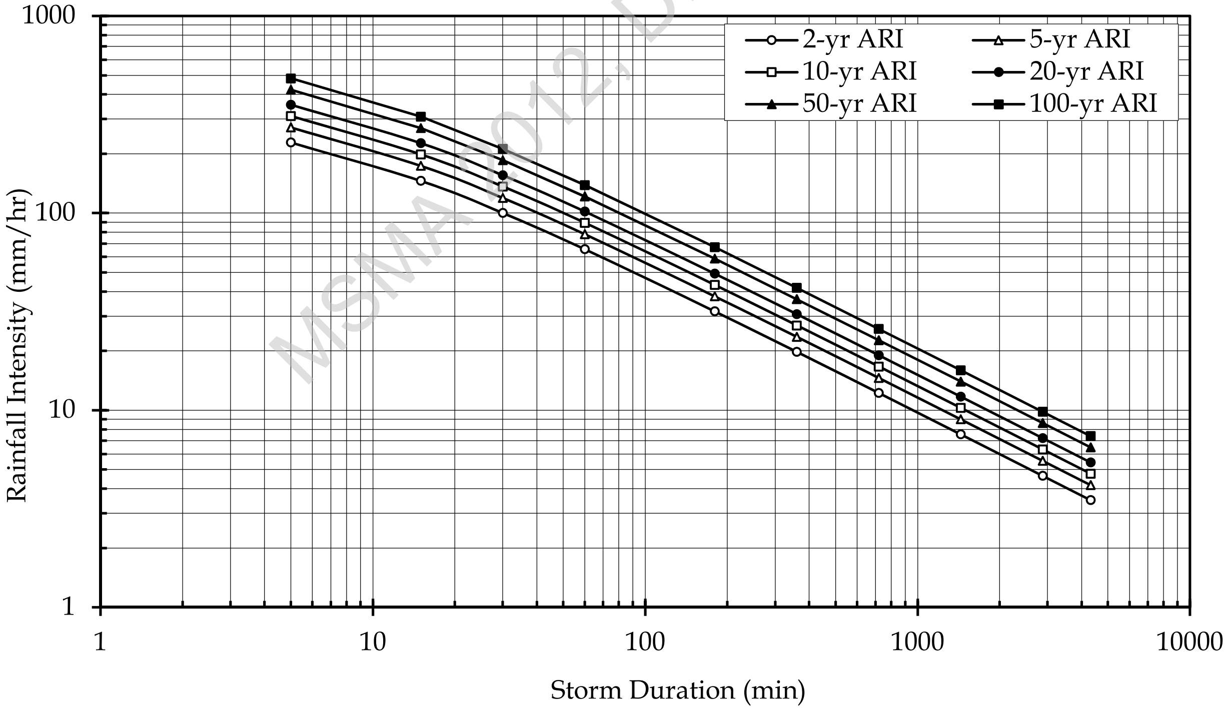


Figure AX3.1.8: Rainfall Station at Simpang Masai K. Sedili -1839196

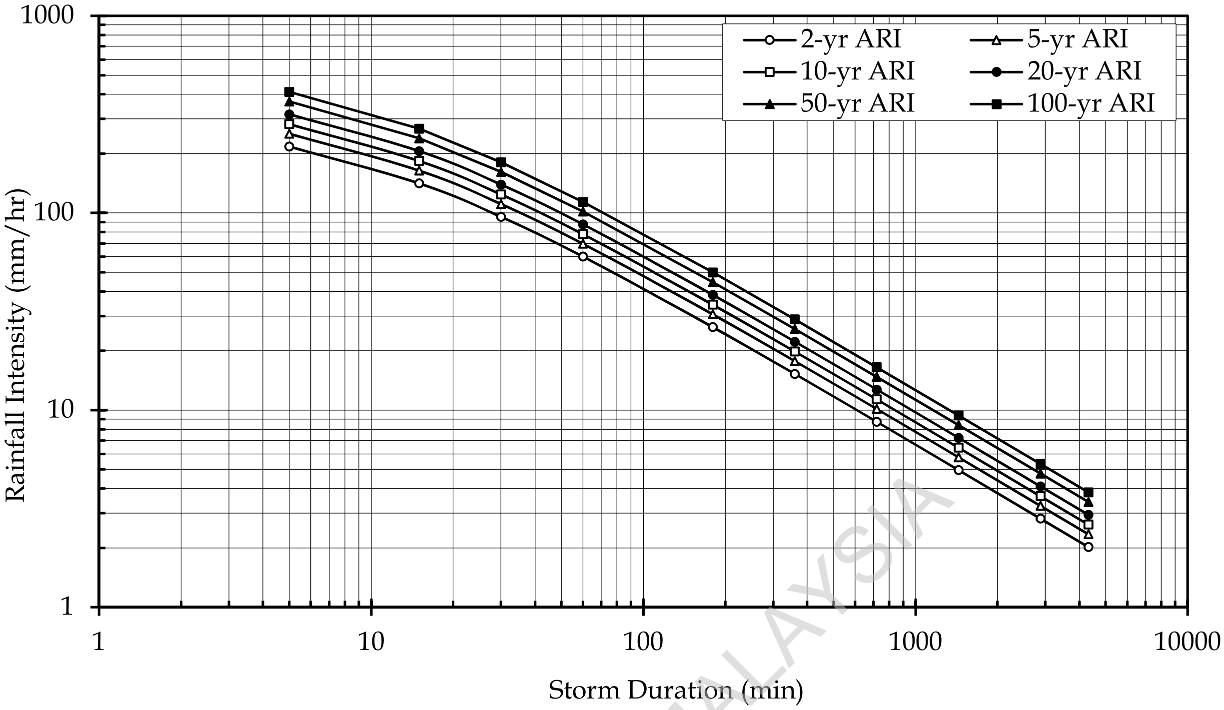


Figure AX3.1.9: Rainfall Station at Emp. Semberong-1931003

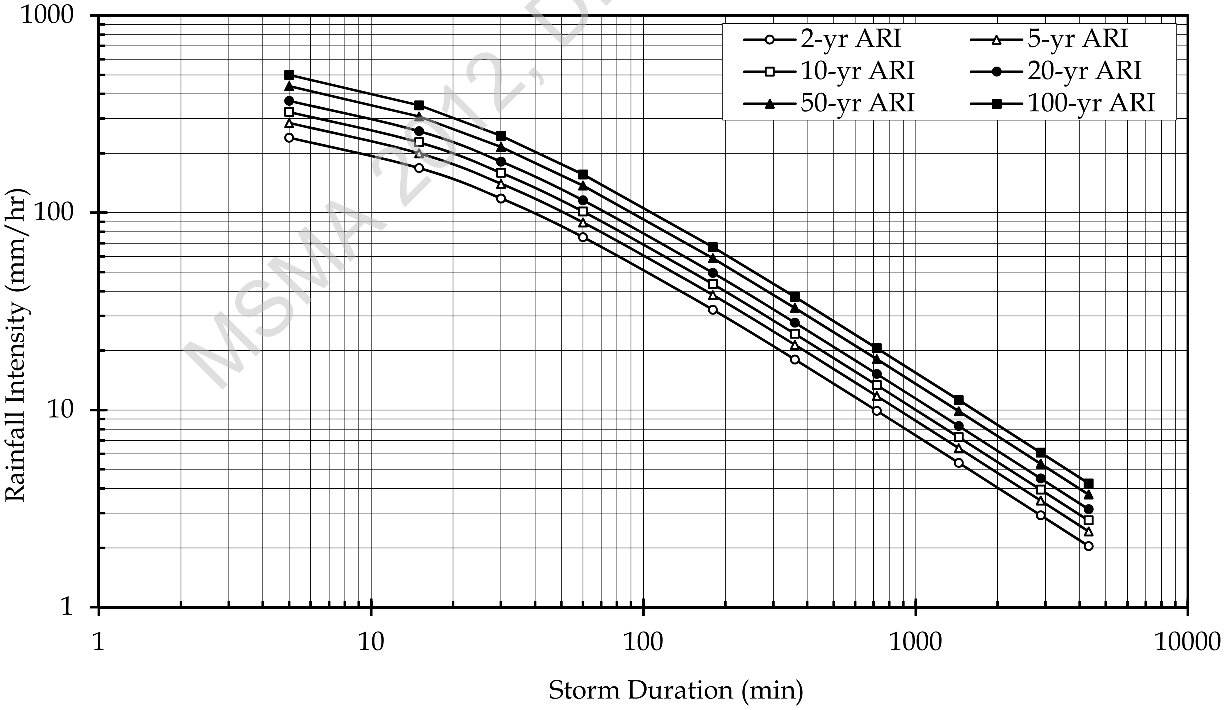


Figure AX3.1.10: Rainfall Station at Pintu Kaw. Tg. Agas-2025001

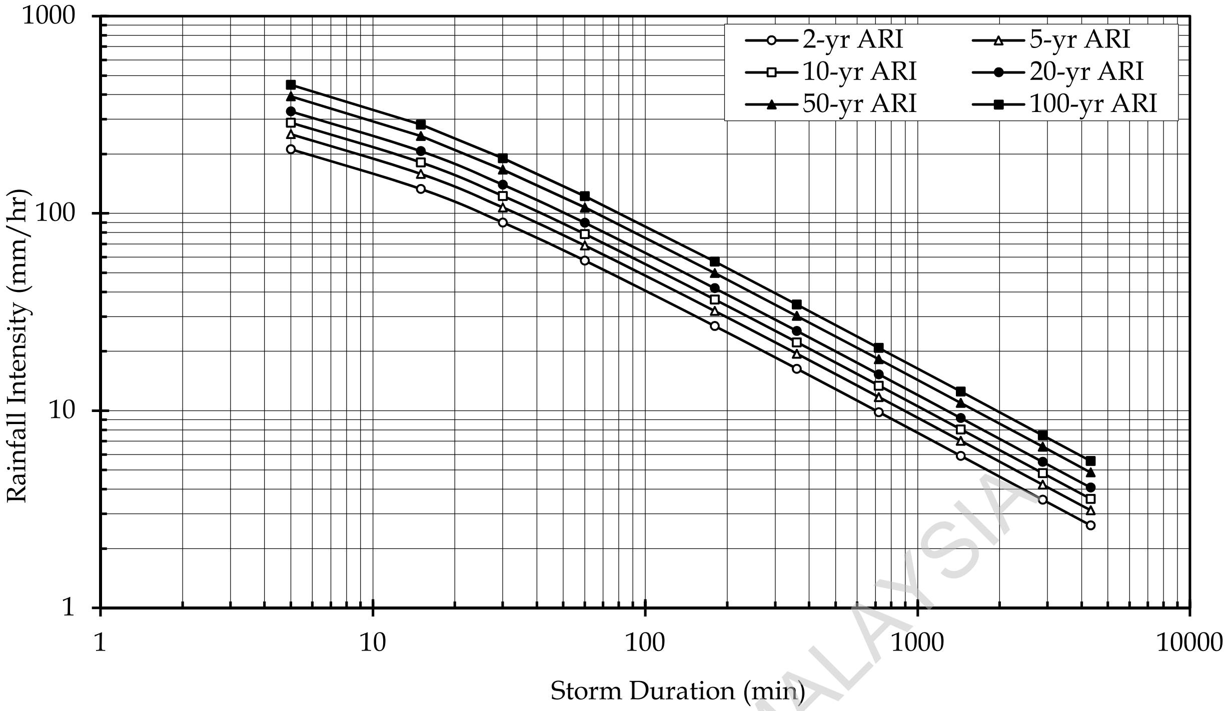


Figure AX3.1.11: Rainfall Station at JPS Kluang -2033001

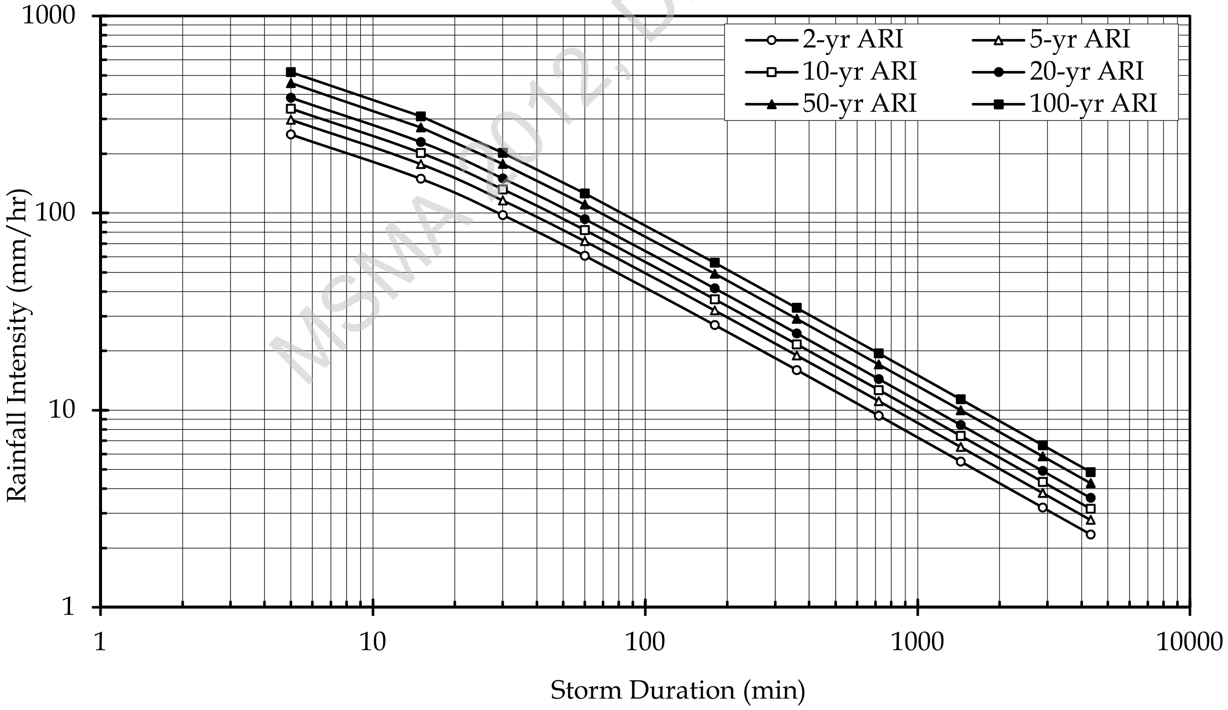


Figure AX3.1.12: Rainfall Station at Ladang Chan Wing -2231001

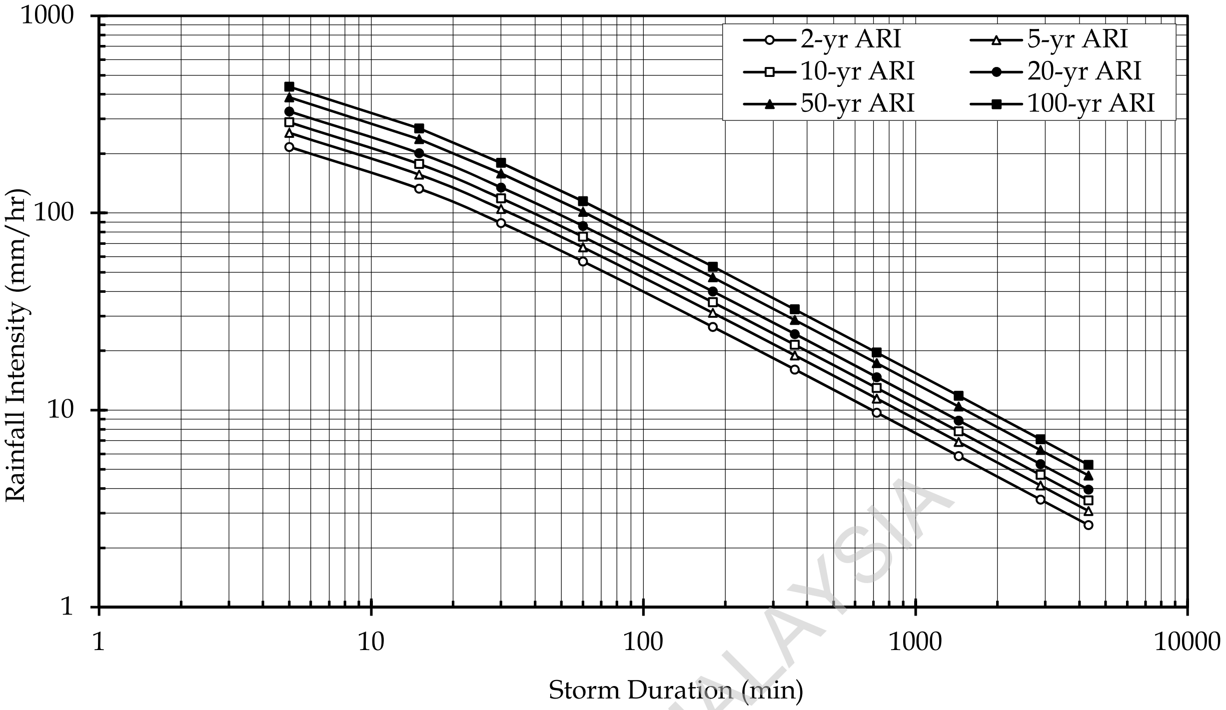


Figure AX3.1.13: Rainfall Station at Ladang Kekayaan -2232001

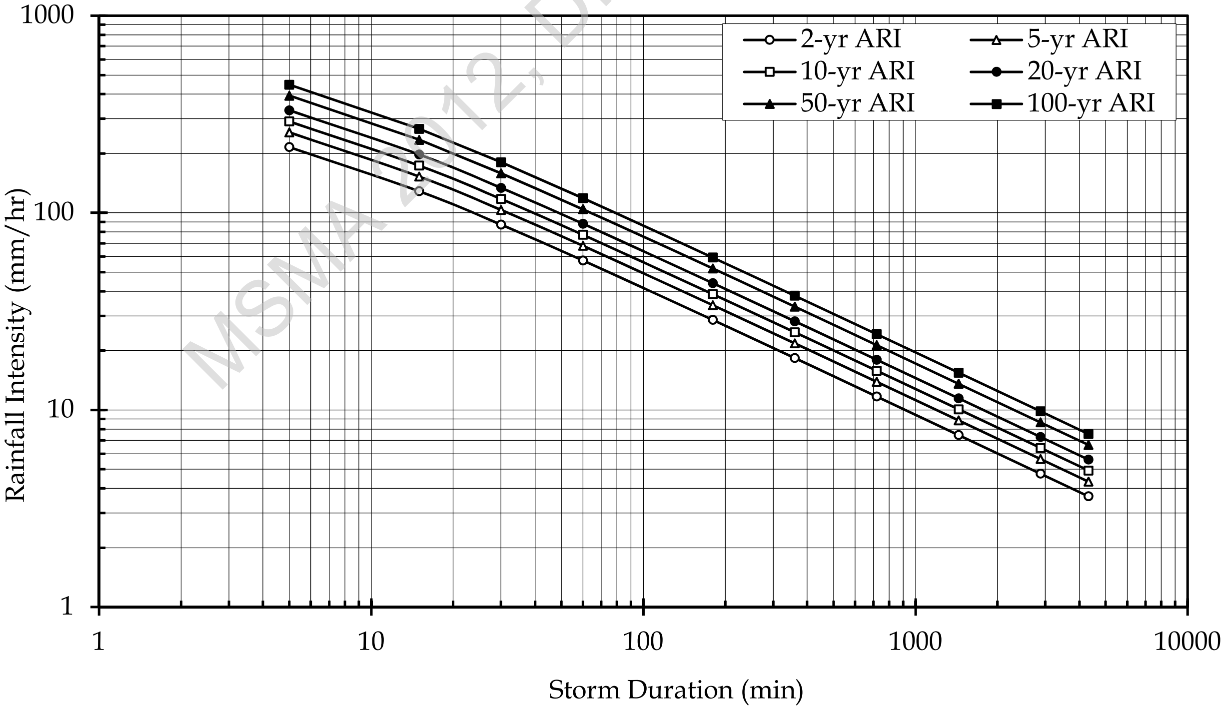


Figure AX3.1.14: Rainfall Station at Ibu Bekalan Kahang-2235163

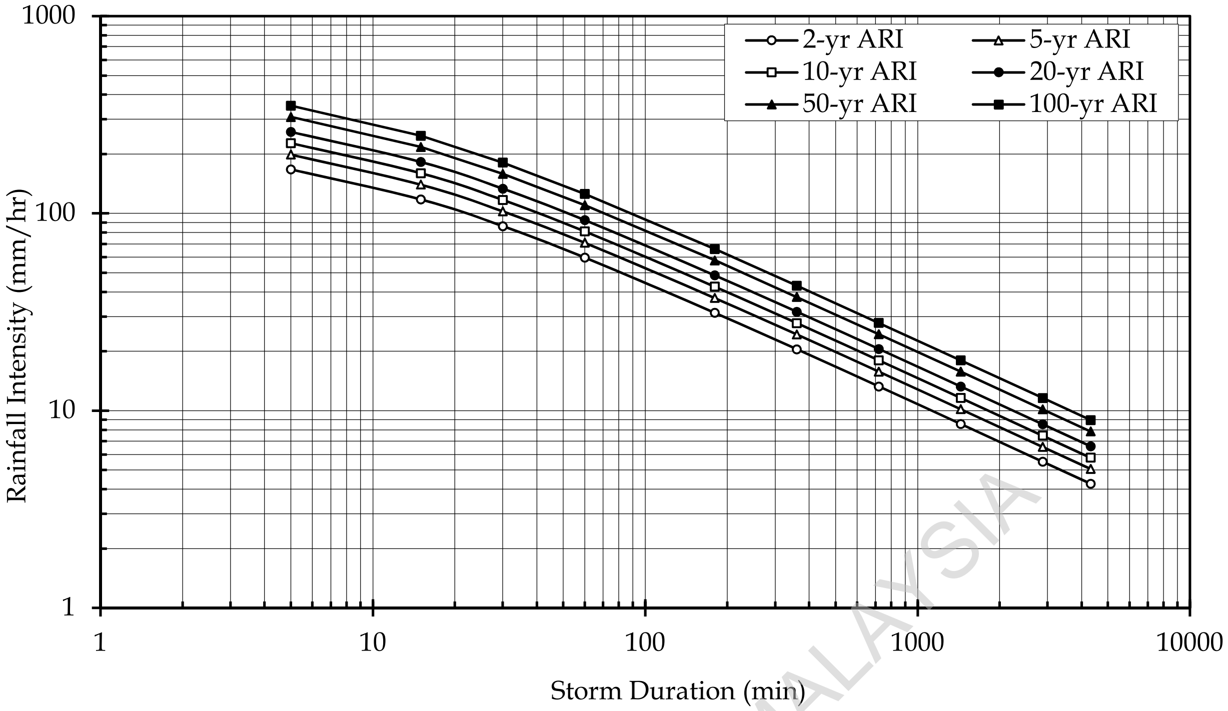


Figure AX3.1.15: Rainfall Station at Jalan Kluang-Mersing-2237164

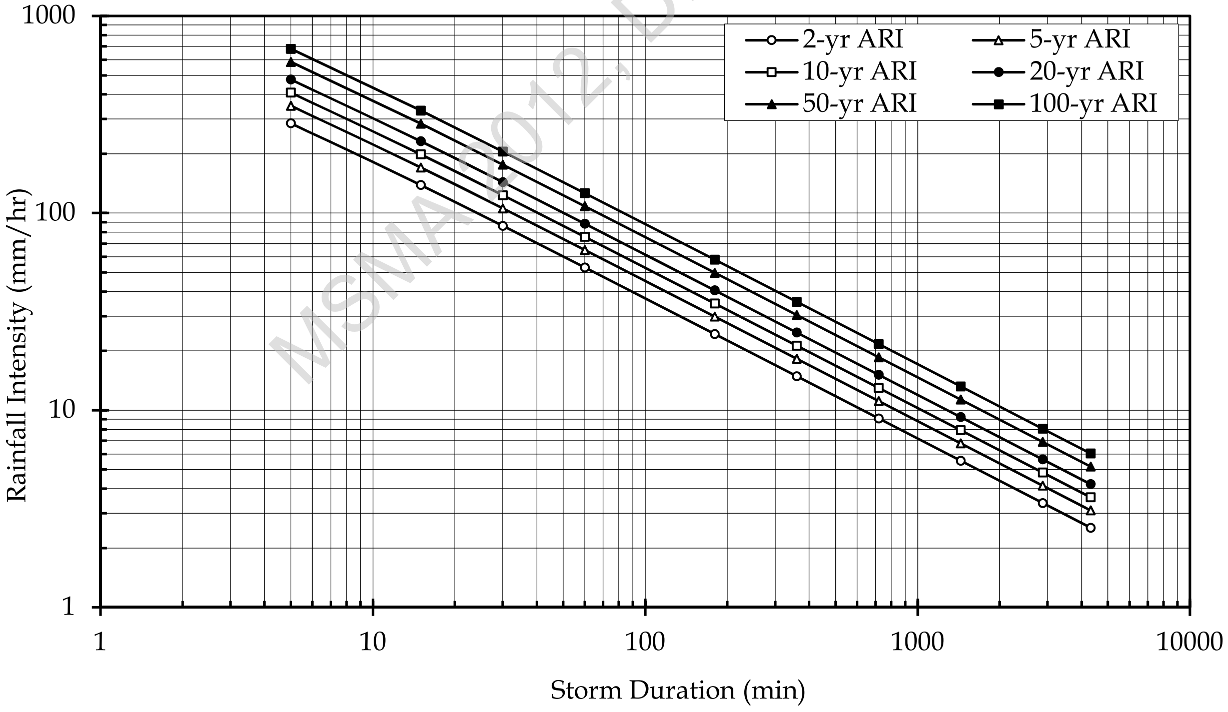


Figure AX3.1.16: Rainfall Station at Ladang Labis-2330009

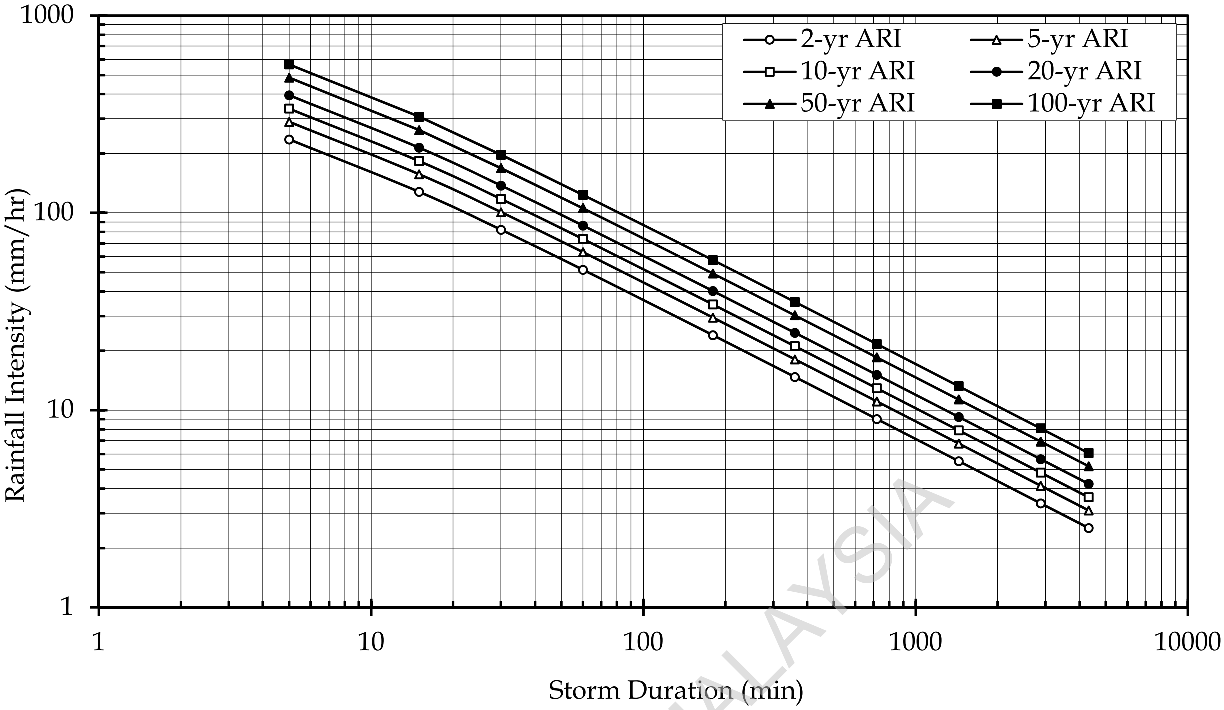


Figure AX3.1.17: Rainfall Station at Rmh. Tapis Segamat-2528012

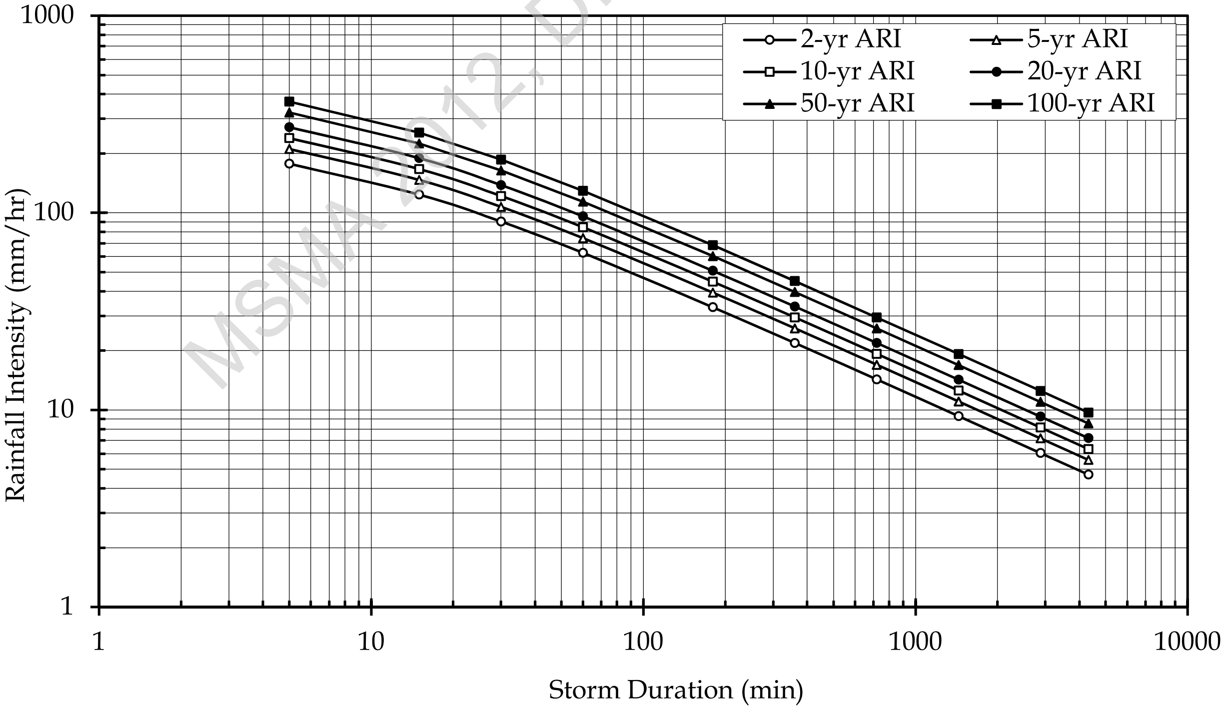


Figure AX3.1.18: Rainfall Station at Kg Peta Hulu Sg Endau-2534160

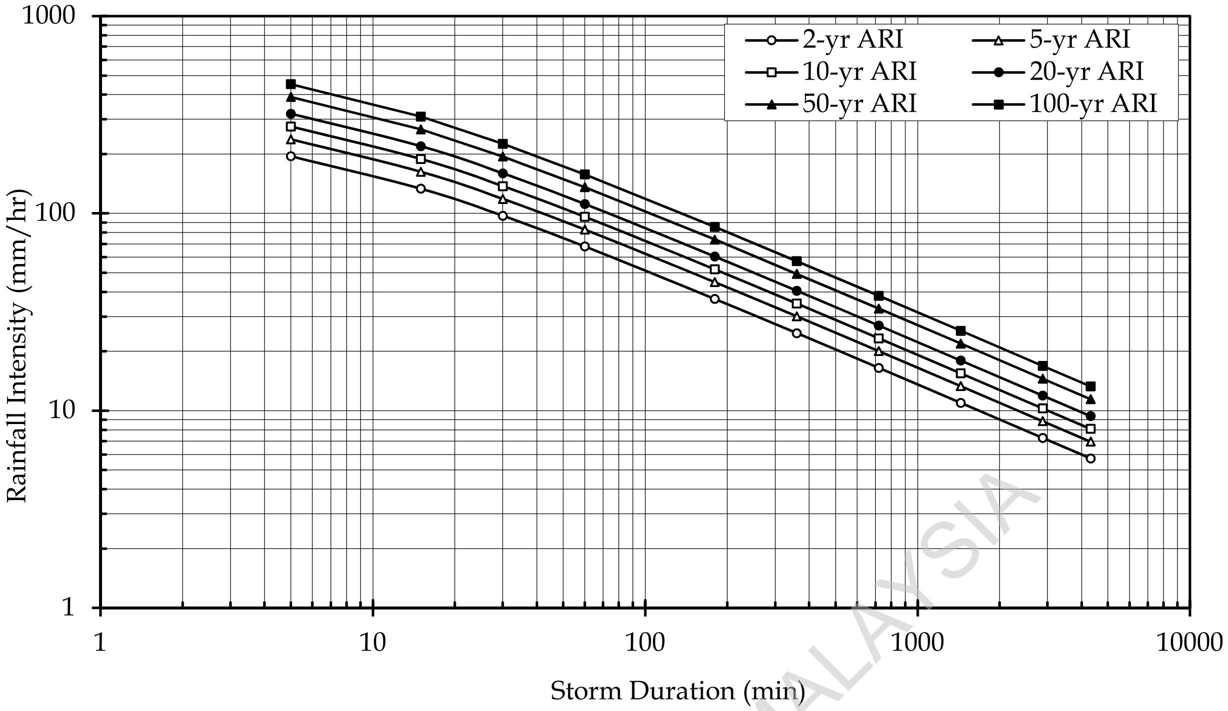


Figure AX3.1.19: Rainfall Station at Setor JPS Endau-2636170

AX3.2 STATE OF KEDAH

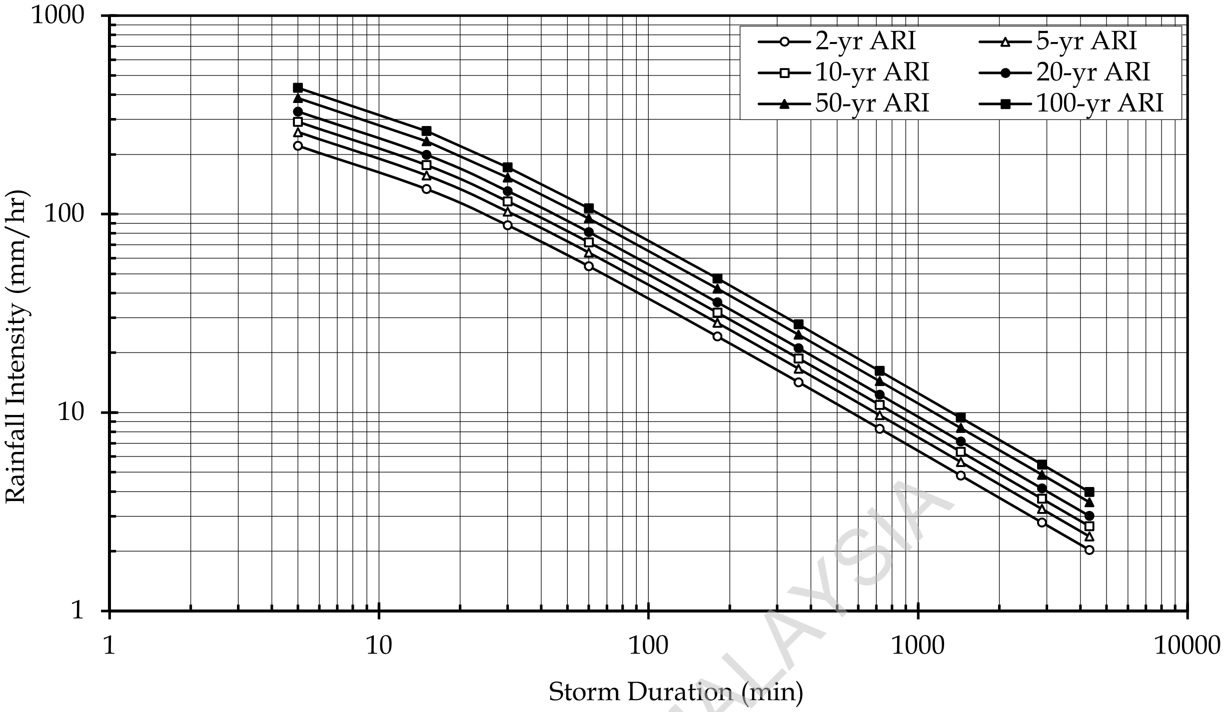


Figure AX3.2.1: Rainfall Station at Bt.27 Jalan Baling-5507076

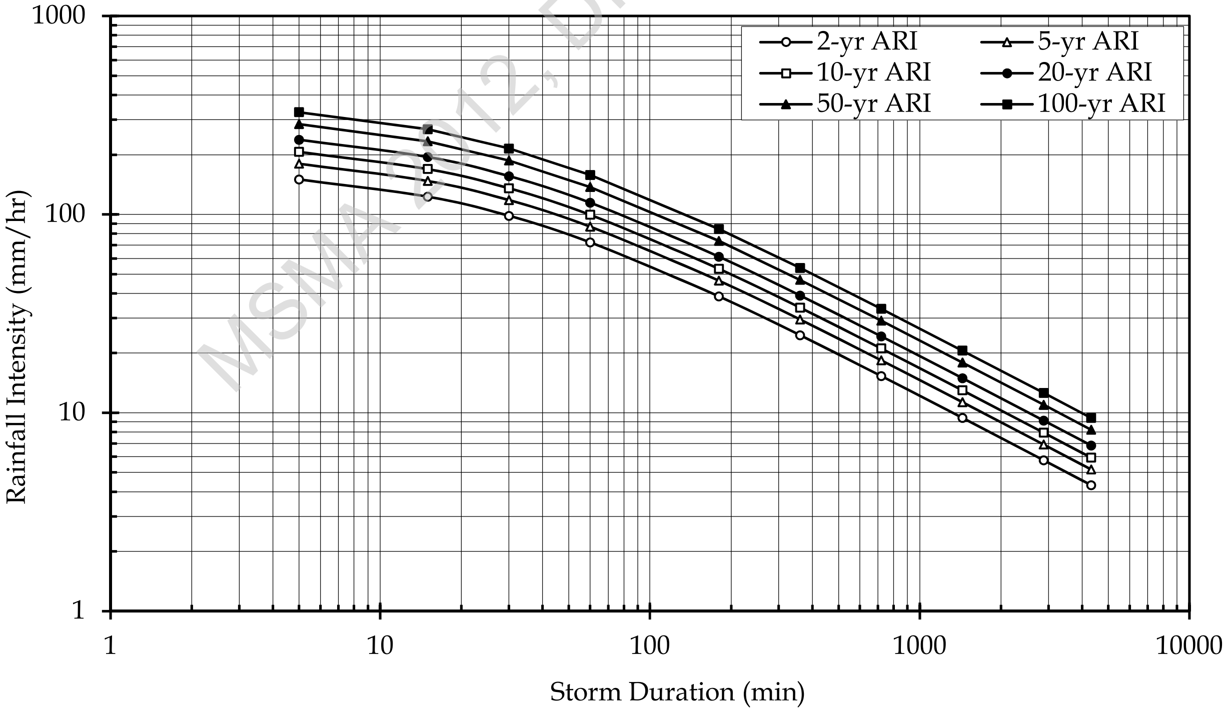


Figure AX3.2.2: Rainfall Station at Kedah Peak- 5704055

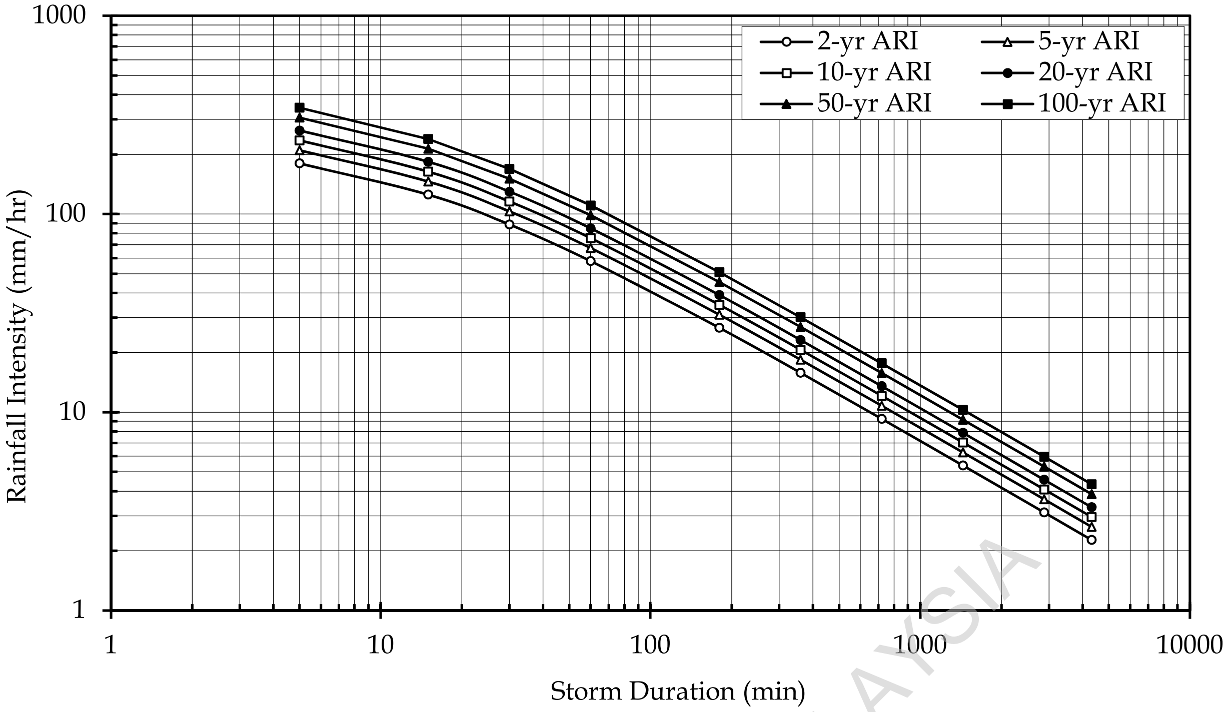


Figure AX3.2.3: Rainfall Station at Klinik Jeniang-5806066

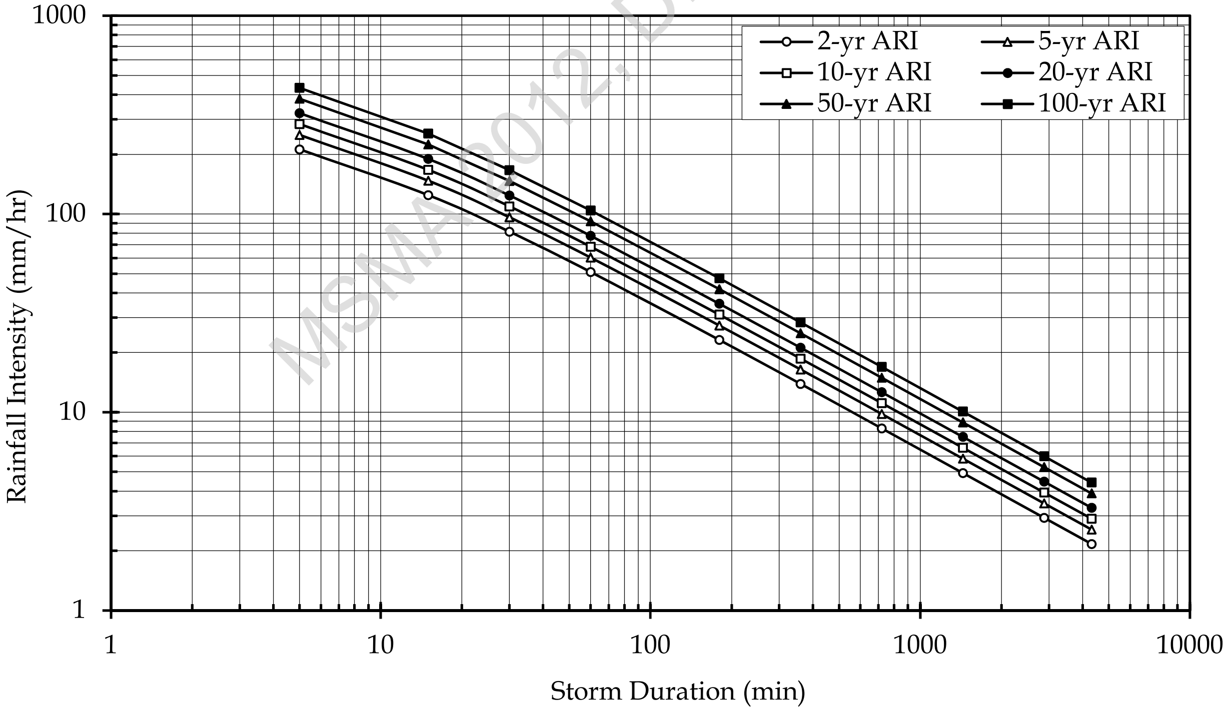


Figure AX3.2.4: Rainfall Station at Bt.61 Jalan Baling -5808001

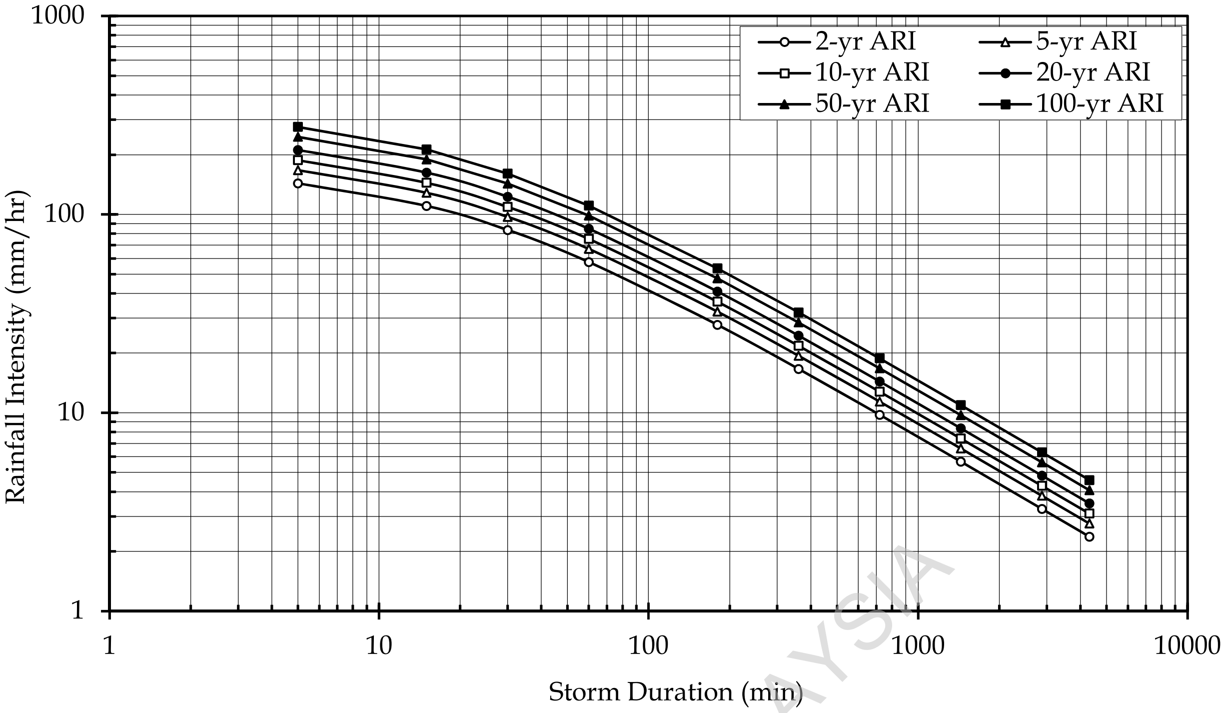


Figure AX3.2.5: Rainfall Station at Setor JPS Alor Setar-6103047

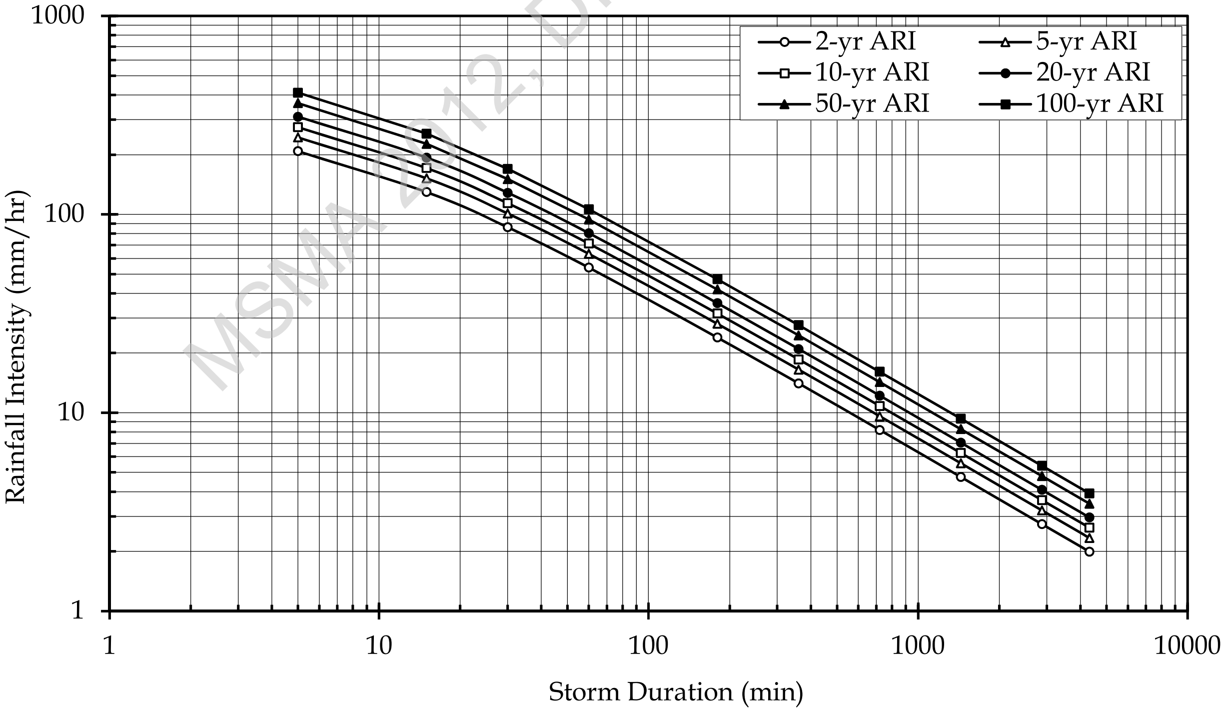


Figure AX3.2.6: Rainfall Station at Kompleks Rumah Muda-6108001

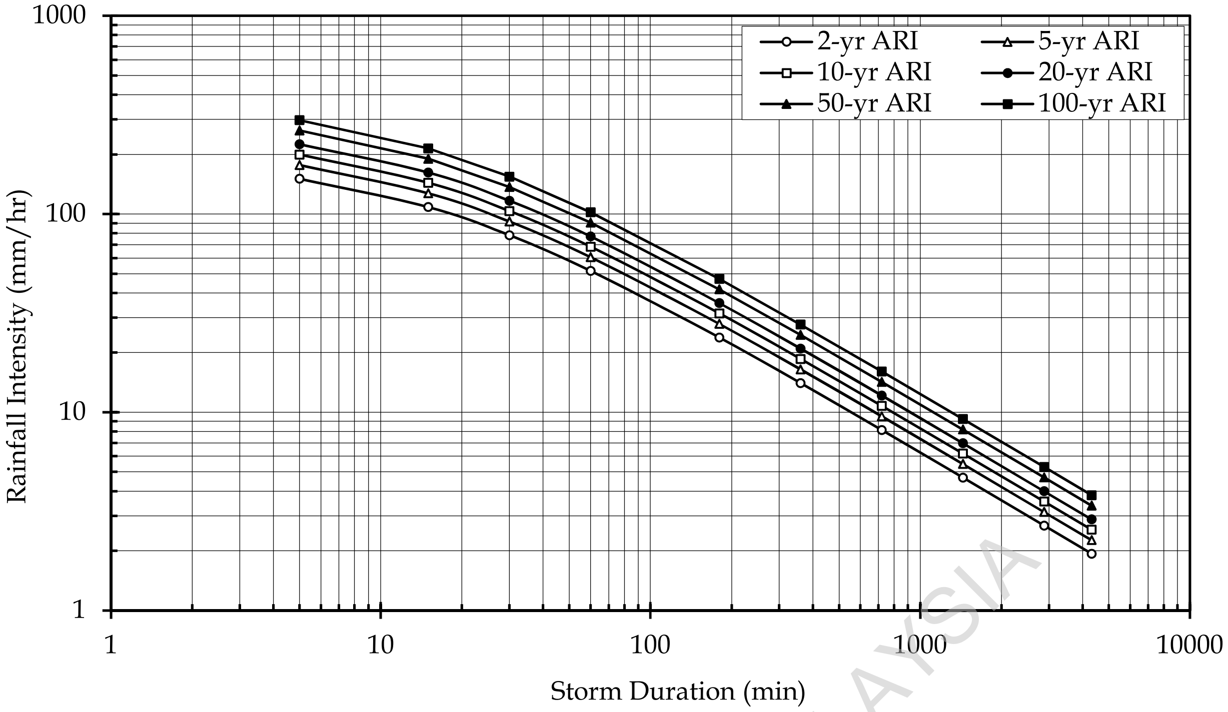


Figure AX3.2.7: Rainfall Station at Kuala Nerang-6206035

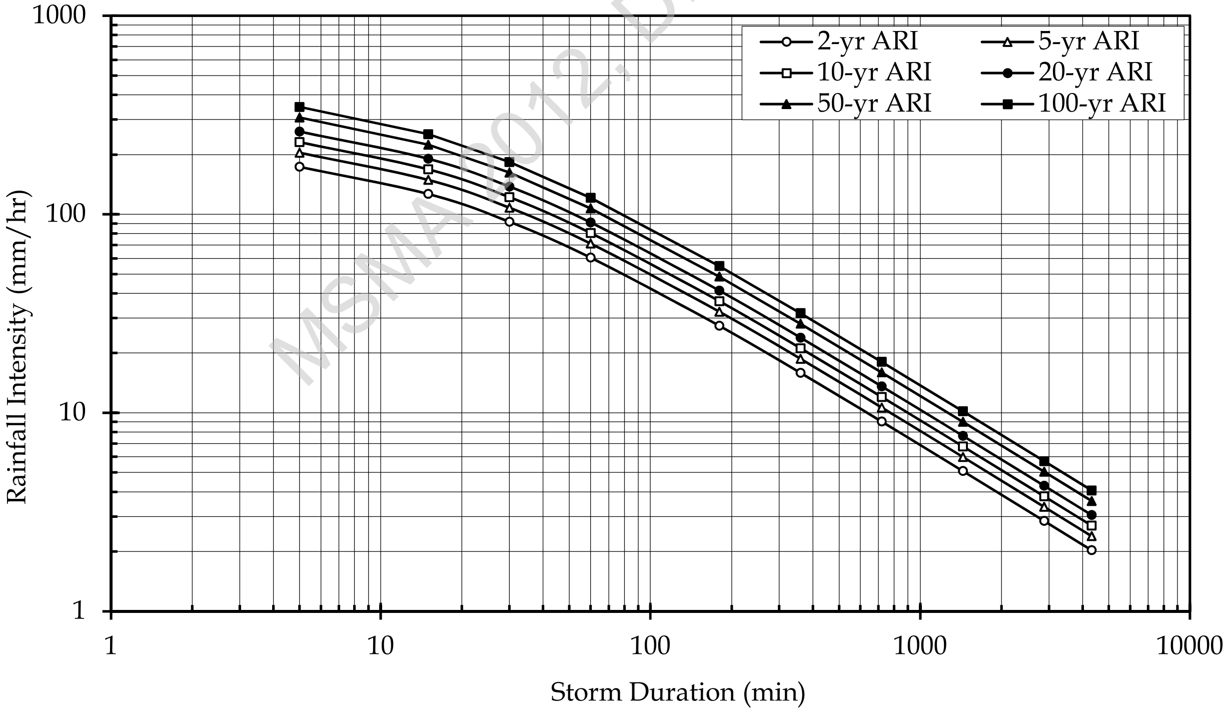


Figure AX3.2.8: Rainfall Station at Ampang Padu -6207032

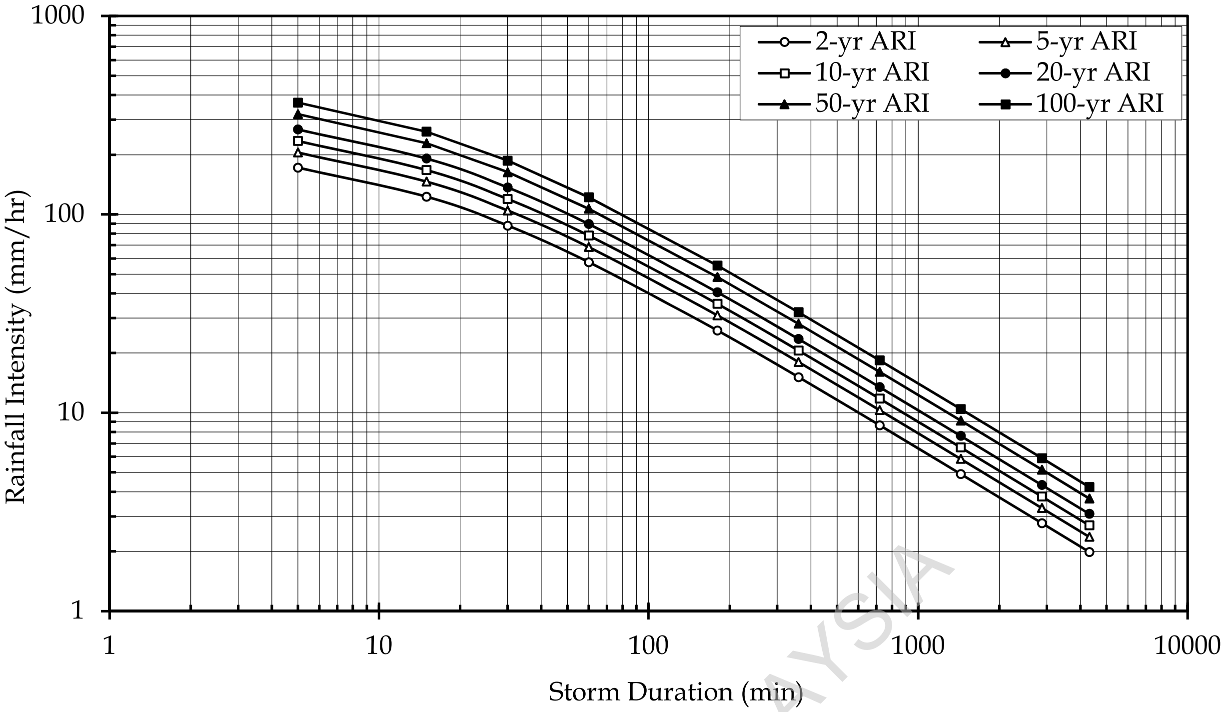


Figure AX3.2.9: Rainfall Station at Padang Sanai-6306031

AX3.3 STATE OF KELANTAN

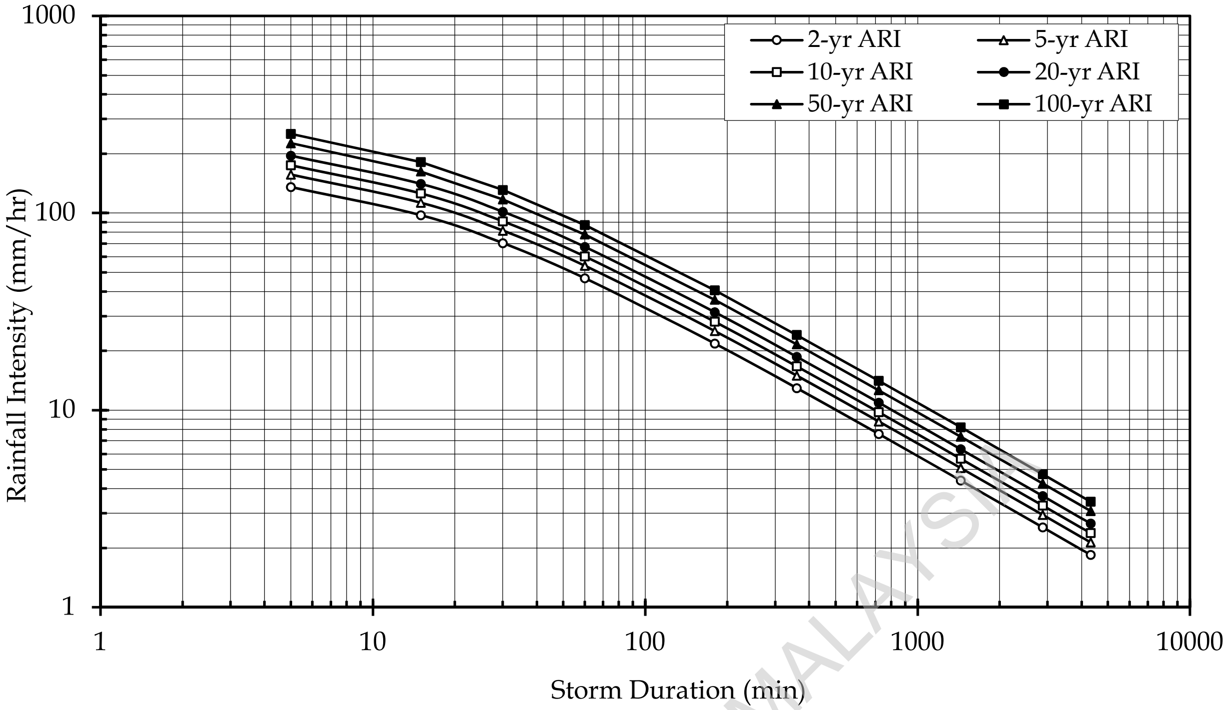


Figure AX3.3.1: Rainfall Station at Brook-4614001

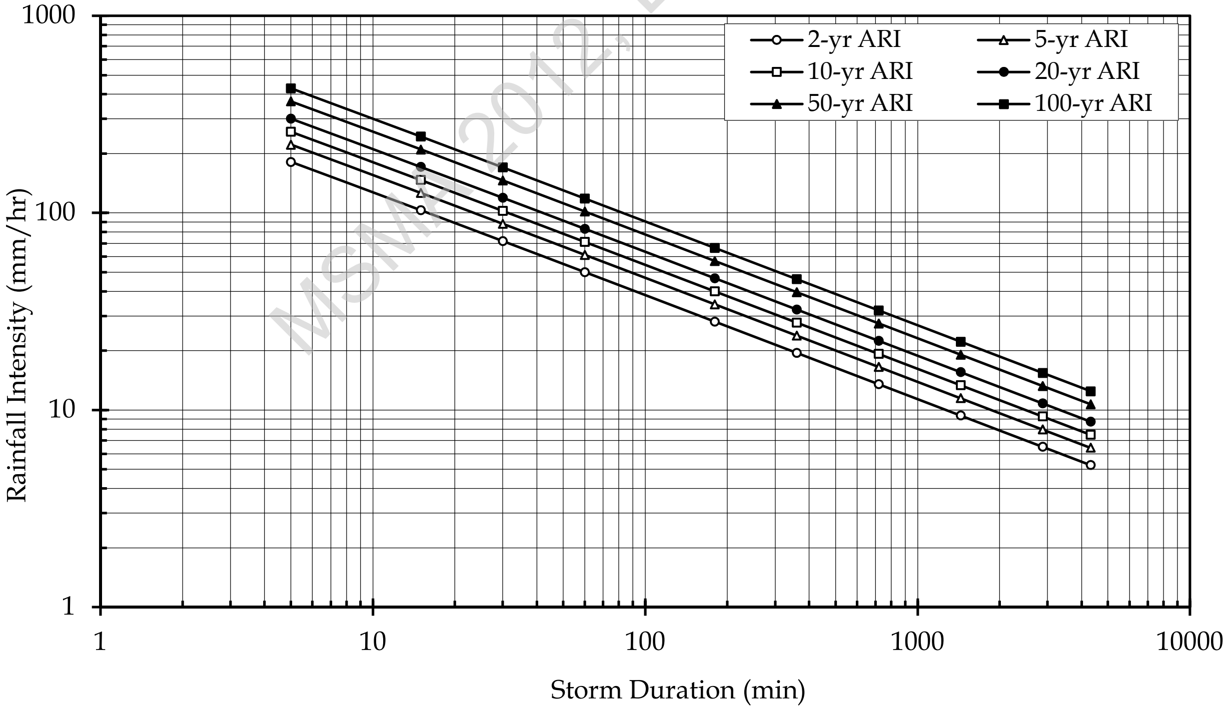


Figure AX3.3.2: Rainfall Station at Gunung Gagau-4726001

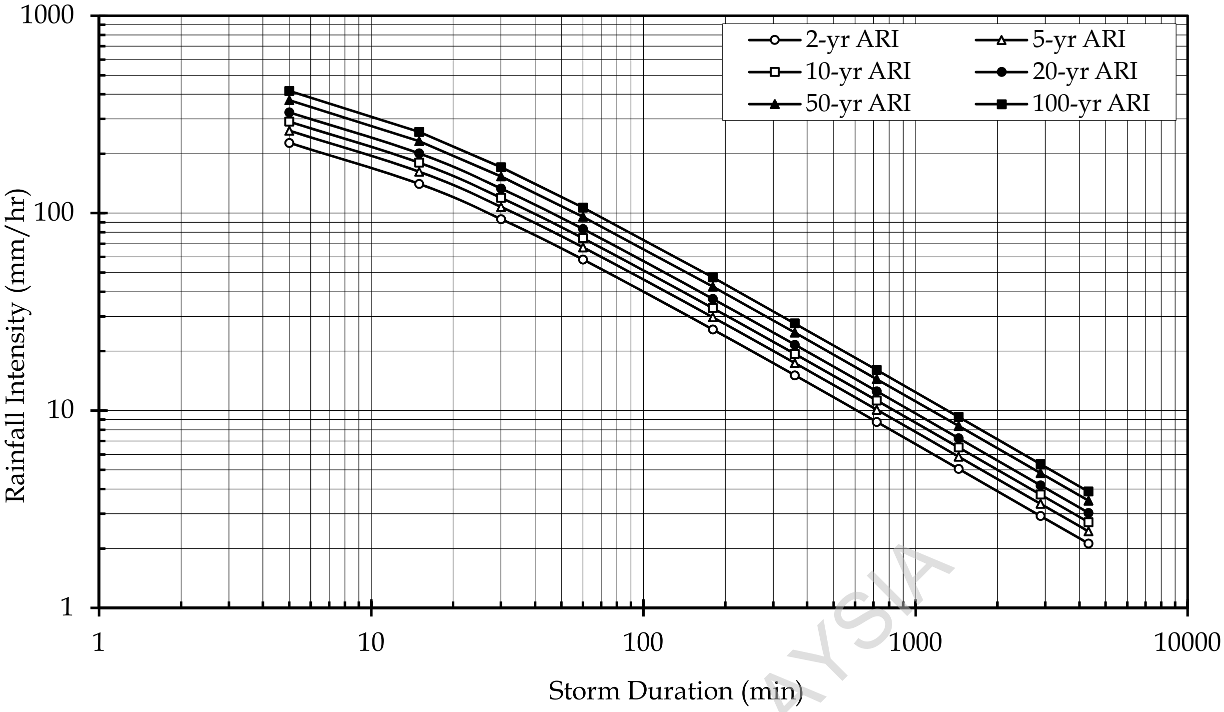


Figure AX3.3.3: Rainfall Station at Gua Musang-4819027

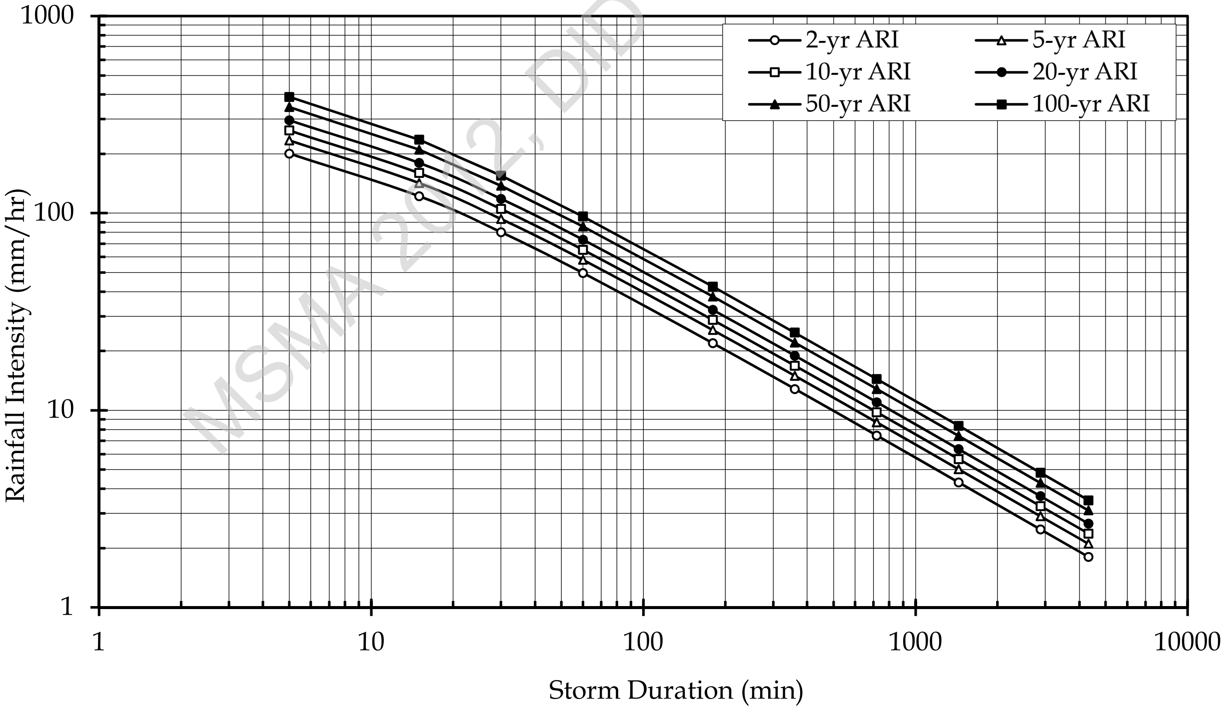


Figure AX3.3.4: Rainfall Station at Chabai-4915001

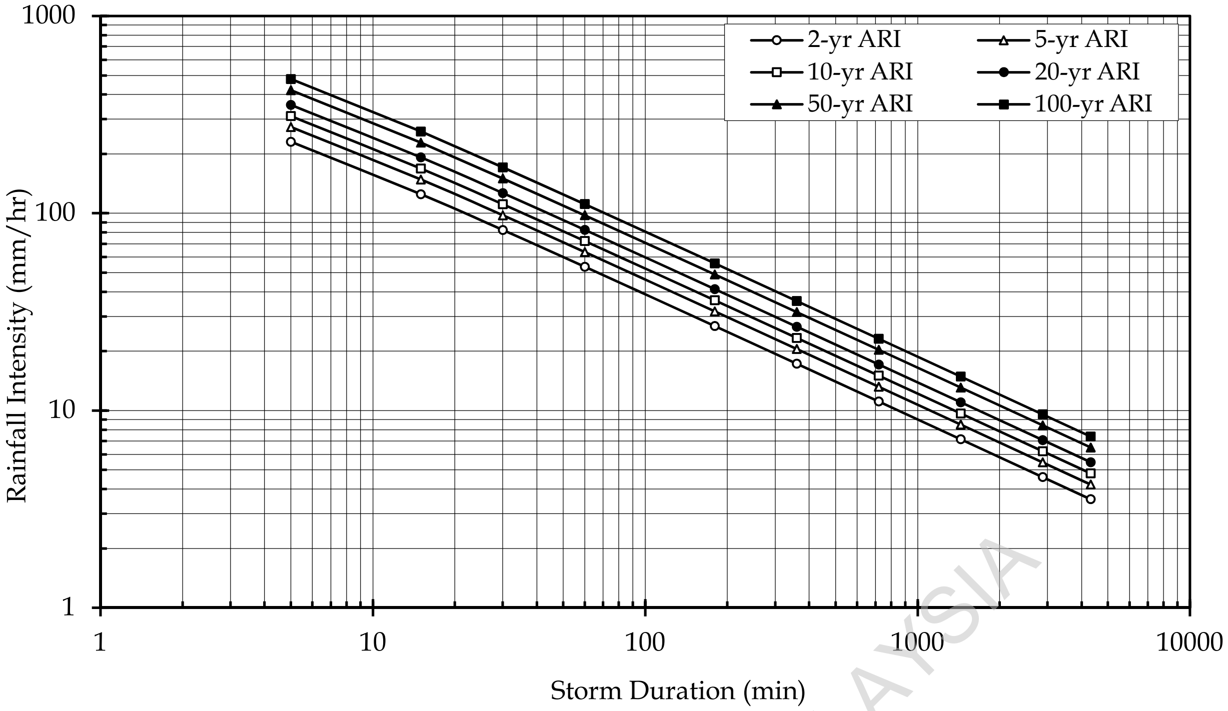


Figure AX3.3.5: Rainfall Station at Kg Aring-4923001

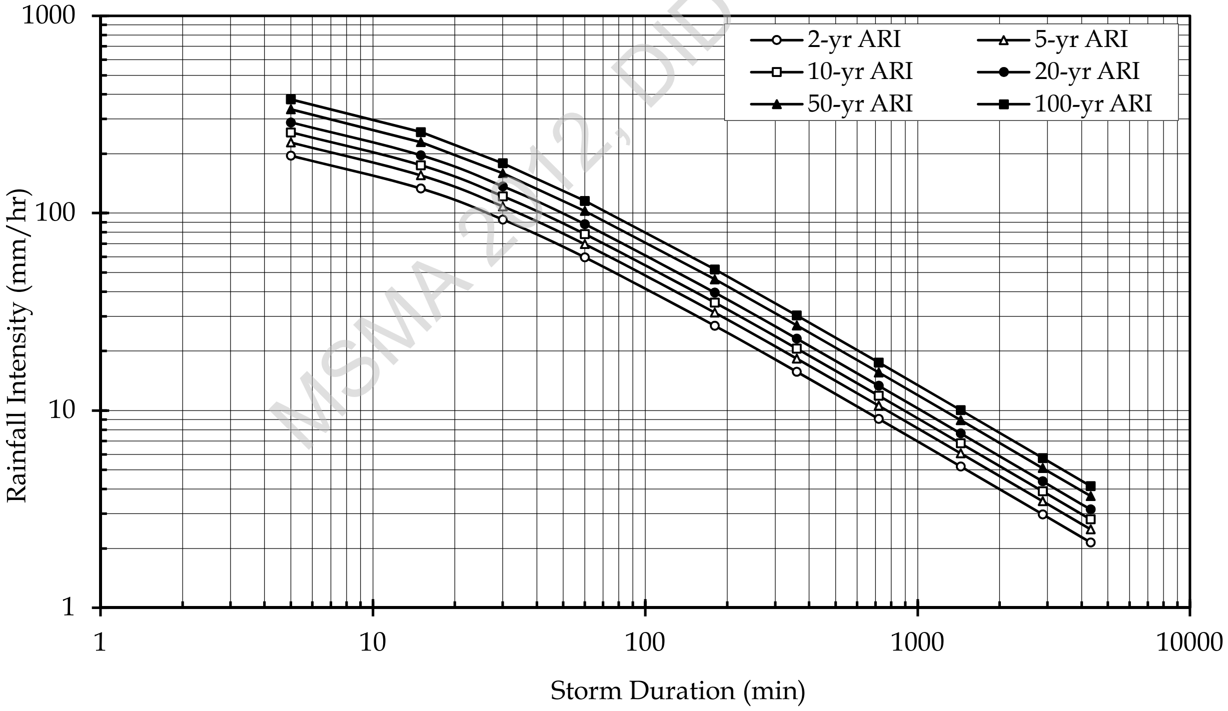


Figure AX3.3.6: Rainfall Station at Balai Polis Bertam-5120025

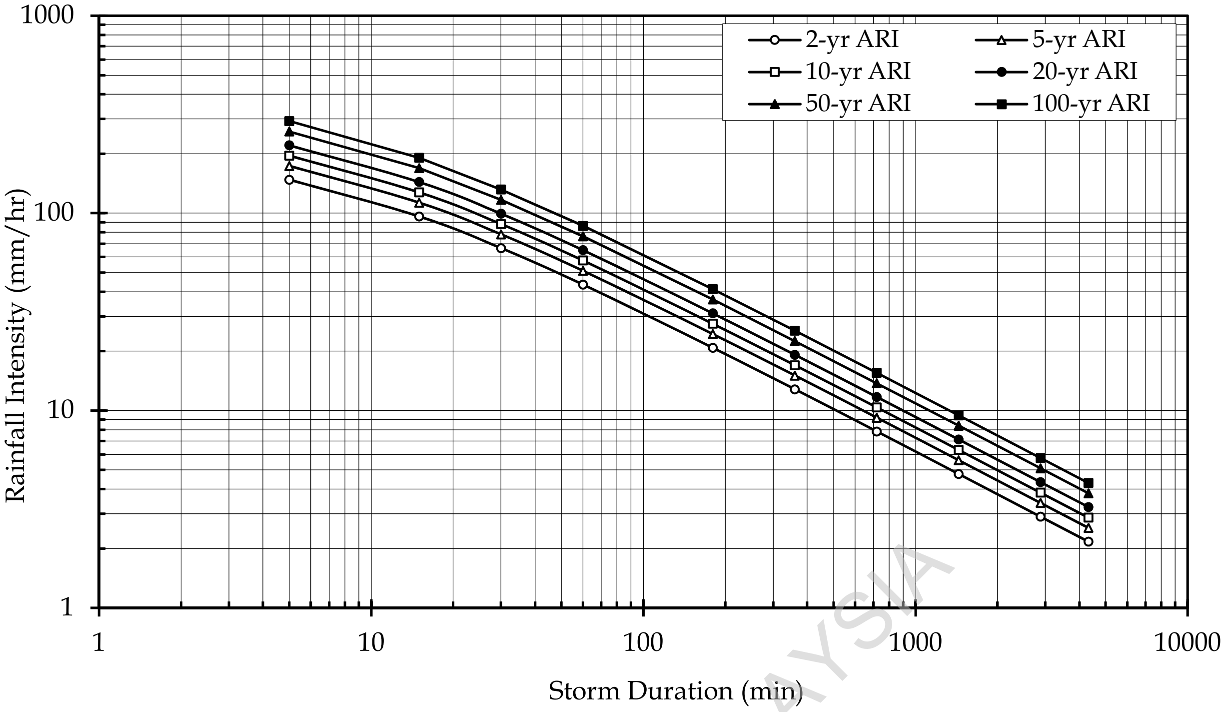


Figure AX3.3.7: Rainfall Station at Gob-5216001

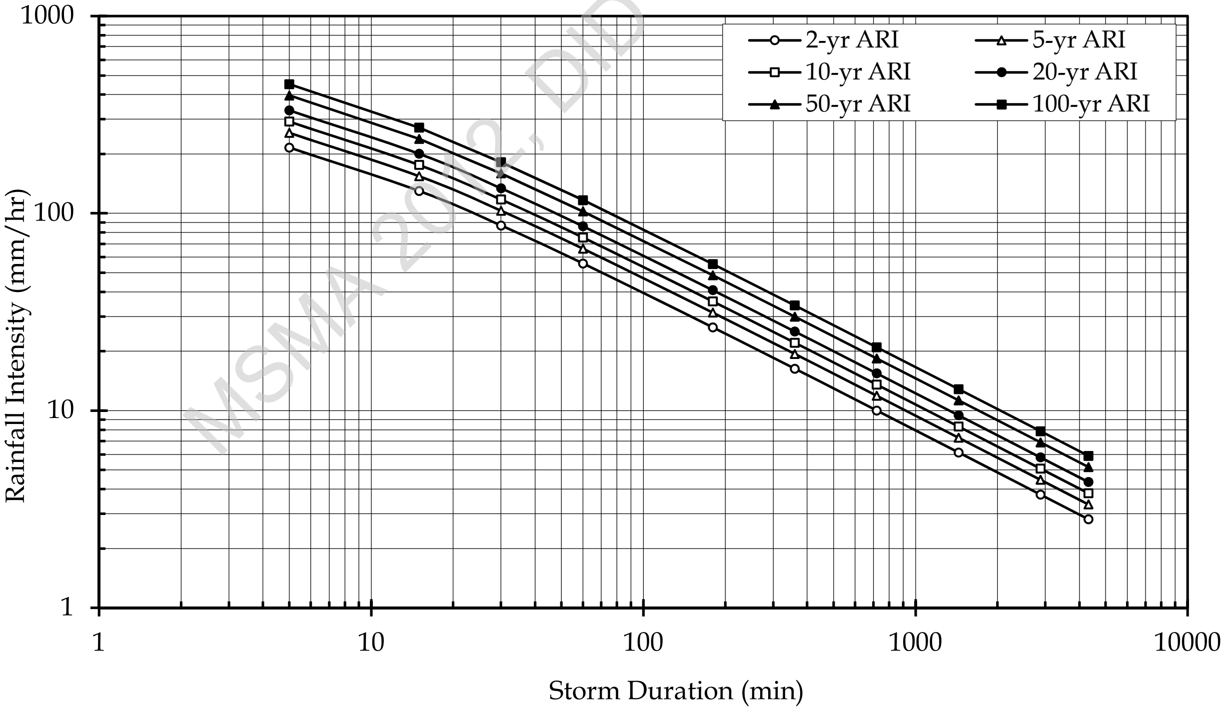


Figure AX3.3.8: Rainfall Station at Dabong-5320038

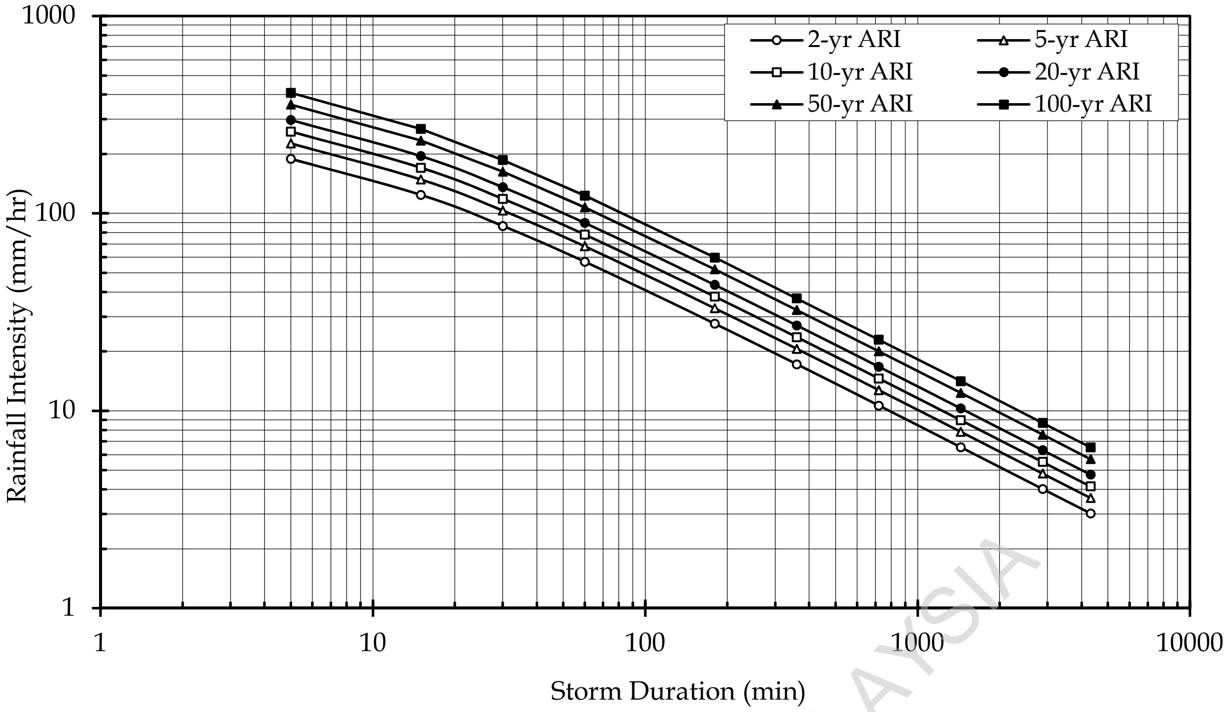


Figure AX3.3.9: Rainfall Station at Kg Lalok-5322044

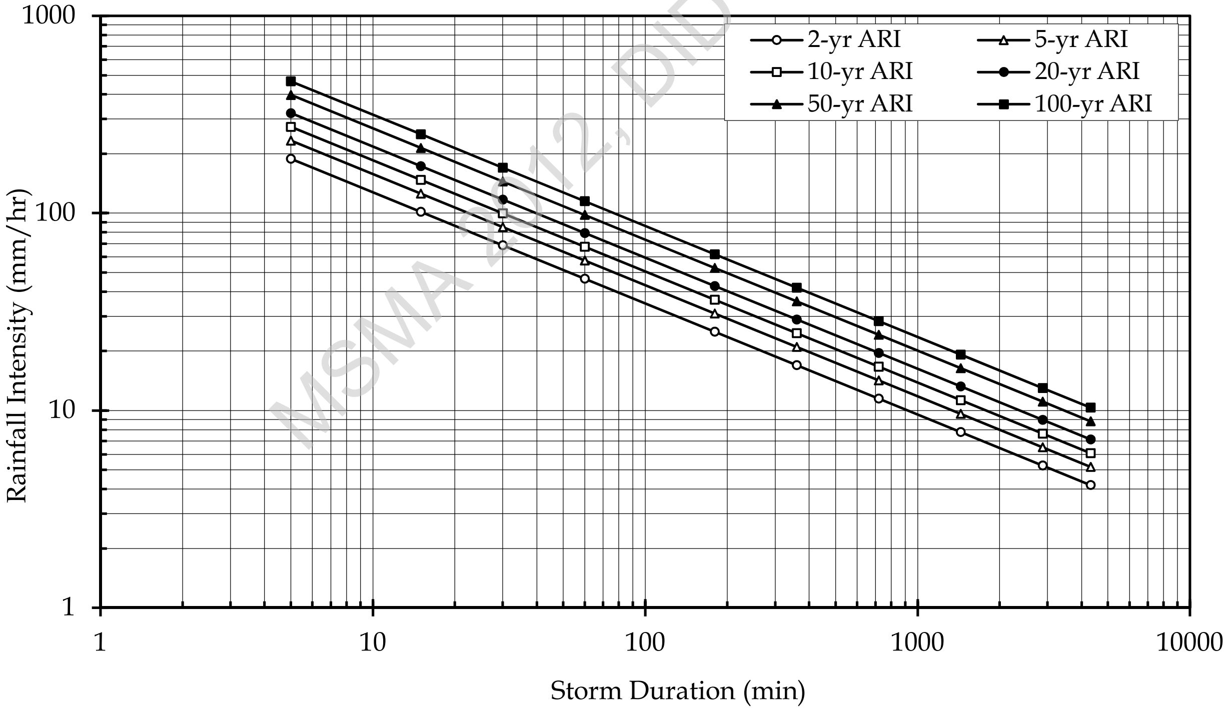


Figure AX3.3.10: Rainfall Station at JPS Kuala Krai-5522047

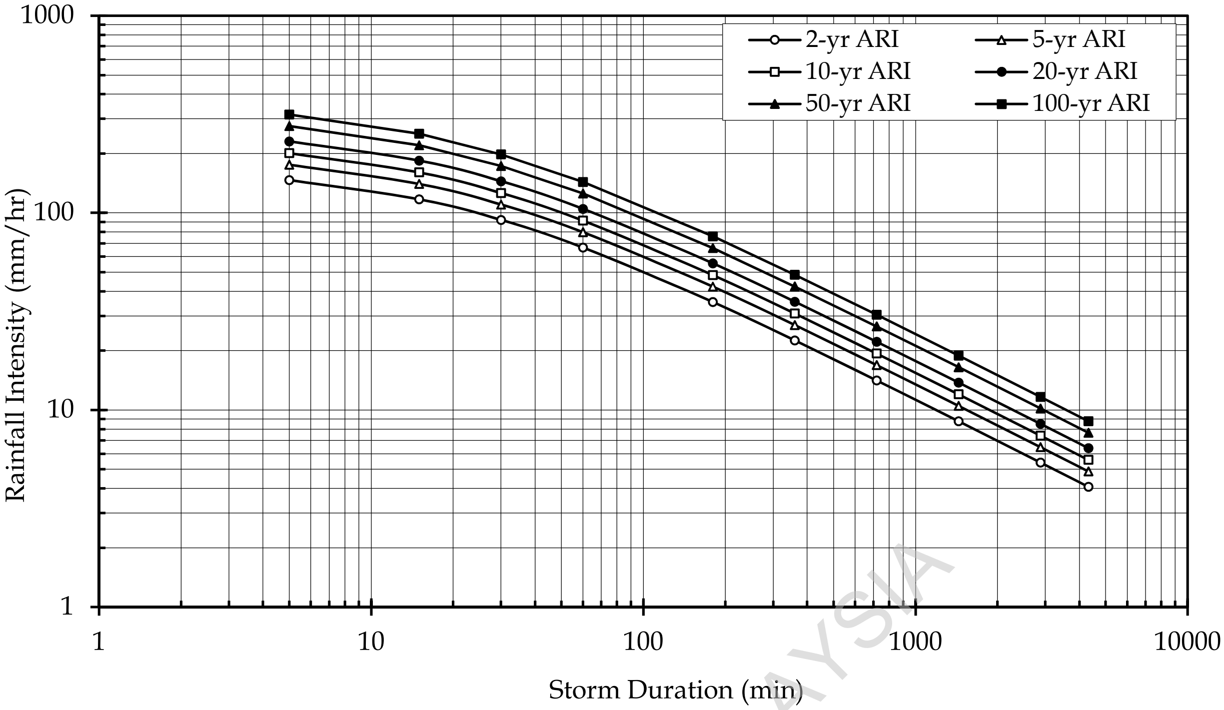


Figure AX3.3.11: Rainfall Station at Kg Jeli, Tanah Merah-5718033

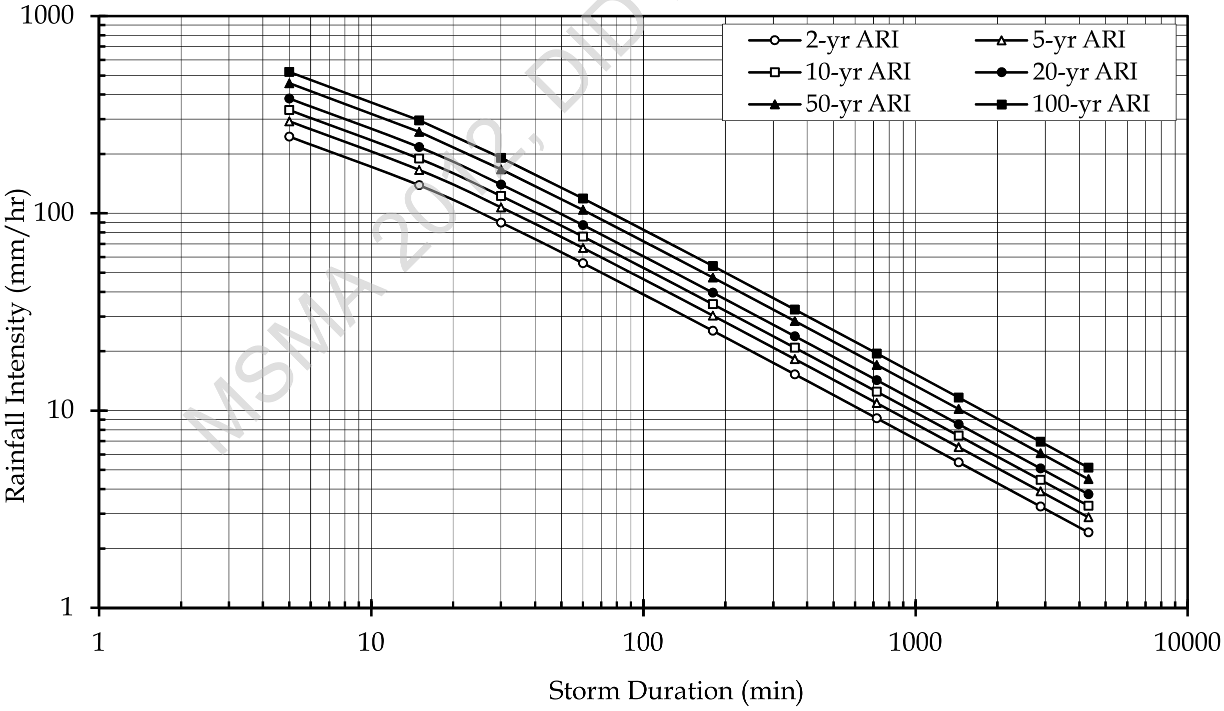


Figure AX3.3.12: Rainfall Station at Kg Durian Daun Lawang-5719001

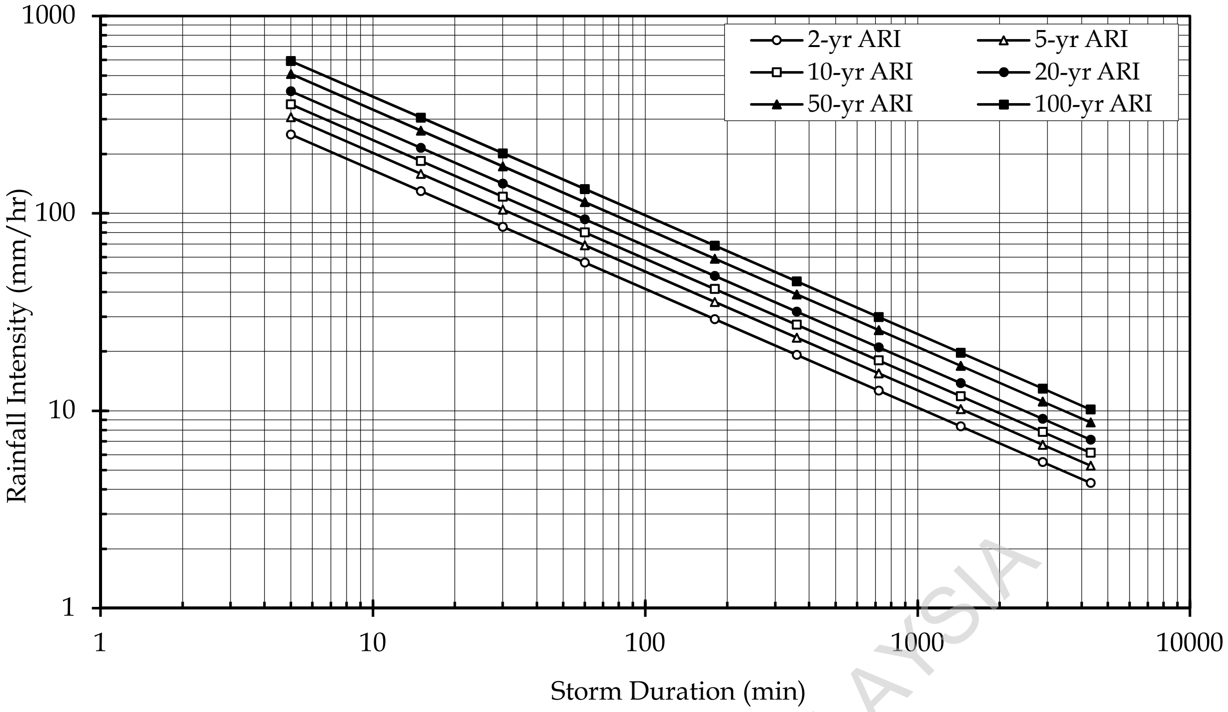


Figure AX3.3.13: Rainfall Station at JPS Machang-5722057

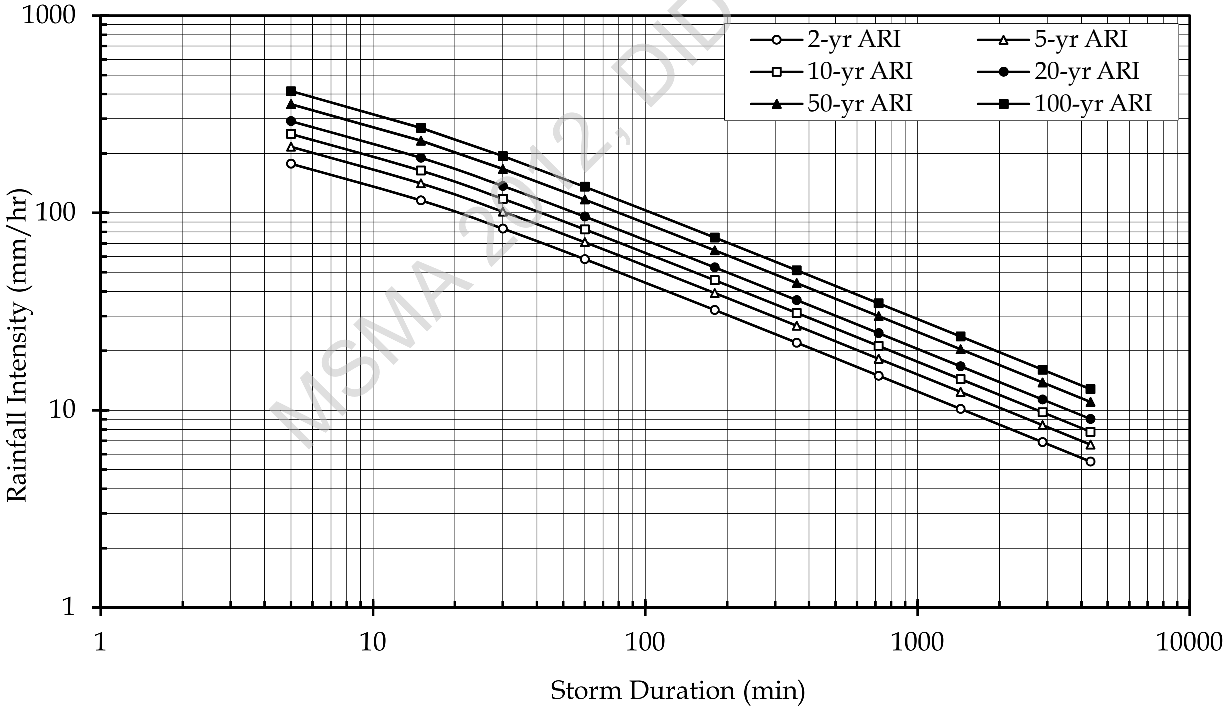


Figure AX3.3.14: Rainfall Station at Sg Rasau Pasir Putih-5824079

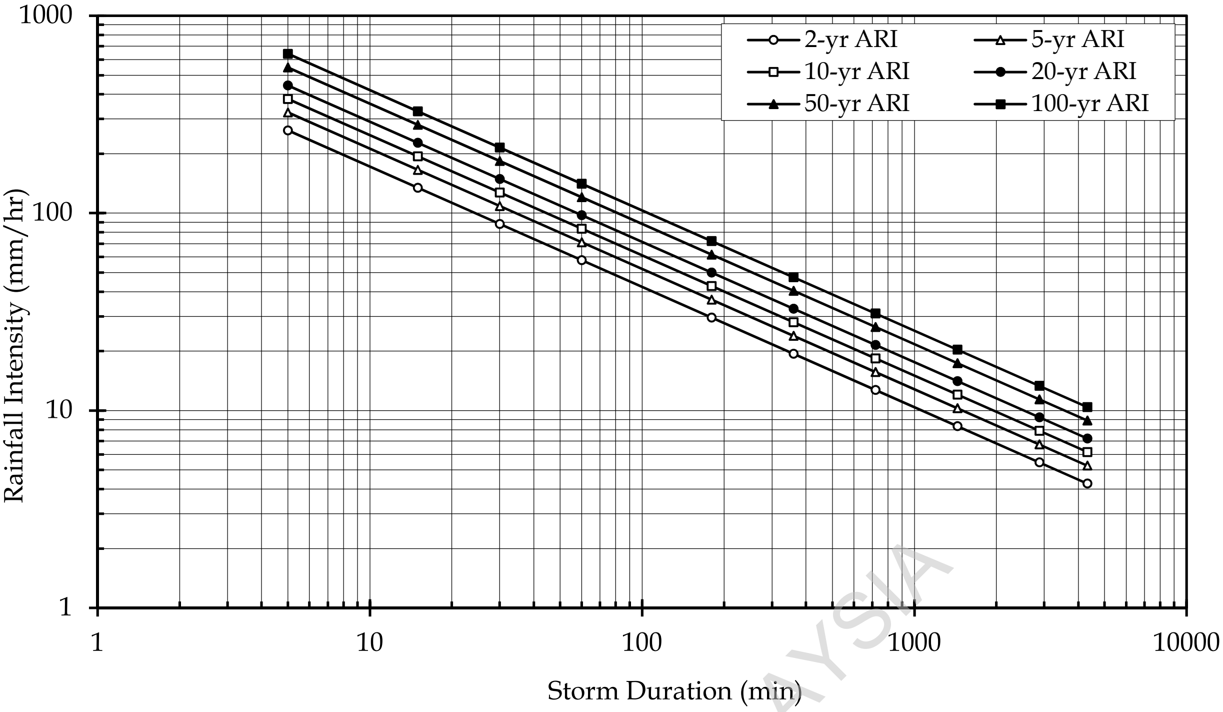


Figure AX3.3.15: Rainfall Station at Rumah Kastam R Pjg-6019004

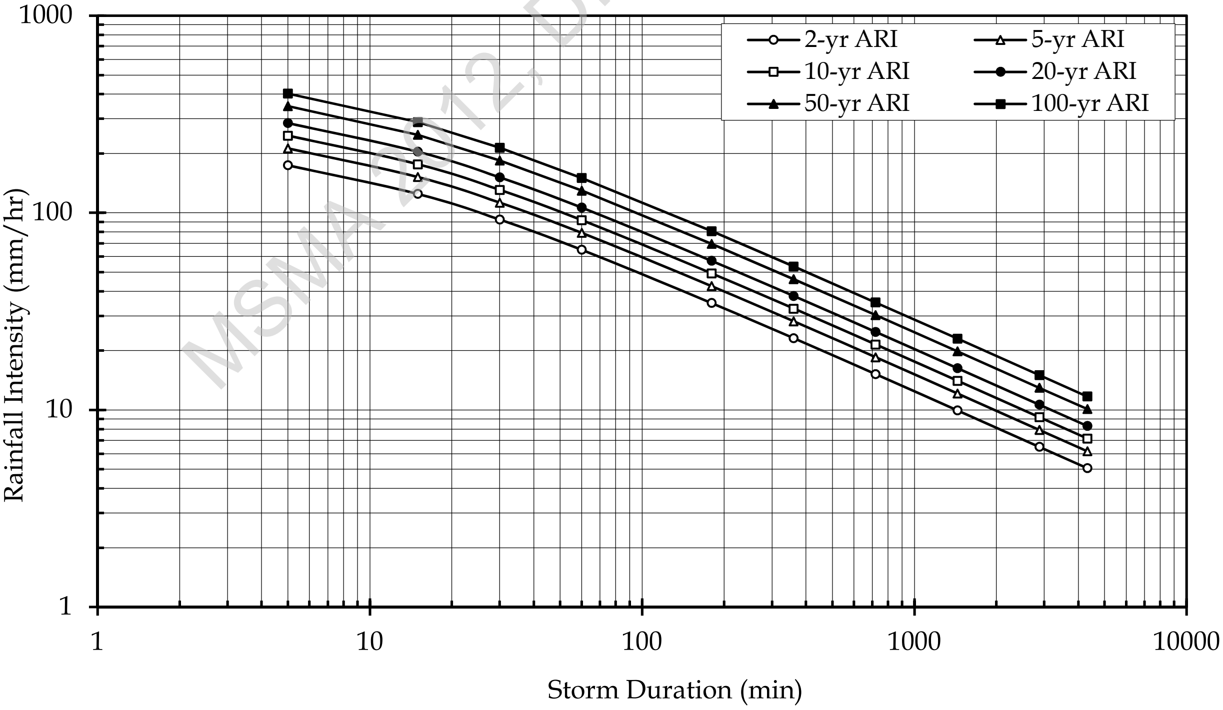


Figure AX3.3.16: Rainfall Station at Setor JPS Kota Bharu-6122064

AX3.4 FEDERAL TERRITORY OF KUALA LUMPUR

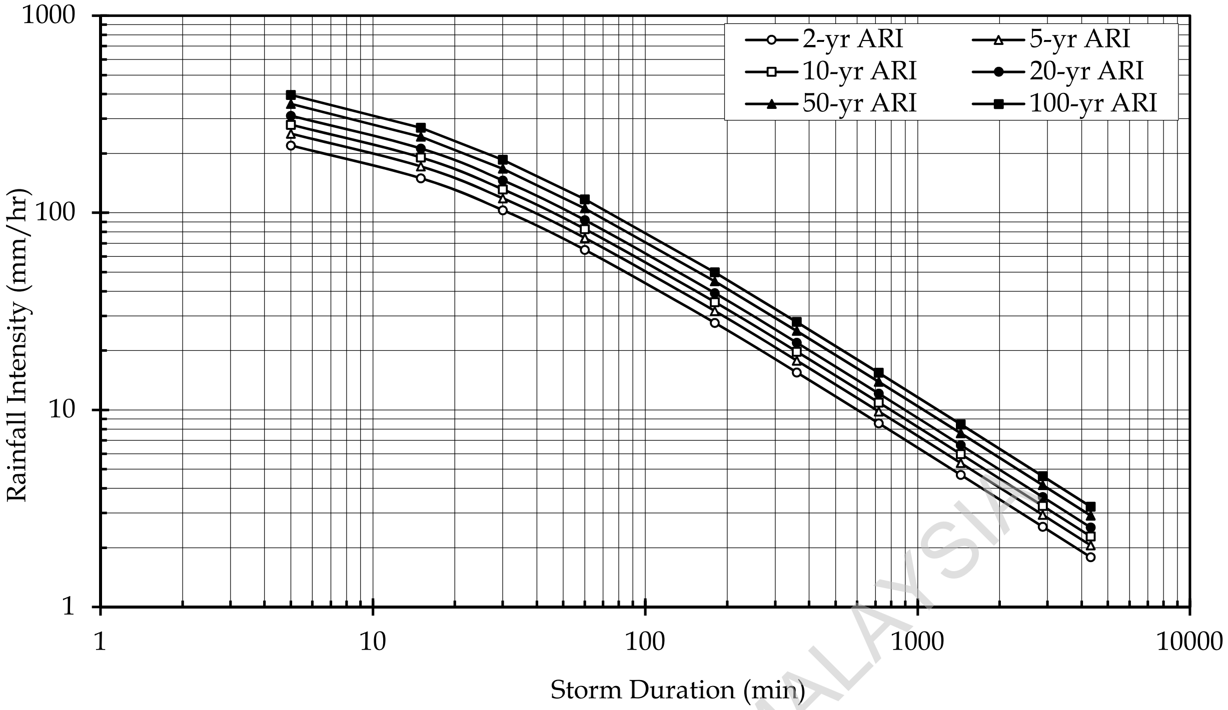


Figure AX3.4.1: Rainfall Station at Puchong Drop, K Lumpur-3015001

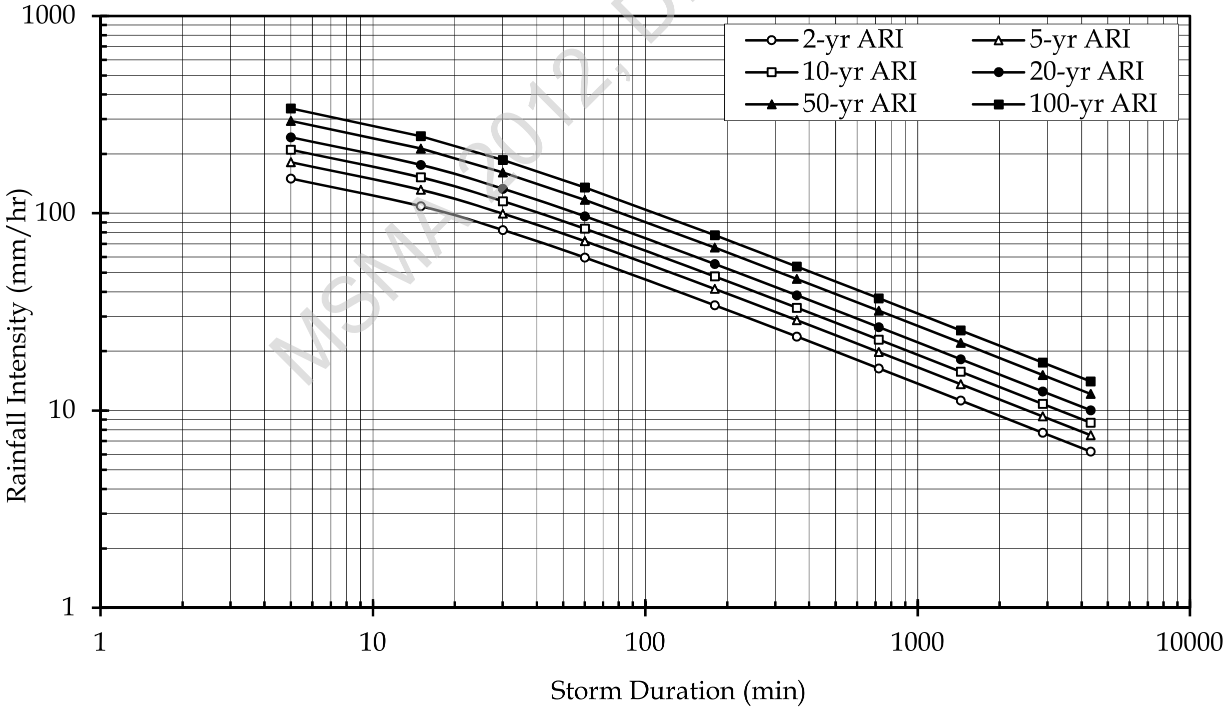


Figure AX3.4.2: Rainfall Station at Ibu Pejabat JPS-3116003

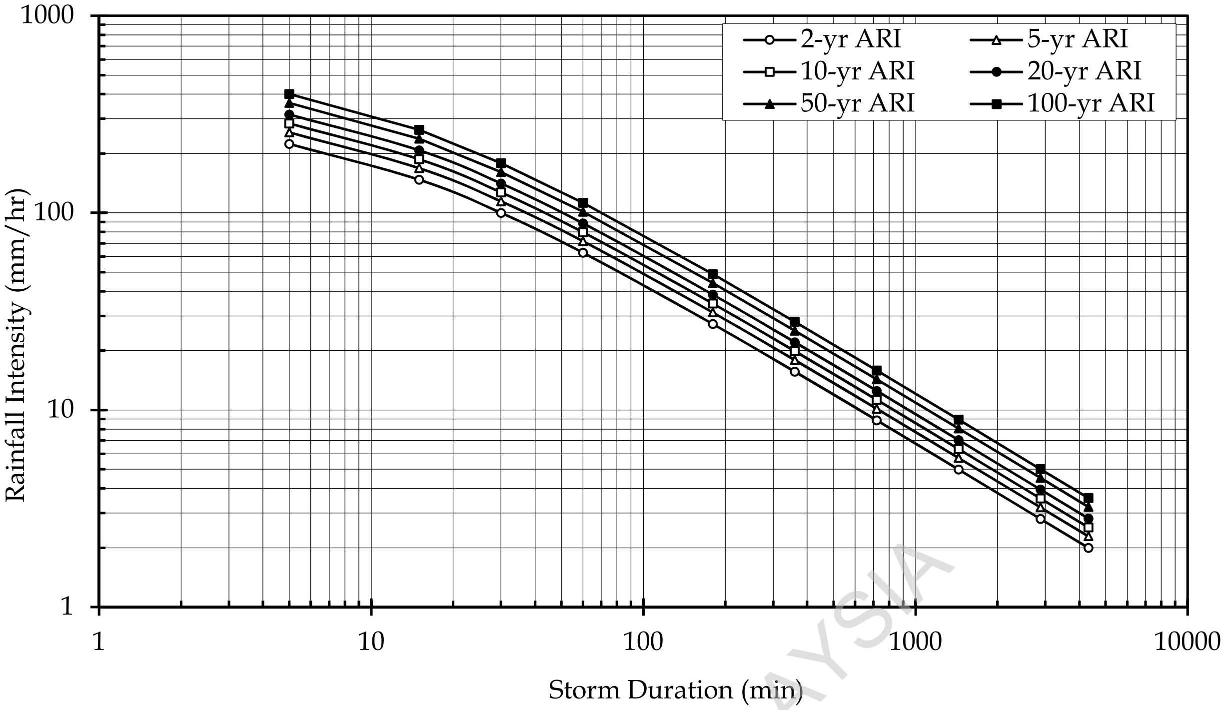


Figure AX3.4.3: Rainfall Station at Ibu Pejabat JPS-3116004

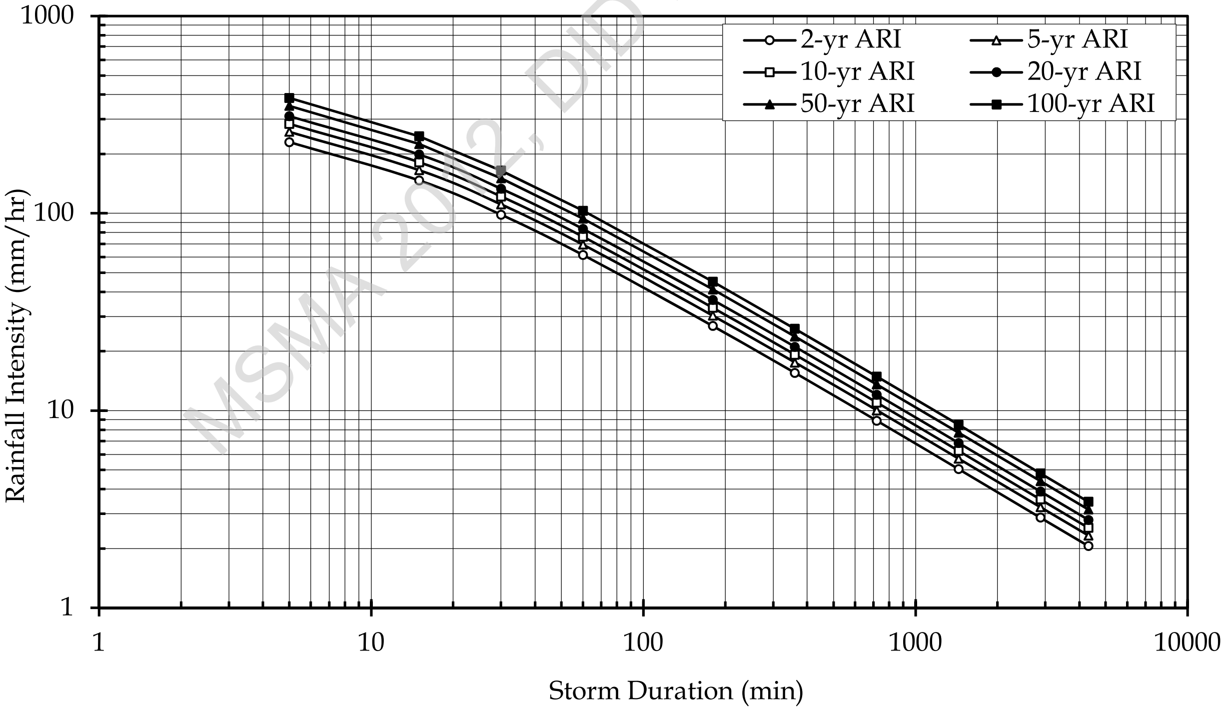


Figure AX3.4.4: Rainfall Station at SK Taman Maluri -3116005

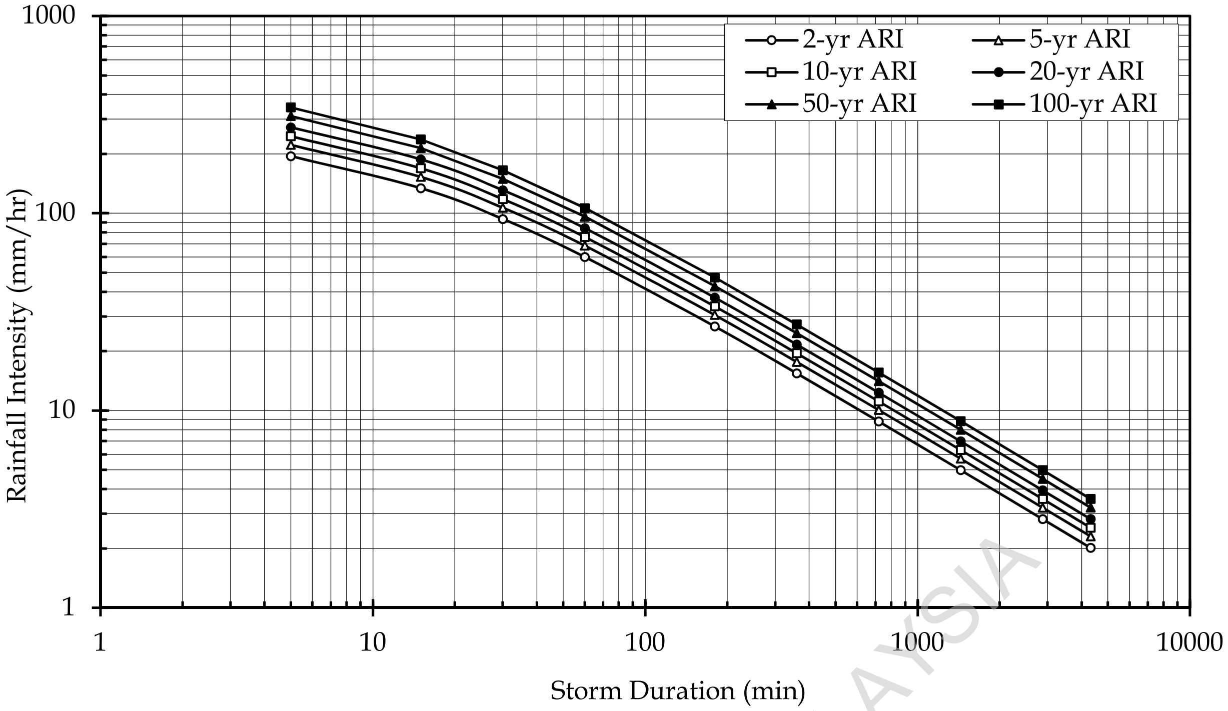


Figure AX3.4.5: Rainfall Station at Edinburgh Kg -3116006

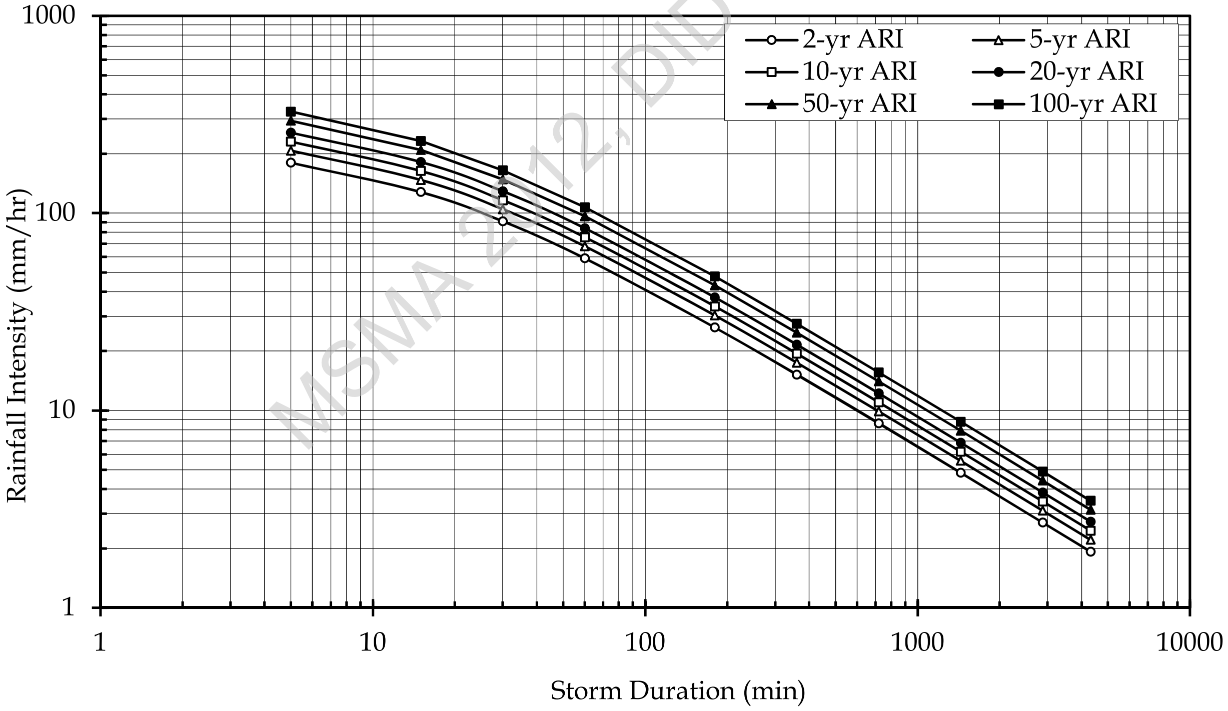


Figure AX3.4.6: Rainfall Station at Sungai Tua-3216001

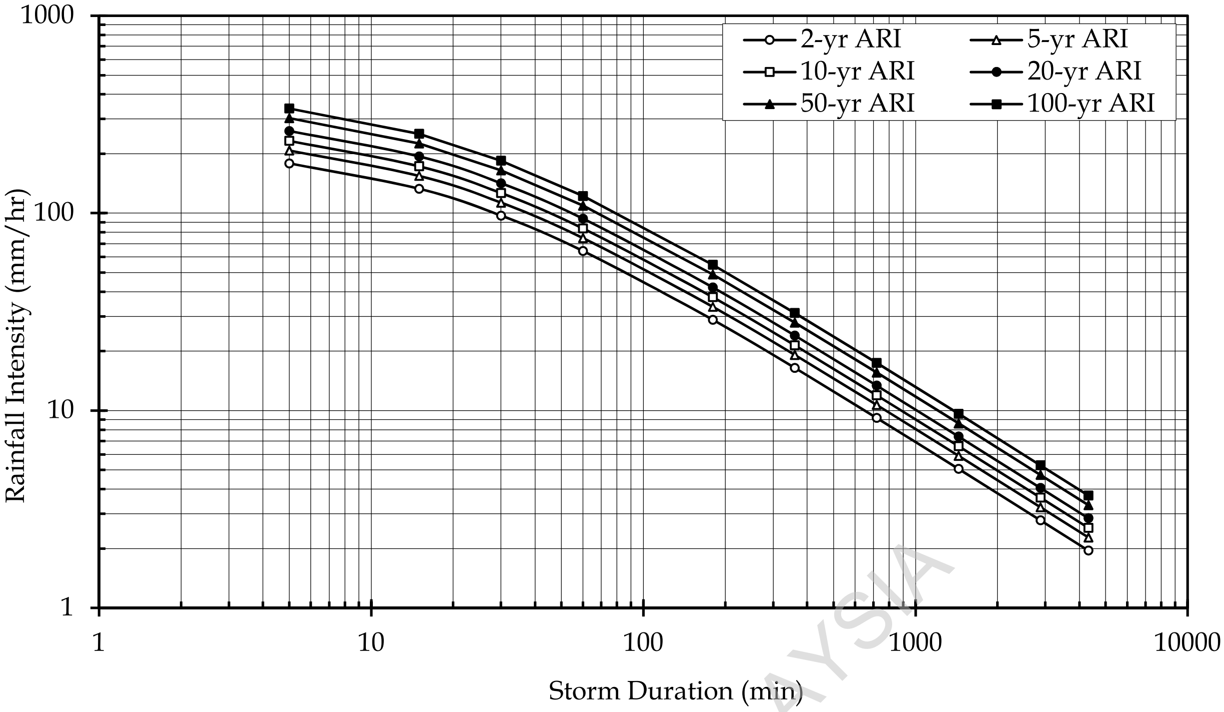


Figure AX3.4.7: Rainfall Station at SK Jenis Keb. Kepong -3216004

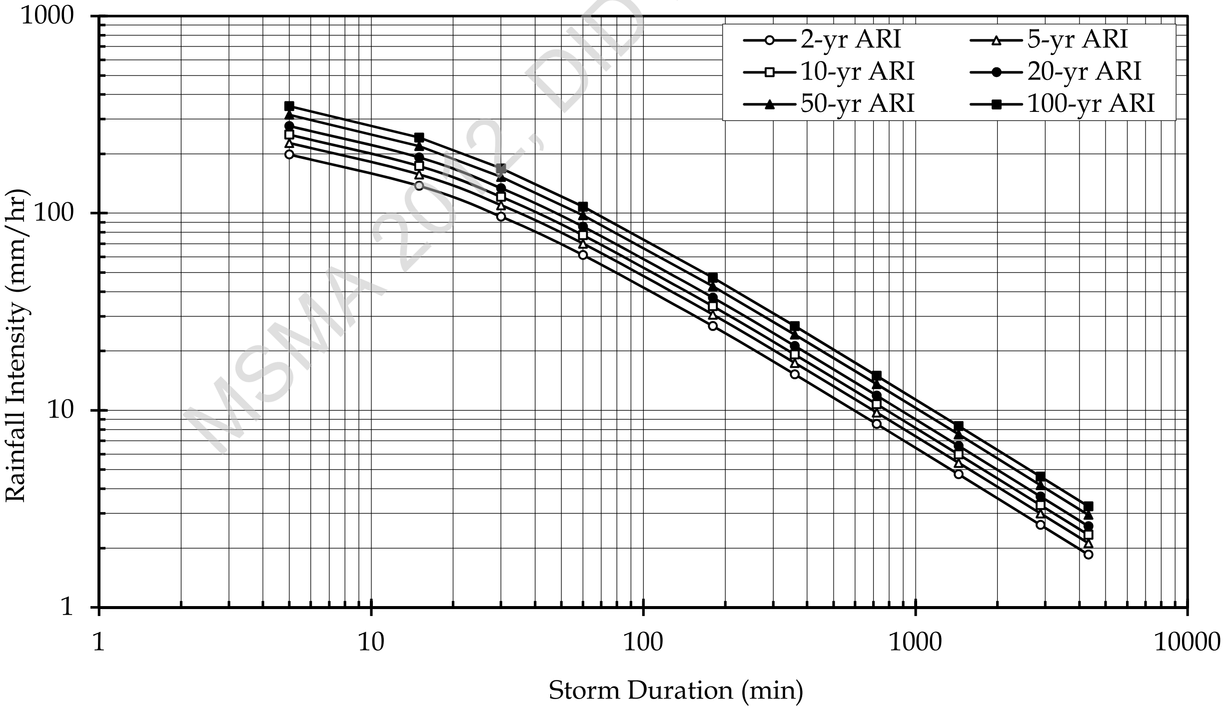


Figure AX3.4.8: Rainfall Station at Ibu Bek. KM16, Gombak-3217001

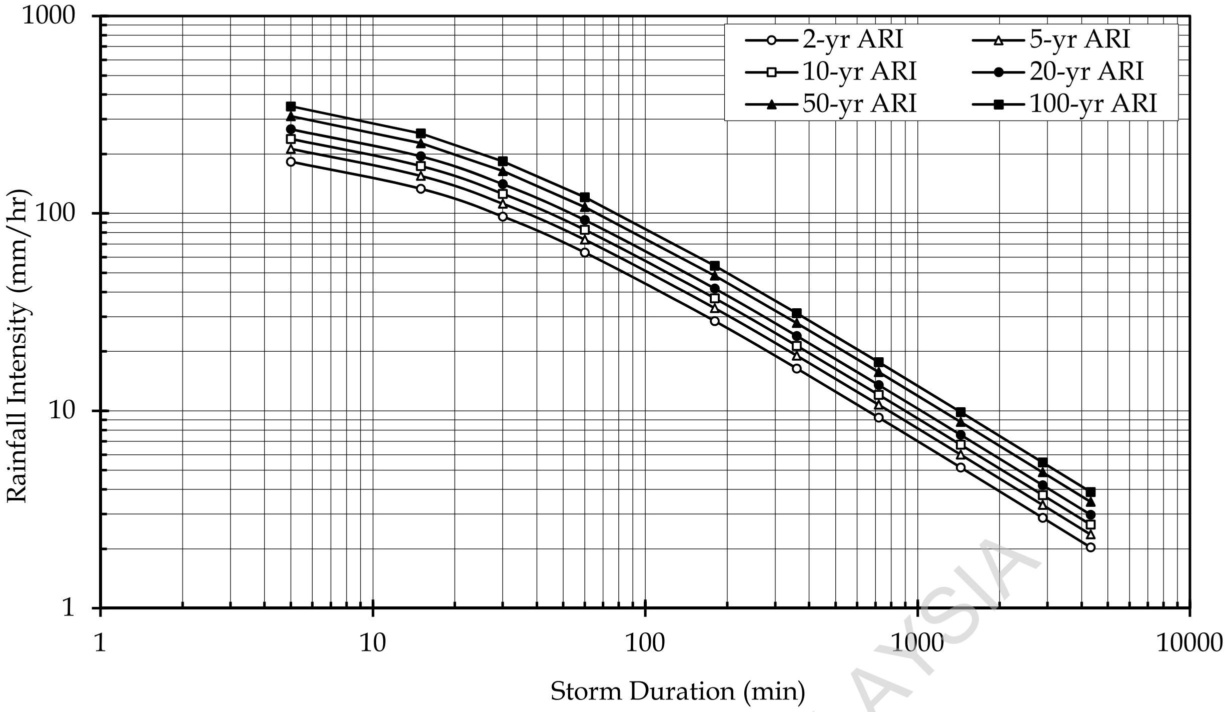


Figure AX3.4.9: Rainfall Station at Emp. Genting Kelang-3217002

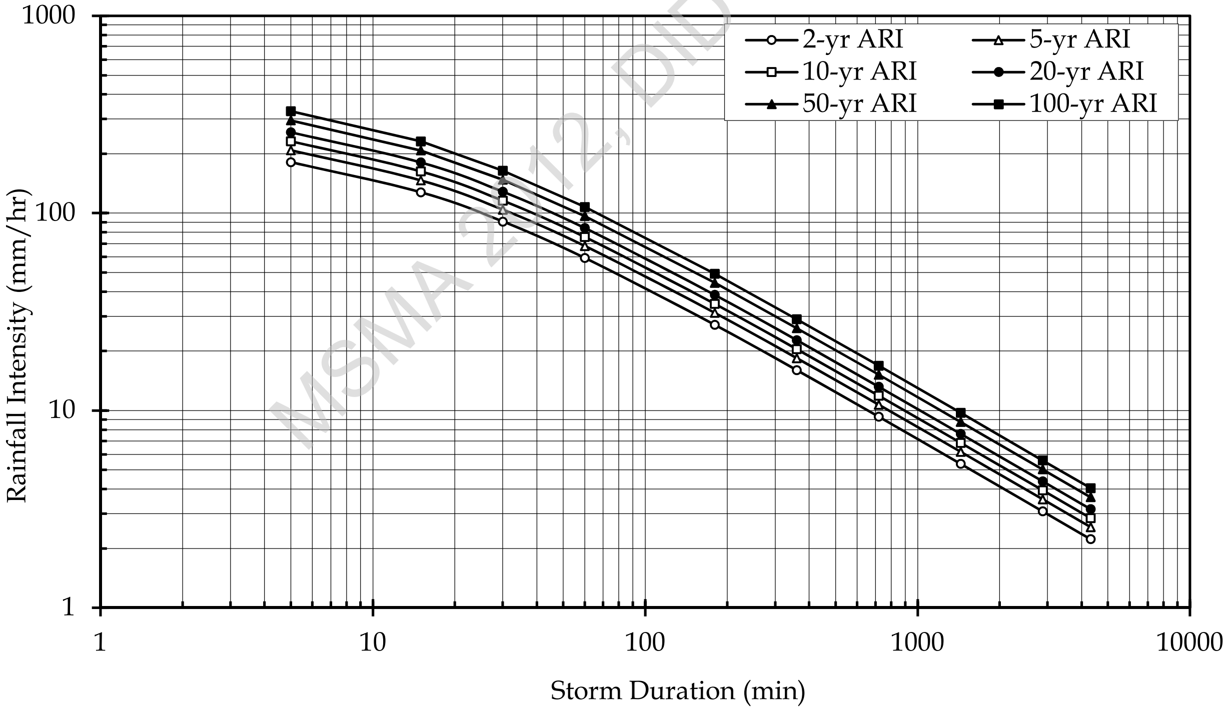


Figure AX3.4.10: Rainfall Station at Ibu Bek. KM11, Gombak-3217003

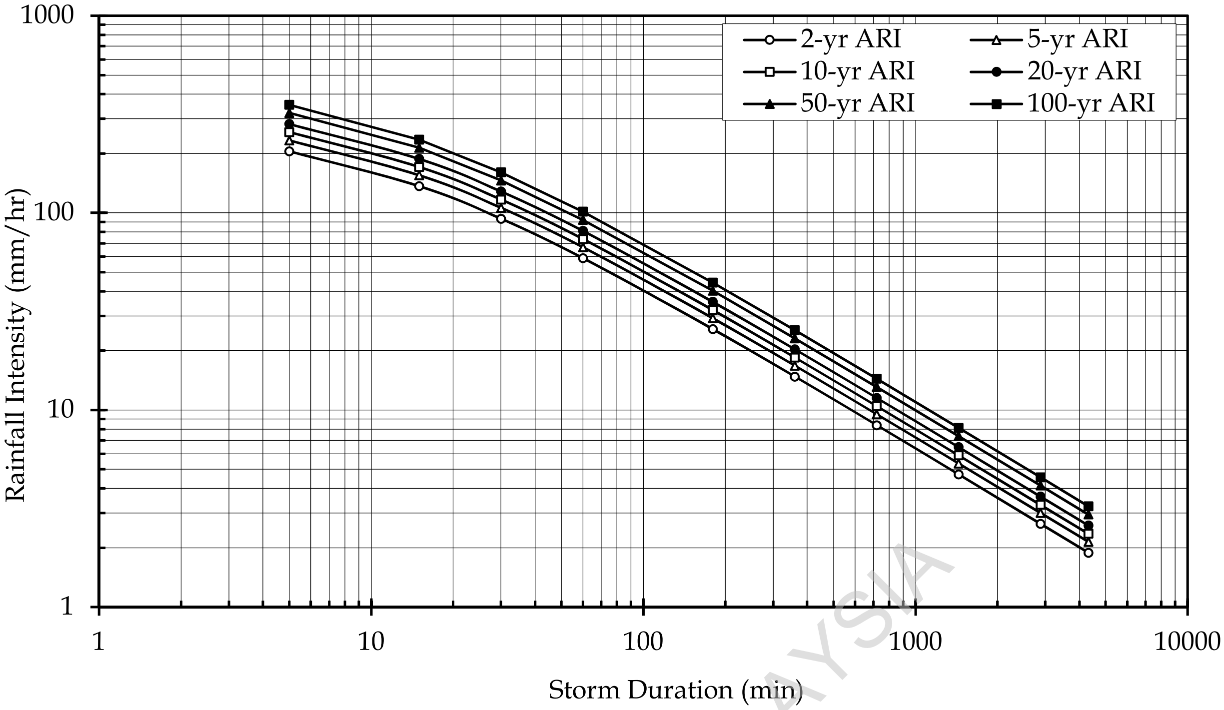


Figure AX3.4.11: Rainfall Station at Kg. Kuala Seleh, H. Klg-3217004

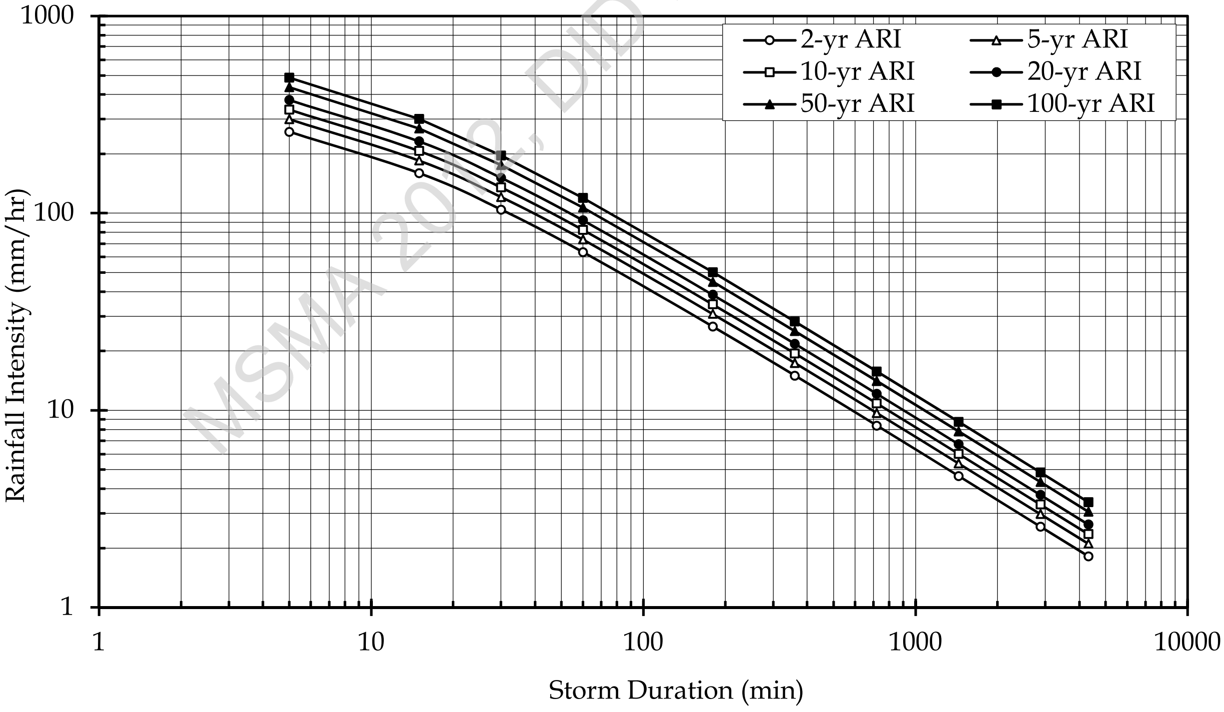


Figure AX3.4.12: Rainfall Station at Kg. Kerdas, Gombak -3217005

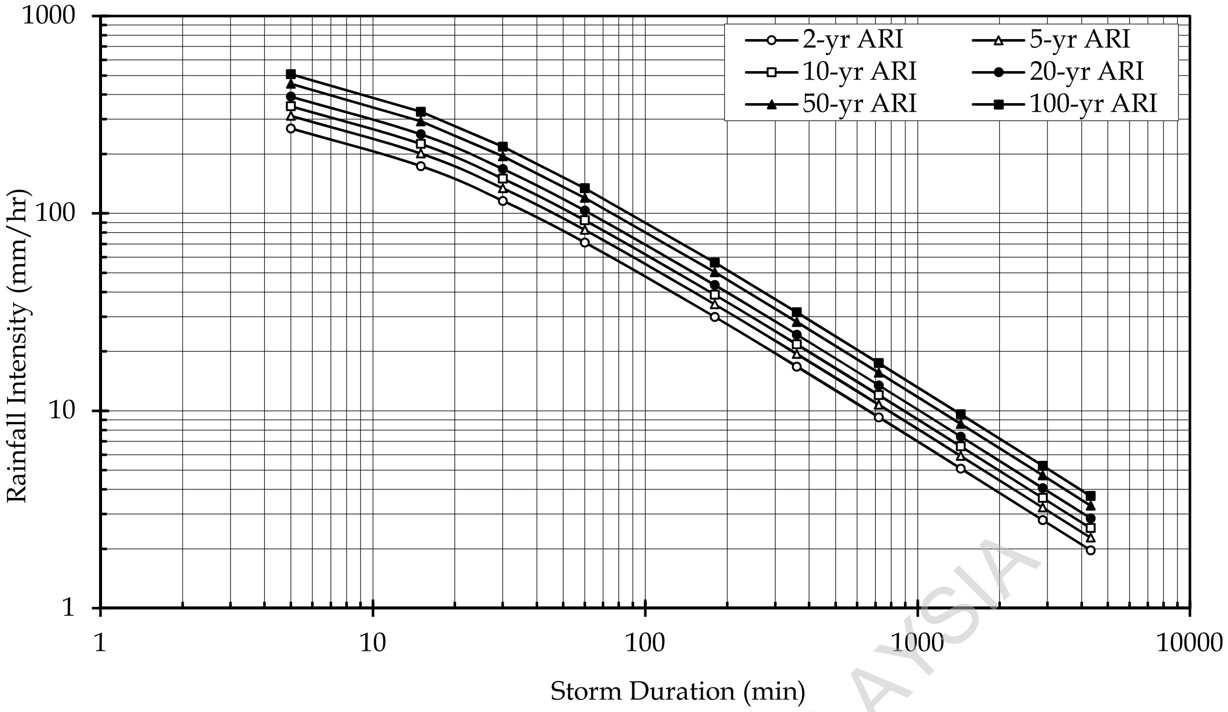


Figure AX3.4.13: Rainfall Station at Air Terjun Sg. Batu -3317001

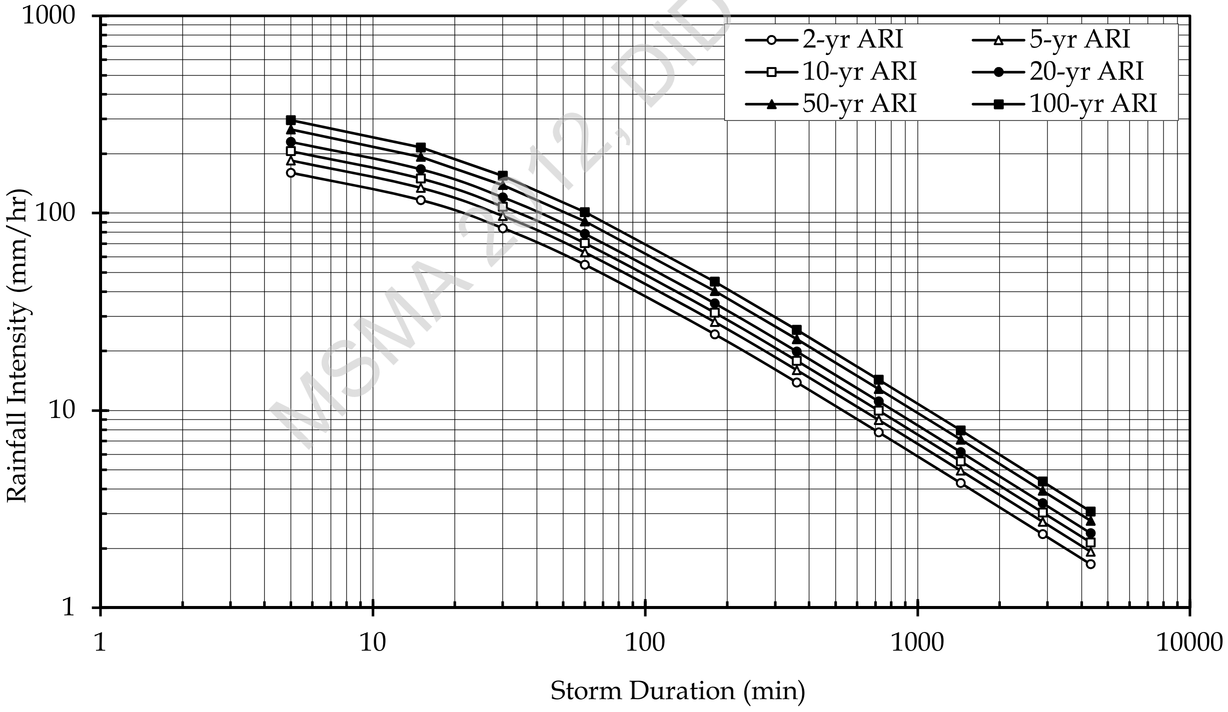


Figure AX3.4.14: Rainfall Station at Genting Sempah-3317004

AX3.5 STATE OF MELAKA

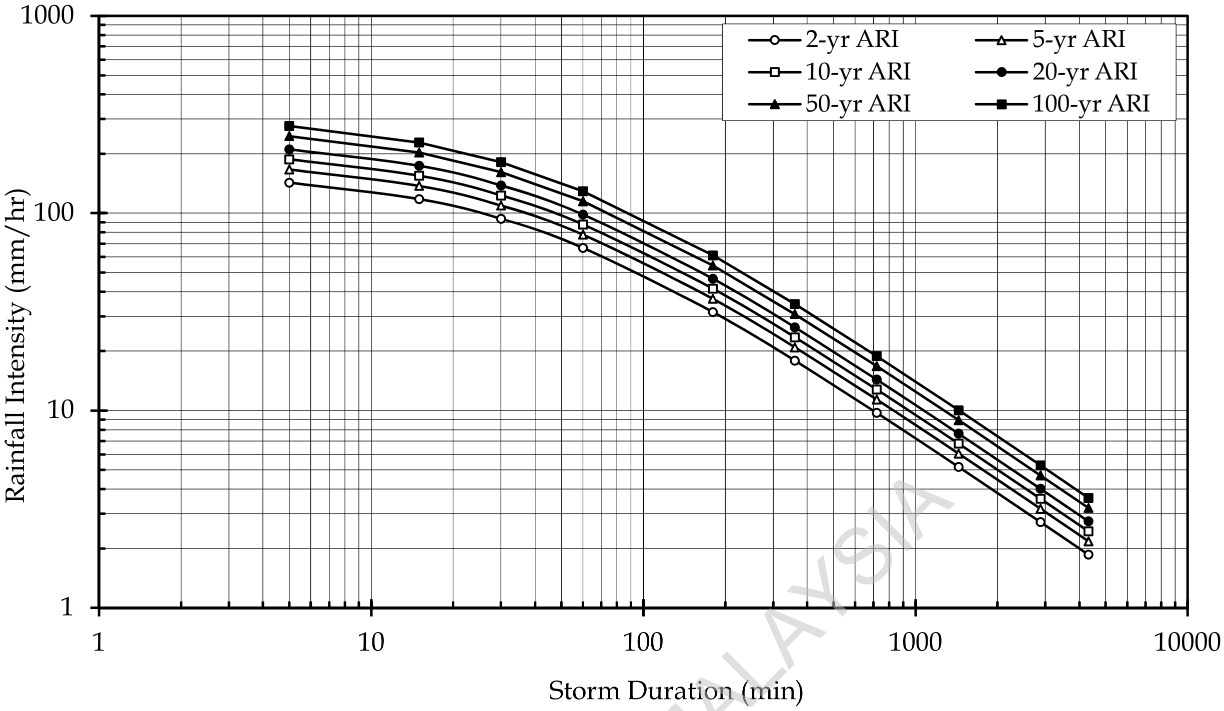


Figure AX3.5.1: Rainfall Station at Bukit Sebukor-2222001

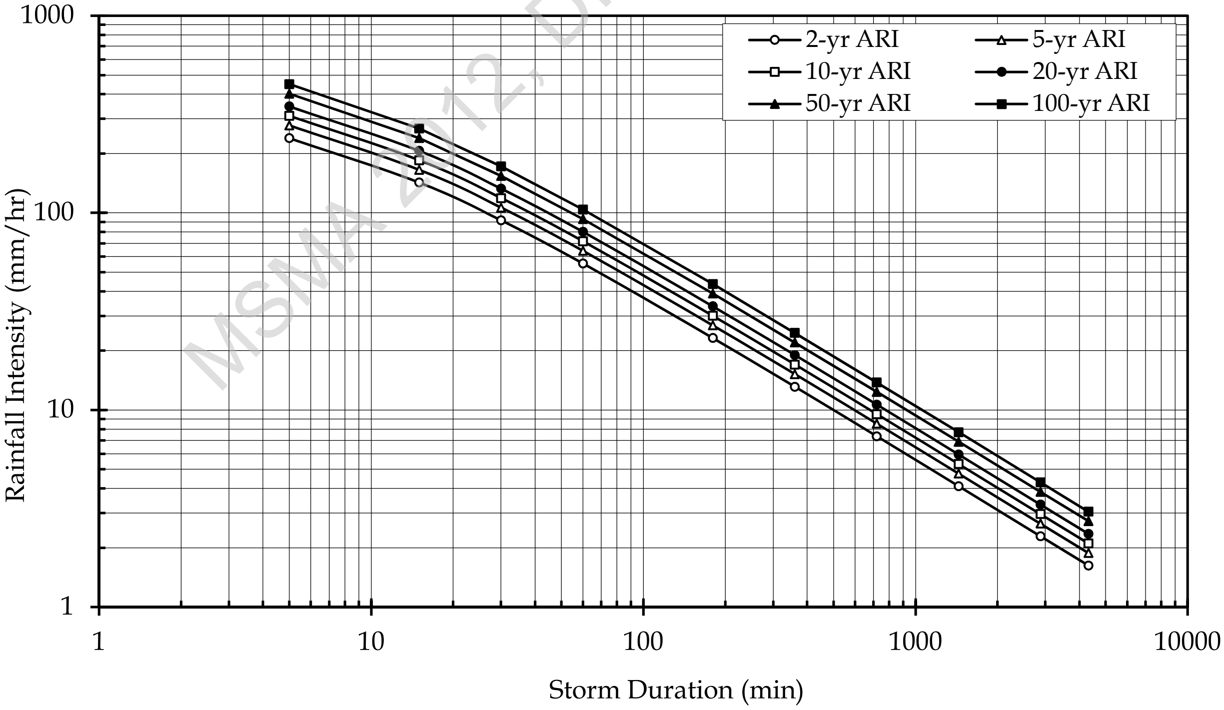


Figure AX3.5.2: Rainfall Station at Chin Chin Tepi Jalan-2224038

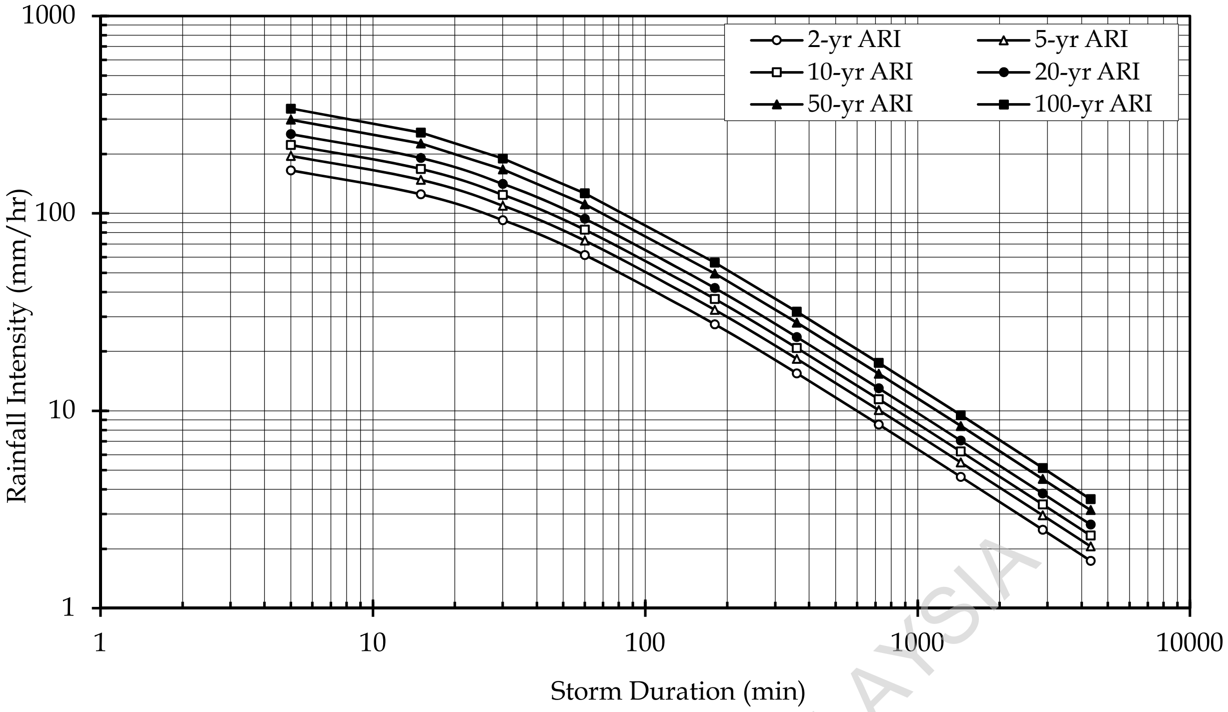


Figure AX3.5.3: Rainfall Station at Ladang Lendu-2321006

AX3.6 STATE OF NEGERI SEMBILAN

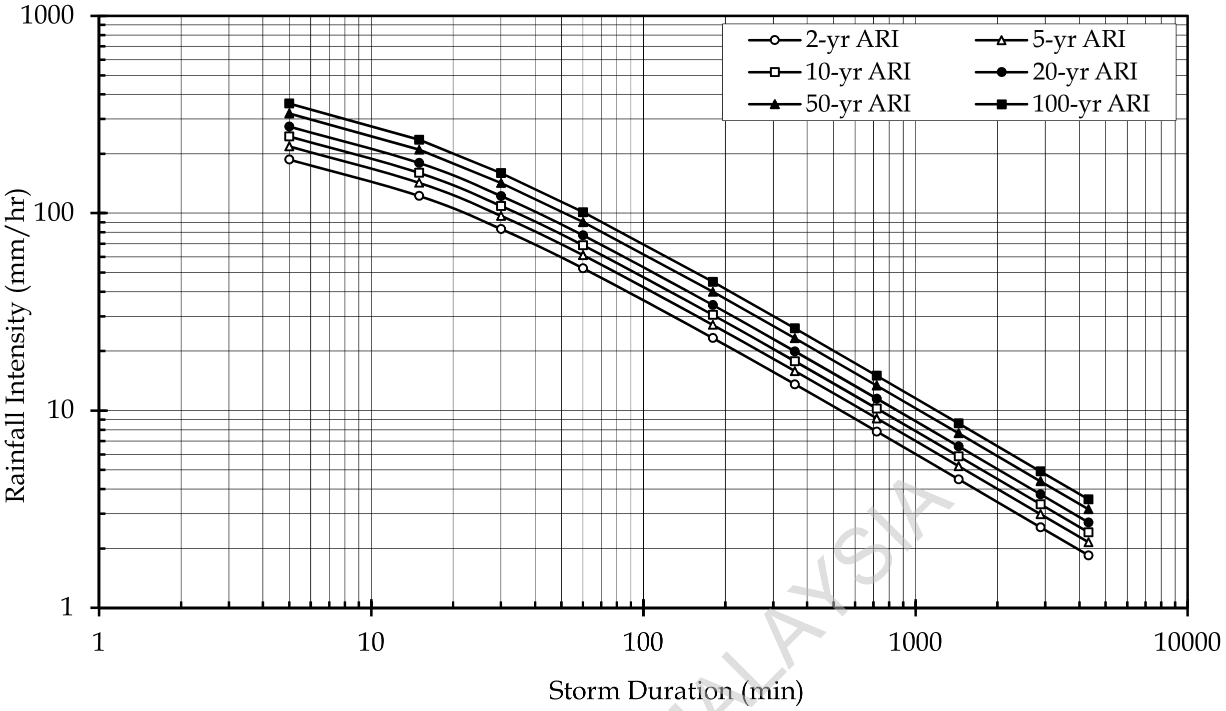


Figure AX3.6.1: Rainfall Station at Setor JPS Sikamat-2719001

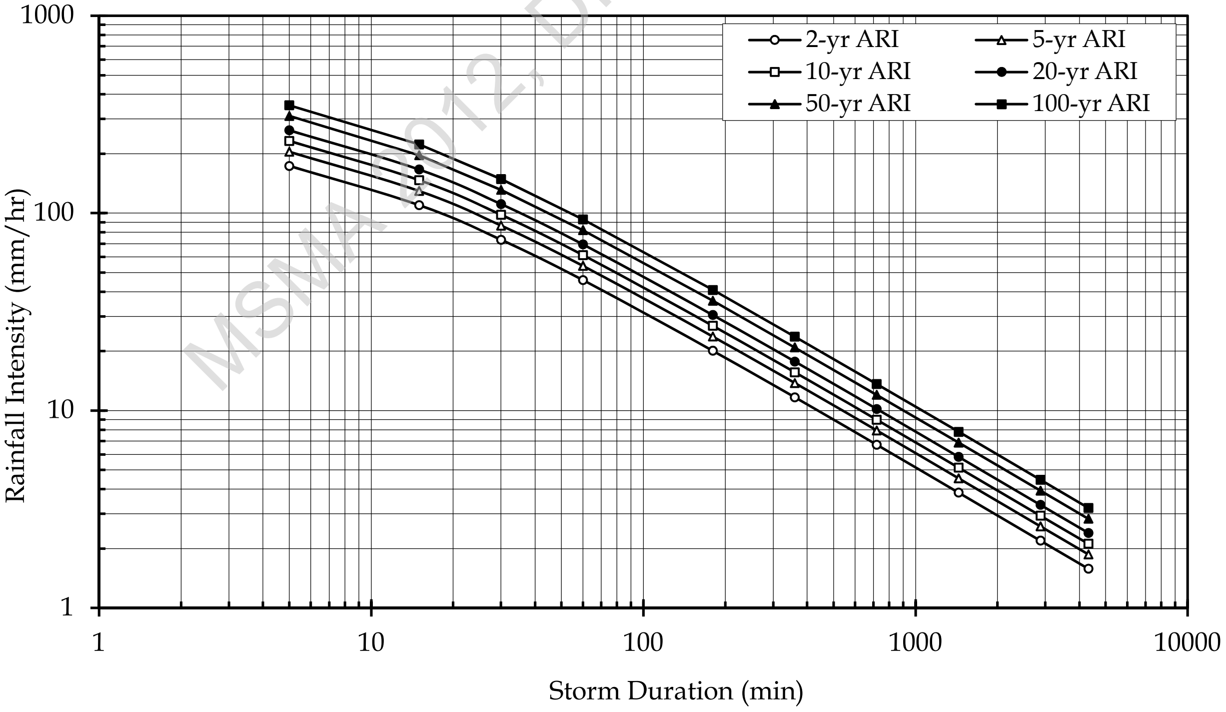


Figure AX3.6.2: Rainfall Station at Kg Sawah Lebar K Pilah-2722202

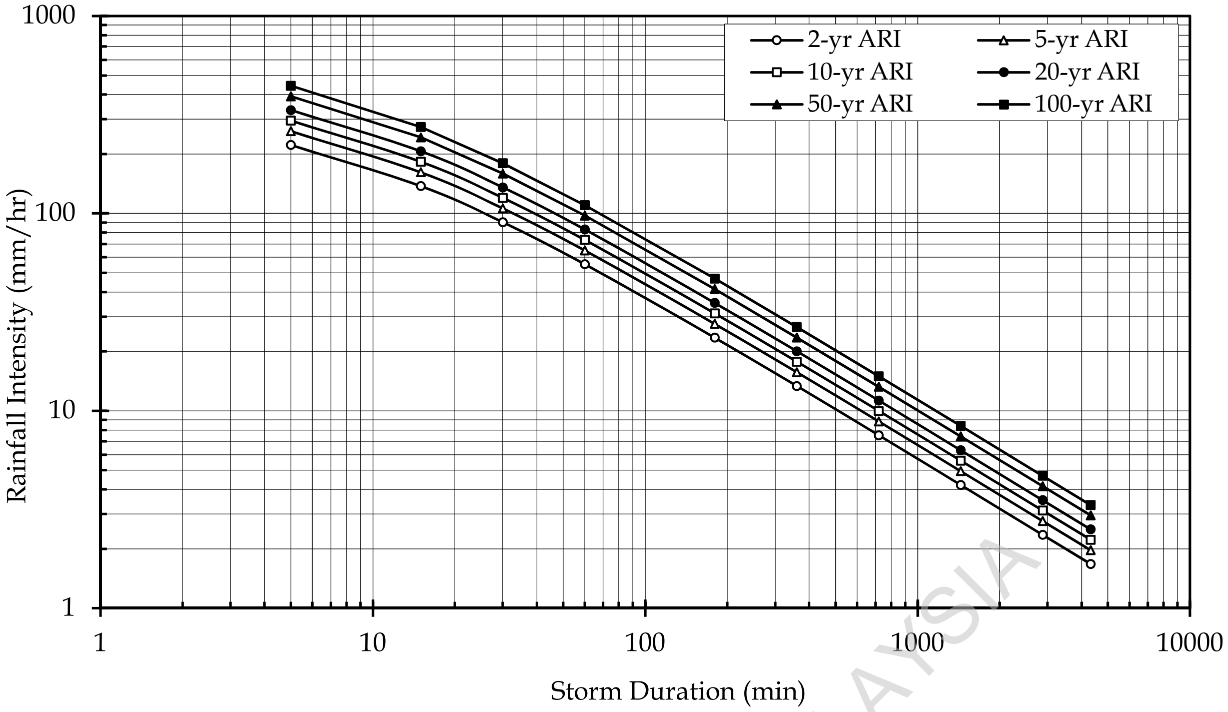


Figure AX3.6.3: Rainfall Station at Sungai Kepis-2723002

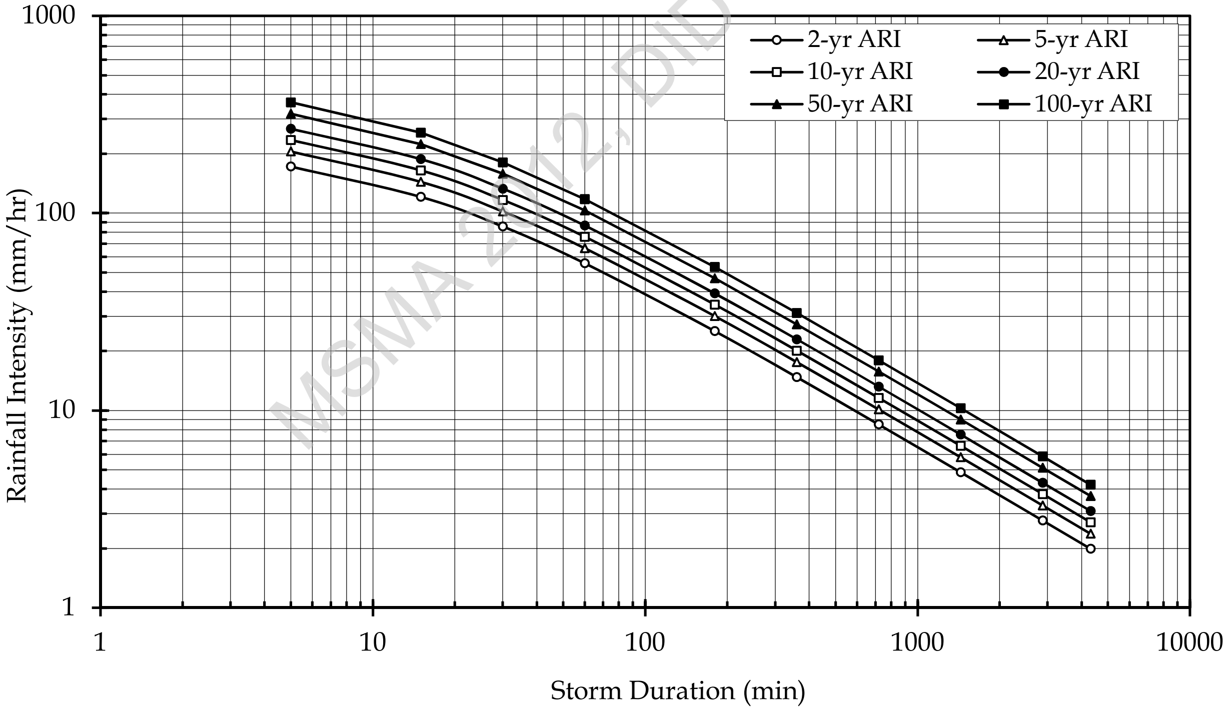


Figure AX3.6.4: Rainfall Station at Ladang New Rompin-2725083

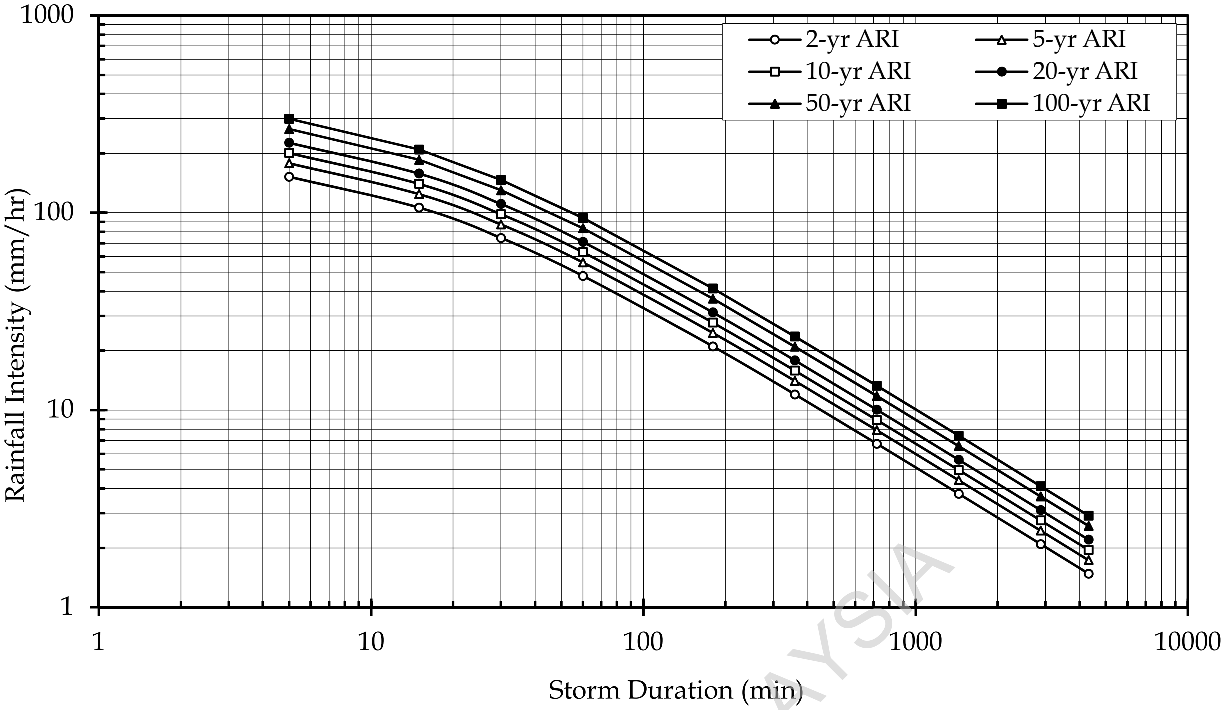


Figure AX3.6.5: Rainfall Station at Petaling K Kelawang-2920012

AX3.7 STATE OF PAHANG

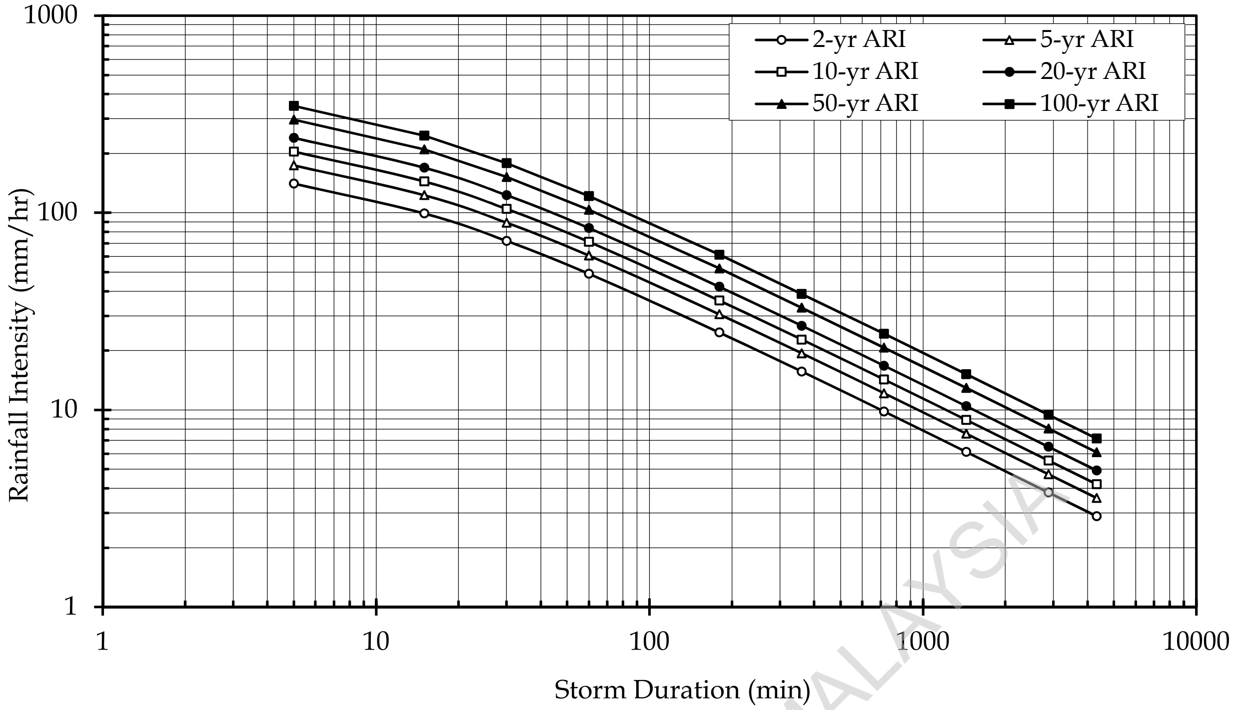


Figure AX3.7.1: Rainfall Station at Sg.Pukim – 2630001

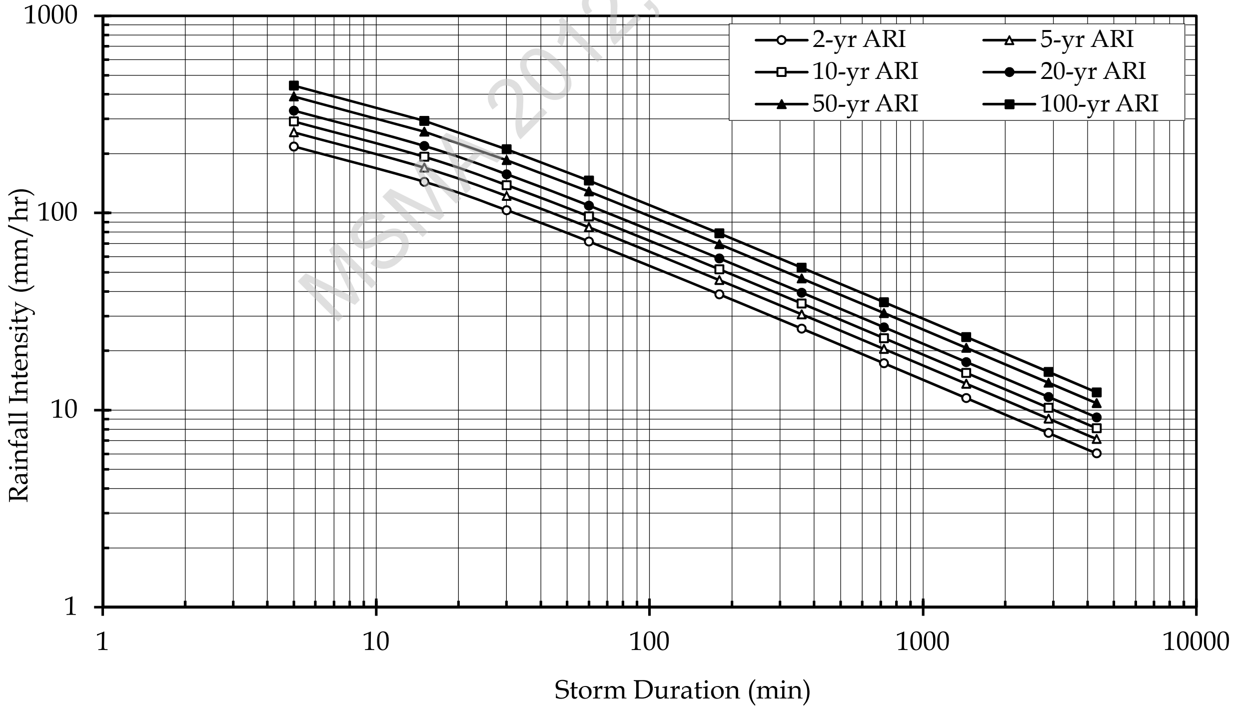


Figure AX3.7.2: Rainfall Station at Sungai Anak Endau -2634193

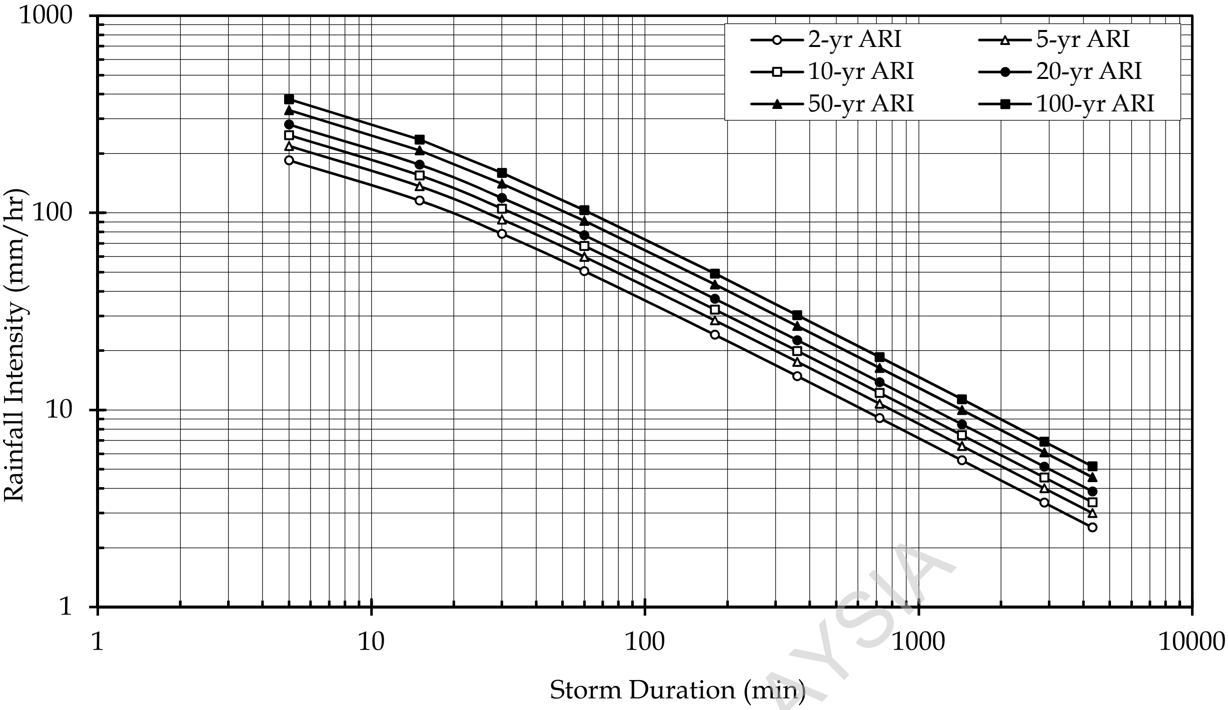


Figure AX3.7.3: Rainfall Station at Kg Gambir – 2828173

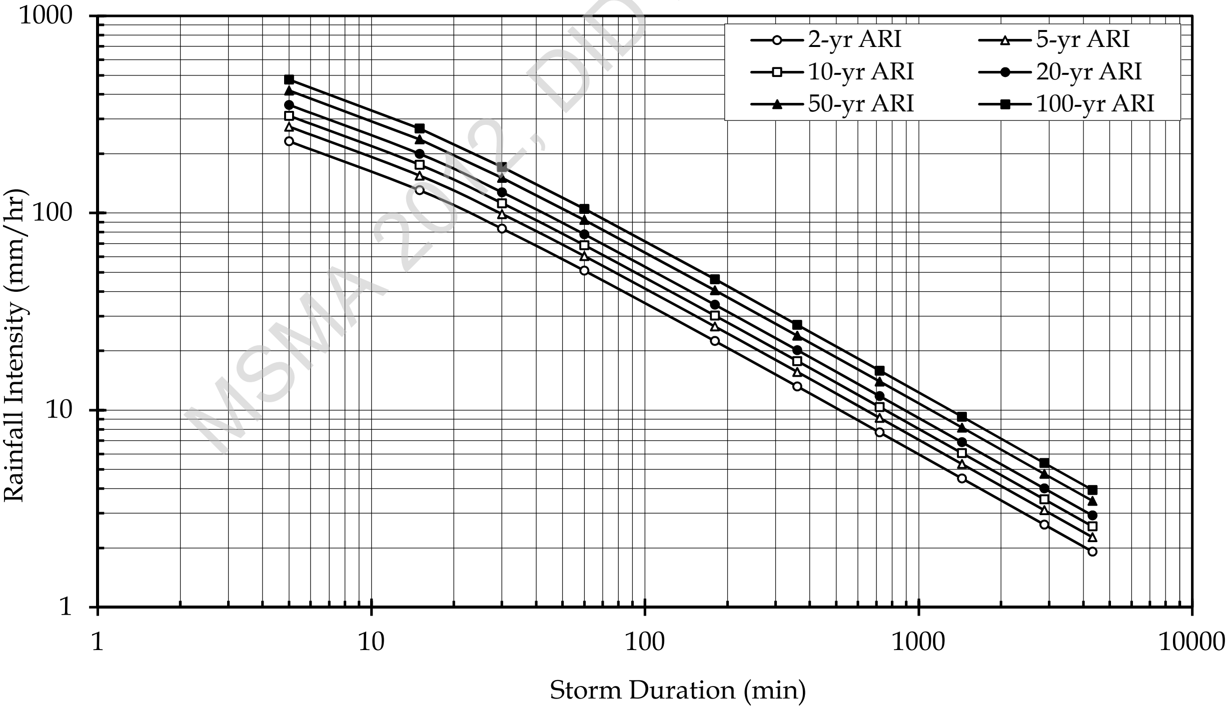


Figure AX3.7.4: Rainfall Station at Pos Iskandar- 3026156

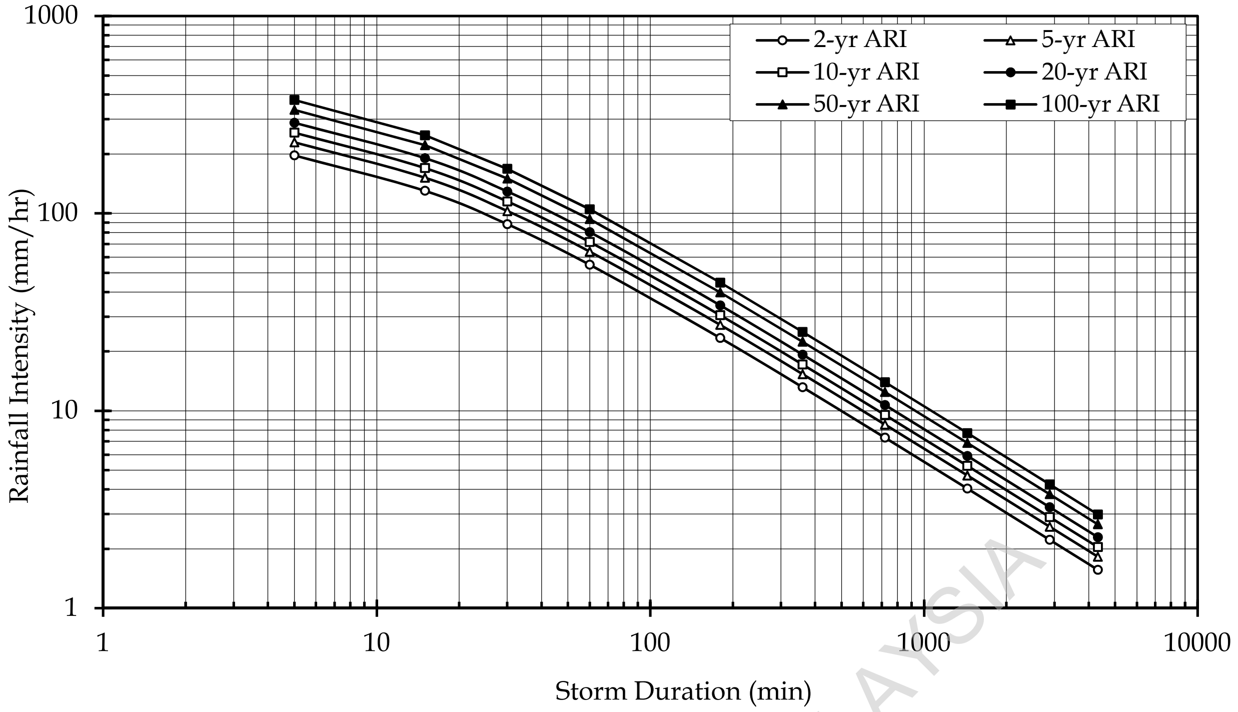


Figure AX3.7.5: Rainfall Station at Simpang Pelangai – 3121143

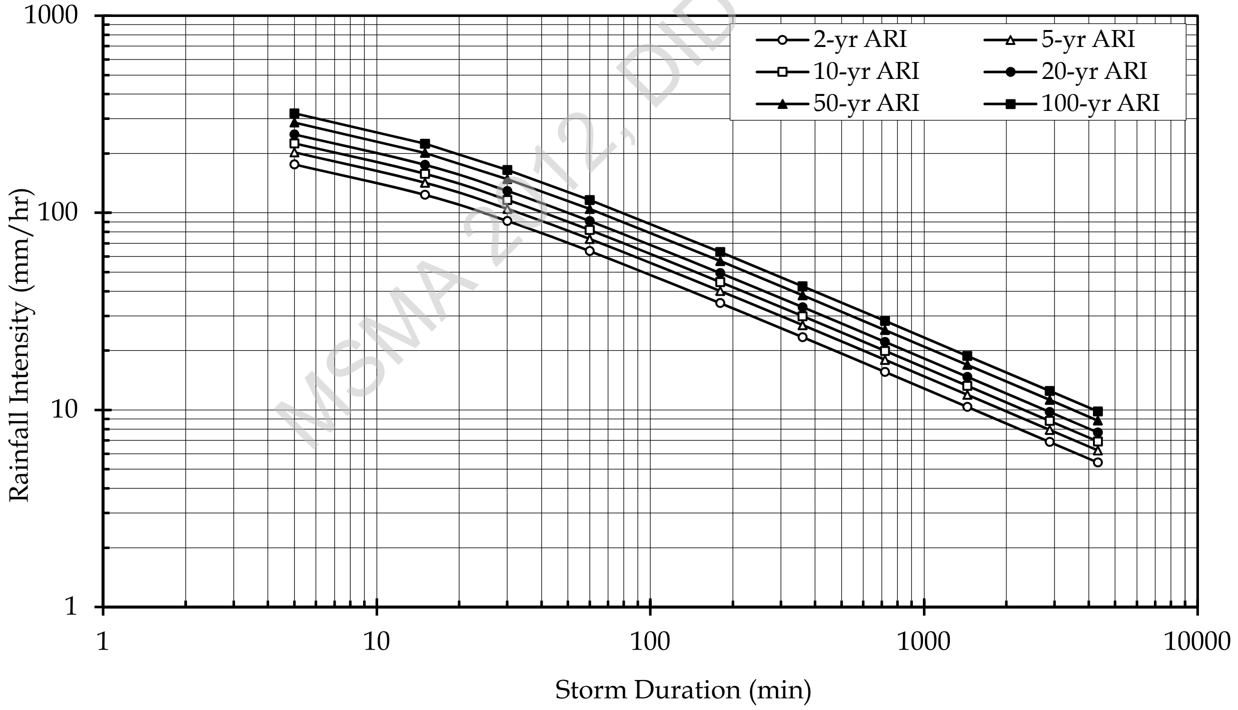


Figure AX3.7.6: Rainfall Station at Dispensari Nenasi - 3134165

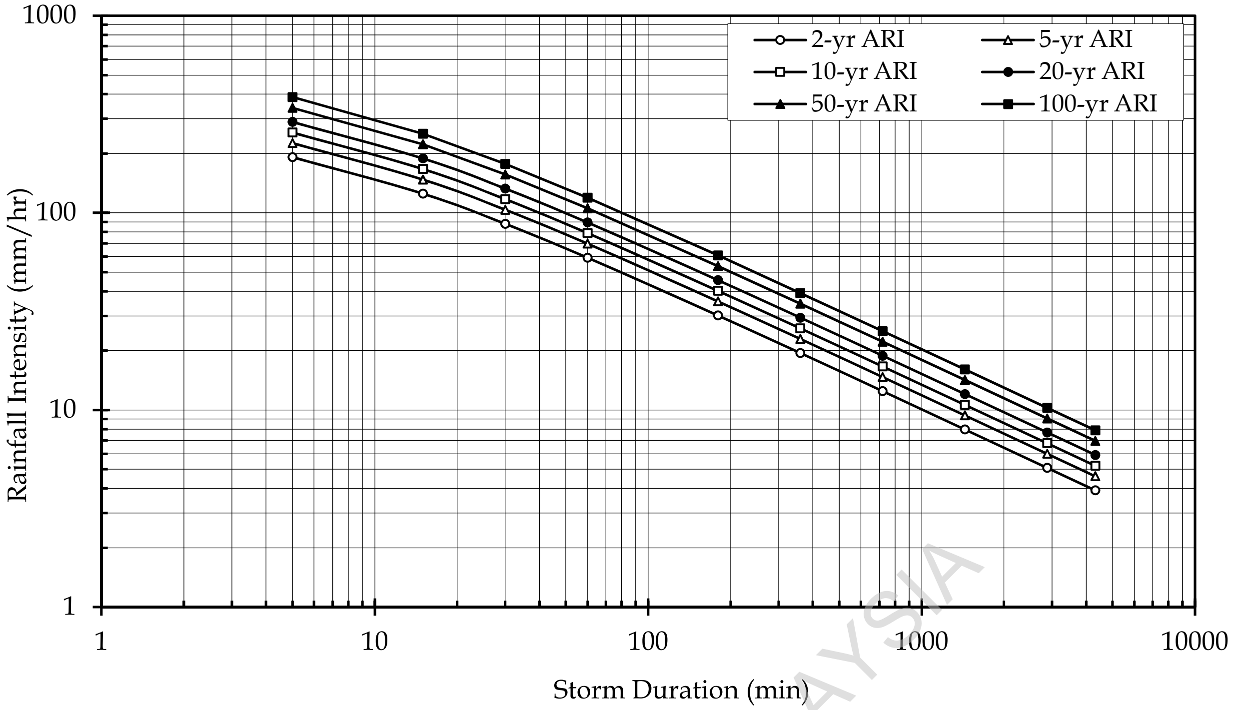


Figure AX3.7.7: Rainfall Station at Kg Unchang – 3231163

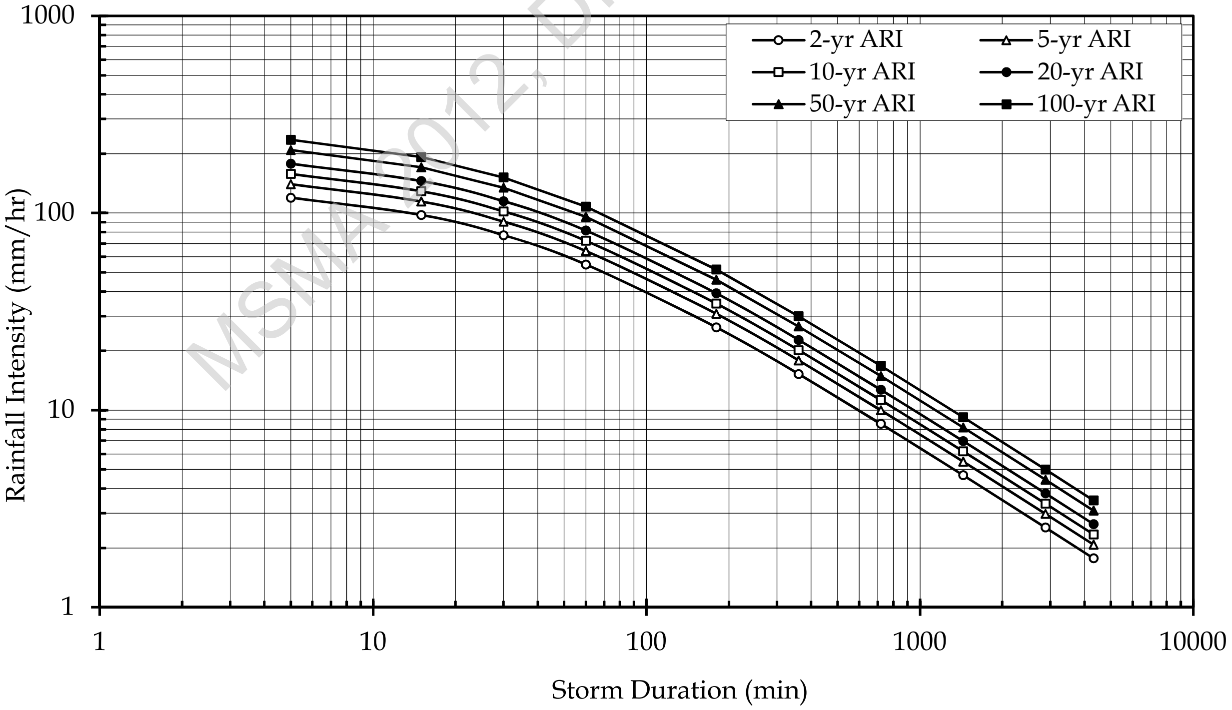


Figure AX3.7.8: Rainfall Station at JPS Temerloh – 3424081

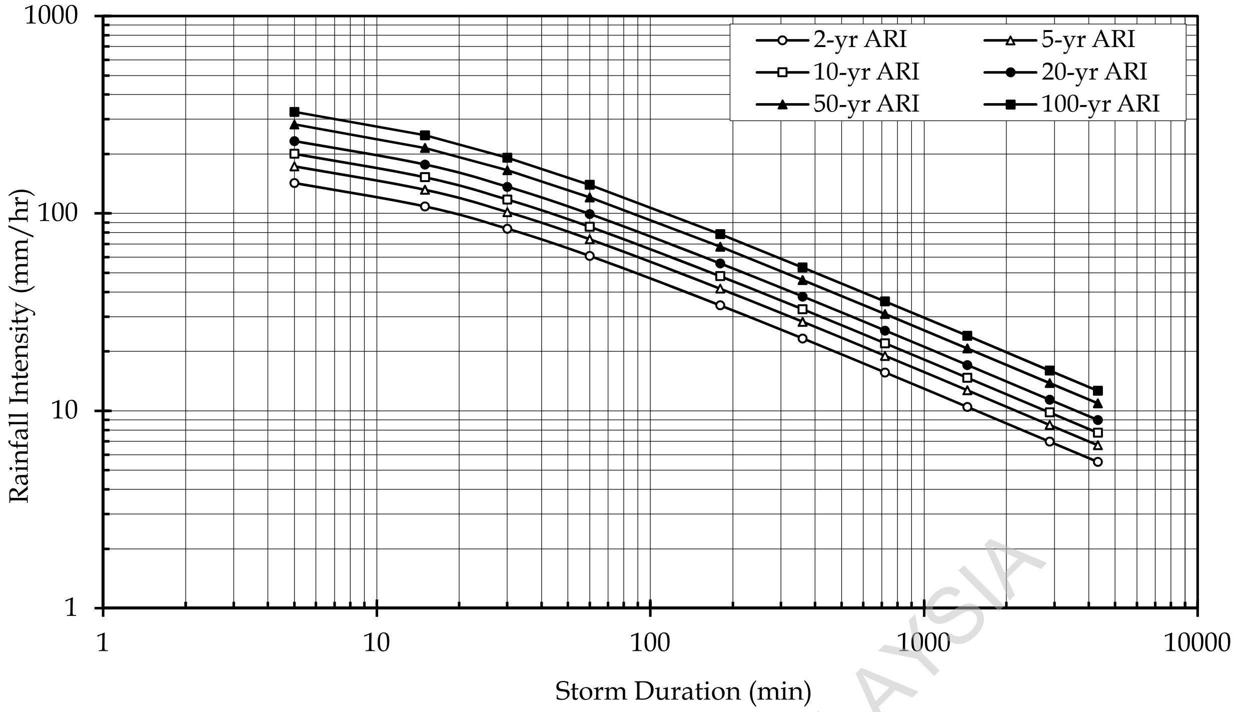


Figure AX3.7.9: Rainfall Station at Rumah Pam Pahang Tua – 3533102

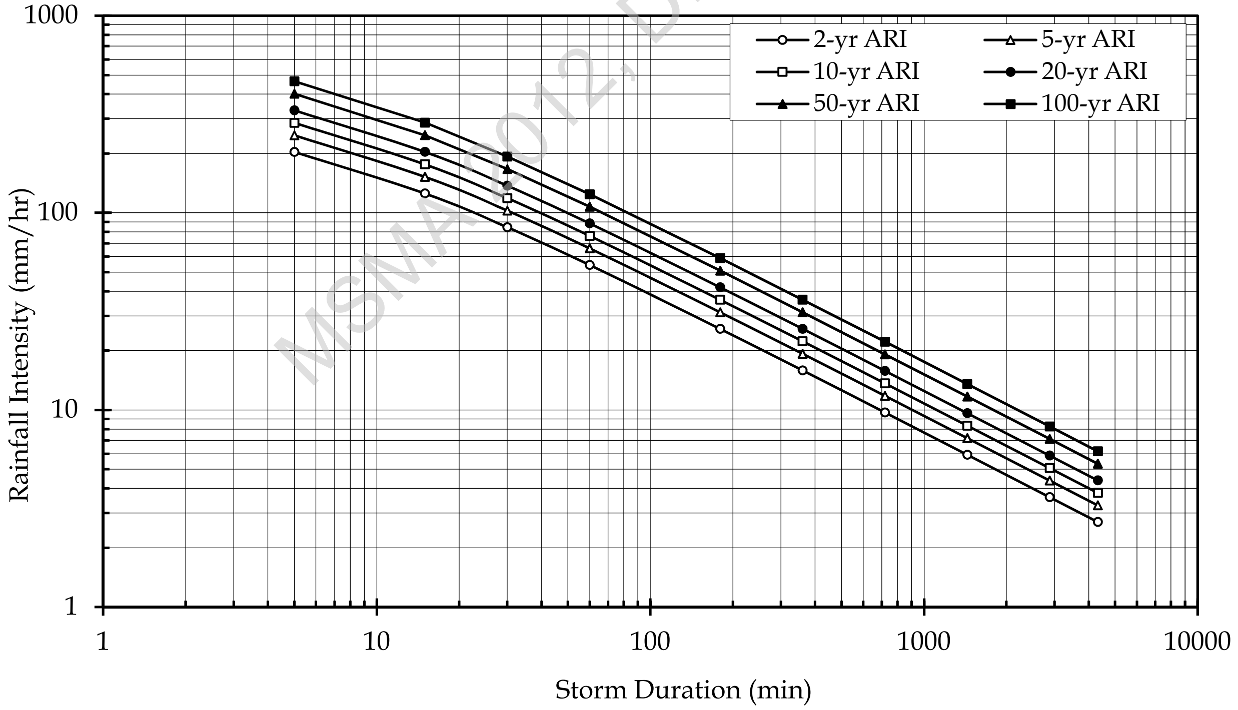


Figure AX3.7.10: Rainfall Station at Pintu Kaw. Pulau Kertam – 3628001

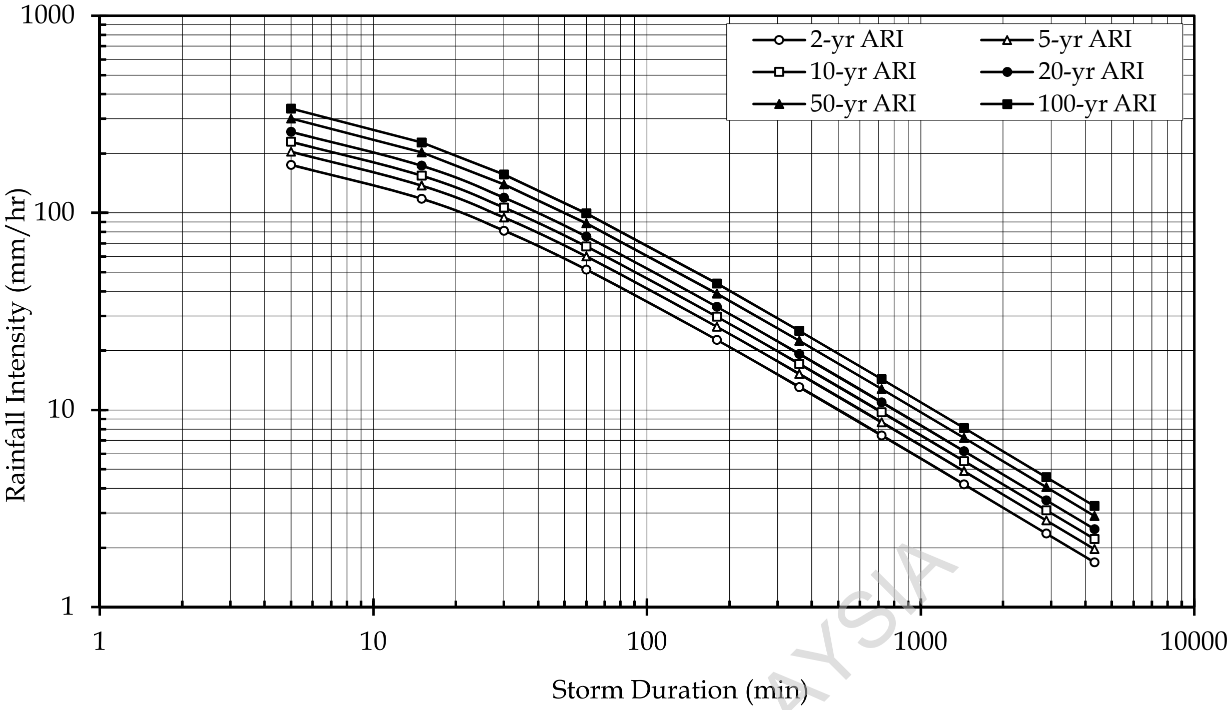


Figure AX3.7.11: Rainfall Station at Setor JPS Raub – 3818054

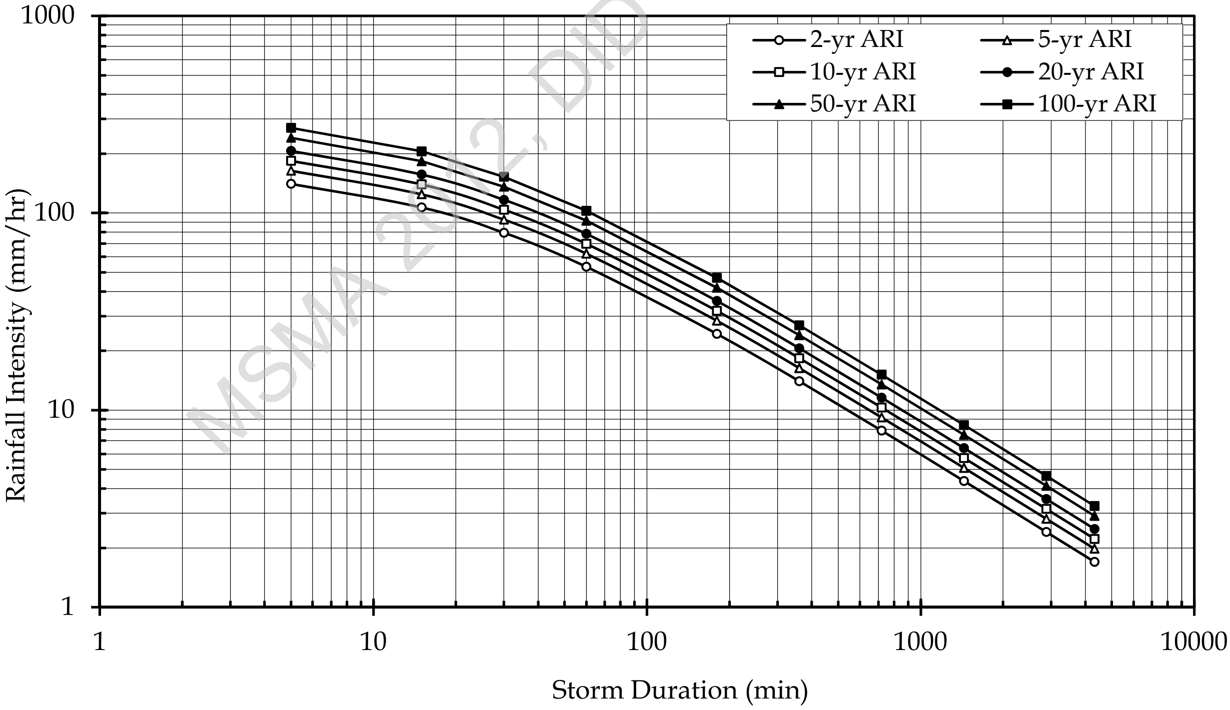


Figure AX3.7.12: Rainfall Station at Rmh Pam Paya Kangsar – 3924072

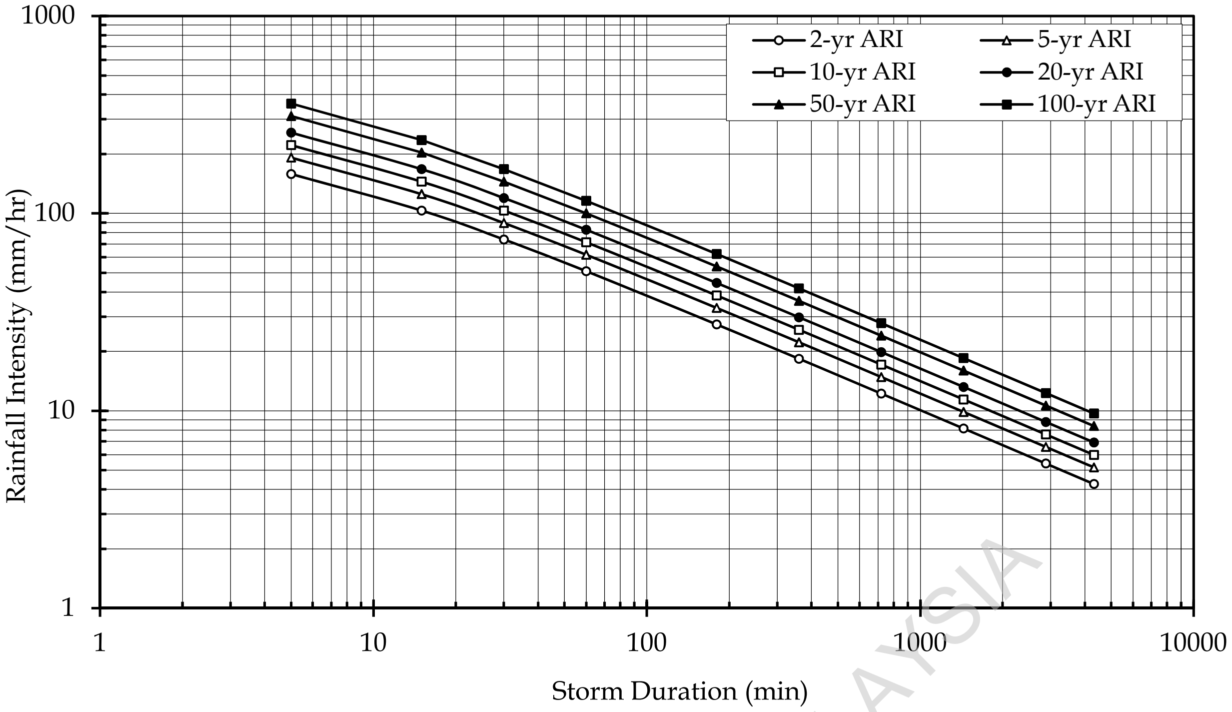


Figure AX3.7.13: Rainfall Station at Sungai Lembing PCC Mill – 3930012

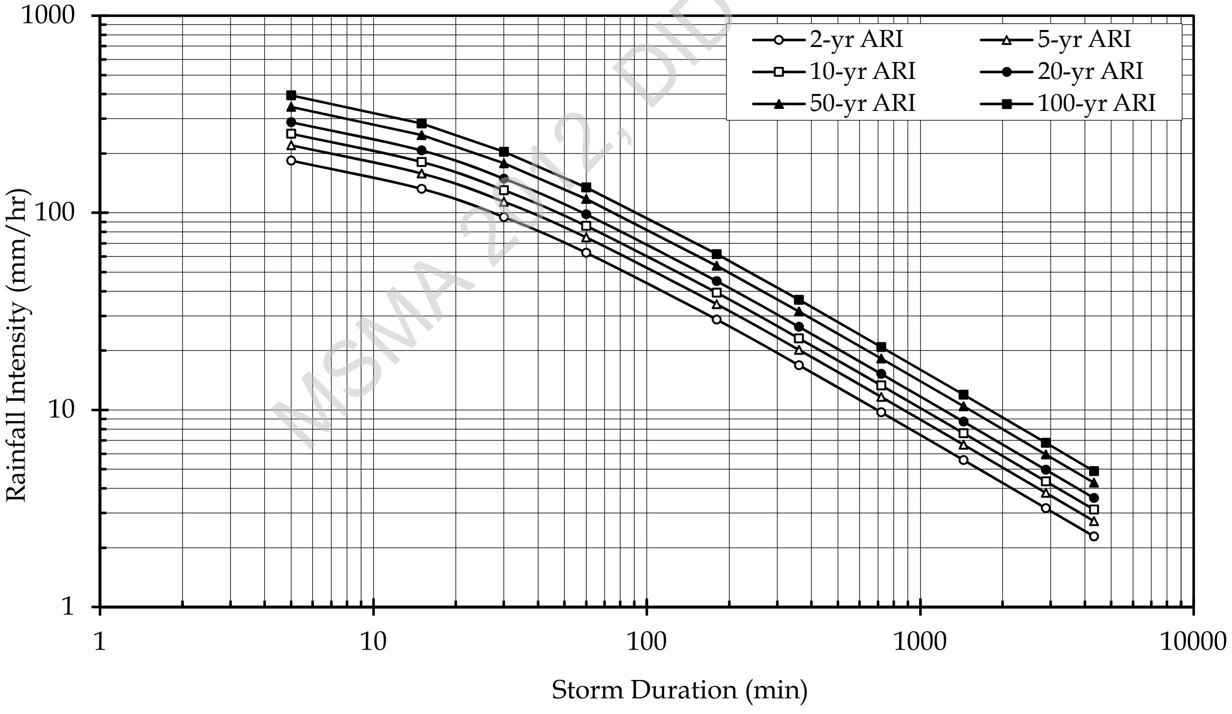


Figure AX3.7.14: Rainfall Station at Kg Sungai Yap - 4023001

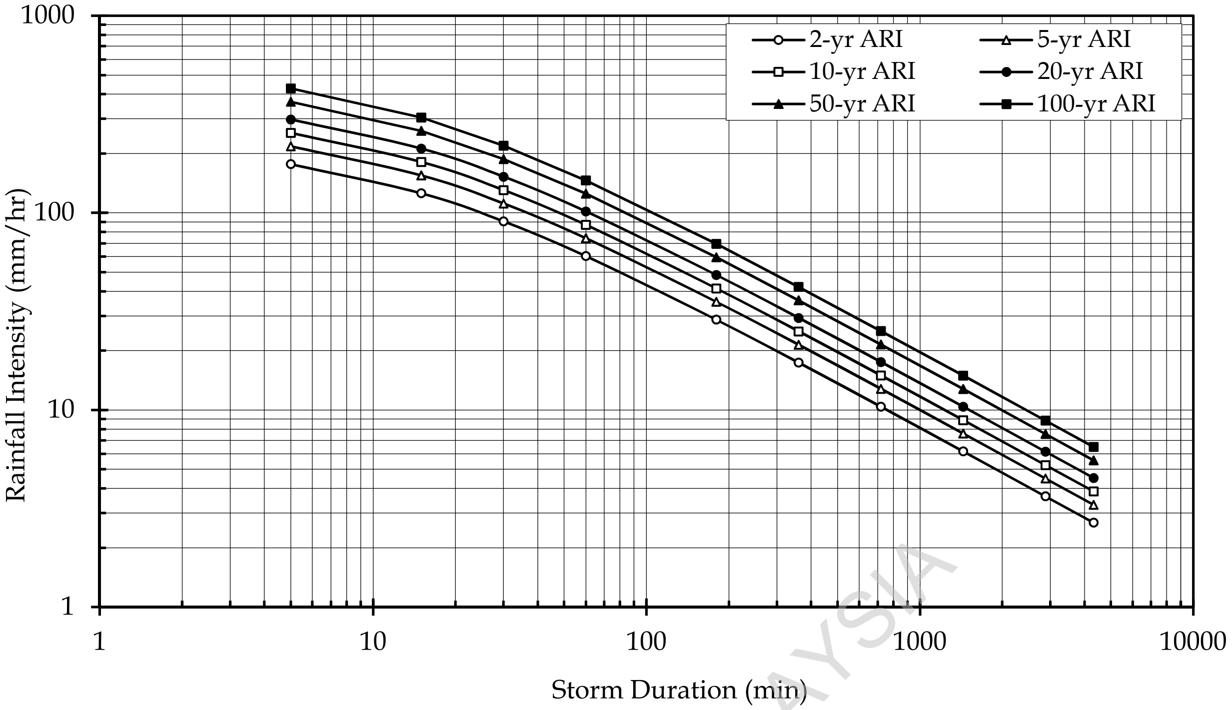


Figure AX3.7.15: Rainfall Station at Hulu Tekai Kwsn."B" – 4127001

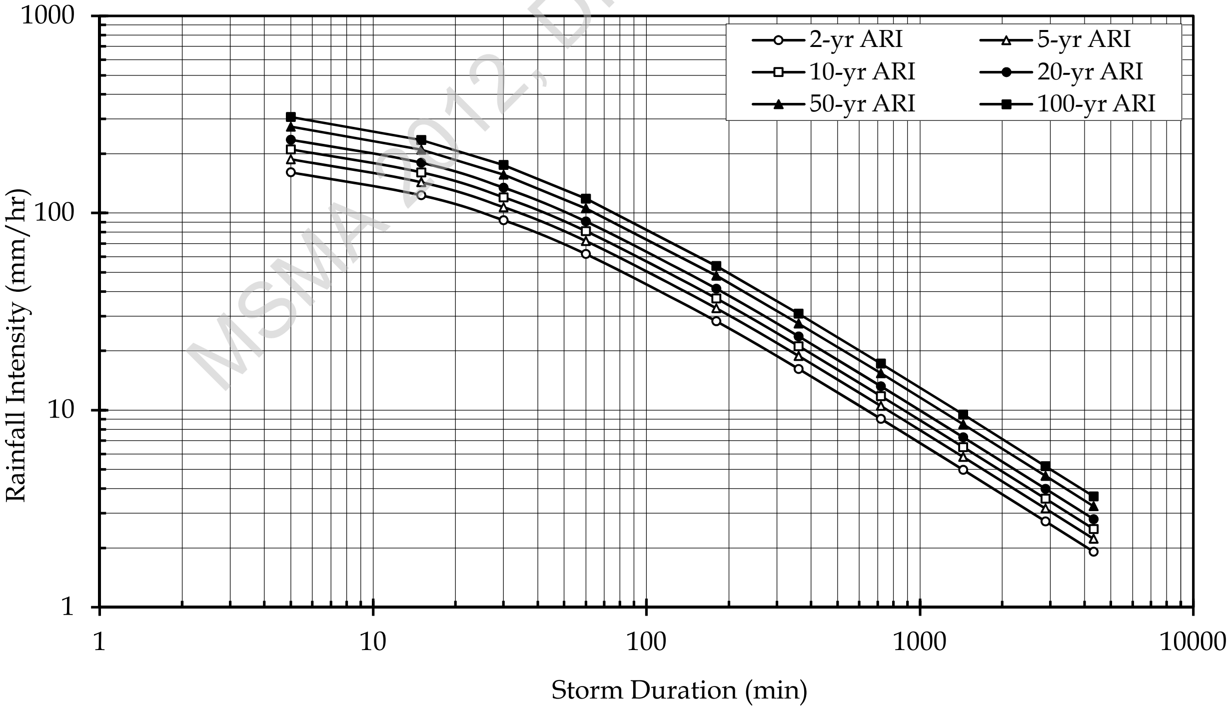


Figure AX3.7.16: Rainfall Station at Bukit Bentong – 4219001

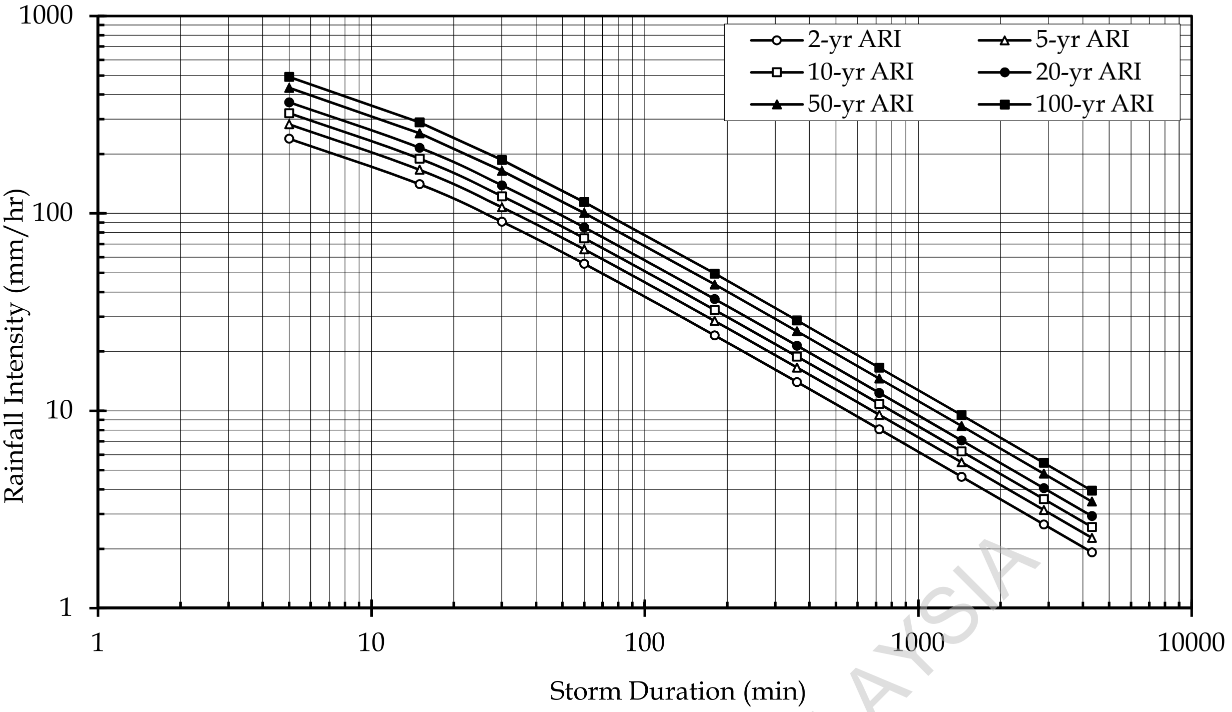


Figure AX3.7.17: Rainfall Station at Kg Merting – 4223115

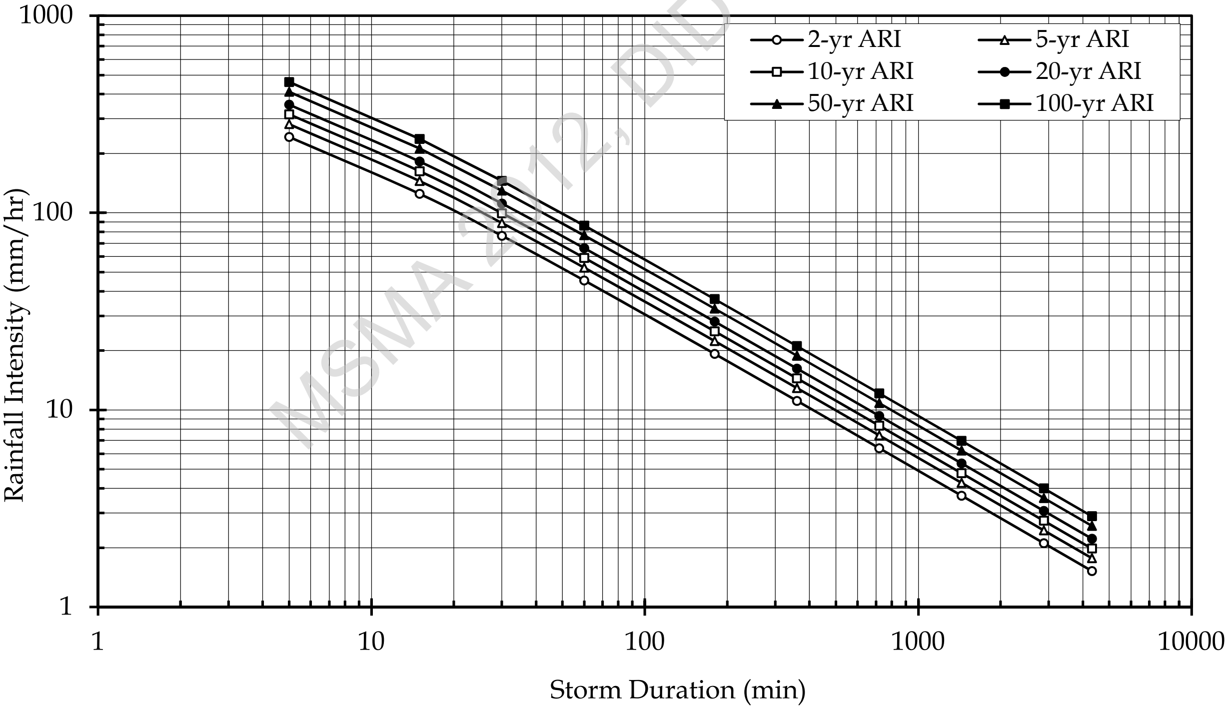


Figure AX3.7.18: Rainfall Station at Gunung Brinchang - 4513033

AX3.8 STATE OF PULAU PINANG

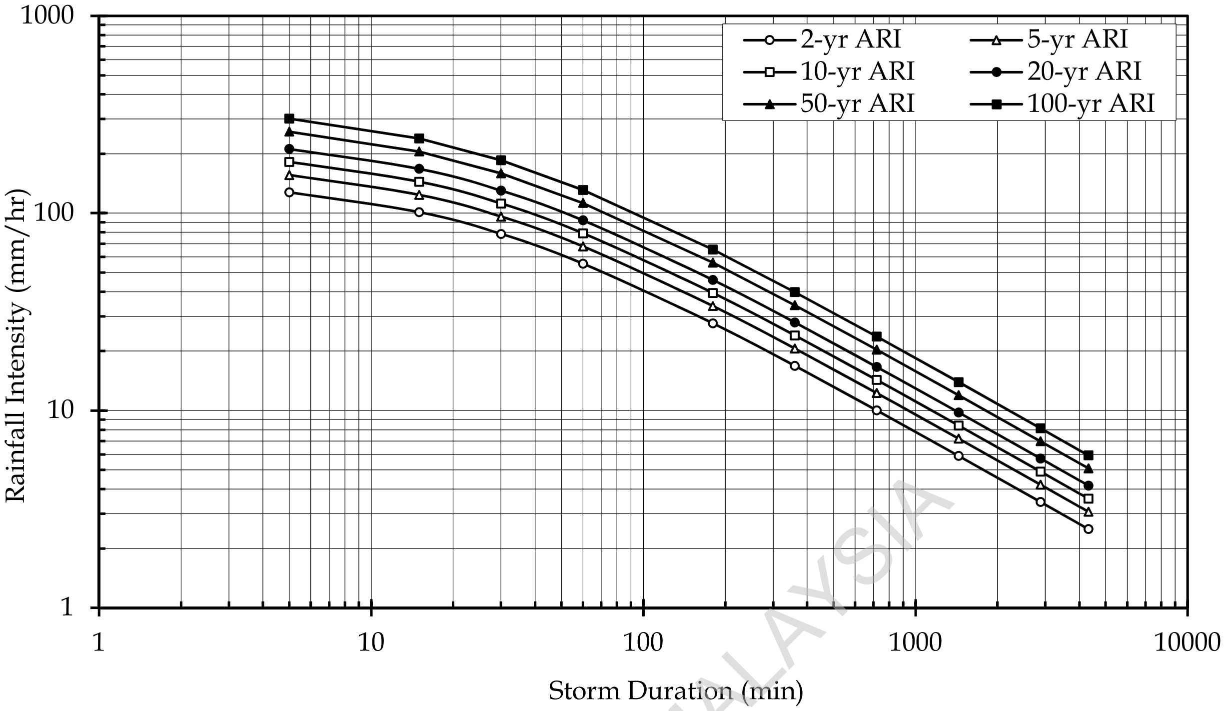


Figure AX3.8.1: Rainfall Station at Sg Simpang Ampat Tangki-5204048

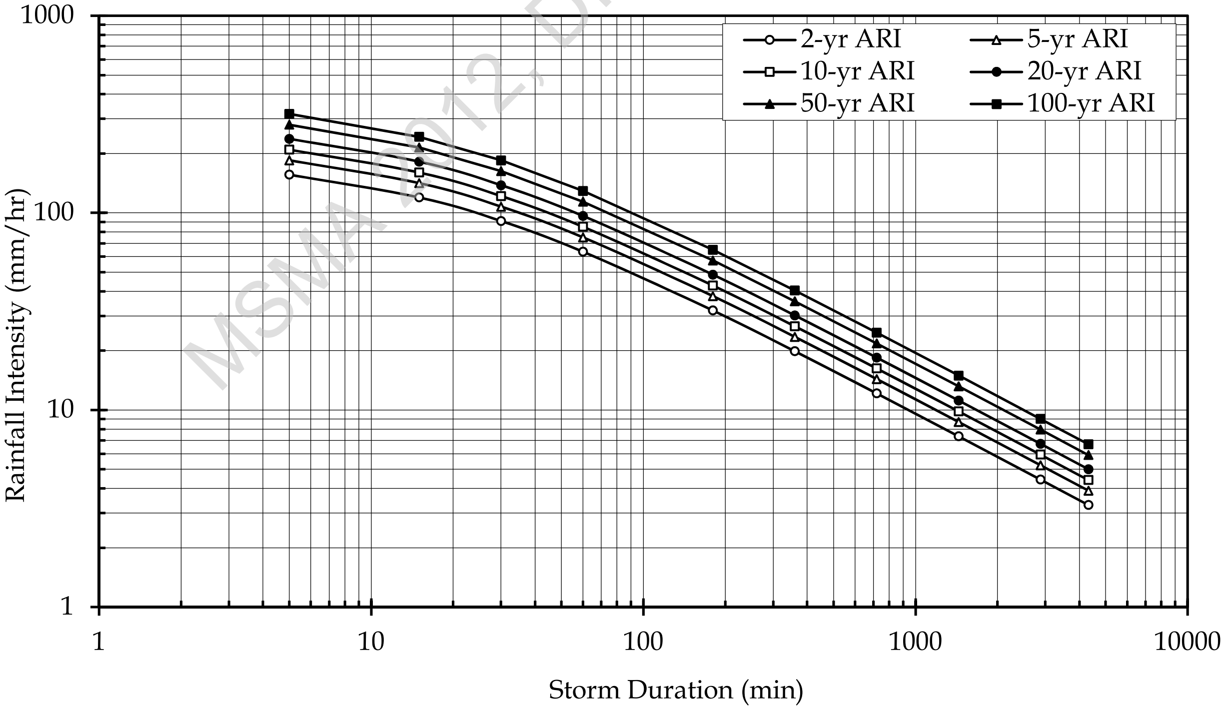


Figure AX3.8.2: Rainfall Station at Air Besar Sg Png -5302001

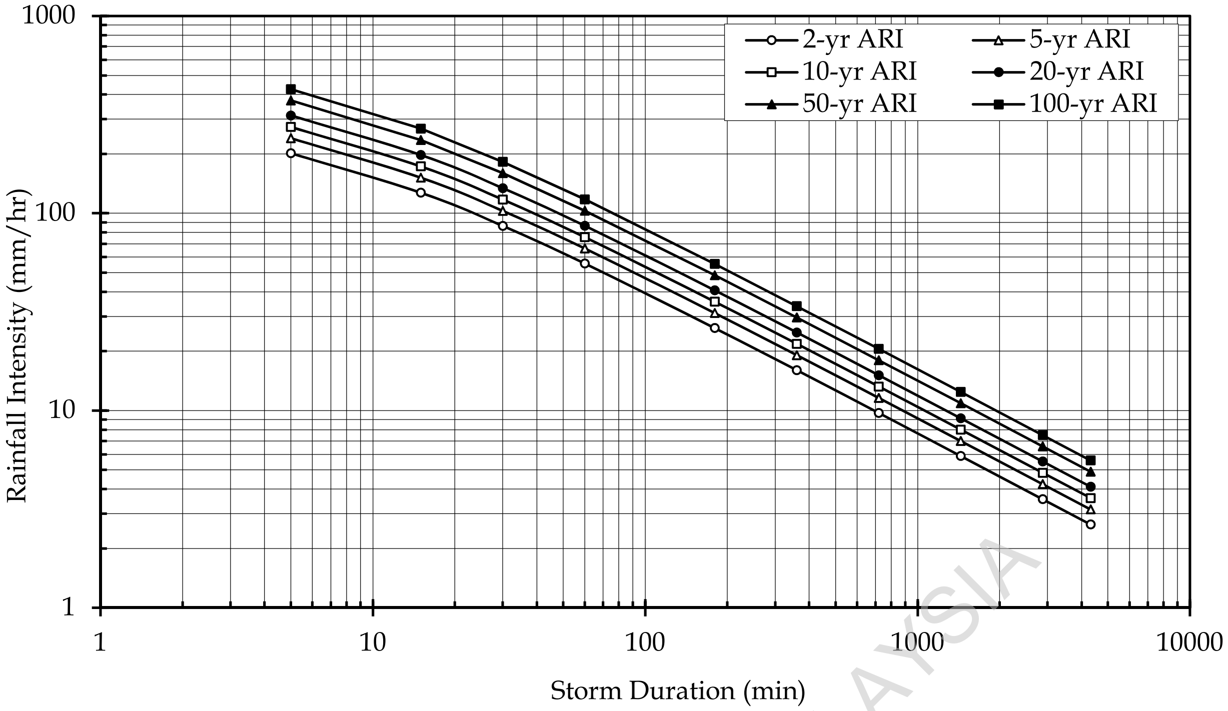


Figure AX3.8.3: Rainfall Station at Kolam Tkgn Air Hitam -5302003

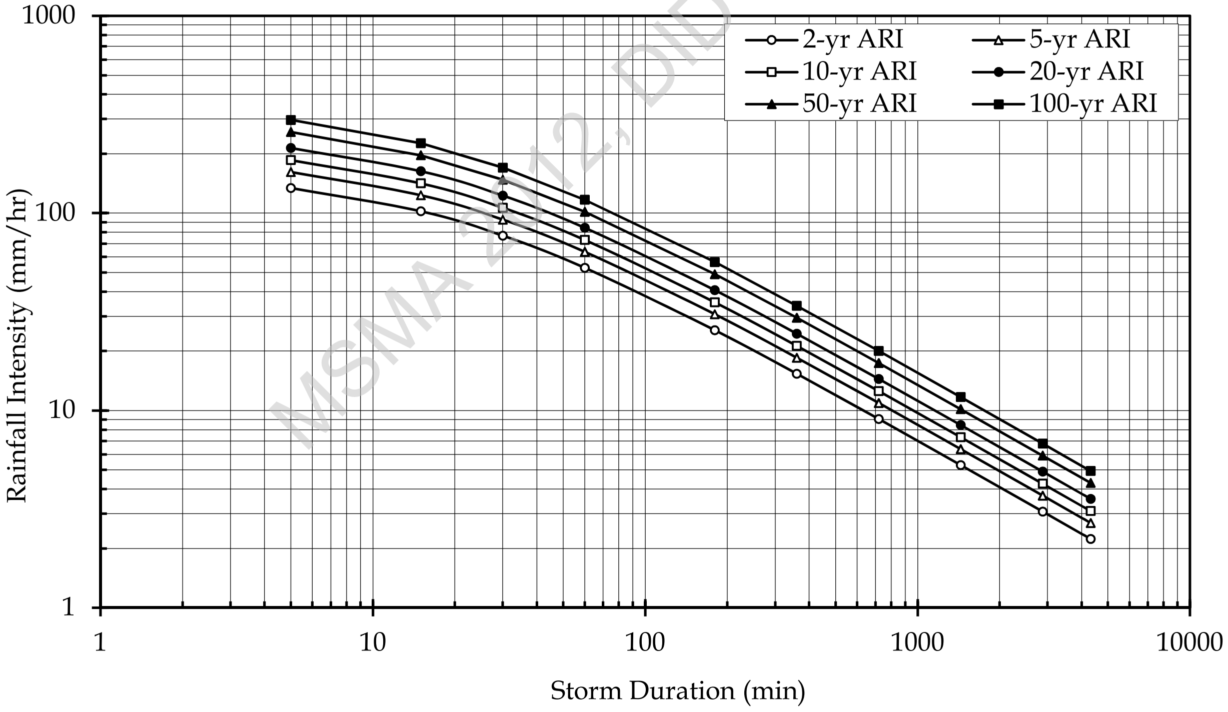


Figure AX3.8.4: Rainfall Station at Rmh Kebajikan P Pinang -5303001

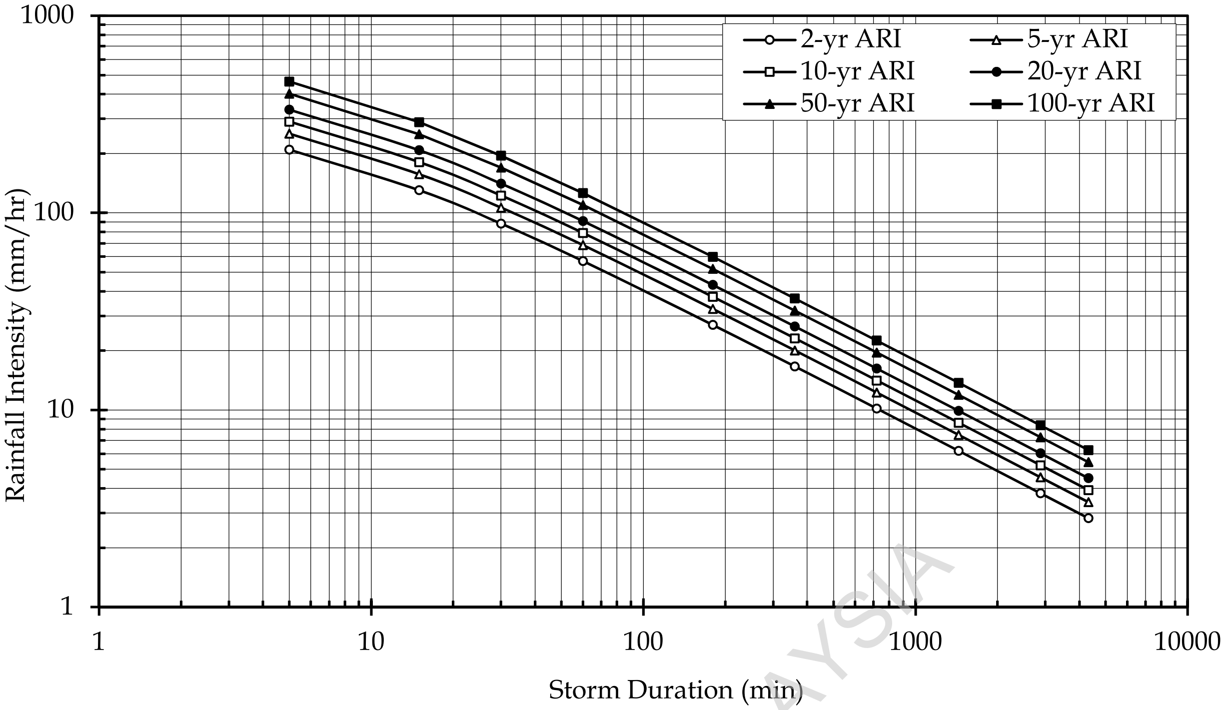
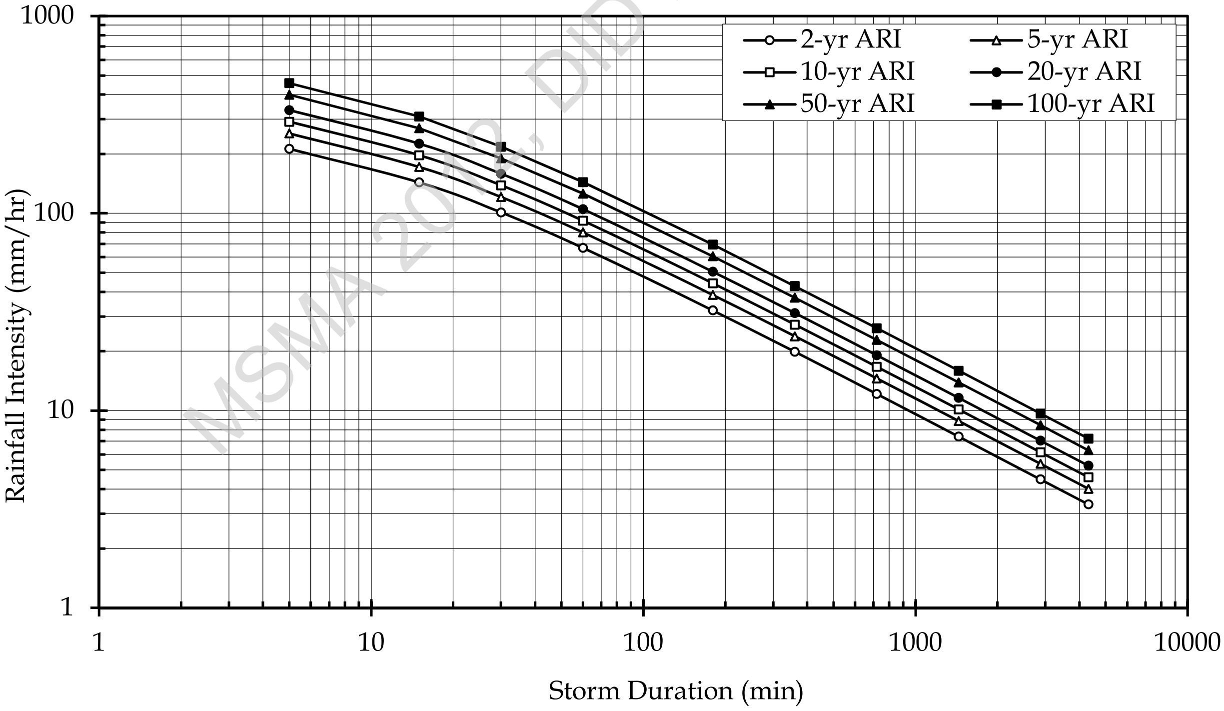


Figure AX3.8.5: Rainfall Station at Komplek Prai-5303053



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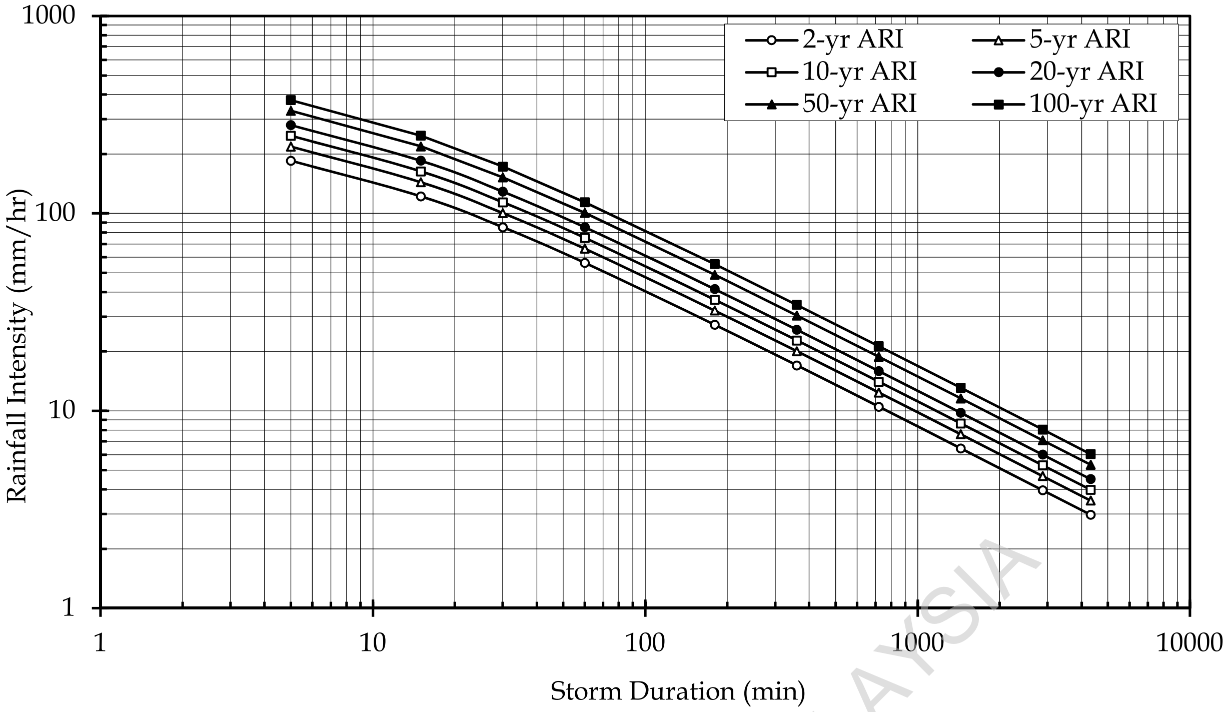


Figure AX3.8.7: Rainfall Station at Kolam Bersih P Pinang-5402002

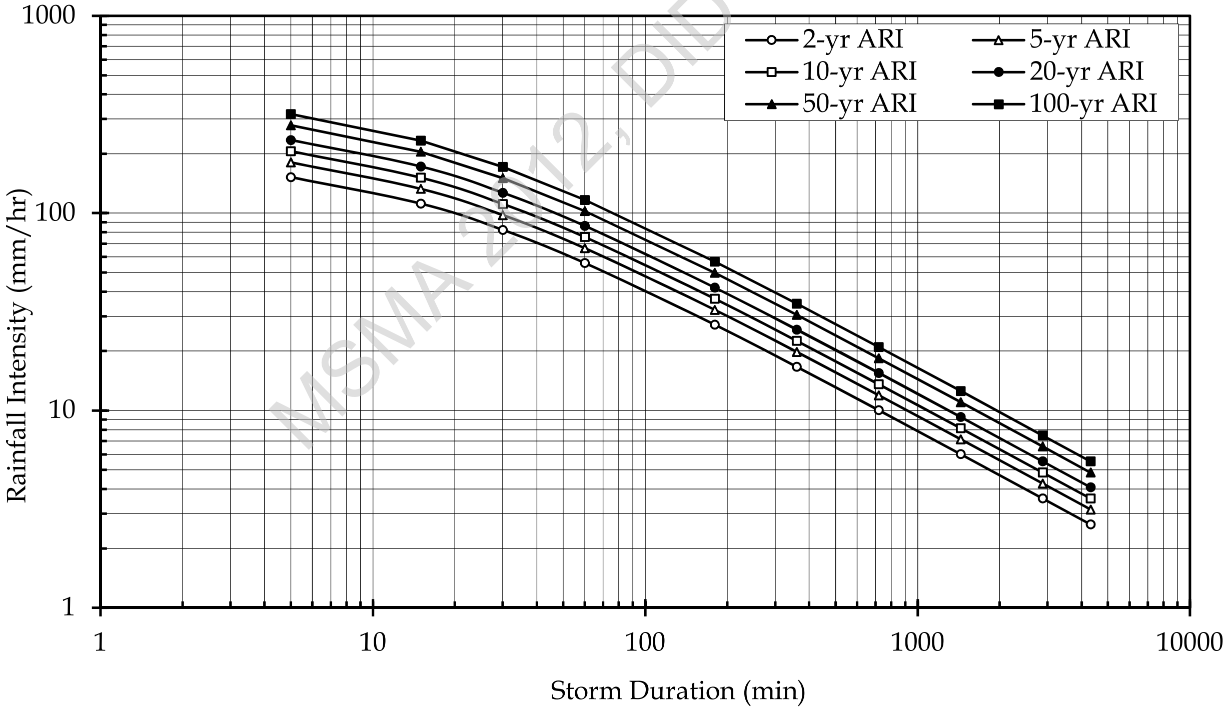


Figure AX3.8.8: Rainfall Station at Ibu Bekalan Sg Kulim -5404043

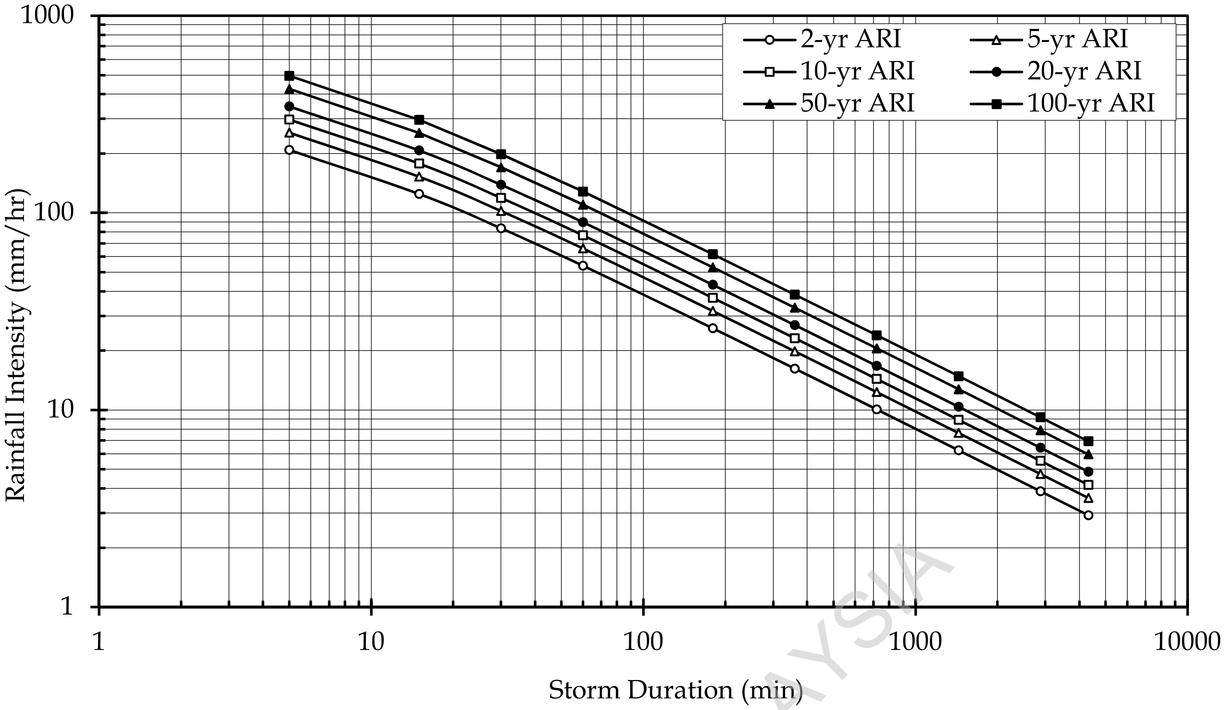


Figure AX3.8.9: Rainfall Station at Lahar Ikan Mati K Batas -5504035

AX3.9 STATE OF PERAK

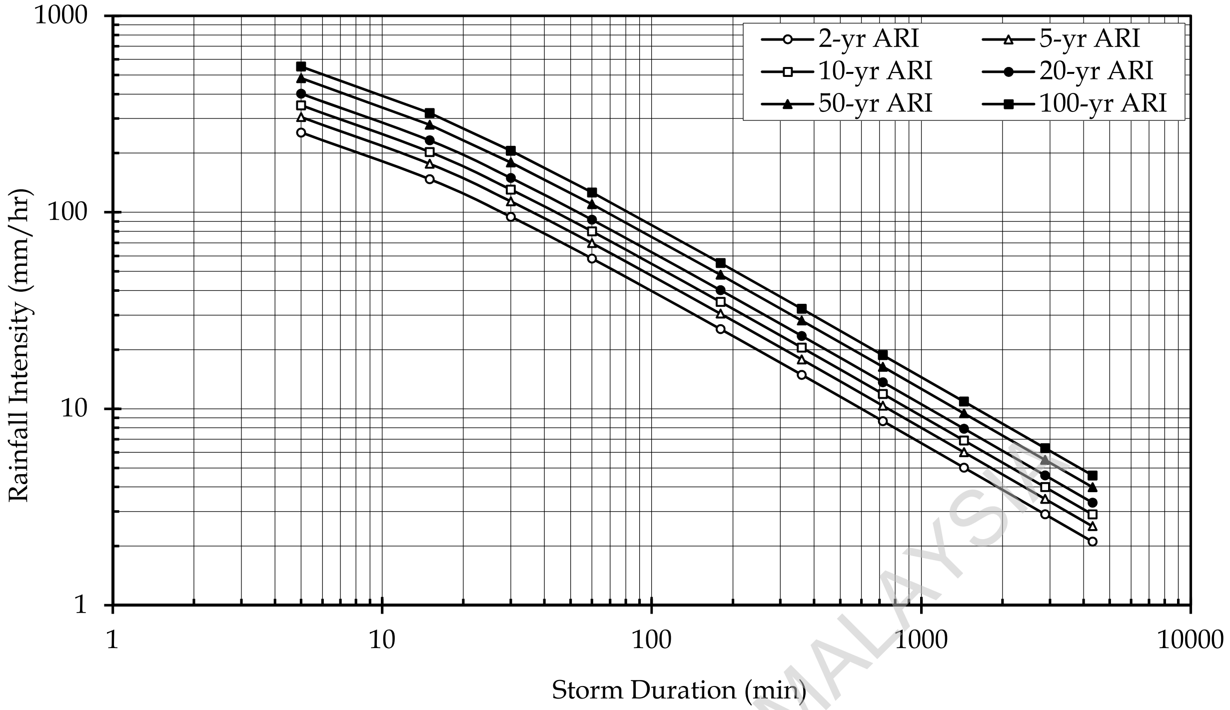


Figure AX3.9.1: Rainfall Station at JPS Teluk Intan - 4010001

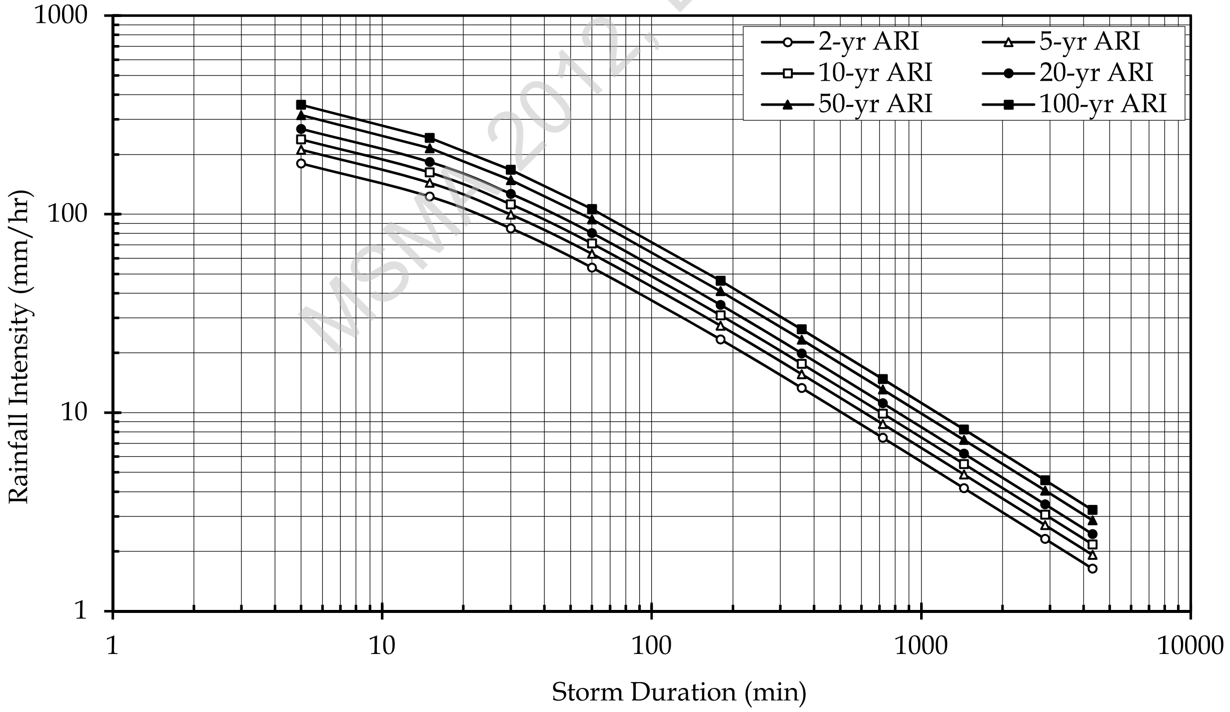


Figure AX3.9.2: Rainfall Station at JPS Setiawan – 4207048

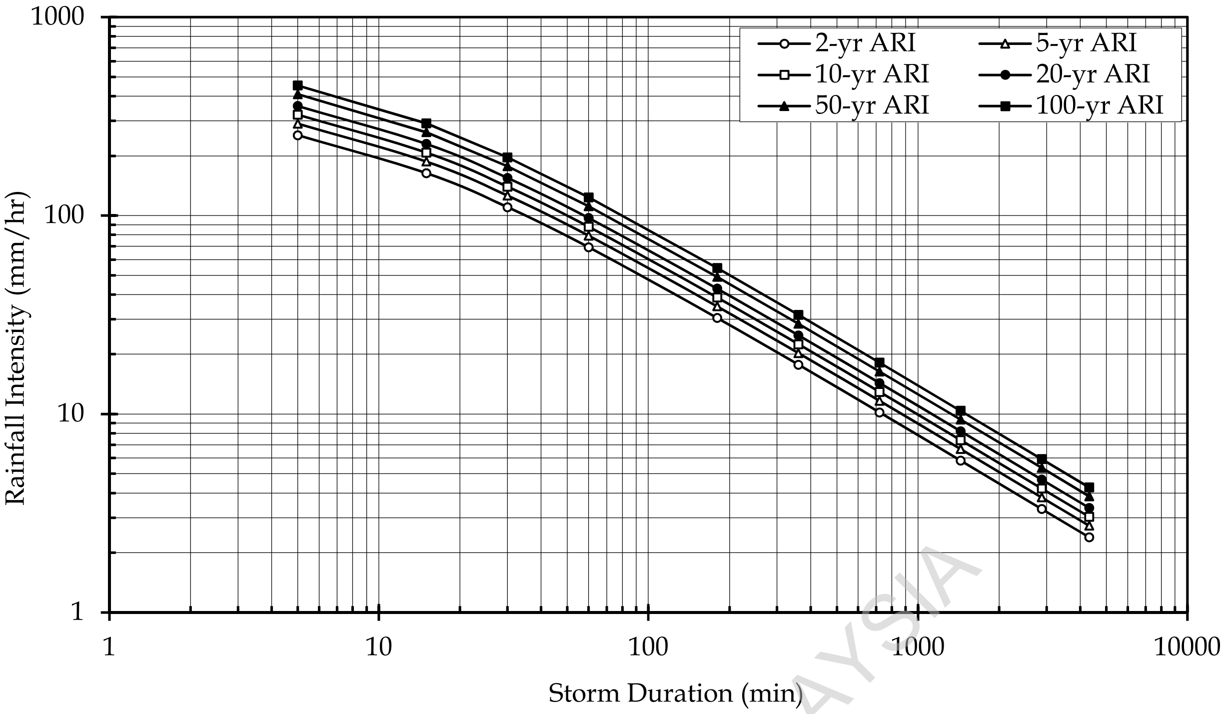


Figure AX3.9.3: Rainfall Station at Pejabat Daerah Kampar - 4311001

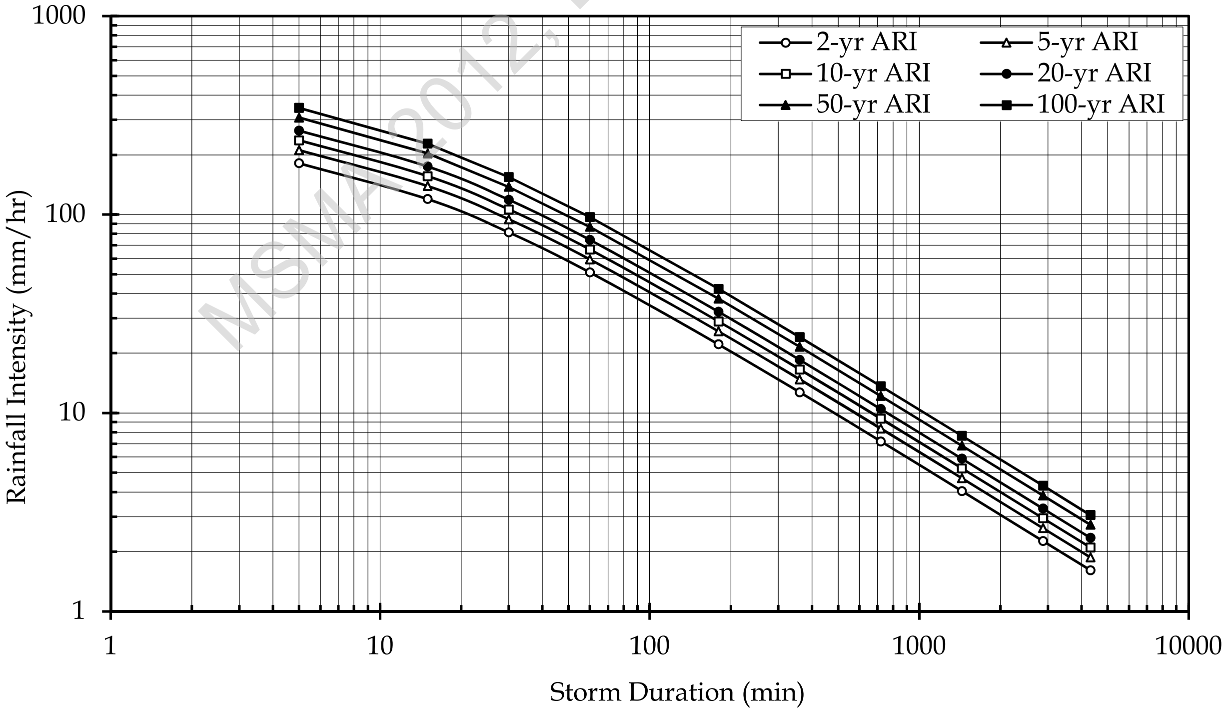


Figure AX3.9.4: Rainfall Station at Rumah Pam Kubang Haji – 4409091

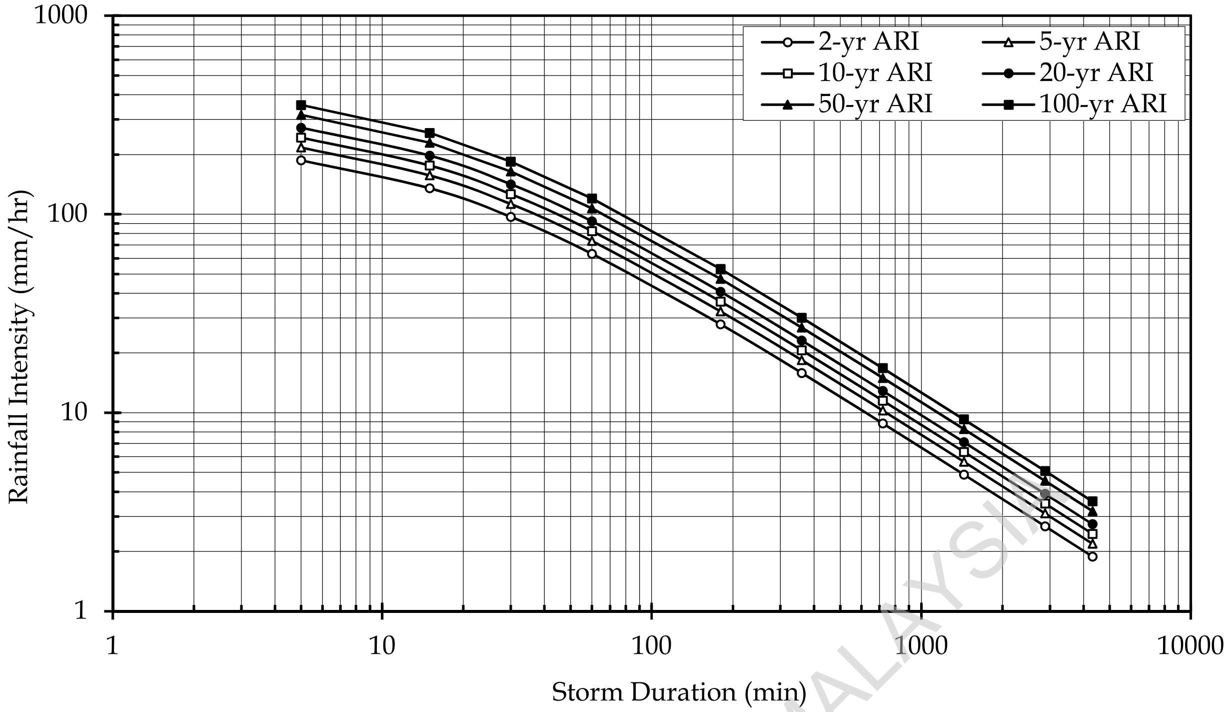


Figure AX3.9.5: Rainfall Station at Politeknik Ungku Umar - 4511111

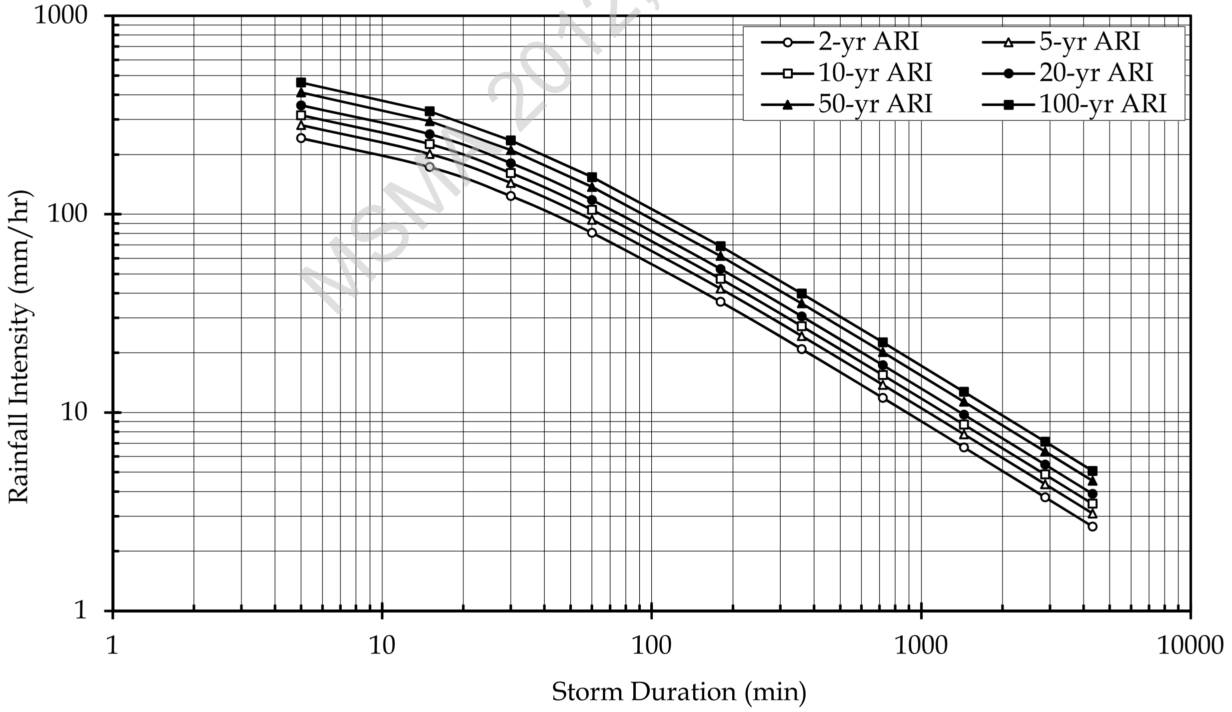


Figure AX3.9.6: Rainfall Station at Bukit Larut Taiping – 4807016

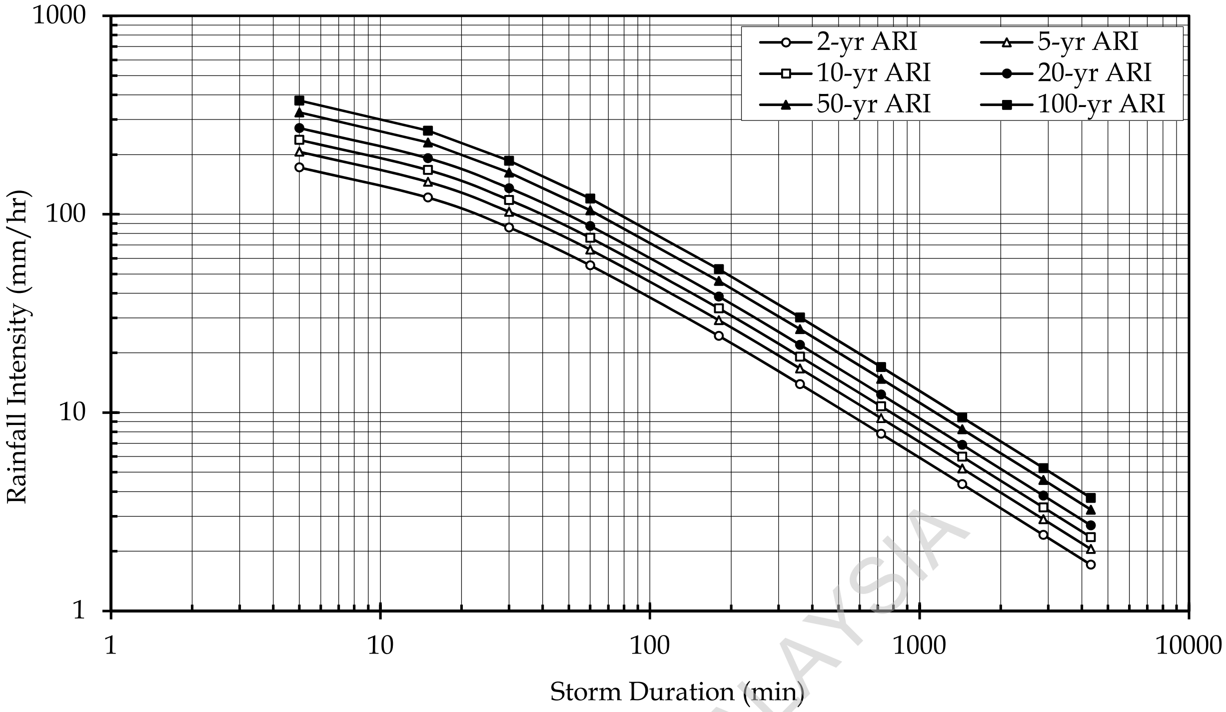


Figure AX3.9.7: Rainfall Station at Rancangan Belia Perlop - 4811075

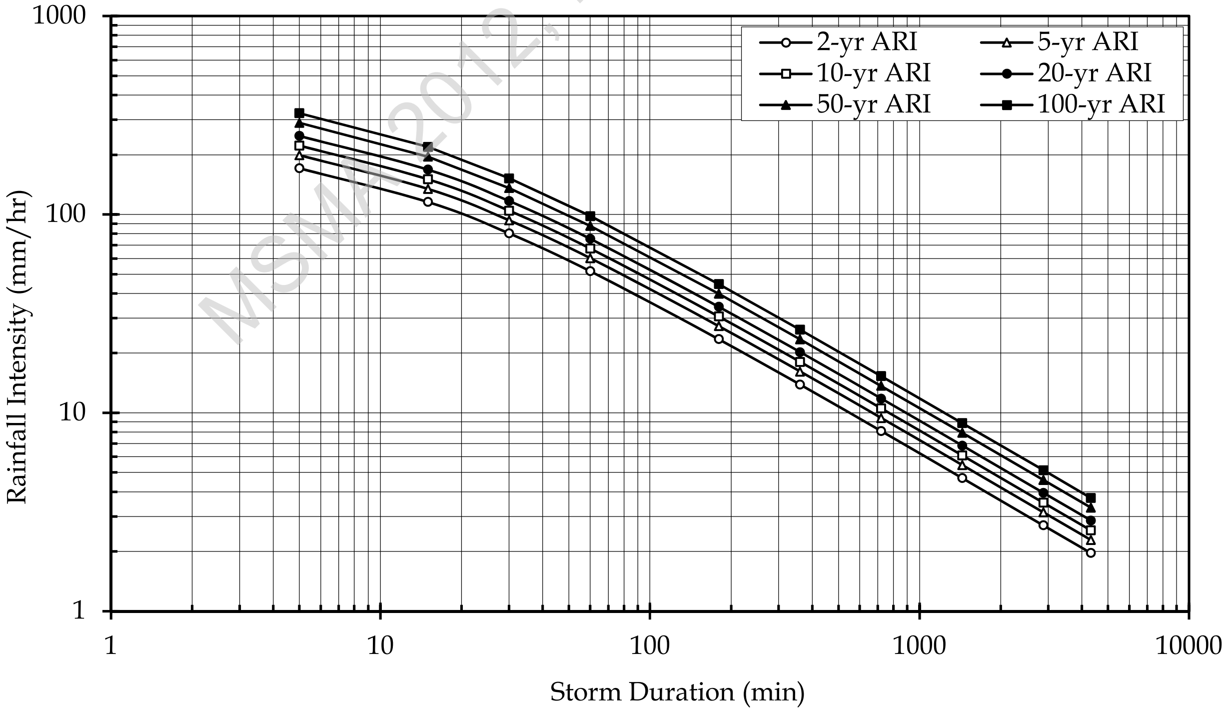


Figure AX3.9.8: Rainfall Station at Jln. Mtg. Buloh Bgn Serai – 5005003

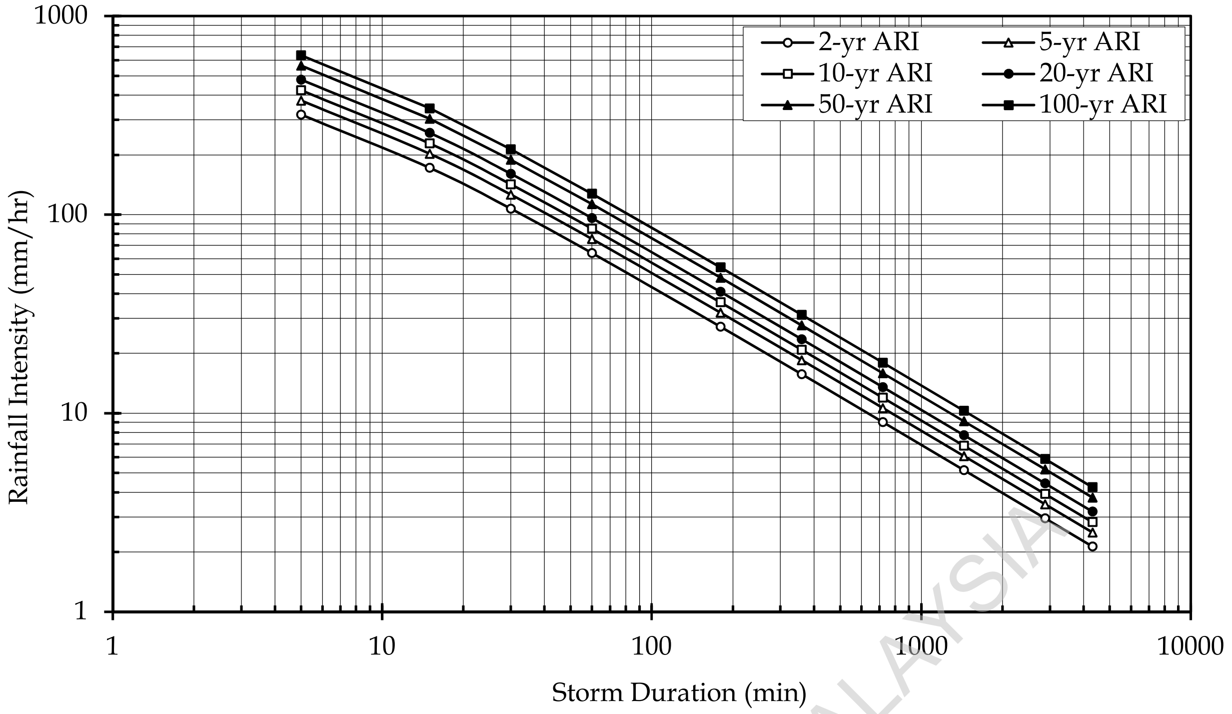


Figure AX3.9.9: Rainfall Station at Kolam Air JKR Selama - 5207001

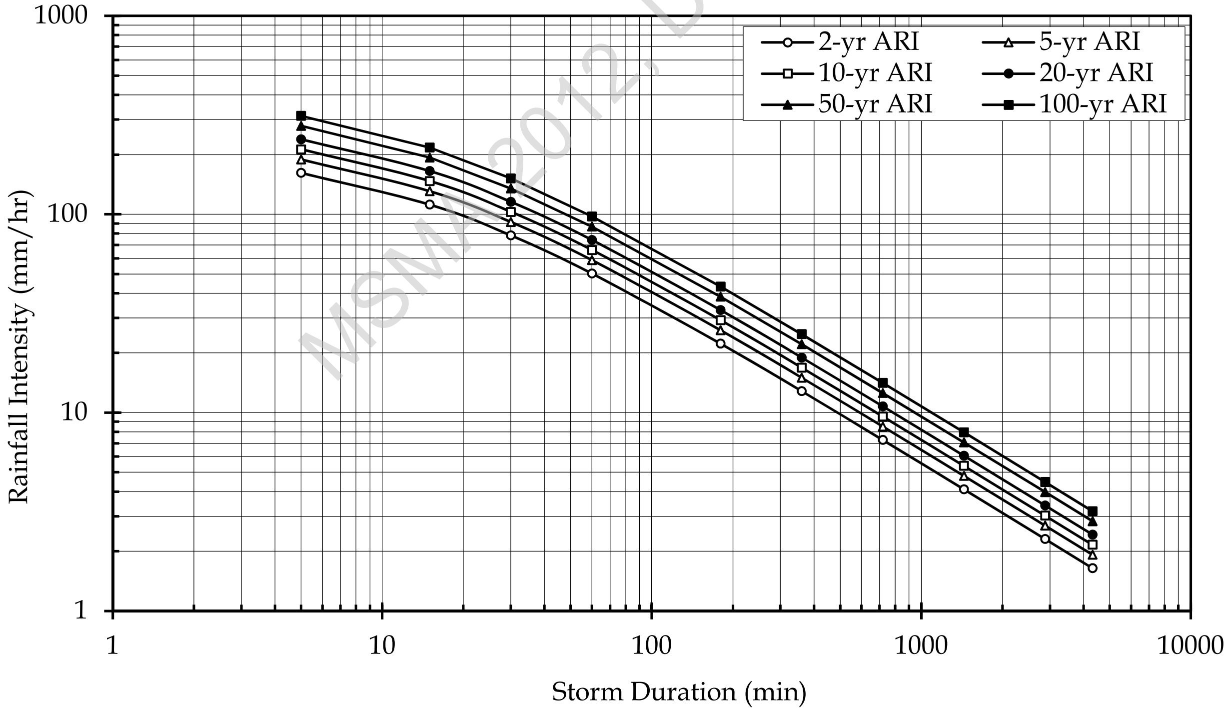


Figure AX3.9.10: Rainfall Station at Stesen Pem. Hutan Lawin – 5210069

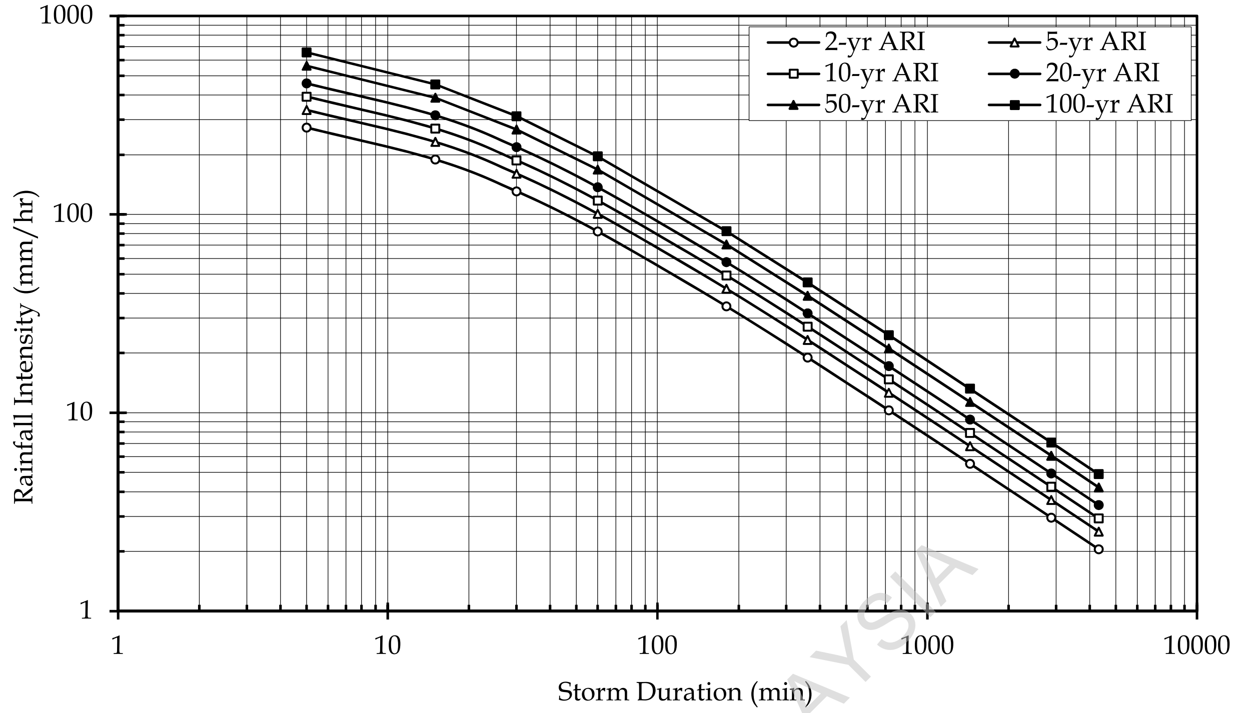


Figure AX3.9.11: Rainfall Station at Kuala Kenderong - 5411066

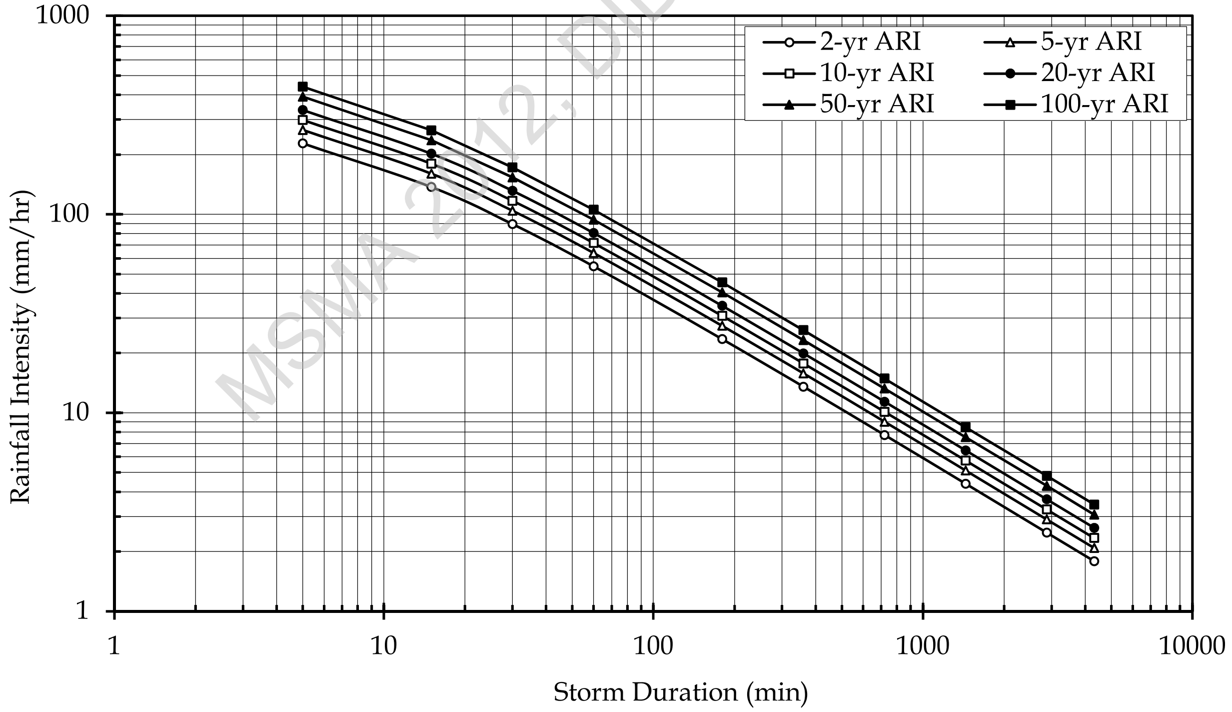


Figure AX3.9.12: Rainfall Station at Dispensari Keroh – 5710061

AX3.10 STATE OF PERLIS

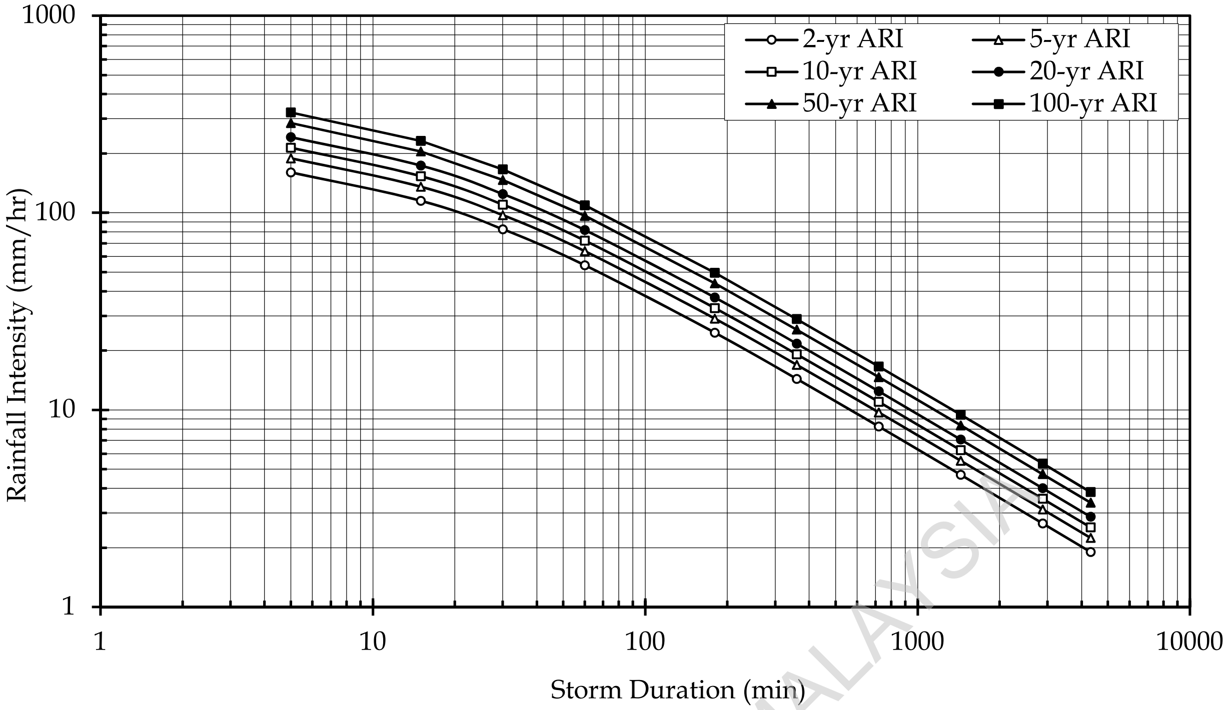


Figure AX3.10.1: Rainfall Station at Padang Katong, Kangar – 6401002

AX3.11 STATE OF SELANGOR

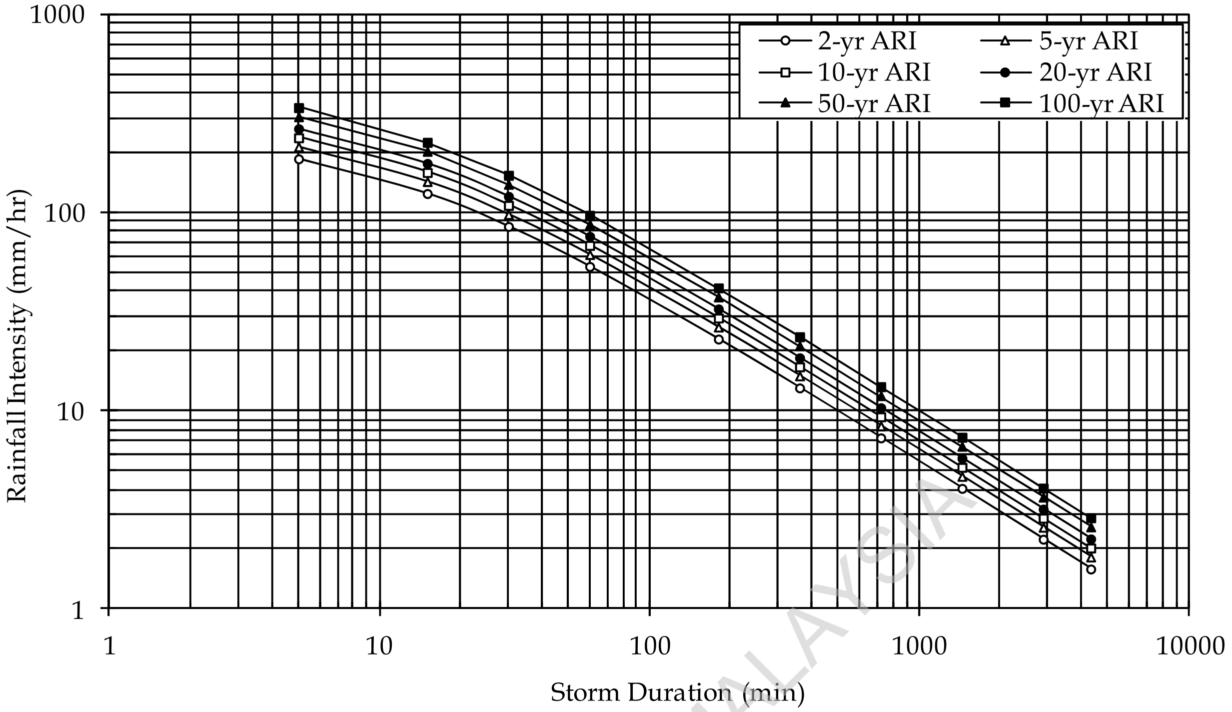
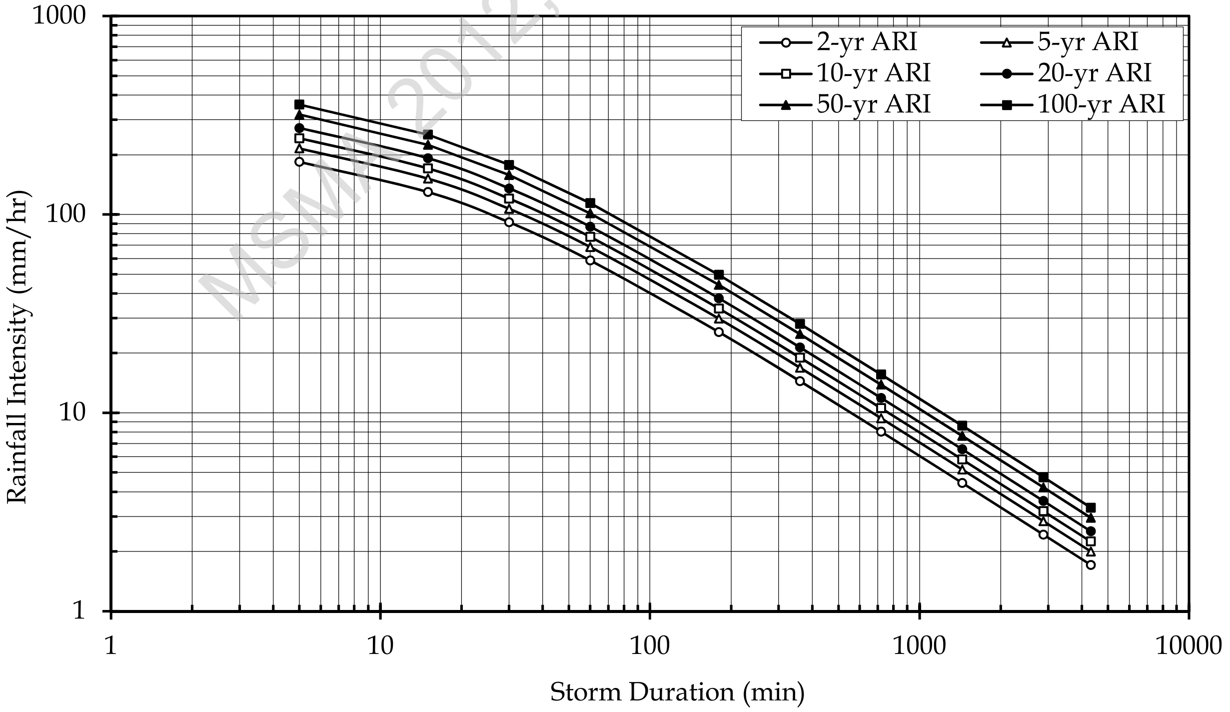


Figure AX3.11.1: Rainfall Station at JPS Sungai Manggis - 2815001



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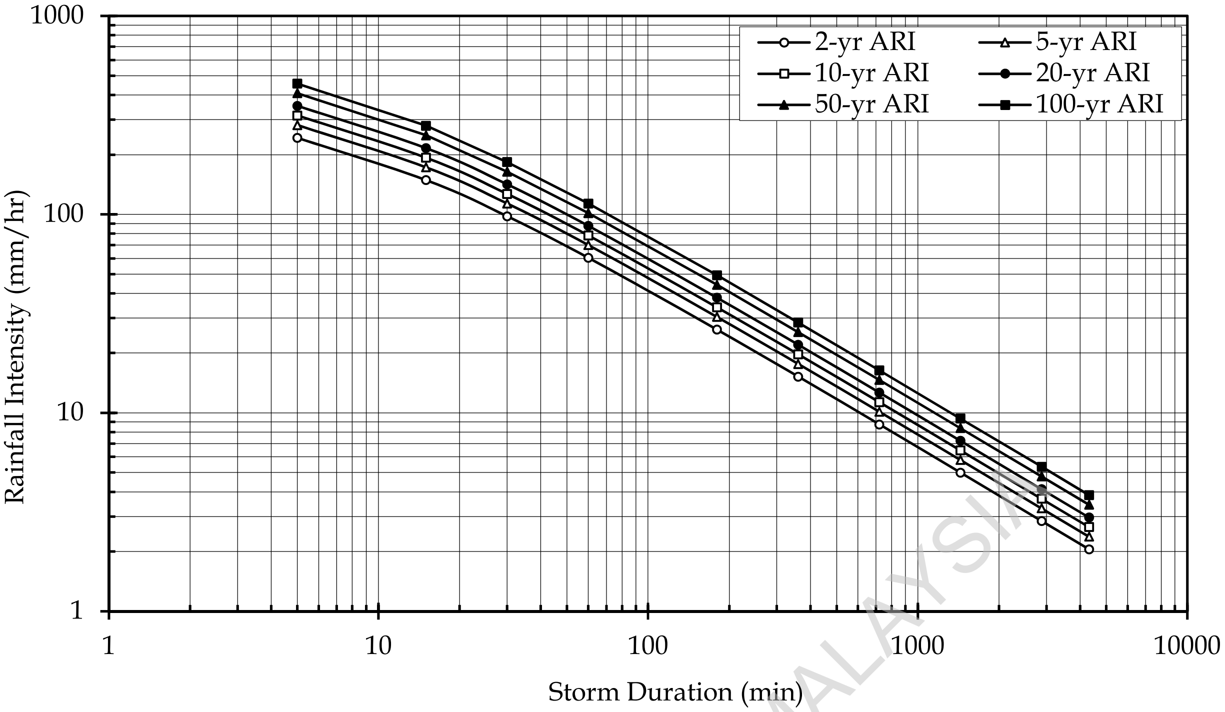
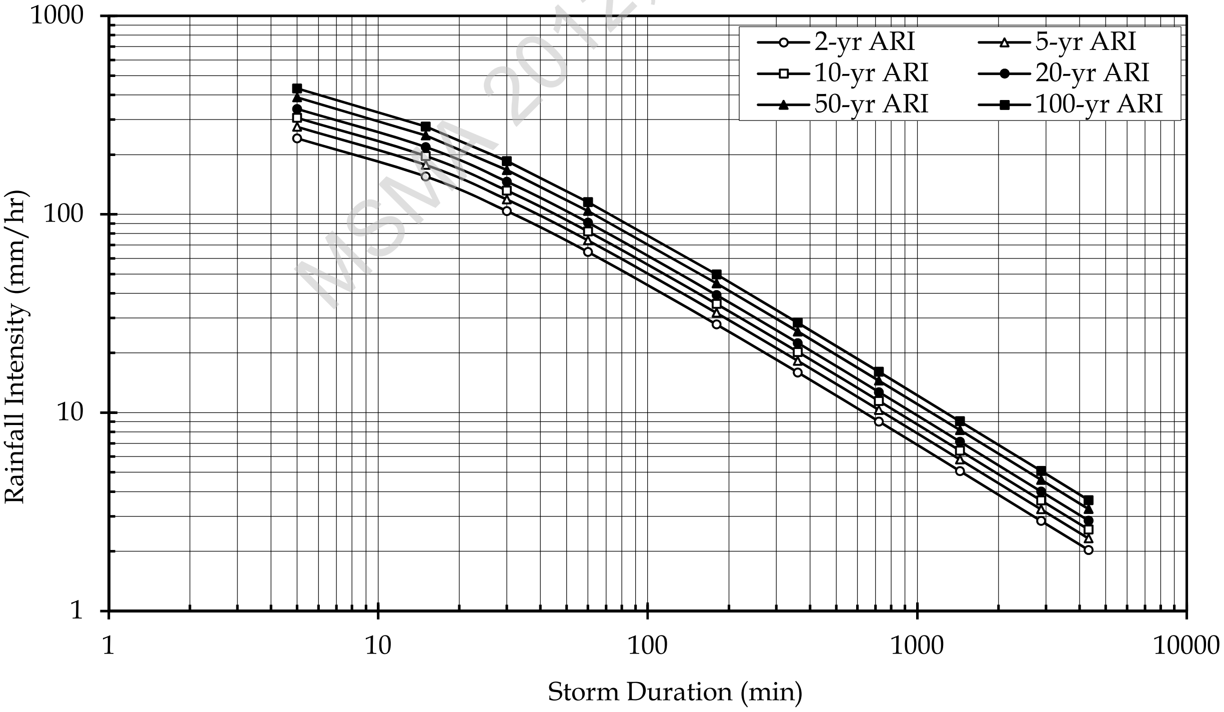


Figure AX3.11.3: Rainfall Station at Setor JPS Kajang - 2917001



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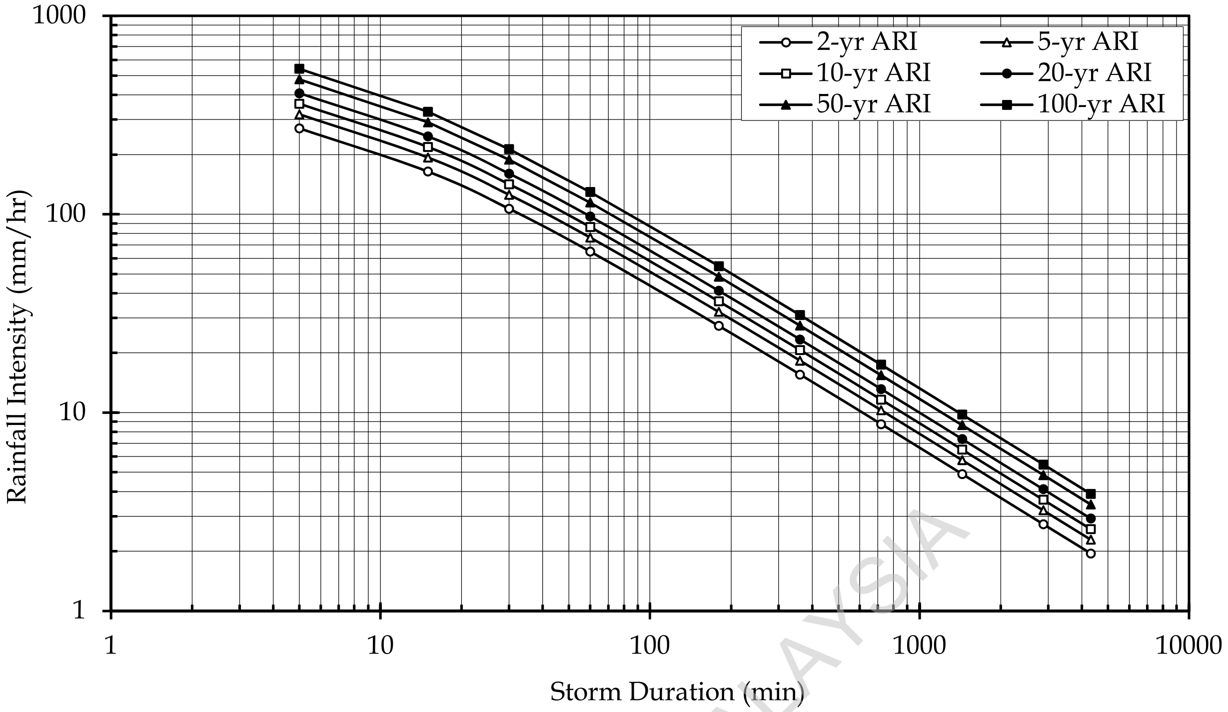


Figure AX3.11.5: Rainfall Station at SK Sungai Lui - 3118102

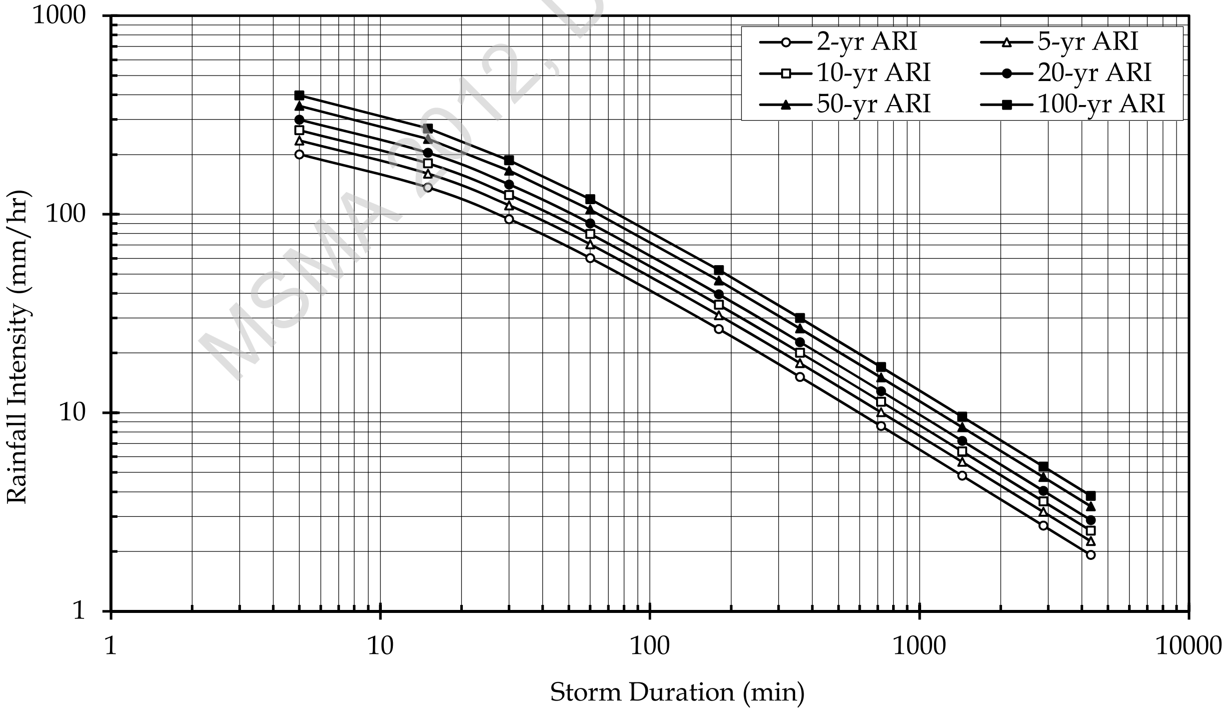


Figure AX3.11.6: Rainfall Station at Rumah Pam JPS P Setia – 3314001

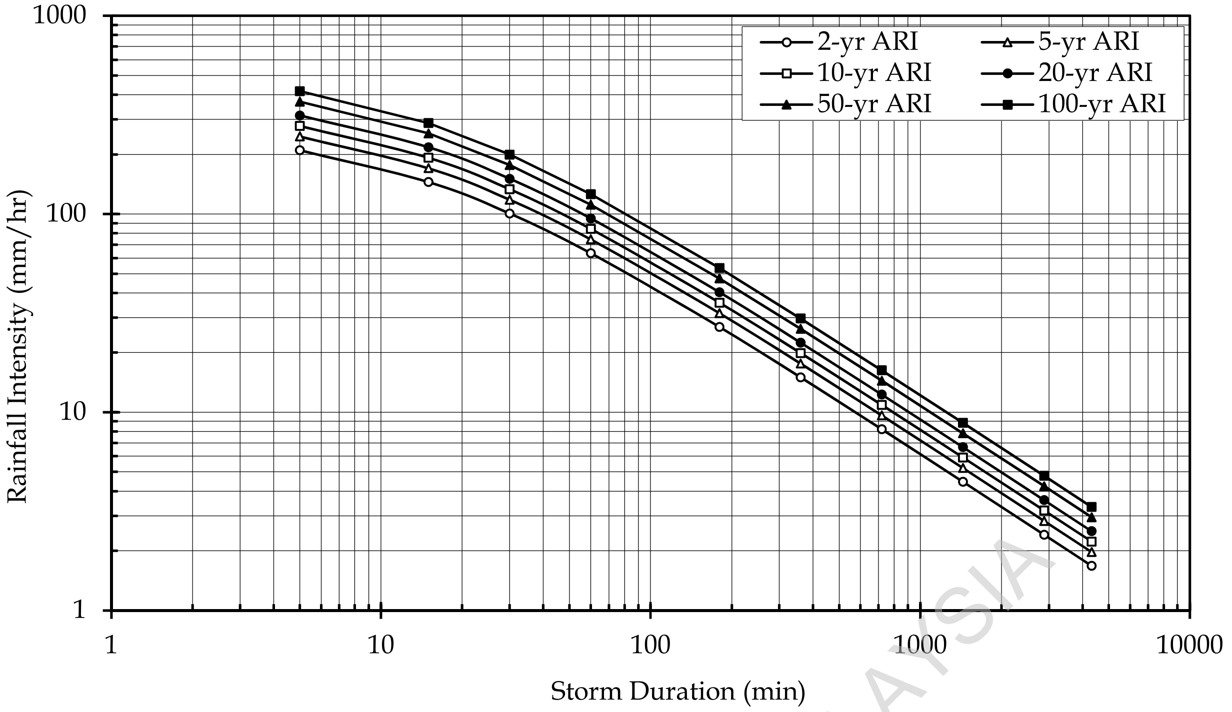


Figure AX3.11.7: Rainfall Station at Setor JPS Tj. Karang - 3411017

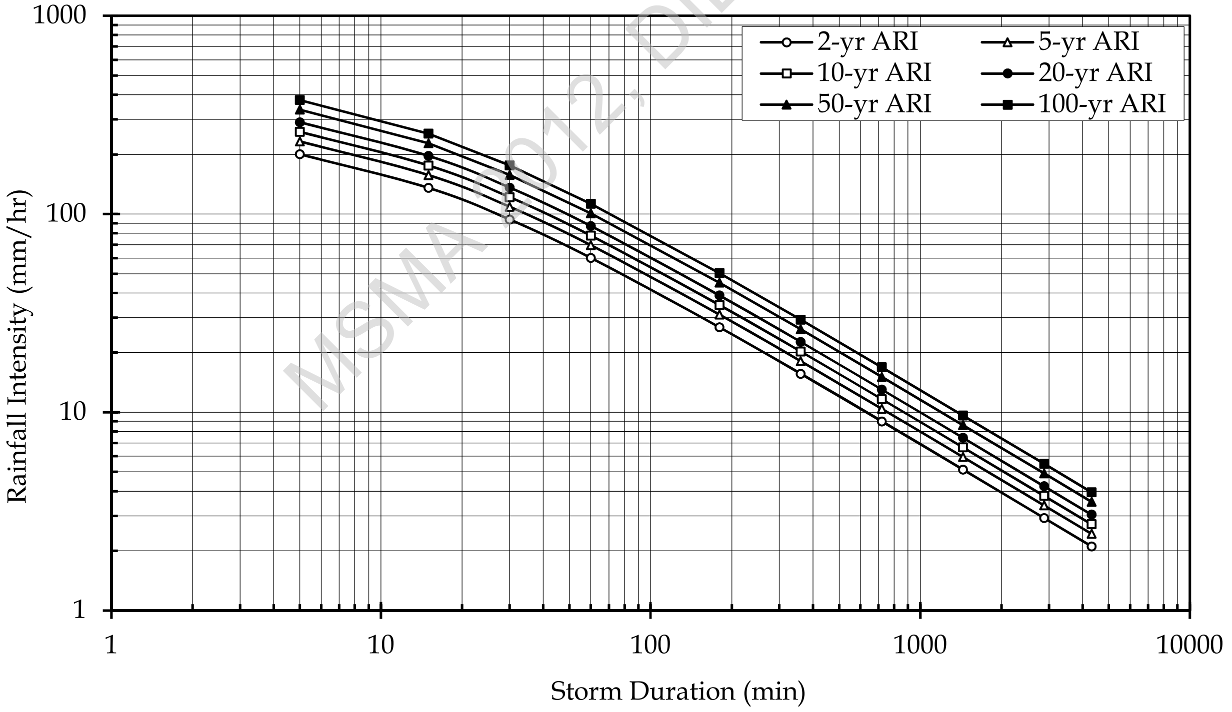


Figure AX3.11.8: Rainfall Station at Kg Kalong Tengah – 3416002

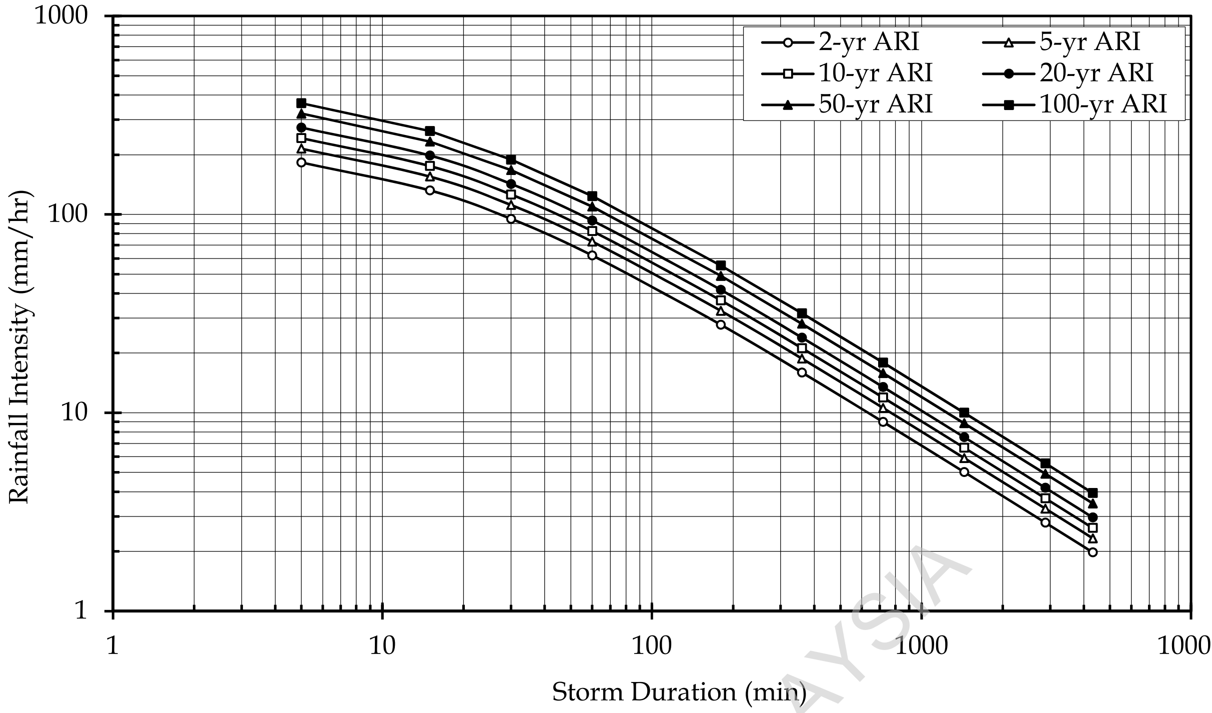


Figure AX3.11.9: Rainfall Station at Loji Air Kuala Kubu Baru - 3516022

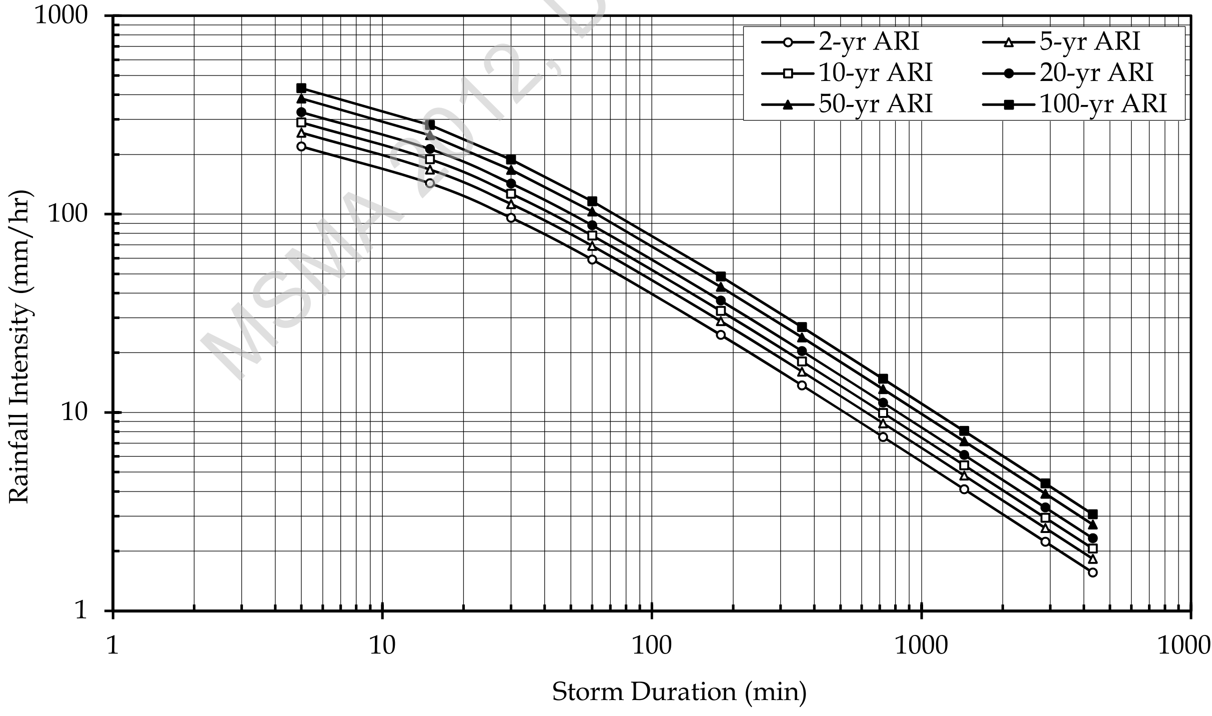


Figure AX3.11.10: Rainfall Station at Rmh Pam Bagan Terap – 3710006

AX3.12 STATE OF TERENGGANU

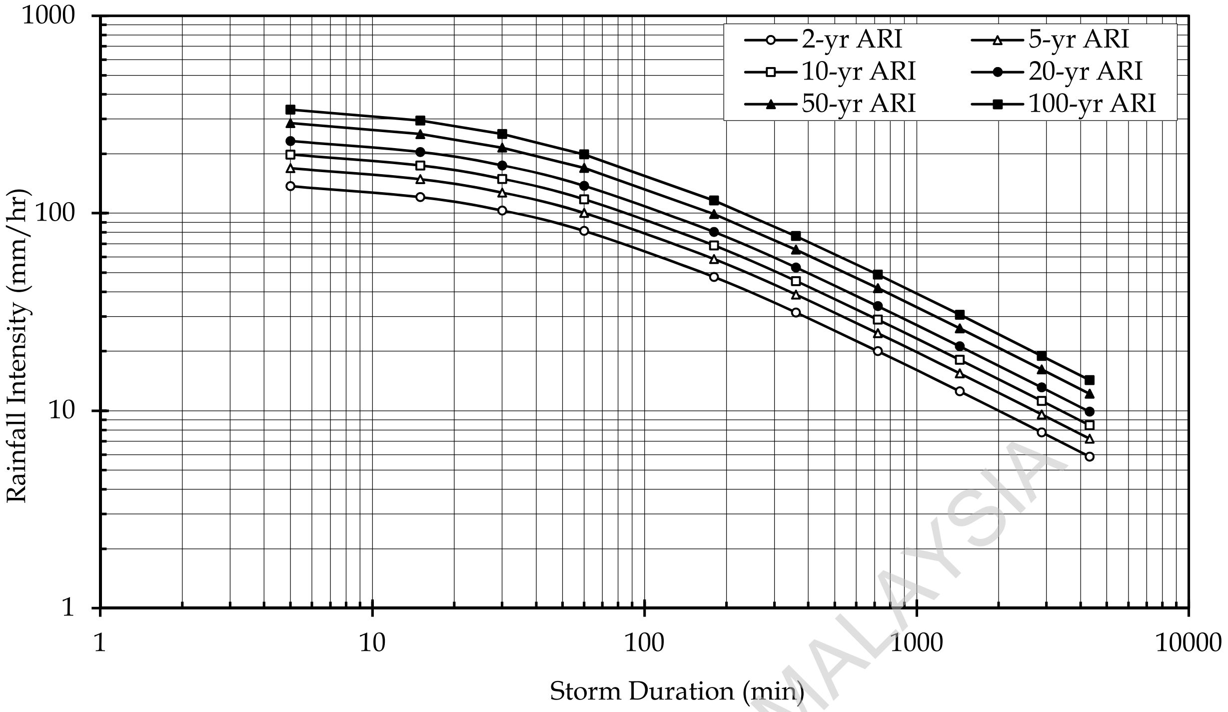


Figure AX3.12.1: Rainfall Station at Hulu Jabor, Kemaman -3933001

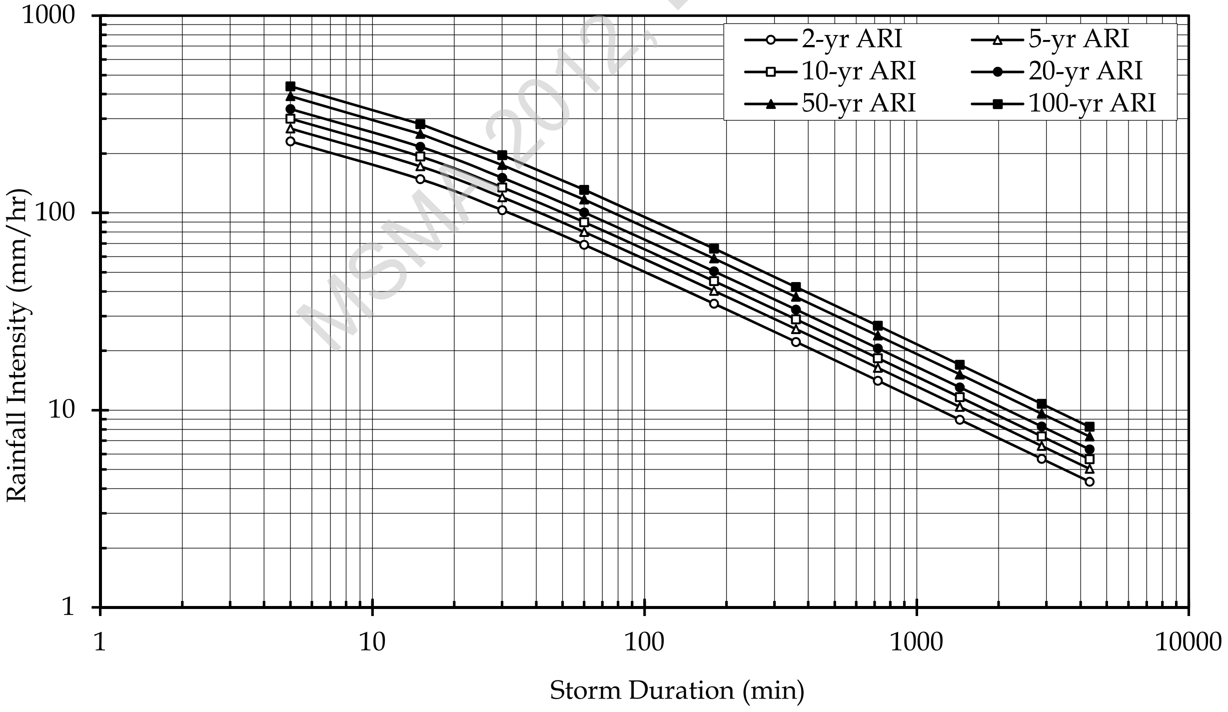
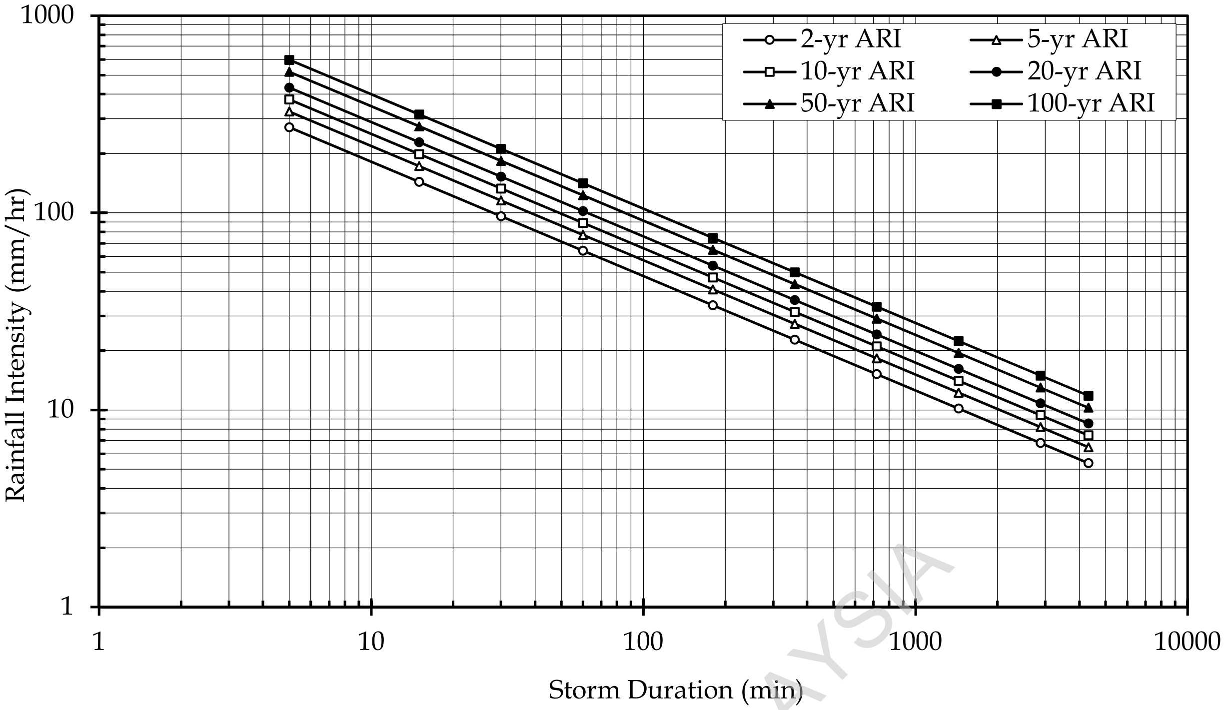


Figure AX3.12.2: Rainfall Station at Kg, Ban Ho, Kemaman-4131001



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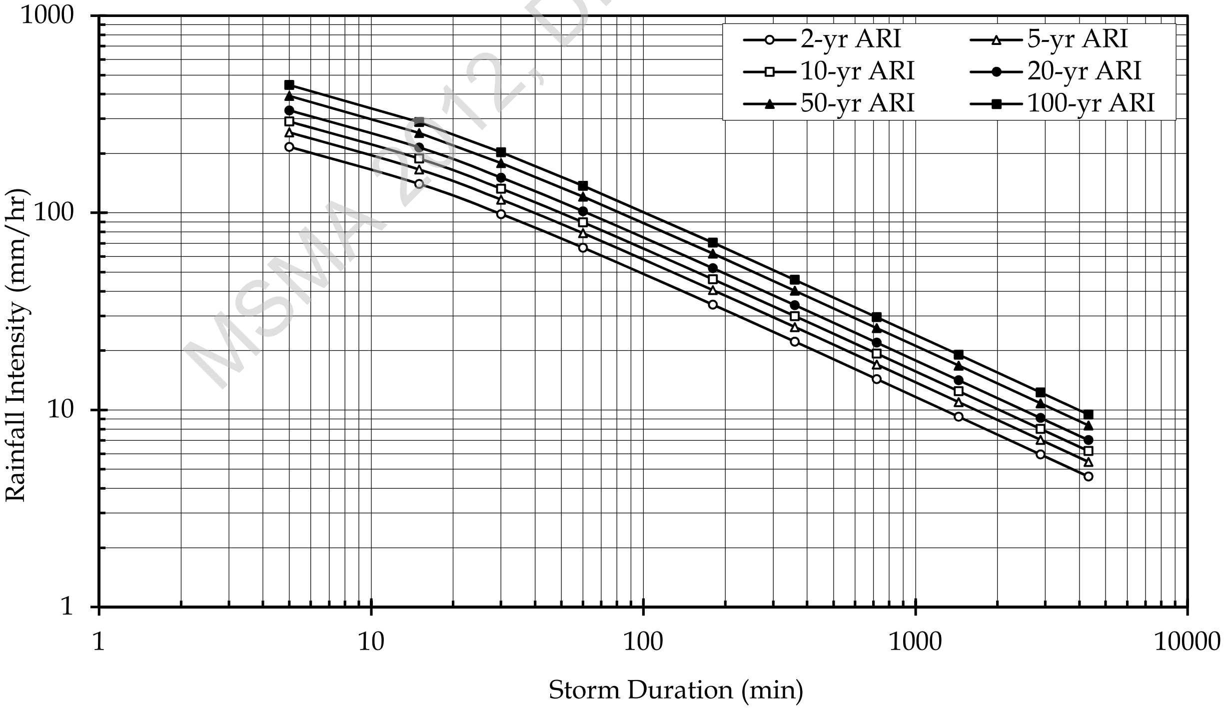

Figure AX3.12.4: Rainfall Station at Jambatan Tebak, Kem.-4332001

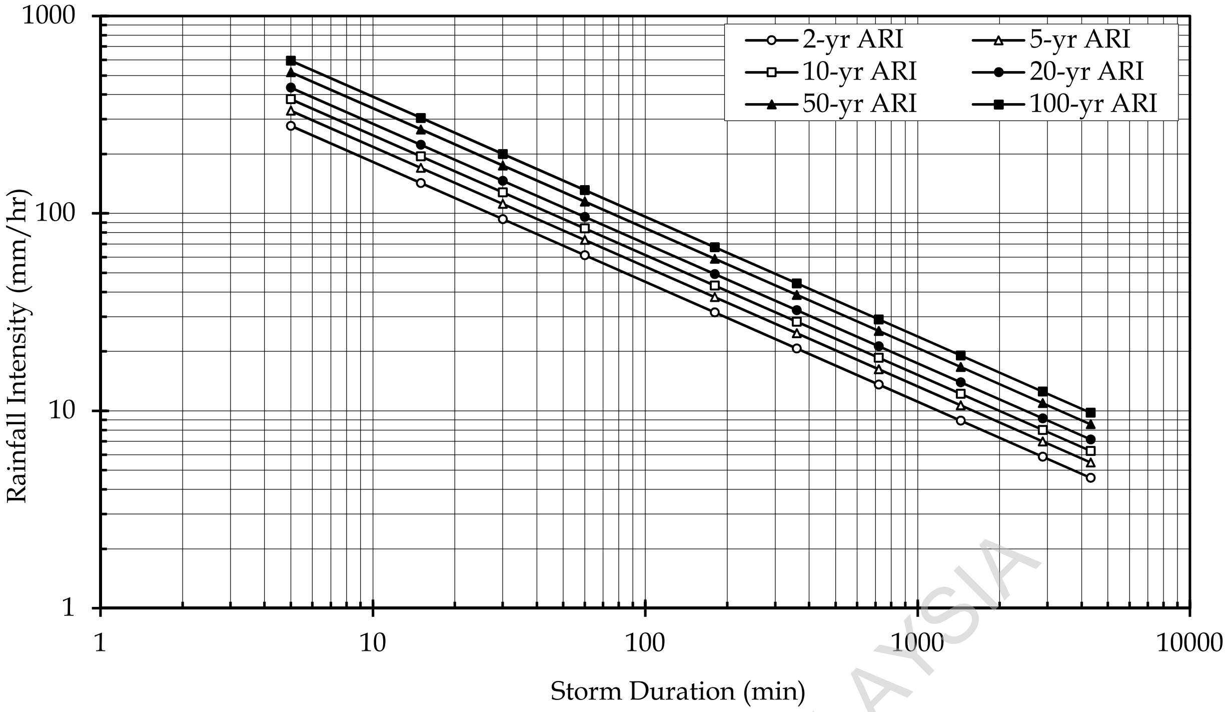

Figure AX3.12.5: Rainfall Station at Rmh Pam Paya Kempian -4529001

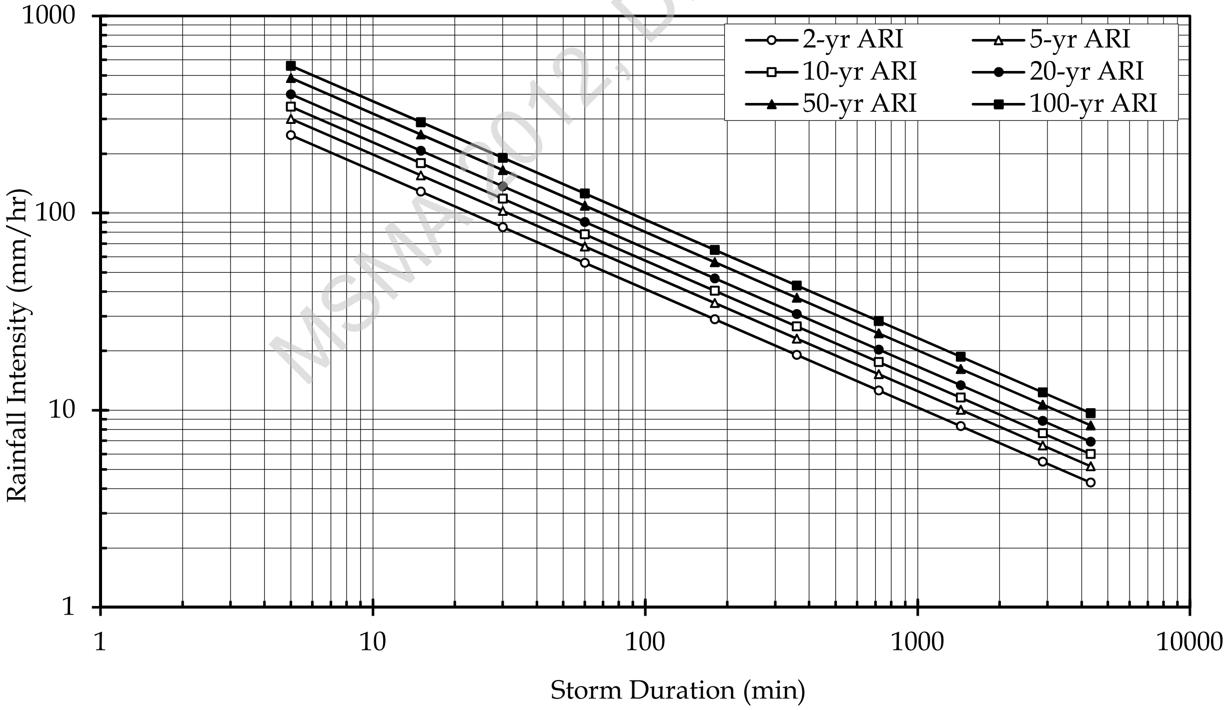

Figure AX3.12.6: Rainfall Station at SK Pasir Raja -4529071

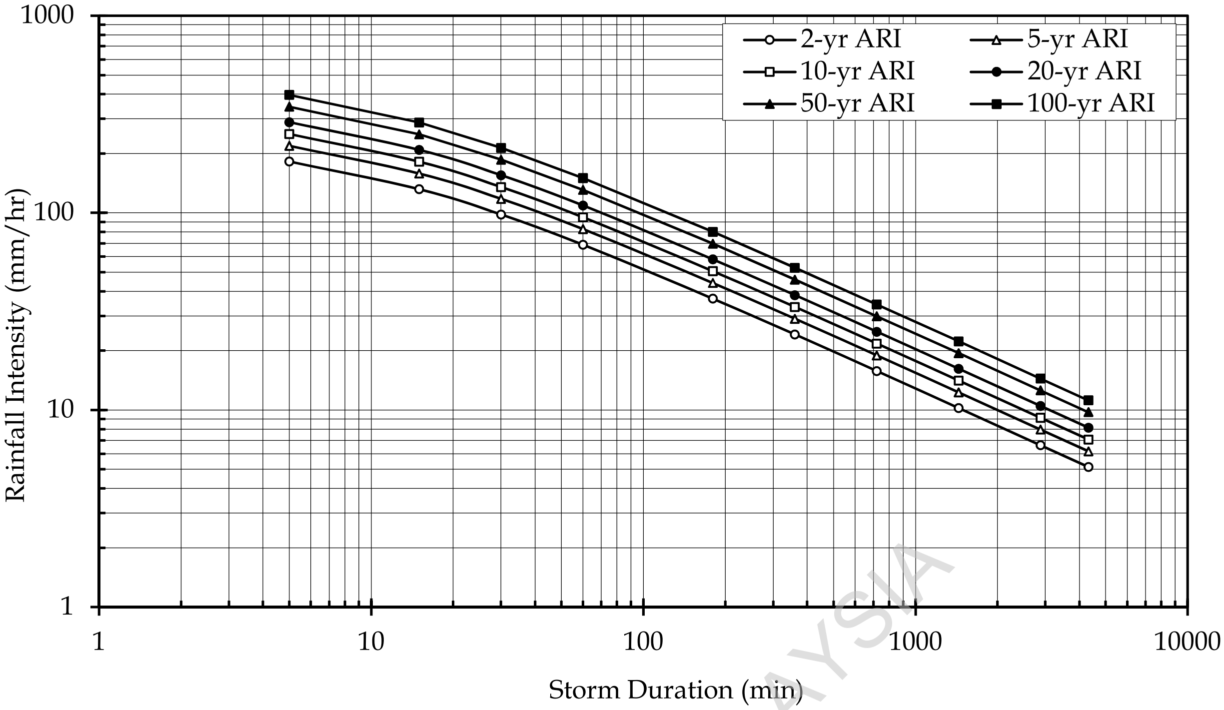

Figure AX3.12.7: Rainfall Station at Almuktafibillah Shah-4631001

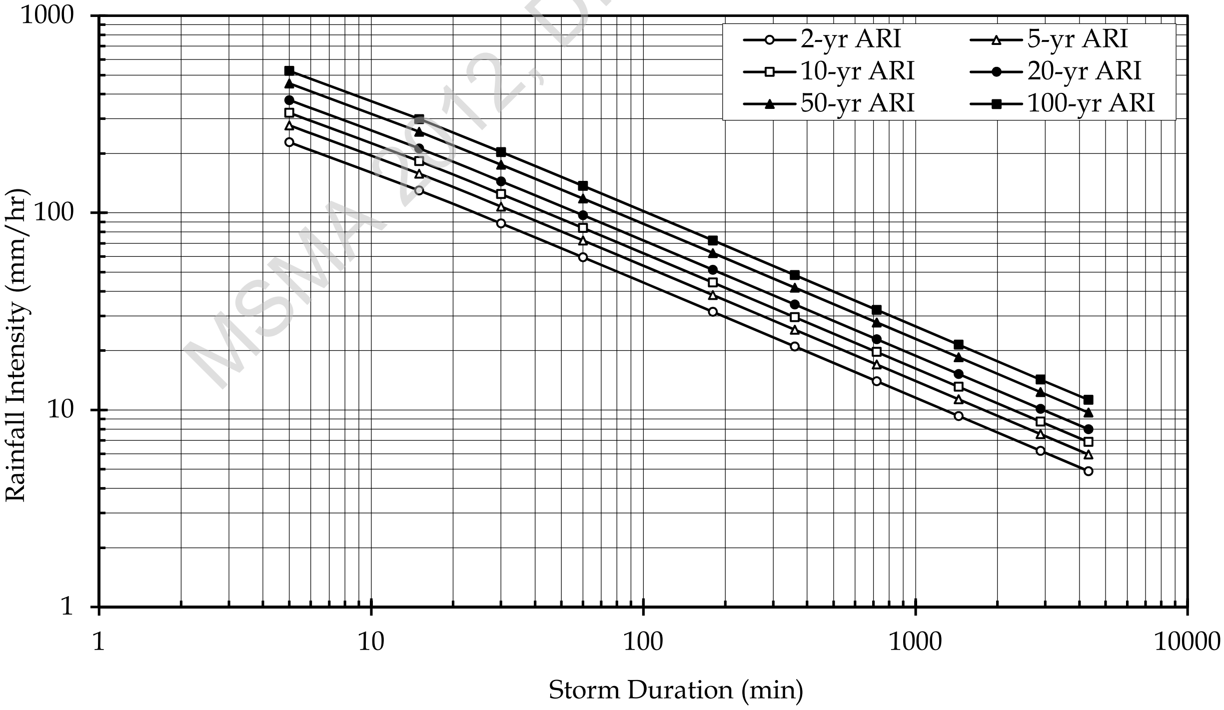

Figure AX3.12.8: Rainfall Station at SM Sultan Omar, Dungun-4734079

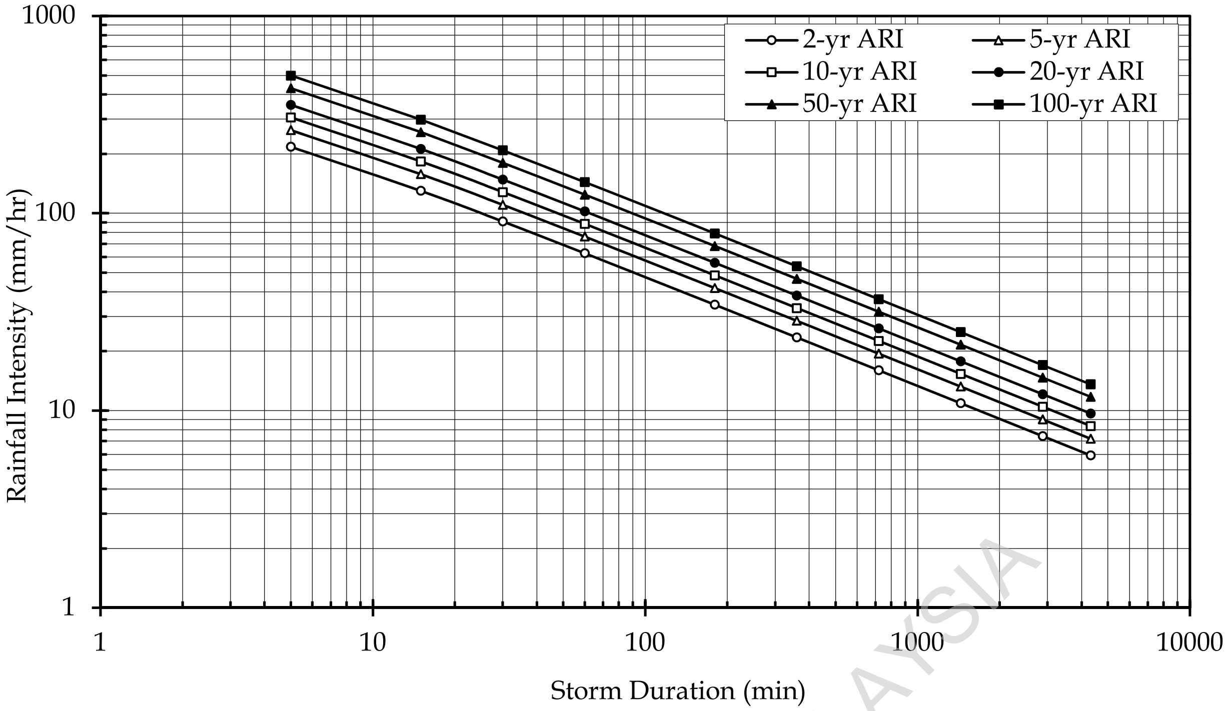

Figure AX3.12.9: Rainfall Station at SK Jerangau-4832077

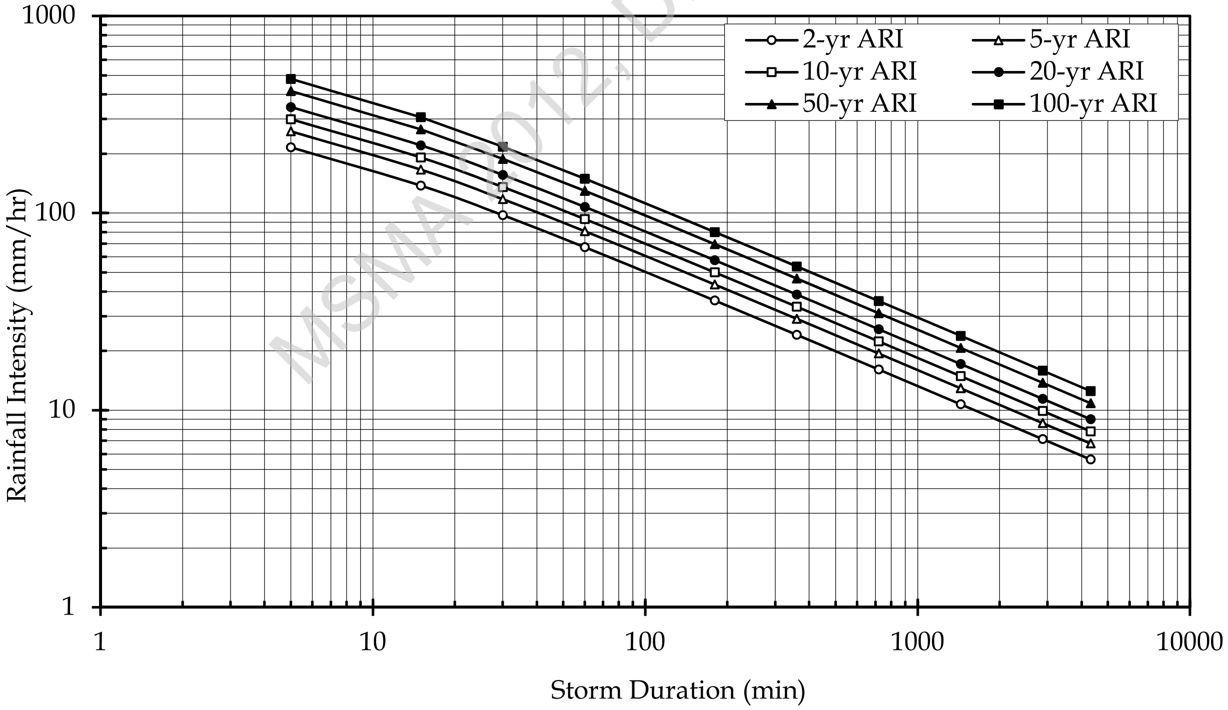

Figure AX3.12.10: Rainfall Station at Kg Menerong, Hulu Trg -4930038

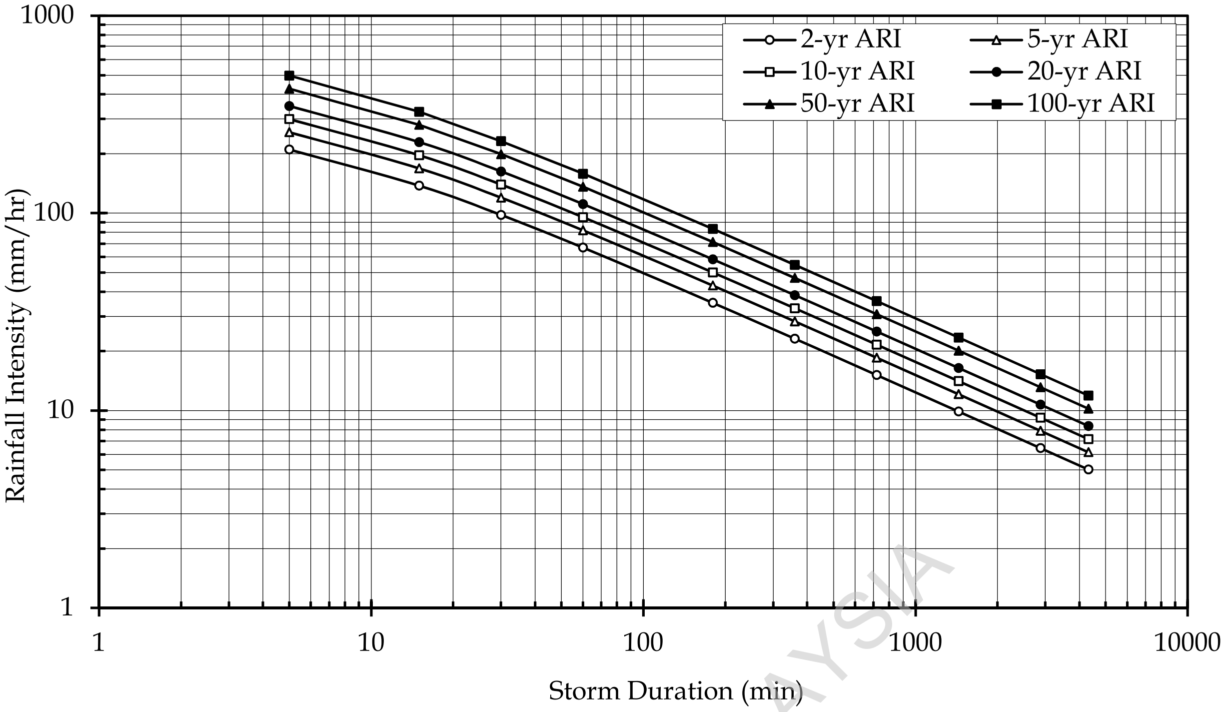

Figure AX3.12.11: Rainfall Station at Kg Dura. Hulu Trg -5029034

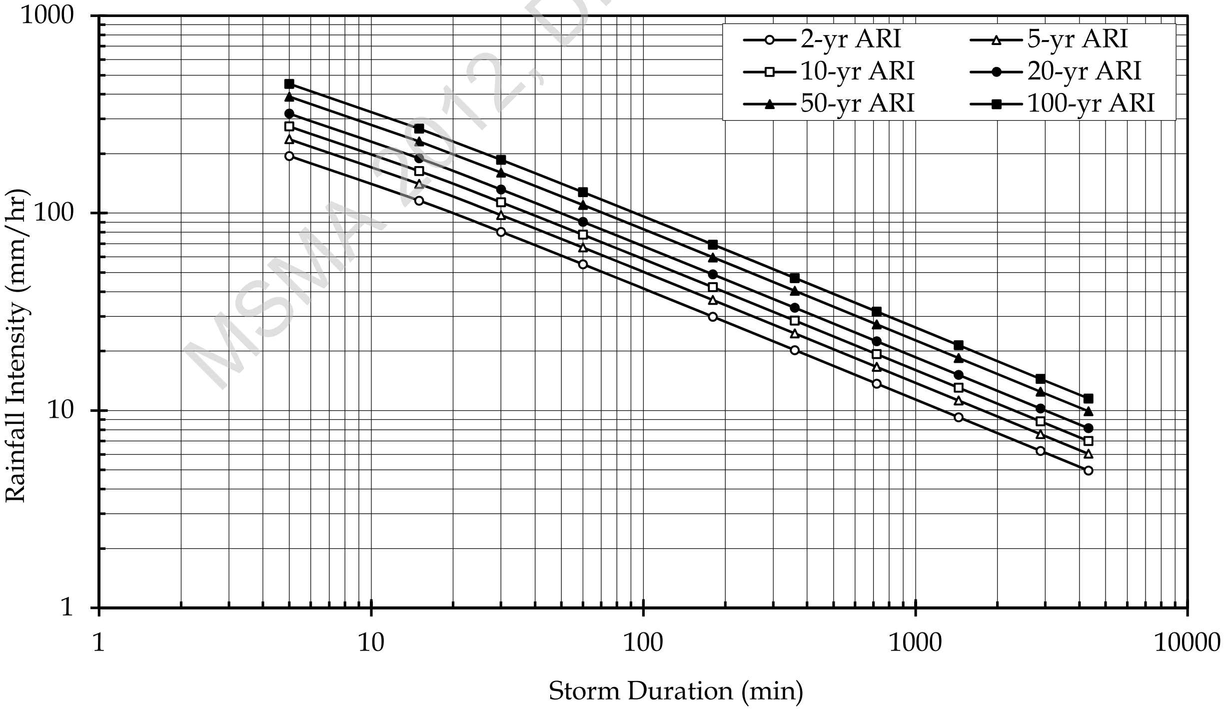

Figure AX3.12.12: Rainfall Station at Sungai Gawi, Hulu Trg -5128001

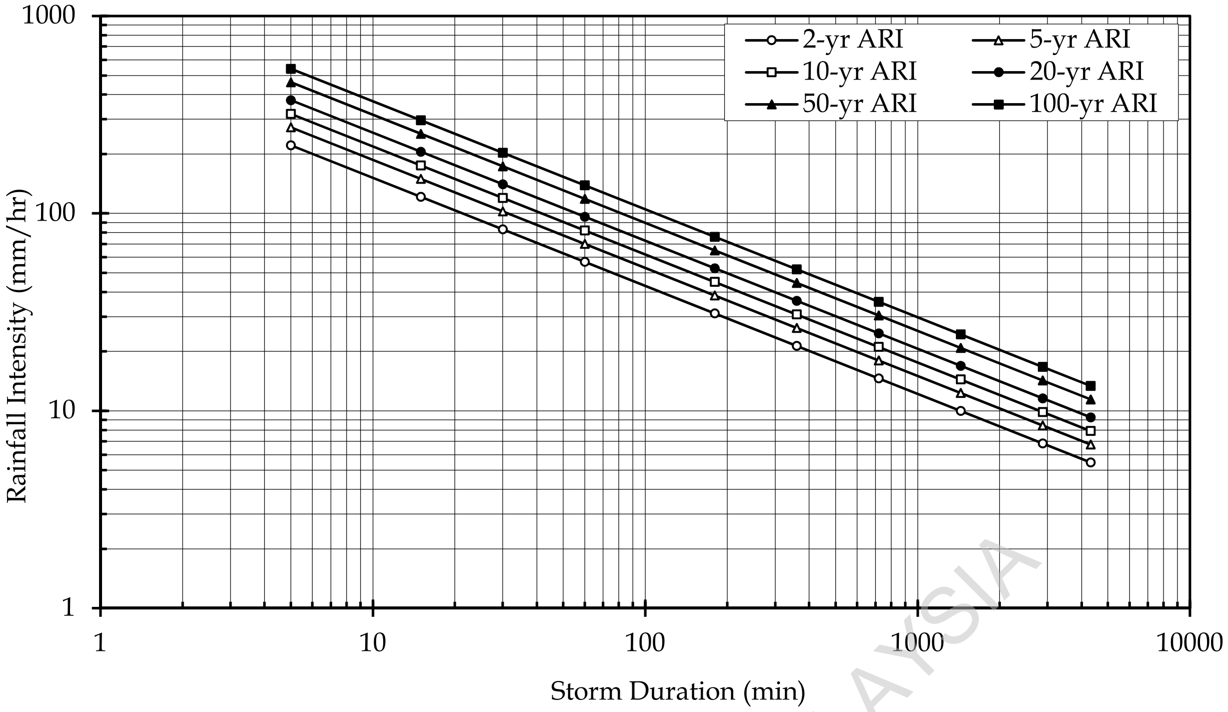

Figure AX3.12.13: Rainfall Station at Sg Petualang, Hulu Trg -5226001

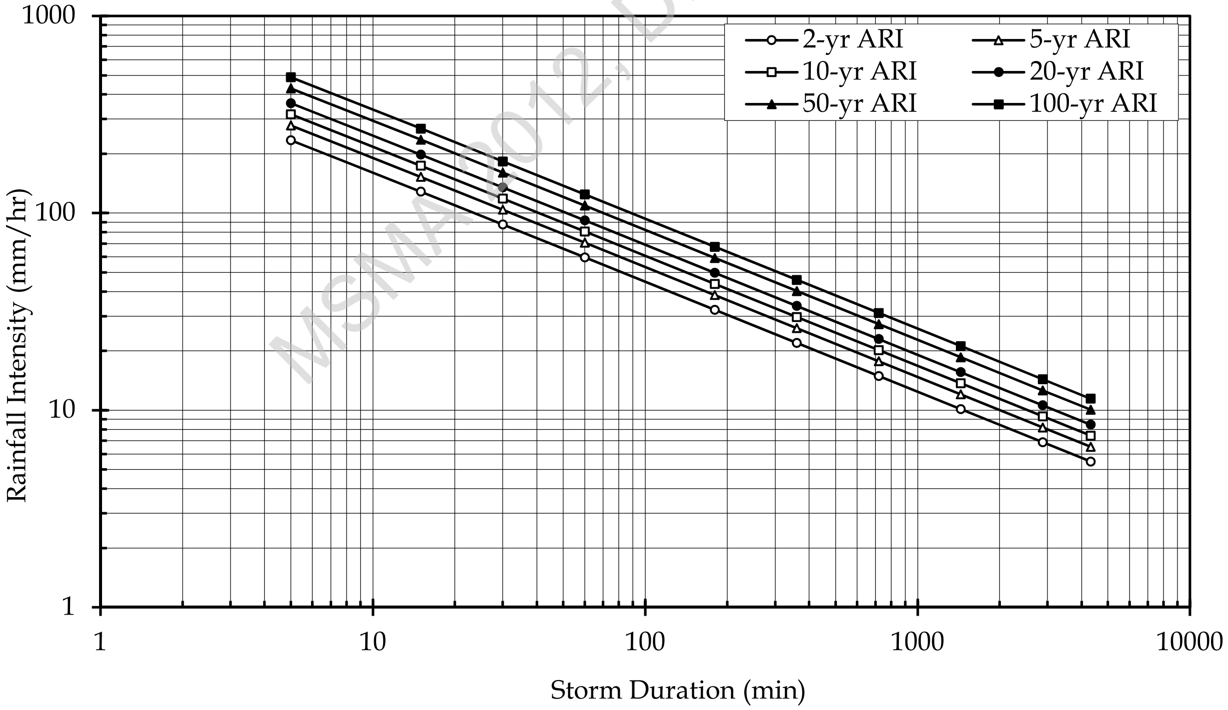

Figure AX3.12.14: Rainfall Station at Sungai Tong, Setiu-5328044

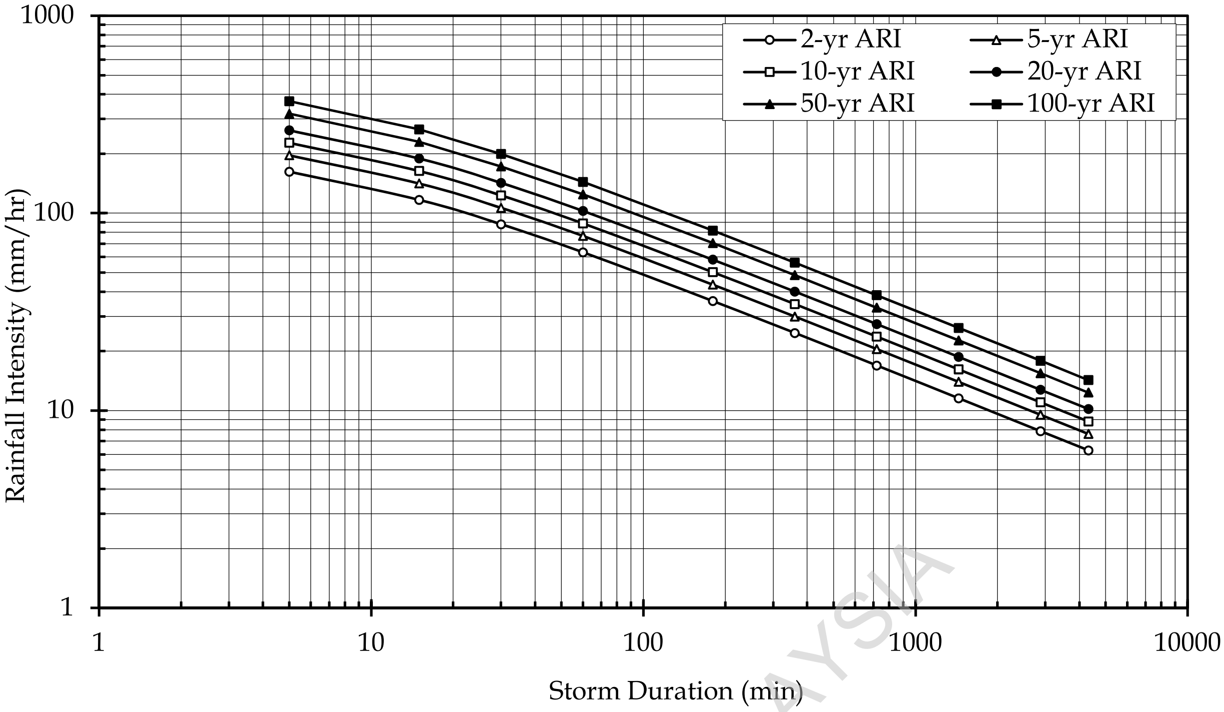

Figure AX3.12.15: Rainfall Station at Setor JPS K Terengganu-5331048

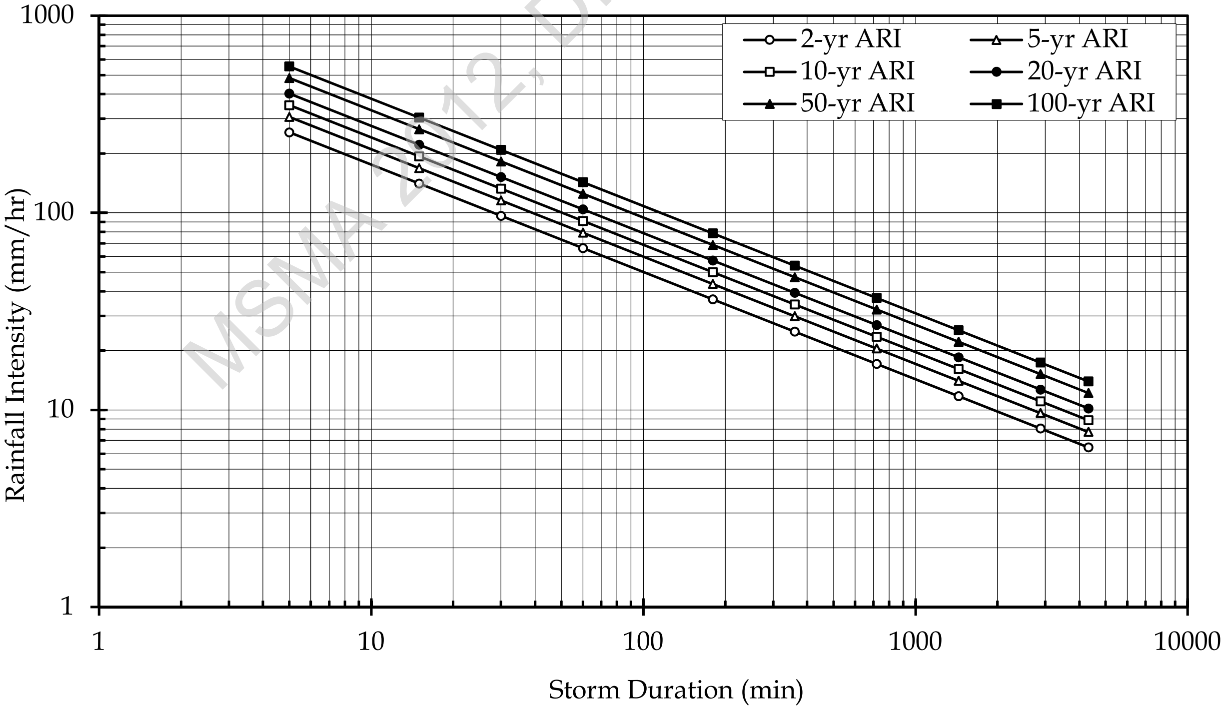

Figure AX3.12.16: Rainfall Station at Kg Seladang, Hulu Setiu-5426001

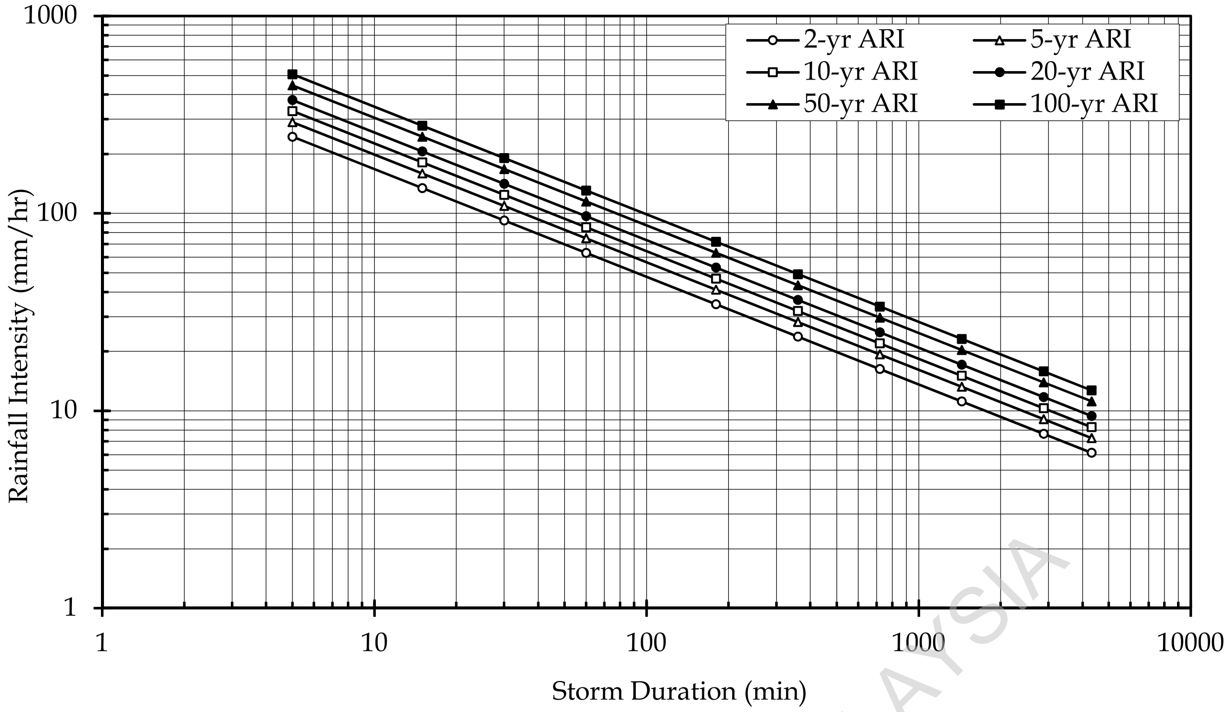

Figure AX3.12.17: Rainfall Station at Kg Bt. Hampar, Setiu-5428001

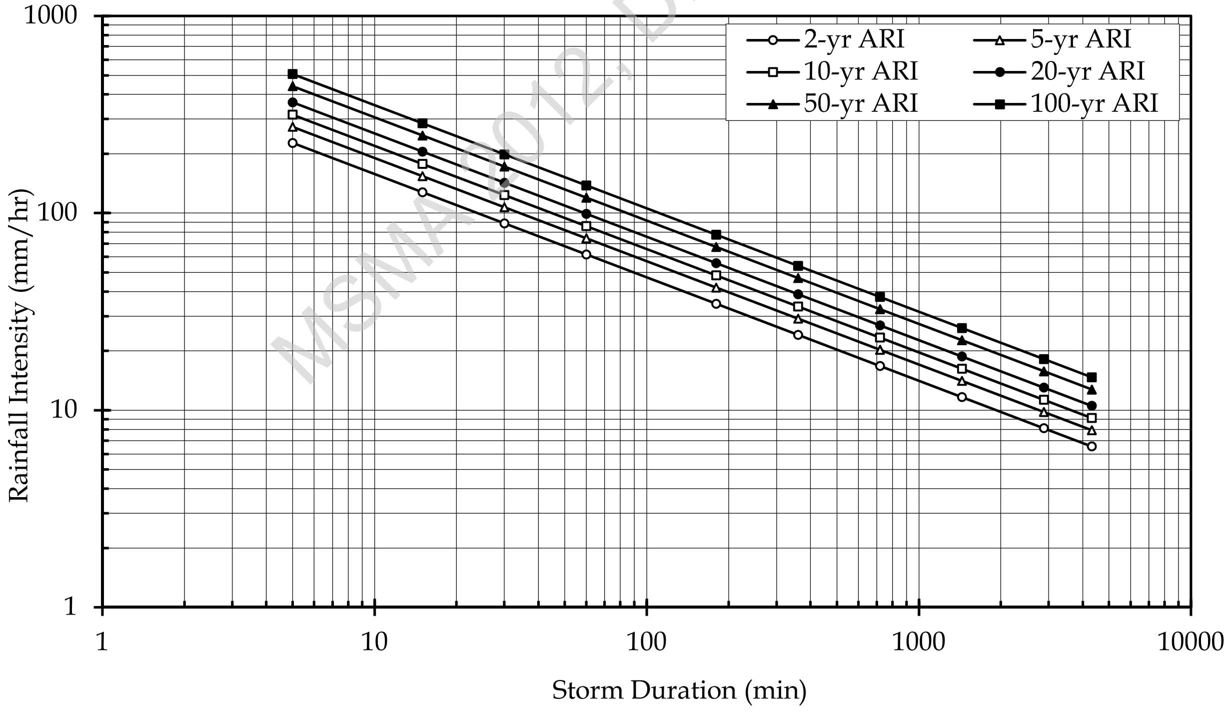

Figure AX3.12.18: Rainfall Station at SK Panchor, Setiu Klinik -5524002

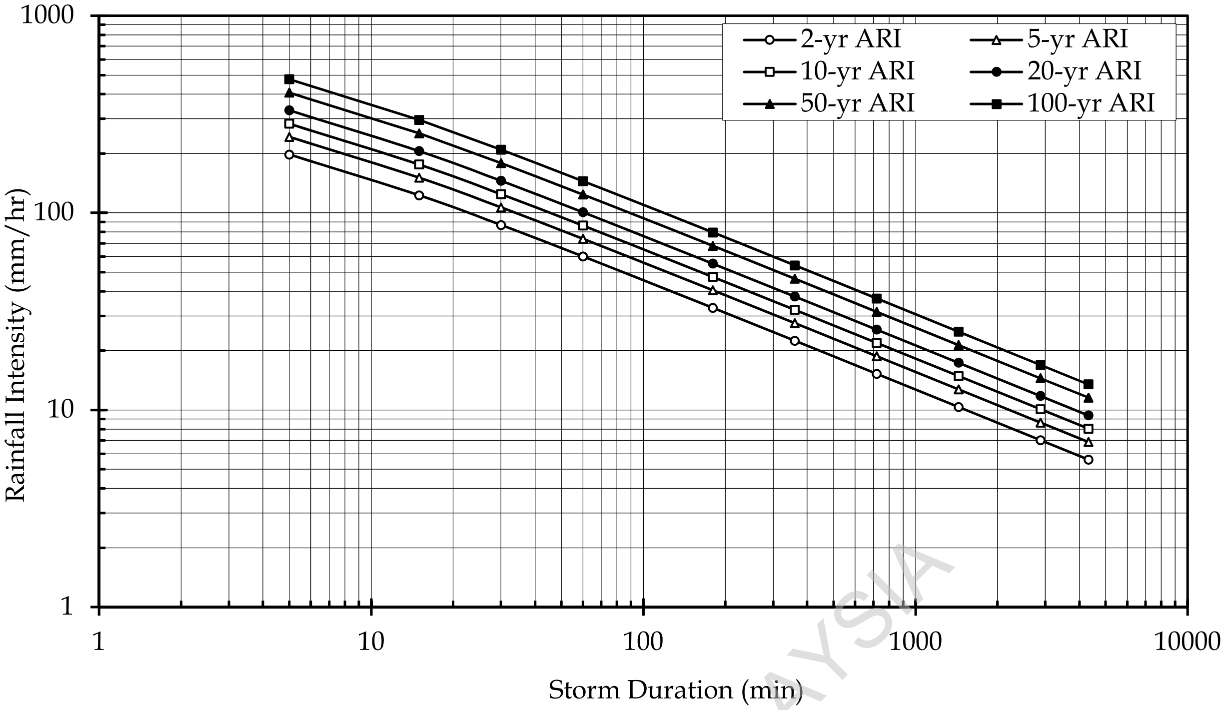

Figure AX3.12.19: Rainfall Station at Kg Raja, Besut-5725006

LIST OF CONTRIBUTORS

4
ANNEX

GLOSSARY

GLOSSARY

Term	Definition
Absorption	A process by which substances in gaseous, or solid form dissolve or mix with other substances.
Access Holes	Access structures and alignment control points in a storm drainage system.
Adsorption	Adherence of gas molecules, ions or molecules to solid surfaces.
Aeration	A process where a substance becomes permeated with air or another gas. The term is usually applied to aqueous liquids being brought into close contact with air by spraying, bubbling or agitating.
Aesthetic	The aspects of water that are perceivable by the senses (such as vision, smell, etc.).
Algae	Comparatively simple chlorophyll-bearing plants, most of which are aquatic and microscopic in size.
Ambient	The natural conditions that would be expected to occur in waters not influenced by man. For stream sampling purposes, those periods of streamflow are not influenced by recent storm events.
Annual Exceedance Probability (AEP)	Refers to the probability or risk of a natural event with a given size occurring or being exceeded in any given year. A 90% AEP event represents a high probability of flood occurring or being exceeded; meaning it would occur quite often and would be relatively small. On the other hand, a 1% AEP event has a low probability of occurrence or being exceeded; therefore it would be fairly rare but it would be relatively large.
Antecedent Moisture Condition (AMC)	A qualitative indication of the moisture content of surficial soils at the beginning of a storm event.
Anti-Seep Collar	A device installed around a culvert, pipe or conduit through an embankment, which lengthens the path of seepage along the exterior of the conduit.
Aquatics	Plants that grow either partly or completely submerged in water.
Average Recurrence Interval (ARI)	The average elapsed time in years between floods of a given size occurring. For example a 1 year flood occurs on average once every year, therefore the ARI value would be relatively small. Contrast to that, a 100 year ARI flood (i.e. occurs on average once every one hundred years and fairly rare) would have a relatively large ARI value.
Bankfull Discharge	A condition where flow from a stream completely fills the stream channel up to the top of the bank. In disturbed watersheds, the discharge condition occurs on average every 1.5 to 2 years and this controls the slope and form of the natural channels.
Barrel	A closed conduit used to convey water under or through an embankment, which is a part of the principal spillway.
Baseflow	The portion of a stream flow that is not due to storm runoff, and is supported by groundwater seepage into a channel.
Berm	A shelf that breaks the continuity of a slope; a linear embankment or dike.

Term	Definition
Best Management Practice (BMP)	A structure or practice designed in stormwater management to prevent the discharge of one or more pollutants to the land surface thus minimising the chance of wash-off by stormwater. It can also be referred to a structure or practice to temporarily store or treat urban stormwater runoff to reduce flooding, remove pollutants, and provide other amenities (such as recreational, fishing spots, etc.).
Biochemical Oxygen Demand (BOD)	The quantity of oxygen consumed during the biochemical oxidation of matter over a specified period of time (See also COD). It is measured in the dark as the decrease in the oxygen content (in terms of mm/L) of a water sample with a certain temperature over a certain period of time. The decrease in oxygen content is brought about by the bacterial breakdown of organic matter. The decomposition usually proceeds as far as after 20 days until no further change occurs. The oxygen demand is measured after 5 days (and is termed BOD_5), where 70% of the final value is expected to be reached.
Bioengineering	Restoration and stabilisation techniques that use plants, often from native species, to mimic the natural functions and benefits.
Biofiltration	The use of a series of vegetated swales to provide filtering treatment for stormwater as it is conveyed through a channel. These swales can either be grassed, contain emergent wetlands or high marsh plants.
Biofiltration Swale	A sloped, vegetated channel or ditch that provides both conveyance and water quality treatment to stormwater runoff. It does not provide stormwater quantity control but may convey runoff to BMPs, which are designed for that purpose.
Biological Indicators	Organisms (e.g. plants, macro-invertebrates and fish) that serve as indicators of the quality and characteristics of that waterbody.
Bioretention	A water quality practice that utilises landscaping and soils to treat urban stormwater runoff by collecting it in shallow depressions before filtering through a fabricated planting-soil media.
Bloom	An unusually large number of organisms in a unit of water, usually made up of one or more algae species.
Box Gutter	A graded channel generally of a rectangular shape used for the conveyance of rainwater located within buildings.
Buffer	The zone contiguous with a sensitive area that requires continual maintenance, function, and structural stability. The critical functions of a riparian buffer (those associated with an aquatic system) include shading, input of organic debris and coarse sediments, uptake of nutrients, stabilisation of banks, interception of fine sediments, overflow during high water events, protection from disturbance by human and domestic animals and maintenance due to hydrologic or climatic effects. The critical functions of terrestrial buffers include protection of slope stability, attenuation of surface water flows from storm water runoff plus precipitation and erosion control.
Bypass Flow	Flow which eludes an inlet on grade and is carried to the next inlet downgrade in the street or channel.

Term	**Definition**
Catchbasin	A chamber or well usually built at the kerb line of a street for the admission of surface water to a sewer or subdrain. At the base of the chamber or well, a sediment sump is designed to retain any grit and detritus located below the point of the overflow.
Catchment	An area draining flow to a particular location or site. It may frequently include an area of tributary streams and flow paths as well as the main stream.
Check Dam	An earthen or gabion structure placed perpendicular across a stream to enhance aquatic habitats. This structure when used in grass swales reduces water velocities, promotes sediment deposition and enhances infiltration.
Check Storm	A lesser frequency event used to assess hazards at critical locations.
Chemical Oxygen Demand (COD)	A monitoring test that measures all the oxidisable matter found in a runoff sample in which a portion of these matters deplete dissolved oxygen in receiving waters.
Combination Inlets	The use of both a curb opening inlet and a grade inlet.
Concentration	The quantifiable amount of chemicals in the surrounding water, food or sediment.
Conservation	The protection, improvement and use of natural resources according to principles resulting in greater economic and social benefits.
Constructed Wetlands	The creation of a wetland on a site, which is designed specifically to remove pollutants from stormwater runoff.
Contributing Watershed Area	A portion of the watershed contributing to its runoff at the point of interest.
Conventional Pollutants	Contaminants other than nutrients (such as sediments, oil, and vehicle fluids).
Conveyance	A mechanism for transporting water from one point to another (which includes pipes, ditches and channels).
Conveyance System	It refers to drainage facilities of both natural and man-made that collect, contain and provide for the flow of surface and stormwater from the highest points of the land right down to receiving waters. The natural elements of the conveyance system include swales and small drainage courses, streams, rivers, lakes and wetlands. The human-made elements of the conveyance system include gutters, ditches, pipes, channels and most retention/detention facilities.
Critical Depth	The depth of flow during critical flow events.
Critical Flow	The flow in an open channel that is at a minimum specific energy and has a Froude Number equal to 1.0.
Cross Slope	The rate of change of roadway elevation with respect to the distance perpendicular to the direction of travel. It is also known as transverse slope.
Cumulative	Brought about or the increased in strength, by successive additions at different times or in different ways.

Term	**Definition**
Darcy's Law	An empirical law based on experimental evidence for the flow of fluids assuming the flow is laminar and inertia can be neglected. It states that the velocity of the flow through a formation is directly proportional to the hydraulic gradient.
Dead Storage	A permanent pool volume located below the outlet structure of a storage device. Dead storage provides water quality treatment but not water quantity treatment.
Design Storm	A selected rainfall event of specified amount, intensity, duration and frequency used as the basis of design.
Detention	A temporary storage of storm runoff in a BMP, which is used to control the peak discharge rates by controlled release rate(s).
Detention Time	The amount of time a volume of water is detained in a BMP.
Detritus	Unconsolidated sediments composed of inorganic (i.e. dead and decaying) and organic material.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Direct Runoff	The streamflow produced in response to a rainfall event and is equal to the total stream flow minus its baseflow.
Discharge	The volume of water that passes a given location within a given period of time (e.g. outfall; the flow of water from a well, a pump, a pipe, a drainage basin or an aquifer in m^3/s).
Discharge Area	An area in which water is lost from the saturated zone.
Discharge Structure	The outlet structure of a structural BMP (such as a pond) designed to release water at a designed flow rate.
Dissolved Constituent	Constituents in a water sample that will pass through a 0.45 µm membrane filter.
Drainage Area	The area of a watershed within which all surface runoff drains by gravity into a stream channel or lake upstream of a given location.
Drainage Easement	A legal encumbrance that is placed against a property's title to reserve specified privileges for the users and beneficiaries of the drainage facilities contained within the boundaries of the easement.
Drainage Inlets	The receptors for surface water collected in ditches and gutters, which serve as a mechanism whereby surface water enters storm drains and this refers to all types of inlets (such as grate inlets, curb inlets, slotted inlets, etc.).
Drawdown	The vertical distance where the free water elevation is lowered, or the reduction of the pressure head due to the removal of free water.
Dry Pond	A facility that provides stormwater quantity control by containing excess runoff in a detention basin and then releasing it at allowable levels.
Dry Vault/Tank	A facility that treats stormwater for water quantity control by detaining runoff in underground storage units and then releasing reduced flows at established standards.

Term	**Definition**
Dry-pit Stations	Pump stations that use both wet and dry wells. Stormwater is stored in the wet well, which is connected to the dry well by horizontal suction piping. The stormwater pumps are located on the floor of the dry well.
Eaves Gutter	A channel, for the conveyance of rainwater, located along the eaves of a roof external to the fascia line. A concealed eaves gutter is located inside the fascia line and can also be called an internal eaves gutter.
Ecology	The study of the habits and modes of life-living organisms (such as plants and animals), and their relationships to each other and their environment.
Effluent	Waste material (e.g. liquid industrial discharge or sewage) that may be discharged into the environment.
Emergency Spillway	The channel of a pond-type BMP, designed to pass a storm event that exceeds the design capacity of the primary discharge structure.
Emergent Plants	Aquatic plants that are rooted in the sediment but whose leaves are at or above the water surface. These wetland plants often have high habitat values for wildlife and waterfowl, and can aid in pollutant uptake.
Endemic	Confined to or originating in a given region as an island. Area or country and found nowhere else.
Energy Dissipater	Any means by which the total energy of flowing water is reduced. In stormwater design, they are usually mechanisms that reduce velocity prior to, or at, discharge from an outfall in order to prevent erosion. They include rock splash pads, drop manholes, concrete stilling basins or baffles, and check dams.
Environmental Values	Particular values or uses of the environment that are conducive to public benefit, welfare, safety or health and that require protection from the effects of pollution, waste discharges and deposits. Several environmental values may be designated for a specific waterbody.
Ephemeral Stream	A stream which does not flow continuously or flows only for short period of time.
Erosion	The wearing of the land surface by water or wind and the subsequent detachment and transportation of soil particles.
Erosion and Sediment Control	Temporary or permanent measures taken to reduce erosion, control siltation and sedimentation, ensuring that sediment-laden water does not leave a site.
Erosive Velocities	Velocities of water that are high enough to wear away land surface. Exposed soil will generally erode faster than stabilised soils. Erosive velocities will vary according to the soil type, slope, structural or vegetative stabilisation used to protect the soil.
Eutrophic	Abundant in nutrients and having high rates of productivity frequently resulting in oxygen depletion below the surface layer of a waterbody.
Eutrophication	Enrichment of water with nutrients, primarily phosphorus, causing abundant aquatic plant growth (mainly algae blooms).
Evaporation	The physical process by which a liquid (such as water) in a stream, lake or moist soil is transformed into a gaseous state. It may be expressed as the total (or the mean) rate in units of mass (or volume) per unit area or as an equivalent depth of water for the period concerned.

Term	Definition
Evapotranspiration	The total water vapour loss from an area by evaporation and transpiration from plants over a given time period. It includes the evaporation of water from soils, dew and intercepted precipitation, as well as transpiration from plants.
Event Mean Concentration (EMC)	The average concentration of an urban pollutant measured during a storm runoff event. The EMC is calculated by flow-weighing each pollutant sample measured during a storm event.
Excess Rainfall	An amount of rainfall greater than what the soil can absorb, resulting in runoff.
Exfiltration	The downward movement of runoff through the bottom of an infiltration BMP into the soil layer.
Extended Detention	A stormwater management BMP that provides for the gradual release of a volume of water over a time interval designed to increase settling of urban pollutants and protect downstream channels from frequent flooding.
Extended Detention Dry Ponds	Depressed basins that temporarily store a portion of the stormwater runoff following a storm event. The extended detention time of the stormwater provides an opportunity for urban pollutants carried by the flow to settle out.
Faecal Coliform Bacteria	Minute living organisms associated with human or animal faeces that are used as an indirect indicator of the presence of other disease-causing bacteria.
Filtration Media	The sand, soil or other organic material in a filtration device used to provide a permeable surface for pollutant and sediment removal.
Floatables	Materials found in runoff that are buoyant such as polystyrene, plastic, cigarette butts and other types of organic materials.
Flood	Relatively high streamflow which overtops the natural or artificial banks in any part of a stream or river.
Flood Routing	Determining the rise and fall of floodwater as it progresses downstream.
Flood Standard (or Designated Flood)	The flood selected for planning purposes. The choice should be based on an understanding of flood behaviour and the associated flood risk. It should also take into account social, economic and ecological considerations.
Flood Storages	Parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
Floodplain	The low land adjacent to a waterbody, which is subjected to flooding.
Floodways	Areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas, which even if only partially blocked would cause a significant redistribution of flood flow that may in turn adversely affect other areas. They are often, but not necessarily the areas of deeper flow or the areas where higher velocities occur.
Flow Splitter	An engineered, hydraulic structure designed to divert a percentage of storm flow to a BMP located out of the primary channel, or to direct stormwater to a parallel pipe system or to bypass a portion of baseflow around a BMP.
Forebay	An extra storage area provided near an inlet of a pond BMP to trap incoming sediment before it accumulates in a pond BMP.
Foreshore	The land between a water body and the dominant ridge line facing the water body.

Term	**Definition**
Freeboard	The space from the top of an embankment to the highest water elevation expected for the largest design storm stored. This space is required as a safety margin in a pond or basin.
Gates	Gates are control device at drainage outlet to avoid backflow during high tides or high flood levels at the receiving water bodies. They are normally opened during low water levels and closed during higher water levels at the receiving water bodies.
Grade	The slope of a land surface, road or channel bottom.
Grass Channel	An open vegetated channel used to convey runoff and to provide treatment by filtering pollutants and sediments.
Grate Inlets	Parallel and/or transverse bars arranged to form an inlet structure.
Gross Pollutant Trap	A device used to intercept gross pollutants being transported in stormwater.
Gross Pollutants	Stormwater laden debris typically larger than 3 mm (includes litter and organic matter).
Groundwater Mound	A round, mound-shaped surface in a water table or other potentiometric surface that builds up as a result of the downward percolation of water.
Groundwater Table	The free surface of the underground water that is frequently subjected to conditions such as fluctuating atmospheric pressure with the season, withdrawal rates and restoration rates. Therefore, the groundwater table is seldom static.
Gully	A channel caused by the concentrated flow of surface and stormwater runoff over unprotected erodible land.
Habitat	The kind of locality in which animal breeds or plants normally grow. It is also the geographic distribution or native home of plant or animals.
Hydraulic Conductivity	For isotropic porous medium and homogenous fluids. The term refers to the volume of water at the existing kinematic viscosity that will move in unit time under a unit hydraulic gradient through a unit area measured at right angle to the direction of flow. Replaces the term coefficient of permeability.
Hydraulic Grade Line (HGL)	A line coinciding with the level of flowing water in an open channel. In a closed conduit flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. It is equal to the energy gradeline elevation minus the velocity head, $V^2/2g$.
Hydraulic Gradient	Slope of the water or potentiometric surface. The change in static head per unit of distance in a given direction. If not specified, the direction generally is understood to be the maximum rate of decrease in head.
Hydraulic Head	The height above a datum plane (such as sea level) of the column of water that can be supported by the hydraulic pressure at a given point in a groundwater system. For a well, the hydraulic head is equal to the distance between the water level in the well and the datum plane.
Hydraulic Jump	A flow discontinuity, which occurs at an abrupt transition from subcritical to supercritical flow.

Term	**Definition**
Hydraulic Radius	This is the ratio of cross sectional area of the flow to the wetted perimeter. For a circular pipe flowing full, the hydraulic radius is one-fourth of the diameter. For a wide rectangular channel, the hydraulic radius is approximately equal to the flow depth.
Hydraulics	The study of water flow; in particular the evaluation of flow parameters such as stage and velocity in a river or stream.
Hydrogeology	The science that deals with subsurface waters and related geologic aspects of surface waters. Also used in the more restricted sense of groundwater geology.
Hydrograph	A graph showing stage, flow, velocity, or other characteristics of water with respect to time. A stream hydrograph commonly shows rate of flow; a groundwater hydrograph shows the water level or head.
Hydrologic Abstractions	Losses of rainfall that do not contribute to direct runoff. These losses include water retained in surface depressions, water intercepted by vegetation, evaporation and infiltration.
Hydrologic Budget (Balance)	An account of the inflow to, outflow from and storage in a hydrologic unit such as a drainage basin, aquifer, soil zone, lake, or reservoir. It is also expressed as the relationship between evaporation, precipitation, runoff and the change in water storage by the hydrologic equation.
Hydrologic Equation	The equation that balances the hydrologic budget.
Hydrology	The study of the rainfall and runoff process and relates to the derivation of hydrographs for given floods, draughts and other water resources aspects.
Hydroperiod	A seasonal occurrence of flooding and/or soil saturation. It encompasses depth, frequency, duration and seasonal pattern of inundation.
Hydroplanning	Separation of the vehicle tire from the roadway surface due to a film of water on the roadway surface.
Illicit Discharge	All non-urban runoff discharges to urban runoff drainage systems that could cause or contribute to a violation of water quality, sediment quality, or groundwater quality standards. This discharge includes sanitary sewer connection, industrial process water, car washing, etc.
Impermeable	A condition where a material is incapable of transmitting significant quantities of water under pressure differences.
Impervious Surface	A hard surface area, which either prevents or retards the entry of water into the soil mantle under natural conditions, and/or causes water to run off the surface in greater quantities or at an increased rate of flow compared to the present flow under natural conditions; prior to development. Common impervious areas include (but are not limited to) rooftops, walkways, patios, driveways, parking lots, storage areas, concrete or asphalt paving, gravel roads, packed earthen materials and oiled macadam.
Imperviousness	The percentage of impervious cover within a defined area.
Impoundment	The body of water retained by a berm, dam or dike.
Indigenous	Native to; belonging naturally to a particular area, country etc. (endemic).

Term	**Definition**
Infiltration	The downward movement of water from the soil surface at ground level into the underlying subsoil. Water infiltrates into the soil profile and percolates through it. The infiltration capacity is expressed in terms of mm/hr. Infiltration depends heavily on the vegetative cover of the soil surface, while permeability depends on the soil texture and compaction.
Infiltration Basin	A facility that provides stormwater quantity control by containing excess runoff in a detention facility, then percolating that runoff into the surrounding soil.
Infiltration Capacity	The maximum or limiting infiltration rate.
Infiltration Rate	The rate at which a soil under specified conditions absorbs falling rain, melting snow, or surface water expressed in depth of water per unit time (centimetres per second, inches per hour).
Inlet	A form of connection between the surface of a ground and a drain or sewer for the admission of surface and stormwater runoff.
Invasive Exotic Plants	Non-native plants having the capacity to compete and proliferate in introduced environments.
Invert	The lowest point on the inside of a culvert or pipe.
Leaching	The loss of nutrients from the existing ground (i.e. when rain dissolve the nutrients and are carried away). There are possibilities that chemical fertilisers leached from the ground are able to pollute streams and other water sources.
Level Spreader	A device used to spread out stormwater runoff uniformly over the ground surface as sheet flows (not through channels). The purpose of level spreaders is to prevent concentrated and erosive flows from occurring, thus enhancing infiltration.
Lowflow Channel	An incised or paved channel from the inlet to the outlet in a dry basin, which is designed to carry low runoff flows and/or baseflow directly to the outlet without detention.
Macrophyte	A member of the macroscopic plant life, especially of a waterbody.
Major Storm	A precipitation event that is higher than the typical largest rainfall for a year.
Major System	A system that provides overland relief for stormwater flows exceeding the capacity of the minor system and is composed of pathways that are provided, knowingly or unknowingly, for the runoff to flow to natural or manmade receiving channels such as streams, creeks or rivers.
Management Plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
Mathematical (Computer) Models	The mathematical representation of physical processes (e.g. rainfall and runoff, or mobilisation and transport of pollutants by runoff). These models usually run on computers due to the complexity of the mathematical relationships.
Mean Depth	The average depth described as the cross-sectional area of the inundated channel divided by its surface width.

Term	**Definition**
Minor System	A system, which consists of the components of the storm drainage system that is normally designed to carry runoff from the more frequent storm events. These components include curbs, gutters, ditches, inlets, manholes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc.
Mulch	Organic material spread on soil to aid moisture retention and prevent weed growth. It also provides nutrients and helps to open soil texture.
Naturalise	The establishment of plants in a manner as though they are of wild species.
New Development	Includes the following activities: land disturbing activities, structural development including construction, installation or expansion of a building or other structure, and creation of impervious surfaces.
Nonpoint Source (NPS) Pollution	Pollution caused by sediment, nutrients, organic and toxic substances originating from land-use activities and/or from the atmosphere, which are carried to surface waterbodies by runoff. (NPS) pollution occurs when the rate at which these materials entering waterbodies exceeds natural levels.
Nutrient	A substance necessary for the growth of organisms.
Observation Well	A test well installed in an infiltration BMP to monitor draining times and sediment accumulation after installation.
Off-line	A stormwater management system designed to manage a storm event by diverting a percentage of stormwater events from a stream or storm drainage system.
Off-line BMP	A water quality facility designed to treat a portion of stormwater, which has been diverted from a stream or storm drain.
Off-line Treatment	A BMP system that is located outside of the stream channel or drainage path. A flow splitter is used to divert runoff from the channel and into the BMP for treatment.
Off-site	Any area lying upstream of the site that drains onto the site and any area lying downstream of the site.
Oil/Grit Separator	A best management practice designed to remove heavy particulate and hydrocarbons.
On-line	A stormwater management system designed to manage stormwater in its original stream or drainage channel.
Orifice	An opening with closed perimeter usually sharp-edged and of regular form in a plate, wall, or partition through which water may flow. Generally used for the purpose of measurement or flow control.
Orifice Flow	Flow of water through a submerged opening and controlled by pressure forces.
Outfall	The point or structure of a conduit discharging to a waterbody.
Outlet	Point of water disposal of a stream, river, lake, tidewater or artificial drain.
Overflow Rate	Detention basin release rate divided by the surface area of the basin. It can be thought of as an average flow rate through the basin.
Overtopping	To flow over the limits of a containment or conveyance element.

Term	**Definition**
Oxidation	The combination of oxygen with a substance, or the removal of hydrogen from it, or more generally, any reaction in which an atom loses electrons.
Oxygenation	The process of adding dissolved oxygen to a solution.
Parameter	A measurable (or quantifiable) characteristic (or feature).
Peak Discharge	The maximum flow for a given hydrologic event at a specified location.
Percolation	The movement under hydrostatic pressure of water through the interstices of rock or soil, except for the movement through large openings such as caves.
Percolation Rate	The rate expressed as either velocity or volume per unit of time at which water percolates through a porous medium.
Perennial	Plant that grow for more than two years.
Permeability	The capacity of a geologic material in transmitting a fluid. The degree of permeability depends upon the size and shape of the openings and the extent of the interconnections of the material.
Pervious	Allowing for the passage of water.
Pesticide	A substance or mixture of substances used to eliminate unwanted species of plants or animals.
pH	Value taken to represent the acidity or alkalinity of an aqueous solution. It is defined as the negative logarithm of the hydrogen ion acidity of the solution.
Point Source	A distinct, identifiable source of pollutants.
Porous Pavement	An alternative to conventional pavement whereby runoff is diverted through a porous asphalt layer or manufactured pavement grid into an underground stone reservoir. Thus, the stored runoff gradually infiltrates into the subsoil.
Post-development Peak Runoff	Maximum instantaneous rate of flow during a storm after a development is completed.
Pretreatment	The removal of material such as gross solids, grit, grease and scum from flows prior to physical, biological or chemical treatment processes to improve treatability. Pretreatment may include screening, grit removal, stormwater and oil separators.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation, see Annual Exceedance Probability.
Pump	Pump is a motor that drive the propeller, impeller or screw to lift flood water to a higher elevation. Pump selection and numbers depend on station layout, required pump rate, wet well depth, and maintenance considerations. The size of each motor depends on the pump size, flow rate, pressure head, and duty cycle.
Pump Station Controls	Control circuitry includes the flood level at which the pump station will be activated, sequence of operation, activation of the standby generator when necessary, deactivation when the flood event has passed, and operation of any night security lighting. Controls may also include automatic communication with a central office on the station's status regarding water levels, pump readiness, utility electrical power, standby generator fuel level, security, or other central office concerns.

Term	Definition
Pump Station Discharge Channels	Conveyance that direct the pump flows into the receiving water bodies and shall be designed to avoid any backflow into the pump sump.
Pump Station Power Sources	The power source is usually provided by the local utility that normally require electrical substation. Every pump station shall have an on-site standby electrical generator because the type of storm that makes a pump station necessary is also the type of storm that interrupts utility power.
Pump Station Side-spill Weir	Side-spill weir is a device to separate the stormwater channel and the pump storage or sump to prevent stormwater from entering the pump storage or sump during normal period where the flow can be discharge through gravity.
Pump Sump and Storage	The sump and storage are small pond to receive the inflow of storm water prior to pumping. The storage can attenuate the storm hydrograph peak to reduce the required pumping rate. In most circumstances, the stormwater that brings along debris can clog and damage the pumps and debris removal system shall be provided before the flows enter the sump and storage. Convenient access shall be provided for the removal of accumulated debris and silt.
Rainfall Intensity	The rate at which precipitation occurs at a given instant.
Rainhead	A collector of rainwater at the end of a box gutter generally of a rectangular shape and external to a building, which is connected to an external downpipe.
Reach	The smallest portion of a drainage system consisting of a uniform shape, cross-section and slope.
Receiving Waters	Bodies of water or surface water systems receiving water from man-made (or natural) upstream streams.
Recharge	Replenishment of groundwater by downward infiltration of water from rainfall, streams and other sources. Natural recharge occurs without assistance or enhancement by man. Artificial recharge occurs when the natural recharge pattern is modified deliberately to increase recharge.
Recharge Basin	A basin constructed on the ground surface to receive discharge from streams, storm drains or other sources for the purpose of replenishing groundwater supply.
Redevelopment	The construction, alteration or improvement exceeding 500 square meters of existing land currently used as commercial, industrial, institutional or multifamily residential.
Release Rate	The rate of discharge in volume per unit time from a detention facility.
Restoration	The reestablishment of wetland functional characteristics and processes to previously defined wetlands that have been lost through alterations, activities or catastrophic events.
Retention	The holding of runoff in a basin without release except through means of evaporation, infiltration or emergency bypass.

Term	**Definition**
Retention/Detention Facility	A drainage facility designed to hold water for a considerable length of time then releasing it by evaporation, plant transpiration and/or infiltration into the ground. It is also designed to hold surface and stormwater runoff for a short period of time and then releasing it to the surface and stormwater management system.
Retrofitting	The renovation of an existing structure or facility to meet changed conditions or to improve performance.
Right-of-way	Right of passage, as over another's property. A route that is lawful to use. A strip of land acquired for transport or utility construction.
Riparian	Pertaining to the banks of streams, wetlands, lakes or tidewater. A relatively narrow strip of land that borders a stream or river, which often coincides with the maximum water surface elevation of the 100 year storm.
Riprap	A facing layer or protective mound of stones placed to prevent erosion or sloughing of a structure or embankment due to the flow of surface and stormwater runoff.
Riser	The vertical portion of an inlet to a conduit, extending from the barrel to the water surface.
Risk	A statistical concept defined as the expected frequency or probability of undesirable effects resulting from a specified exposure to known or potential environmental concentrations of a material. A material is considered safe if the risks associated with its exposure are judged to be acceptable. Estimates of risk may be expressed in absolute or relative terms. Absolute risk is the excess risk due to exposure. Relative risk is the ratio of the risk in an exposed population to the risk in an unexposed population.
Runoff	A portion of rainfall which ends up as streamflow; also known as rainfall excess.
Safety Bench	A flat area above the permanent pool and surrounding a stormwater pond designed to provide a separation to adjacent slopes.
Saturated Zone	Part of a water-bearing material in which all voids, both large and small, are ideally filled with water under pressure greater than atmospheric.
Sediment	Mineral and organic soil material that is transported in suspension by wind or water flow from its origin to another location.
Separate Sewer Overflow (SSO)	An event where wastewater entering sanitary sewers may be so great the collection system or sewage treatment plant cannot handle the increased flow and may be due to blockage, a lack of capacity, inflow and infiltration or other reasons. As a result, untreated sewage empties directly into receiving waters, often from manholes or up through sewer connections.
Settleable Solids	Suspended solids in stormwater that separate by settling when the stormwater is held in a quiescent condition for a specified time.
Sheet Flow	Runoff, which flows over the ground surface as a thin, even and unconcentrated layer in a channel.
Short Circuiting	The passage of runoff through a BMP in less than the theoretical or design treatment time.

Term	**Definition**
Shrub	Plant with many woody stems, the main ones rising from near the base.
Slope	A ratio of run (horizontal) to rise (vertical).
Slotted Inlets	A section of pipe cut along the longitudinal axis with transverse bars spaced drainage form slots.
Soil Groups	The great soil group system is one system that can be used to classify soils. The grouping depends on the presence and type of morphological features observed in the field, selection of these features and the weighting they receive based on the concepts of soil genesis.
Soil Moisture	Water or moisture contained in the soil mantle.
Soil Porosity	The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.
Soil Stabilisation	The use of measures such as rock lining, vegetation or other engineering structures to prevent the movement of soil when loads are applied.
Source Control BMP	BMP that is intended to prevent pollutants from entering the stormwater. A few examples of source control BMPs are erosion control practices, maintenance of stormwater facilities, constructing roofs over storage and working areas, and directing wash water and similar discharges to the sanitary sewer or a dead end sump.
Species	The basic unit of biological classification; a group of individual plants resembling each other by a combination of constant characteristics with inter-breeding possible within the species but generally not between species.
Spillway	A passage (such as a paved apron or channel) for surplus water over or around a dam or similar obstruction. An open or closed channel, or both, used to convey excess water from a reservoir. It may contain gates, either manually or automatically controlled to regulate the discharge of excess water.
Spreading Water	Discharging native or imported water to a permeable area for the purpose of encouraging it to percolate to the saturated zone. Spreading, artificial recharge and replenishment all refer to operations used to place water in a groundwater basin.
Steady Flow	Flow that remains constant with respect to time.
Storm Drain	A particular storm drainage system component that receives runoff from inlets and conveys the runoff to some point. They are either closed conduits or open channels connecting to two or more inlets.
Stormwater	Water resulting from runoff from a storm event. During a rainfall event some water remains on the surface or is held in the soil or underground aquifer as groundwater, a portion of the water is used directly by plants and the remainder flows over the surface. This overland flow is called stormwater. It usually moves as overland (sheet) flow or channel (concentrated) flow.
Stormwater Management	The process of controlling the quality and quantity of stormwater to protect the downstream environment.
Stormwater Ponds	A land depression or impoundment created for the detention or retention of stormwater runoff.

Term	**Definition**
Stormwater Site Plan	A plan, which shows the measures taken during and after project construction to provide erosion and sediment control, and stormwater control.
Stormwater Treatment Train	A series of BMPs or natural features, each designated to treat a different constituent, component, or aspect of runoff, implemented together to maximise pollutant removal effectiveness.
Stormwater Utility	An utility established to generate a dedicated source of funding for stormwater pollution prevention activities where users pay a fee based on land-use and contribution of runoff to the stormwater system.
Stormwater Wetlands	Shallow, constructed pools that capture stormwater and allow for the growth of characteristic wetland vegetation.
Structural BMPs	Devices which are constructed to provide temporary storage and treatment of stormwater runoff.
Subcritical Flow	Flow characterised by high velocities, large depths, mild slopes and a Froude number less than 1.0.
Sump	A collector of rainwater, generally of rectangular shape, in the sole of a box gutter and connected to a downpipe within the building perimeter. Its function is to increase the head of water at the entry to the downpipe and thus increasing its capacity.
Surface Drainage System (Property Drainage)	A system for the collection and conveyance of stormwater, the elements which includes kerbs and gutters, site stormwater drains or channels and appurtenances and pumped systems.
Suspended constituent	The constituents in a water sample (the residue) that are retained on a filter medium. The type of filter must be specified.
Suspension	A system in which very small particles (e.g. solid, semi-solid or liquid) are more or less uniformly dispersed in a liquid or gaseous medium. If the particles are small enough to pass through filter membranes, the system is termed a colloidal suspension. If the particles are larger than colloidal dimensions they will tend to precipitate if heavier than the suspending medium, or if lighter, to agglomerate and rise to the surface.
Swale	A natural or human-made open depression or wide, shallow ditch that intermittently contains or conveys runoff. Can be used as a BMP to detain and filter runoff.
Threshold Concentration	A concentration, where if above will produce some effect (or response) and vice versa.
Time of Concentration	The time required for water to travel from the hydraulically most distant point to the outlet of a drainage area.
Toxicity	The inherent potential or capacity of a material to cause adverse effects in a living organism.
Toxicity test	The means by which the toxicity of a chemical or other test material is determined. It is used to measure the degree of response produced by exposure to a specific level of stimulus (or concentration of chemicals).

Term	**Definition**
Trash Rack	A protective structural device installed to protect outlet structures from inflowing debris.
Travel Time	The time interval required for water to travel from one point to another through a part (reach) of a watershed.
Treatment Control BMPs	These are methods of treatment to remove pollutants from the stormwater. Treatment control BMPs are also known as "structural controls". These controls do require maintenance.
Turbidity	Cloudiness of water due to suspended solids.
Turbulence	Unorganised movement in liquids and gases resulting from the eddy formation.
Uniform Flow	A state of steady flow where the mean velocity and cross-sectional area remain constant.
Unit Hydrograph	The direct runoff hydrograph produced by a storm of given duration such that the volume of excess rainfall and direct runoff is 1 cm.
Unsaturated Zone	The zone between the land surface and the water table. It includes the capillary fringe and may contain water under pressure less than that of the atmosphere.
Unsteady Flow	Flow that changes with respect to time.
Uptake	A process by which materials are absorbed and incorporated into a living organism.
Vadose Zone	See Unsaturated Zone.
Valley Gutter	Inclined channels placed at the intersecting sloping surfaces of the adjacent roof for the conveyance of rainwater.
Varied Flow	Flow in an open channel where the flow rate and depth change along the length of the channel.
Vegetated Filter Strip (VFS)	A facility that is designed to provide stormwater quality treatment of conventional pollutants but not nutrients through the process of biofiltration.
Water Quality BMP	A BMP specifically designed for pollutant removal.
Water Quality Criteria	Scientific data evaluated to derive the recommended limits for water uses.
Water Quality Inlets	Pre-cast storm drain inlets (oil and grit separators) that remove sediment, oil and grease, and large particulates from paved area runoff before it reaches storm drainage systems or infiltration BMPs.
Water Table	The upper surface of a saturated zone except where that surface is formed by an impermeable body; or locus of points in soil water at which the pressure is equal to atmospheric pressure; or the surface where groundwater is encountered in a well in an unconfined aquifer. The water table is a particular potentiometric surface.
Weed	Generally a plant which rapidly reproduces itself in large numbers, and if not checked, supersedes or destroy cultivated crops or interferes with their cultivation.

Term	**Definition**
Weir Flow	Flow over a horizontal obstruction controlled by gravity.
Wet Detention Ponds	A BMP consisting of a permanent pool of water designed to treat runoff by detaining water long enough for settling, filtering, and biological uptake. Wet ponds are also often designed to have an aesthetic or recreational value.
Wet Pond	A facility that treats stormwater for water quality by utilising a permanent pool of water to remove conventional pollutants from runoff through sedimentation, biological uptake and plant filtration.
Wetlands	Areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, at under normal circumstances, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes and bogs. This includes wetlands, which are created, restored or enhanced as part of a mitigation procedure. This does not include constructed wetlands or surface waters intentionally constructed from sites that are not wet-lands such as irrigation and drainage ditches, grass-lined swales, canals, agricultural detention facilities, farm ponds and landscape amenities.
Wet-pit Stations	Pump stations designed such that the pumps are submerged in a wet well or sump with the motors and the controls located overhead.