حكـــومـــة أبـــوظـــبي
GOVERNMENT OF ABU DHABI

STORMWATER & SUBSOIL DRAINAGE SYSTEMS

VOLUME I: DESIGN MANUAL

WA-726-1

Standards Guideline
For

STORMWATER & SUBSOIL DRAINAGE SYSTEMS

VOLUME 1

DESIGN MANUAL

Document No: WA-726-1
Third Edition
April- 2022
Department of Municipalities and Transport
PO Box 20
Abu Dhabi, United Arab Emirates

1. Amendment Page

To ensure that each copy of this technical document (Abu Dhabi Guideline) contains a complete record of amendments, the Amendment Page is updated and issued with each set of revised/new pages of the document. This ADG/ISGL is a live document which can be amended when necessary. QCC operates VOLUME I, DESIGN MANUAL Group which prepared this document and can review stakeholder comments in order to review and amend this document and issue an updated version when necessary.

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
01	00	DEC 2016			
02	01	JULY 2020	WA-726-1: 2.3 (PAGE-5)	Modify as below: Structure (With Reinforcement) - 50 years. Structure (Without Reinforcement) -60 years	
02	01	JULY 2020	WA-726-1: 3.4.1 (PAGE-20)	Add the following statement: “Recommended type of curve to be used for hydraulic modelling should be SCS Type II Cumulative type curve, unless otherwise specified by DEPARTMENT”.	
02	01	JULY 2020	WA-726-1: 3.4.2 (PAGE-23)	Modify as below: The hydraulic model CIVILSTORM (for ADM), SewerGEMS, INFOWORKS (for AACM) **or** shape file exports are to be submitted to DEPARTMENT for input by DEPARTMENT into their hydraulic model to verify the results and extend the overall Abu Dhabi hydraulic models. Section 8 details DEPARTMENT’s requirements for the submission of model files.	
02	01	JULY 2020	WA-726-1: 4.3.4 (PAGE -31)	To be added as follows: All Materials bearing the AD QCC Trust Mark are preferable and recommended.	
02	01	JULY 2020	WA-726-1: 4.3.1 (PAGE-29)	Modify as below: HDPE Pipes (plain and corrugated)	
02	01	JULY 2020	WA-726-1: 4.3.4 (PAGE-31)	Modify as below: Structured Wall / Corrugated HDPE Pipes (SWPP)	
02	01	JULY 2020	WA-726-1: 4.3.4 (PAGE-31)	Modify as below: As all these materials have acceptable properties, subject to correct design and allowable deflection and ovalization as per specifications, the choice will be determined by bedding and construction cost and size availability. Other pipe materials may be considered, subject to approval of DEPARTMENT	
02	01	JULY 2020	WA-726-1: 4.3.4 (PAGE-31)	Add this part in the comments column: Any stormwater pipe under the road	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
				shall comply with DIN 8061/8062 or ISO 4422 (2.5)	
02	01	JULY 2020	WA-726-1: 4.4.2 (PAGE-34)	Replace with the following: • intervals not exceeding 100m for pipes ≤ 300mm diameter (perforated lines) • intervals not exceeding 120m for pipes 300mm to 600mm diameter.	
02	01	JULY 2020	WA-726-1: 4.12 (PAGE-39)	Modify as below: The use of soakaways and soakaway trenches are not permitted in the design of new storm water systems within DEPARTMENT jurisdiction. However, the use of Geo-cells can be considered only for selected areas after approval from DEPARTMENT. Soakaways are used within Al Ain City Municipality region, please refer to Appendix-A4 for further details.	
02	01	JULY 2020	WA-726-1: 4.14 (PAGE-39)	4.13 Open Channels & Ditches please refer to Appendix-A4 for further details.	
02	01	JULY 2020	WA-726-1: 5.2 (PAGE-41)	"conditions includes"	
02	01	JULY 2020	WA-726-1:TABLE 5.2 (PAGE-43)	Added Note: * - for information only; to be verified by the ADSSC approved consultant prior to any construction activities	
02	01	JULY 2020	WA-726-1: 5.4.2 (PAGE-48)	Replace with the following: Minimum size for perforated pipes is 160mm OD.	
02	01	JULY 2020	WA-726-1: 8.3 (3.C) (PAGE-89)	Modify as below: Hydraulic modelling and Shape files as per including input and output data sheets and drawings presenting water surface and HGL profile. These shall be submitted with a format that is stated in Section - 3.4.2.	
02	01	JULY 2020	WA-726-1: 8.4.1 (PAGE-89)	Submissions can only be accepted through qualified stormwater drainage Consultant. No design submittals by Contractors or non-qualified stormwater drainage Consultant can be acceptable.	
02	01	JULY 2020	WA-726-1: 8.4.2.E 8.4.2.P 8.4.3 K (PAGE-90&91 &91)	AutoCAD and Excel Justification Justification	
02	01	JULY 2020	WA-726-1: 8.7 (#4) (PAGE-94)	Modify as below: One soft copy of the Hydraulic modelling Shape files that is compatible with a format that is stated in Section - 3.4.2.	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
02	01	JULY 2020	WA-726-1: REFERENCE (PAGE-101)	28. Abu Dhabi Sewerage Services Company (ADSSC) Design Guidelines, Specification & Drawings	
03	02	April 2022	WA-726-1: 2.7 Page (8)	Added as follows: For further details, refer to Value Engineering Manual.	
03	02	April 2022	WA-726-1: 3.1 Page (10)	Added as follows: Prepare 2D model and submit if requested by the DEPARTMENT to realize actual floods	
03	02	April 2022	WA-726-1: 3.2.1 Page (11)	Added this part in the Area column: Airports, Seaports (check for 1 in 50 years storm)	
03	02	April 2022	WA-726-1: 3.2.2 Page (11)	Added this part in the Catchment type column: Airports; Seaports	
03	02	April 2022	WA-726-1: 3.2.2 Page (11)	Added as follows: For SCS method, multiplication factors shall be applied only for the initial time of concentration.	
03	02	April 2022	WA-726-1: 3.2.4 Page (14)	Entire section modified.	
03	02	April 2022	WA-726-1: 3.4.2 Page (27)	Added as follows: Consultant shall integrate the project model within existing model if requested by DEPARTMENT (Existing hydraulic model shall be provided by DEPARTMENT upon submission of official request).	
03	02	April 2022	WA-726-1: 3.6 Page (28)	Added as follows: rational method	
03	02	April 2022	WA-726-1: 4.1.4 Page (32)	To be added as follows: In sites where the clear width (B) needs to be less than the value given in Figure 4-1, a geotechnical investigation must be submitted and approved by the concerned authority.	
03	02	April 2022	WA-726-1: 4.2 Page (32)	To be added as follows: A geo-cellular storm water management system	
03	02	April 2022	WA-726-1: 4.2.1 Page (33)	New item added	
03	02	April 2022	WA-726-1: 4.3.1 Page (35)	To be added as follows: Surcharge conditions of the flow are encouraged for all pipe systems, where applicable, if a clearance of 0.3 m (for the 1 in 5 – year storm event) is provided from manhole covers.	
03	02	April 2022	WA-726-1: 4.4 Page (39)	To be added as follows: On special cases (such as areas subject to minimal or no sand storms), and depending on the project limitations, inlets could be arranged to discharge directly to the storm water manhole structure without catch basin considering Abu Dhabi Standard dimensions for inlets and Catch basins.	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
03	02	April 2022	WA-726-1: 4.5 Page (41)	Added as follows: • The oil interceptor compartment shall be incorporated by a Hydrocarbon layer thickness sensor to detect when the maximum thickness is reached. • The sludge trap compartment shall be incorporated with a Sludge layer sensor to alarm when maintenance is necessary. The sensor should be placed such that the end of the probe is 2 cm under the alarming level	
03	02	April 2022	WA-726-1: 4.8.1 Page (43)	Modified this part in (Level to New Abu Dhabi Datum (m) (NADD)) as below: 0.8 to 1.3.	
03	02	April 2022	WA-726-1: 4.8.2 Page (43)	Modify as below: Number modified from 0.8 to 1.3.	
03	02	April 2022	WA-726-1: 4.8.2 Page (43)	Added as follows: Outfall structure may also be provided with a manual or automatic trash screen for removal of debris. Trash bar screen shall be housed in a separate chamber.	
03	02	April 2022	WA-726-1: 4.12 Page (45)	Added as follows: Generally, the use of soakaways and soakaway trenches are not permitted in the design of new storm water systems. However, where the groundwater table is deep, the soil's permeability is high, and the maintenance requirements are practical, infiltration systems can be considered as an alternative based on a study and on a case-by-case basis CIRIA	
03	02	April 2022	WA-726-1: 4.12.1 & 4.12.2 Page (46)	Sections were moved from appendix A3	
03	02	April 2022	WA-726-1: 4.12 Page (45)	Added as follows: In addition, the use of Geo-cellular units can be considered, the total areas of the base and sides of the blocks shall be designed to enable the rate of infiltration of the stored water through the geotextile soil interface to be developed.	
03	02	April 2022	WA-726-1: 4.12.1 Page (46)	Point 3 modified as follows: Determine the catchment area and calculate the volume of runoff for the design return period storm using either (i) the NRCS depth of runoff equation, (ii) the modified rational equation, or the Depth-Duration-Frequency (DDF) where available (refer to Equation A-4.1 and Equation 4.3, for additional information.	
03	02	April 2022	WA-726-1: 4.12 Page (48)	Point 9 added	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
03	02	April 2022	WA-726-1: 4.14 Page (49)	Entire section added	
03	02	April 2022	WA-726-1: 5.3.2 Page (56)	Added as follows: A proper network shall be provided to control the ground water rise if required based on the study with all supporting calculations	
03	02	April 2022	WA-726-1: 5.3.2 Page (57)	Modify as below: From water table at the midway point must also be at least 15 cm to water table at the midway point must also be at least 50 cm	
03	02	April 2022	WA-726-1: 5.3.2 Page (57)	Added as follows: root zone	
03	02	April 2022	WA-726-1: 5.3.2 Page (57)	Modify as below: From street medians the water table must be at least 1.0 m to street medians the water table must be at least 2.0 m	
03	02	April 2022	WA-726-1: 5.3.2 Page (57)	Added as follows: Minimum depth for the ground water control system shall be 1.20 m from the finished road level.	
03	02	April 2022	WA-726-1: 5.5 Page (65)	Added as follows: Please note that minimum diameter for field drain shall be 150 mm (nominal diameter)	
03	02	April 2022	WA-726-1: 6.2.2 Page (73)	Added as follows: The minimum required sump volume can be calculated, and it depends on the inflow to the pump station, the pump capacities, their allowed cycle time and their operating sequence. It is Important to know the required active volume. It is the volume defined by the highest start level and lowest stop level in the pump sump.	
03	02	April 2022	WA-726-1: 6.2.2 Page (73)	Added as follows: In general, the wet well should have a storage of a minimum cycle time of peak flow (6:10 starts per hour or as per the minimum allowable cycle time ‘t’ designated by the pump manufacturer)	
03	02	April 2022	WA-726-1: 6.2.2 Page (74)	Added as follows: The wet well volume when multiple pumps are installed is calculated as for a single pump where the minimum volume is the capacity between the start and stop level for each pump. However, additional capacity is required to allow a vertical distance of 150mm between the start or stop levels of consecutive pumps For more details, refer to guidance provided in ANSI-HI_9.8.	
03	02	April 2022	WA-726-1: 6.3.6 Page (77)	Entire section added	
03	02	April 2022	WA-726-1: 7.5.8 Page (101)	Entire section modified	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
03	02	April 2022	WA-726-1: 8.4.1 Page (107)	Added as follows:2. Master Plans and/or Concept Designs approval shall be obtained and reviewed by DEPARTMENT.	
03	02	April 2022	WA-726-1: 8.4.1 Page (107)	Added as follows: For the discharge to Marine Environment. A specific approval related to outfall discharging to marine water (Sea/Canal) is required from EAD clearing mentioning outfall location during the design stage (Not to be confused with Environmental Permit Application).	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: iii.(Environmental Permit for overall project and Specific approval for outfall to discharge to marine environment)	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: iv. ADM Design Section and	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: f. infiltration rates etc.).	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: h. for all project components including pumping stations.	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: d. plans for gravity and pressure pipelines	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: d. outfalls	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: g. invert and cover	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: g. runoff	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: i. using the mentioned software	
03	02	April 2022	WA-726-1: 8.4.2 Page (108)	Added as follows: j. Hydrology report shall be prepared and submitted for the for urban and rural highways/expressways. A watershed model shall be prepared and submitted with all detailed calculations. A layout shall be submitted showing the delineated catchment areas, location of stream, location of culverts, preliminary sizing of culverts. Culverts shall be analyzed using international software such as HY-8 or compatible including the culver report.	
03	02	April 2022	WA-726-1: 8.4.2 Page (109)	Added as follows: k. These calculations shall be based on actual soil parameters obtained through geotechnical investigations.	
03	02	April 2022	WA-726-1: 8.4.3 Page (109)	Added as follows: v. showing all other utilities crossing the storm water	
03	02	April 2022	WA-726-1: 8.4.3 Page (109)	Added as follows: vii. Structural design of manholes (where applicable),	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
				outfalls, culverts with final detailed calculations,	
03	02	April 2022	WA-726-1: 8.4.3 Page (110)	Added as follows: ix. Surge analysis report and calculations	
03	02	April 2022	WA-726-1: 8.7 Page (112)	Added this part in the Description column :2. Preliminary Design / 4. ADM software and / 18. Hydrology Report, watershed model, calculations, watershed catchment area, location plan showing culverts and other hydraulic structures / 21. marked on the layout plans.	
03	02	April 2022	WA-726-1: 9 Page (120)	13.CIRIA Report C737- structural and geotechnical design of modular geocellular drainage systems	
03	02	April 2022	WA-726-1: A3.4.2 Page (146)	Added as follows: 22	
03	02	April 2022	WA-726-1: A3.5.1 Page (156)	Added new inlets type: Figure A3-5: Perspective Views of Gutter and Curb-opening Inlets	
03	02	April 2022	WA-726-1: A3.5.1 Page (157)	Added new inlets type: Figure A3-6 – Perspective Views of Combination and Slotted Drain Inlets*	
03	02	April 2022	WA-726-1: A3.5.2 Page (159)	Added as follows: Grates come in many different shapes these shapes include but are not limited rectangular, rhomboidal, and honey comb (refer to Figures A 3-5 and A 3-6). Detailed hydraulic design should be carried out by an experienced engineer and approved by the ADM [added text for stormwater and subsoil drainage systems design manual].	
03	02	April 2022	WA-726-1: A3.5.2 Page (160)	Added as follows: W net = net width of gutter inlet or grate (see Figure A3-5), in m = W – W bars [modified for stormwater and subsoil drainage systems design manual]	
03	02	April 2022	WA-726-1: A3.5.2 Page (160)	Added as follows: L net = net length of grate, in m = L – L bars Where, L is the overall length of the grate and L bars is the total length of the bars. [modified for stormwater and subsoil drainage systems design manual]	

EDITION NO.	REVISION NO.	DATE	PAGE NO.	AMENDMENTS	NOTES
03	02	April 2022	WA-726-1: A4.2 Page (167)	Added as follows: On special cases (such as areas subject to minimal or no sand storms), and depending on the project limitations, inlets could be arranged to discharge directly to the storm water manhole structure without catch basin considering Abu Dhabi Standard dimensions for inlets and Catch basins.	
03	02	April 2022	WA-726-1: A5.1.1.5 Page (176)	Entire section added	
03	02	April 2022	WA-726-1: A5.2.1.3 Page (182)	Added as follows: f) a. / geo-cell panels/ concrete geo-synthetic cementicious composite sheets	
03	02	April 2022	WA-726-1: A5.2.1.3 Page (182)	Added as follows: f) b. / geo-cell panels/ concrete geo-synthetic cementicious composite sheets	

2. About the Abu Dhabi Quality and Conformity Council

Abu Dhabi Quality and Conformity Council (QCC) is an Abu Dhabi government entity established in accordance with Local Law No. (3) of 2009 to raise the quality of Abu Dhabi's exports and products traded locally. QCC consists of a council of regulators and industry with a mandate to ensure provision of quality infrastructure in line with global standards.

- QCC's functions are divided into six key areas:
  - Developing standards and specifications
  - Capacity building of metrology systems
  - Strengthening testing infrastructure
  - Launching conformity schemes
  - Protecting consumer interests
  - Ensuring fair trade

- QCC's key stakeholders include regulatory authorities, consumers, retailers and wholesalers, industry, conformity assessment bodies (CABs) and importers.

QCC supports regulators and government organizations through offering quality and conformity facilities, expertise and resources that allow them to implement products safety and compliance requirements and regulations. Additionally, QCC works towards promoting a culture of quality and protecting the interests of consumers. In doing this, QCC seeks to promote the Emirate's competitiveness to become one of the world's most attractive regions for investments and human capital, and to support the competitiveness of national industries in world markets.

3. Acknowledgement

QCC would like to thank the members of the Working Group listed below.

S.#	Name	Entity
1	**Eng. Hamad Muhsen Abdullah**	Abu Dhabi City Municipality
2	**Dr. Amr Adel El-Agroudy**	Abu Dhabi City Municipality
3	**Eng. Mohamed Darwish Jaddawi**	Abu Dhabi City Municipality
4	**Eng. Bakhita Mubarak Saeed Al Ketbi**	Al Ain City Municipality
5	**Eng. Aysha Saeed Ali Al Sharyani**	Al Ain City Municipality
6	**Eng. Anwar Mahmoud Sediqee**	Al Ain City Municipality
7	**Eng. Mohammed Ali Al Ghanem**	Al Dhafrah Region Municipality
8	**Eng.Wael Taher Salem Suleiman**	Environment Agency -Abu Dhabi
9	**Eng. Ehab.Nessim**	Musanada
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17	**Eng.Nizar Kouzar**	Parsons
18	**Eng.Hashim Majeed**	Aldar

4. Foreword

This Storm water and Sub-soil Drainage Systems Design Manual covers the key technical requirements for designing storm water and sub-soil drainage systems within the jurisdiction of Abu Dhabi Emirate.

Principal guidance is contained in the main body of the manual which includes references to established and recognised documents. These documents include national and international standards. Supporting detail such as formulae, information and methodologies are given in the Appendices.

The safety of construction and maintenance workers and of the general public is an important factor for projects in Abu Dhabi and it is essential that safety is considered throughout the design of each project. The reader's attention is drawn to various safety considerations identified by red text.

The Manual is intended to be used by experienced, professional drainage engineers, qualified professionals with specialised knowledge of geology and the application of soil mechanics and other suitably qualified and experienced professionals.

5. Working Group

The Professional Working Group was organized by Abu Dhabi Quality and Conformity Council and established in 2022, which was requested by Department of Municipalities and Transport, to prepare Design Manuals for Storm Water Network in cooperation with the related stakeholders including representatives from government and private sectors in order to establish operating procedures and technical guidelines for the relevant Authorities, Project Developers, Planners, Engineers etc., a step-by-step guidance on how to prepare the waste management aspect of a planning application for new development in the Emirate of Abu Dhabi at the Inception, Planning & Design stages of the Project.

6. Purpose

The purpose of the Project is to review the current design manuals (VOLUME I, DESIGN MANUAL) for storm water network and update the same to count for the Department of Municipalities and Transport current requirements.

7. Scope

This Manual is one of four volumes relating to the design, construction, operation and maintenance of storm water and sub-soil drainage systems for Municipal controlled roads and developments in Emirate of Abu Dhabi:

- Volume 1 – Design Manual
- Volume 2 –Standard Drawings
- Volume 3 – Operation and Maintenance Manual
- Volume 4 – Standard Specifications

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GLOSSARY

AACM	Al Ain City Municipality
ADM	Abu Dhabi City Municipality
ADEHSMS	Abu Dhabi Environment, Health and Safety Management System
ASTM	American Society for Testing Materials
AWWA	American Water Works Association
BS EN	British Standard European Norm
CAPEX	Capital Expenditure
CIRIA	Construction Industry Research and Information Association (CIRIA) (UK)
CoP	Code of Practice
Consultant	An engineering or other professional person or company who translates the Developer's proposals into reports, calculations, drawings and other documents to allow the project to proceed
DEPARTMENT	Department of Municipalities and Transport Abu Dhabi City/ Al Ain Region/ Dhafra Region
Developer	Generally, a property developer who creates new built environment which may include storm water and sub-soil drainage systems
DDP	Design Documentation Package
FHWA	Federal Highway Administration
GPS	Global Positioning System
H&S	Health & Safety
ha	Hectare
HAT	Highest Astronomical Tide
HAZOP	Hazard and Operability Study
HEC	Hydraulic Engineering Circular
HGL	Hydraulic Gradient Line
IDF	Intensity-Duration-Frequency
ISD	Infrastructure Support Division
ITC	Integrated Transportation Centre
MEICA	Mechanical, electrical, instrumentation, control and automation
MHWS	Mean High Water Spring Tide
NADD	New Abu Dhabi Datum
NCMS	National Centre of Meteorology and Seismology
NPC	Net Present Cost
NPV	Net Present Value
NRCS	Natural Resources Conservation Service
O&M	Operation and Maintenance
OSHAD	Abu Dhabi Environment, Health and Safety Centre
OPEX	Operational Expenditure
PLC	Programmable Logic Controller
PIMP	Percentage Impermeability
RTU	Remote Terminal Unit
SSDDM	Storm water and Sub-soil Drainage Design Manual
SCS	Soil Conservation Service, (now US Department of Agriculture Natural Resources Conservation Service)
SuDS	Sustainable Drainage Systems
UPS	Uninterruptible Power Supply

USBR	United States Bureau of Reclamation
VE	Value Engineering
VFD	Variable Frequency Drives
VM	Value Management
WIS	Water Industry Standards (UK)
WRC	Water Research Centre (UK)

1. OBJECTIVES

1.1. Introduction

The purpose of this Design Manual is to identify the requirements for the design of storm water and sub-soil drainage projects arising from municipal development in Emirate of Abu Dhabi.

The Manual limits the use of detailed design theories and methods of calculation and requires that users refer to other documents as necessary. A list of principle reference documents is shown at the end of the Manual.

In addition to rain water run-off from developed areas, including side roads and paved areas and overland flow from open areas, the municipal storm water network may also receive run-off from main highways.

This Manual is one of four volumes relating to the design, construction, operation and maintenance of storm water and sub-soil drainage systems for Municipal controlled roads and developments in Emirate of Abu Dhabi:

- Volume 1 – Design Manual
- Volume 2 –Standard Drawings
- Volume 3 – Operation and Maintenance Manual
- Volume 4 – Standard Specifications

These documents are interdependent and shall be read and applied as a single entity.

Engineers and other disciplines using this Manual must be experienced and appropriately qualified professionals who are familiar with the planning, design, construction, operation and maintenance of storm water drainage networks.

Compliance with the Manual does not absolve users of their professional and contractual responsibilities.

Where a Developer or other party believes that a change to the Manual may be advantageous for a particular project, the written approval of DEPARTMENT shall be sought. Supporting documentation shall be submitted to DEPARTMENT in sufficient detail to allow the proposal to be appraised. A copy of DEPARTMENT's approval letter to be part of any submittal.

Where reference is made to the 'Developer' this also includes all his professional advisers.

It should be noted that the design of foul sewer systems is not covered by this Manual.

1.2. Approvals Process

Any design submittal shall be by a qualified Consultant. The Consultant shall submit his proposals for the site layout and associated surface water and sub-soil drainage to DEPARTMENT in the stages and level of detail set out in Section 8. This section also describes the requirements for the submission of reports, calculations, drawings and other documents for approval.

The Developer shall note that following completion of construction there are further stages to be completed to the satisfaction of DEPARTMENT before the new works can be accepted by DEPARTMENT and ultimately taken over. It should be noted that DEPARTMENT design approval does not guarantee taking over of assets

The following diagram illustrates the general progression of a storm water and sub-soil drainage project, for which the detail is given in Section 8.

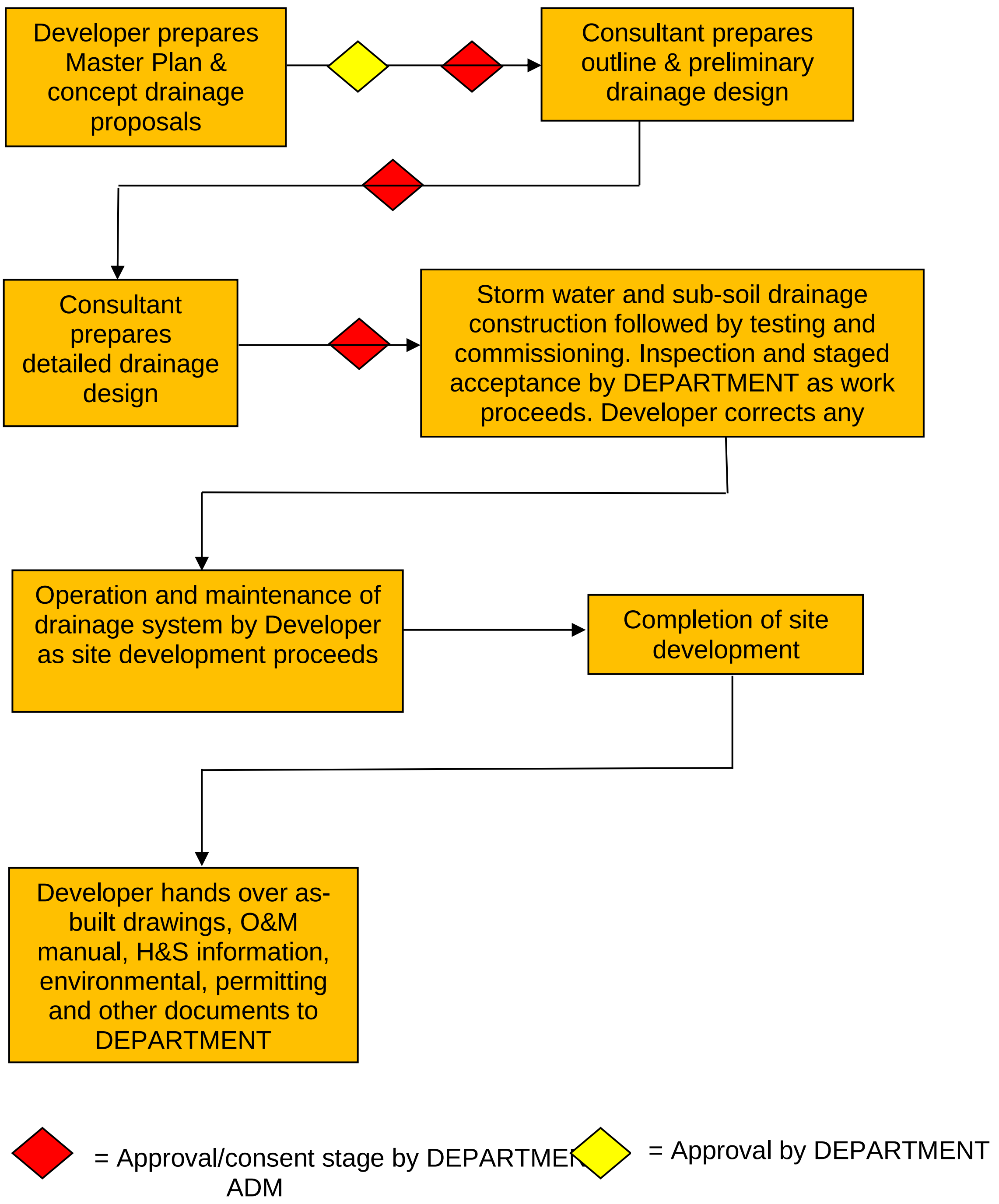


Figure 1-1 - DEPARTMENT and Developer-Led Drainage Project Stages

1.3. Permanent Works

This Manual applies only to the design of the 'permanent works. This also includes the design of any interim measures, such as temporary storage, which may be required until a long-term solution is available. The design of all temporary works required for the construction of the permanent and interim works shall be the responsibility of the Consultant who shall ensure that any such works do not adversely impact on the permanent works.

1.4. Inspection, Testing and Commissioning

The requirements for inspection, testing and commissioning of the works are given in Volume 4: Standard Specifications.

1.5. Innovation

The Developer shall encourage the parties involved in the planning, design and construction of a storm water and sub-soil drainage system to devise innovative, resilient, and sustainable drainage systems(SuDS) solutions and challenge conventional thinking where this could be beneficial to the project and DEPARTMENT.

Documentation in support of this shall be submitted to DEPARTMENT in sufficient detail to allow the proposal to be appraised.

1.6. Copyright

Copyright of the Design Manual, Specification, Standard Drawings and Operation and Maintenance Manual is the property of DEPARTMENT.

1.7. Updates

This manual will be revised by DEPARTMENT from time to time to keep up to date with technical developments and improved practices. It is the responsibility of the user to ensure that they are working to the latest issue.

Any errors that are found or recommendations for improvement should be notified to DEPARTMENT.

2. DESIGN CONSIDERATIONS

2.1. Network Integration

DEPARTMENT's Planning Department development plans will show the existing and future land use within which the Developer shall identify his proposals.

At any point in time the storm water and sub-soil drainage systems in Abu Dhabi will need to accommodate flows from the following contributing sources:

- The existing networks being operated by DEPARTMENT
- Proposed future planning:
    - Extensions to the trunk main network being planned and/or constructed by DEPARTMENT to accommodate surface water from new developments and changes in sub-soil flows in accordance with the overall Municipal Development Plan. Manholes are to be located at strategic points to allow connections from new developments to be made in the future,
    - New developments with their associated sub-catchment drainage, and
- Stormwater drainage from new highways being planned and/or constructed by the Department of Transport Overland run-off from open areas.
- Excess irrigation water collected from the soft landscaped areas
- Discharge from Dewatering system arising from construction sites
- Flow from ground water control system

The Consultant shall take account of all the above when preparing his drainage proposals.

DEPARTMENT has long-term responsibility for the storm water and sub-soil networks and in order to optimise the ownership and maintenance costs, it requires the design proposals to:

- Optimise for gravity solutions, utilizing non-disruptive and trenchless pipeline installation technologies when large depths are encountered. No electromechanical solutions shall be utilized unless a gravity solution does not connect at the required elevations.
- Optimise the size of pipes versus additional storage provided for flow attenuation. Attenuation reduces the peak flow rate and may allow the use of smaller pipes downstream. A detailed appraisal, including NPV calculations, shall be carried out to justify the solution adopted.
- Optimise the number of pumping stations when pumping is absolutely necessary. The number and location of pumping stations shall be balanced against other considerations. A detailed appraisal including Net Present Value (NPV) calculations shall be carried out to justify the number, size and location of pumping stations.

2.2. Design Philosophy

DEPARTMENT requires storm water and sub-soil drainage systems to be provided which:

- Fulfil its requirements for minimizing the risk of flooding
- Drain water-logged areas
- Are of reasonable cost to construct, operate and maintain
- Do not involve undue health and safety risks during construction, operation and maintenance
- Comply with environmental legislation and protocols

The Consultant shall use this Manual together with their professional experience and expertise to achieve a sensible balance of the above. Achieving the right balance will be assisted by the Consultant having regular interaction with DEPARTMENT as the design proceeds.

2.3. Design Life

The minimum periods of time for new assets to last before replacement are shown in Table 2-1.

Asset type	Minimum Design Life (years)
Pipelines	60
Structure (With Reinforcement) Structure (Without Reinforcement)	50 60
Mechanical and Electrical Equipment	15
Instrumentation, computer hardware & sensors	5

Table 2-1 – Design Life

It is recognised that maintenance, repairs and in the case of MEICA equipment, parts replacement, will be required during the above periods. A planned maintenance schedule and spare parts list shall form part of the design submission documentation.

The design life given in Table 2-1 is to be used in the NPV calculations for asset replacement.

2.4. Investigations

The Consultant shall note that he may need to carry out site surveys and investigations during the design process which may take a significant amount of time to complete. These may include:

- Environmental studies, e.g., hydrodynamic modelling
- Marine studies
- Ecological studies
- Traffic studies
- Geotechnical and groundwater investigations

2.5. Environmental Impact

2.5.1. Environmental Legislation

The Consultant's drainage proposals shall comply with all relevant local and regional legislation and international conventions and protocols to which AbuDhabi is a signatory.

The relevant requirements are given in Appendix A1.

The Consultant shall note that no project can commence prior to securing the appropriate approval in accordance with the regulations determined by the Competent Authority.

The Consultant shall comply with the Standard Operating Procedures and Technical Guidelines produced by the Environment Agency Abu Dhabi.

Where there is any conflict between the legislation or standards a decision shall be sought from DEPARTMENT on the correct application.

2.5.2. Cultural Heritage Permitting Requirements

It is the responsibility of the Consultant to ensure that all current requirements relating to cultural heritage permitting are complied with during the project.

2.6. Health and Safety in Design

The Consultant shall be aware of his statutory Health and Safety (H&S) responsibilities in carrying out a drainage project. Those responsibilities start at the inception of a project and continue until the asset is adopted by DEPARTMENT.

The Consultant shall ensure that other parties to the project are aware of their statutory H&S responsibilities.

All parties shall adhere to the Abu Dhabi Environment, Health and Safety Management System (ADEHSMS) published by OSHAD and the associated Codes of Practice, which together form the framework under which H&S is managed. Consultants should in particular take account of Code of Practice No 20 – 'Safety in Design'.

The key requirements for consultants are to:

- ensure they are sufficiently competent and adequately resourced to address H&S issues during design
- minimise foreseeable risks and hazards to people and property directly or indirectly involved or affected by construction work and the future use of the asset
- provide a record of all H&S considerations during the design process
- include information on the drawings of any significant risks that could be present during construction
- provide information for inclusion in the O&M Manual on design-related matters that could impact on the safety of operation and maintenance
- coordinate with others to ensure that risks are comprehensively managed
- promote a learning environment in H&S matters

2.7. Value Management and Value Engineering

Value Management (VM) is a means of maximizing the value of a project through the structured examination of decisions about benefits, risks and costs. The VM process requires a high degree of challenge to standard solutions and depends on a clear understanding of the problems to be solved. Maximum benefit from VM occurs in the early stages of a project.

Value Engineering (VE) is a structured process which examines the functionality of a project element and assesses whether it fulfils the required function at best value.

VM and VE are carried out through formal workshops, as well as being an integral part of the day-to-day design development. Formal workshops are to be run by a trained facilitator and include all relevant stakeholders.

VM reviews should be carried out throughout the project life cycle, as set out in the value management plan, and the number required will depend on the project complexity. The reviews should generally follow the following sequence:

i. VM1 Project definition.
ii. VM2 Concept design.
iii. VE1 Preliminary design and engineering.
iv. VE2 a,b,c Detailed design.
v. VM3 Procurement and contract strategy.
vi. VM4 Post project feedback.

The Consultant shall prepare and submit report at the end of each review summarising the scope of the review, the process adopted, the changes made and the ensuring benefits.

A formal VM and VE approach involves significant time and cost. The Consultant shall carry out a preliminary assessment to determine whether such an approach would be beneficial to a particular project. DEPARTMENT has a major interest in optimising the content of drainage projects and the Consultant shall obtain DEPARTMENT's agreement where he considers that formal VM and VE are not justified. For further details, refer to Value Engineering Manual.

2.8. Options Appraisal

In many cases of storm water and sub-soil drainage design there will be several layout options.

DEPARTMENT's approval of the Developer's proposals requires that there is a cost-effective balance between initial capital cost and ongoing operational and maintenance costs. To support this evaluation the Consultant shall carry out an options appraisal exercise based on the layout options agreed with DEPARTMENT. The optimum balance will normally be given by the lowest NPV based on capital and annual costs. However, other factors such as construction risk, programme risk, health and safety risk and environmental impact shall also be considered.

In the case of a simple drainage network with a clear solution the Developer may request DEPARTMENT's approval to not carrying out an options appraisal exercise.

2.8.1. Capital Cost (CAPEX)

The capital cost of each option shall be estimated using the latest market rates, preferably obtained from DEPARTMENT tenders. The estimating method will depend on the stage of the project; unit rates will be adequate for the Master Plan stage with increasing accuracy and detail through to the final estimate at detailed design which is to be based on a detailed bill of quantities. The estimating method is to be approved by DEPARTMENT before commencing the stage.

2.8.2. Operational and Maintenance Cost (OPEX)

The operating and maintenance costs of each option shall be estimated taking advice from DEPARTMENT as the long-term owner. Costs will include:

- Labour
- Vehicles and plant
- Power
- Parts and consumables
- Chemicals
- O&M management

For each option the Consultant shall prepare a spreadsheet of estimated daily, weekly, monthly and annual operating and maintenance quantities to which DEPARTMENT will add its unit costs to give annual estimates of OPEX. These values shall be used by the Consultant in appraising the options.

2.8.3. Economic Appraisal of Options

At various points in the Manual reference is made to NPV (Net Present Value) calculations. This is the means of making an economic comparison of the whole life cost of competing options. The option with the lowest NPV gives the most economically advantageous solution.

NPV costs shall be calculated from the time of construction commencement and all costs are to be at the current price base.

DEPARTMENT will provide the evaluation period to be used in the financial appraisal, and the Consultant will decide on the discount rate in coordination with DEPARTMENT. A sensitivity check with higher and lower discount rates may also be requested by DEPARTMENT.

In addition to the NPV whole life costing, the capital and O & M costs used in the evaluation have to be clearly set out all at current price base.

2.9. Drawings

2.9.1. Standard Drawings

Standard drawings are included in Volume 2: Standard Drawings. Where these are used, they shall not be altered except where indicated.

2.9.2. Typical Drawings

Typical drawings are included in Volume 2: Standard Drawings. These drawings are for guidance and may be adapted to suit the particular requirements of the project.

2.10. Materials Selection

All materials shall comply with the Volume 4: Standard Specifications and shall be approved by the Materials Section of DEPARTMENT.

3. RAINFALL AND RUNOFF

3.1. Introduction

This section provides information about key rainfall parameters, catchment characteristics and modelling methods to be used in storm water design in Emirate of Abu Dhabi. The methods should be used when an estimate of flow from an urban, non-urban or mixed catchment in response to storm rainfall is required. The method that should be used is summarised in Table 3-1, and described in detail in the rest of this chapter. If flood volumes are also required, the recommended approach for calculating these is given in Section 3.6.

Prepare 2D model and submit if requested by the DEPARTMENT to realize actual floods

<table>
<tr><th colspan="3">Summary of rainfall runoff methods</th></tr>
<tr><td>1</td><td colspan="2">Select required design standard. In case the Rational Method will be used, select the time of concentration multiplier.</td></tr>
<tr><td>2</td><td colspan="2">Calculate time of concentration t_c, as appropriate to the rural or urban nature of the catchment</td></tr>
<tr><td>3</td><td colspan="2">Calculate rainfall depth and intensity from IDF relationships</td></tr>
<tr><td colspan="2">Rational Method
For catchment areas < 80 ha and / or t_c < 30 mins and no significant retention and / or back water effects, the rational method will be used.</td><td>SCS Method
For catchment areas > 80 ha and / or t_c >30 mins, the SCS method is recommended for use in the design of new networks. However, the SCS method can be used in the design and analysis of networks for all sizes of catchments.</td></tr>
<tr><td>4</td><td>Estimate runoff coefficients based on land use.</td><td>Estimate curve number based on land use. Check runoff and replace with coefficient of runoff estimate if necessary.</td></tr>
<tr><td>5</td><td>Apply rational equation for network calculations for sub-catchments.</td><td>Apply SCS unit hydrograph method. Use hydrodynamic model for network calculations.</td></tr>
<tr><td>6</td><td>Calculate runoff volumes if necessary.</td><td>Calculate runoff volumes.</td></tr>
</table>

Table 3-1 - Summary of rainfall-runoff methods

Other design storms, as appropriate for local conditions or specific projects may be used as agreed with by DEPARTMENT.

3.2. Design Storms

3.2.1. Design Return Period

The first step in generating the design rainfall is to determine the required design standard. These should be taken as:

Event	Area
1 in 5 Years Storm	This is the design standard for all areas served by storm water networks, with the exceptions highlighted in the rows below.
1 in 10 Years Storm	Major roads, palaces, soakaways and other critical areas, as approved by DEPARTMENT.

Event	Area
1 in 25 Years Storm	Culverts on minor roads (and check for 1 in 50 years storm); lagoons & soakaways that do not have an outlet and where overtopping may lead to flooding of properties* Airports, Seaports (check for 1 in 50 years storm)
1 in 50 Year Storm	Underpasses and underground car parks (Ramps); Culverts on major roads (and check for 1 in 100 years storm)

* Lagoons that have an outlet or overland flow track to sea that does not cause disruption to properties or road users will be sized for the next higher event in the above table. For example, a lagoon serving an area which is generally a 1 in 5-year catchment type shall be sized for a 1 in 10-year storm before overflow occurs

Table 3-2 – Design storm standard for various facilities

3.2.2. Duration of Rainfall

To determine the storm duration, the multipliers given in Table 3-3 should be applied to the time of concentration, according to the catchment type. This means longer storms, and therefore lower rainfall intensities are used for lower priority sites; design flows for these catchments will therefore be lower. The recommended methods for calculation of time of concentration are described in the following sections.

Priority	Catchment type	T_c Multiplier M
1	Major Roads; expressways; arterials; underpasses; Airports; Seaports; road crossings and culverts; underground car parks (Ramps)	T_c
2	High density sectors (≥ 50% impermeable area)	2 x T_c
3	Low density sectors (≤ 50% impermeable area)	3 x T_c
4	Industrial sectors*	5 x T_c
5	Open areas, parks and areas of infrequent use and not subject to building flooding	12 x T_c

* If the Industrial sectors are more than 50% impervious, then they should be treated as Low density sectors.

Table 3-3–Priorities and time of concentration multipliers used to determine design storm durations

Please note that the above is applicable only to calculations done using the Rational Method. For SCS method, multiplication factors shall be applied only for the initial time of concentration.

3.2.3. Design Storm Duration

Design storm duration and a specified design return period are required to determine design rainfall intensity, i (mm/hr) from IDF relationships as given in the following section.

For storm water networks and drainage design the time of concentration T_c, along with the multipliers of Table 3-3, should be taken as the design storm duration. Time of concentration is defined as the interval in time from the beginning of the rainfall to the time when water from the furthest point in the catchment reaches the point under consideration. It is one of the key characteristics of a catchment and care is required in its calculation.

The response times of urban and rural catchments can be very different, and different methods are required for each. Dealing with mixed urban/rural catchments is discussed in Section 3.5.

Rural (undeveloped) catchments

The following formula for t_c is based on kinematic flow assumptions and should be used for rural catchments:

$$\boldsymbol{T_c = \frac{0.12n^{0.6}L^{0.6}}{S^{0.3}i^{0.4}}}$$

Where:

T_c = time of concentration (hours)

n = Manning's roughness coefficient

L = length of flow from furthest point (meters)

S = average slope (meters/meters)

i = average rainfall intensity (mm/hr)

The rainfall intensity,'*i*', is related to the storm duration by a relationship of the form:

$$i = \frac{\alpha}{t^{\beta}}$$

The values of α and β for this equation for each return period are given in Table 3-5.

Because the required storm duration is equal to the time of concentration, combining these equations gives:

$$T_c = \left(\frac{0.12n^{0.6}L^{0.6}}{S^{0.3}\alpha^{0.4}}\right)^{\frac{1}{1-0.4\beta}}$$

This equation should be used to calculate time of concentration. The appropriate multiplier given in Table 3-3 is then applied to give the design storm duration for the storm water network.

It is recommended that an appropriate 'n' value to use for undeveloped areas of Abu Dhabi lies between 0.020 and 0.035. This range has been established using a method for developing Manning's 'n' values for floodplains. Where catchment response is expected to be dominated by channel flow, then an appropriate value of Manning's n based on the channel type should be used.

Urban (developed) catchments

Time of concentration for urban catchments is made up of two chief components:

- Time taken for water to enter the storm water drainage network (entry time)
- Time taken for water to pass through the storm water drainage network (travel time)

The total time of concentration is the sum of these.

For calculation of entry time, use

Figure 3-1 and an average value for catchments slope to calculate the flow velocity, then use this with the average distance water must travel to reach the storm water network to calculate the entry time. A minimum value of 10 minutes should be used for impervious catchments.

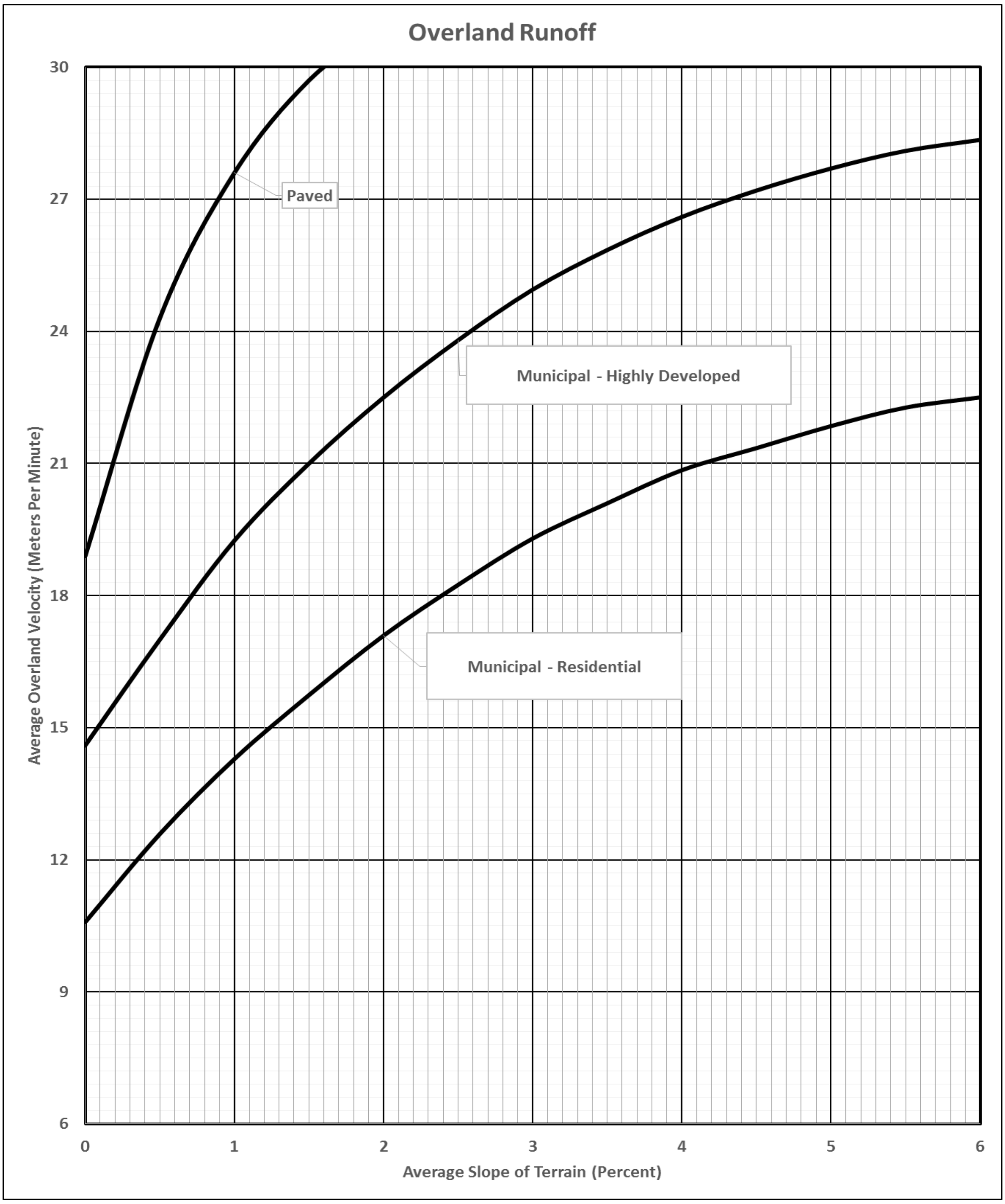


Figure 3-1 - Overland flow velocities for various land types

To calculate the time taken to pass through the storm water system, use the following method:

1. Prepare a preliminary layout of storm water network, taking account of the available discharge points. Mark pipe numbers on plan in accordance with numbering convention
2. Estimate the approximate gradients and pipe diameters for each pipe.
3. Estimate the pipe full velocity (V_f) using Manning's equation and a Manning's *n* value of 0.015:

$$V_f = \frac{1}{n}\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

D is the pipe diameter and S is the slope. Alternatively, the Colebrook White equation and the pipe roughness as specified in Section 4.3.1 can be used to estimate the pipe full velocity.

4. Calculate the time of flow for each main branch using T_f=L/V_f where L is the length of the branch
5. Calculate the time of concentration for each main branch by adding the 'time of entry' to the 'time of flow' using V_f (i.e., $t_c = T_e + T_f$) and choose the branch giving the maximum t_c from the head of the storm water networks to the point of discharge. This time is then used to give the storm duration to be adopted.

3.2.4. Rainfall Intensity Duration Frequency (IDF)

The rainfall Intensity-Duration-Frequency (IDF) relationship provides the average intensity of rainfall during a storm event with a specified duration and frequency of occurrence (return period).

The rainfall pattern of the northeast part of the study area is different than the rainfall pattern of other locations inside the study area covered by this manual. Additionally, the topography of the North-East part of the study area is a mountainous area while the remaining area of the study area covered by this manual is classified as a flat area. Based on that, the study area is divided into two zones. The first zone (Zone A) is devoted to the North-East part while the second zone (Zone B) is devoted to the remaining part of the study area. The dividing line between the two zones is around the contour line equal to 200 m above mean sea level. The boundaries of the two zones are presented in Figure 3-2.

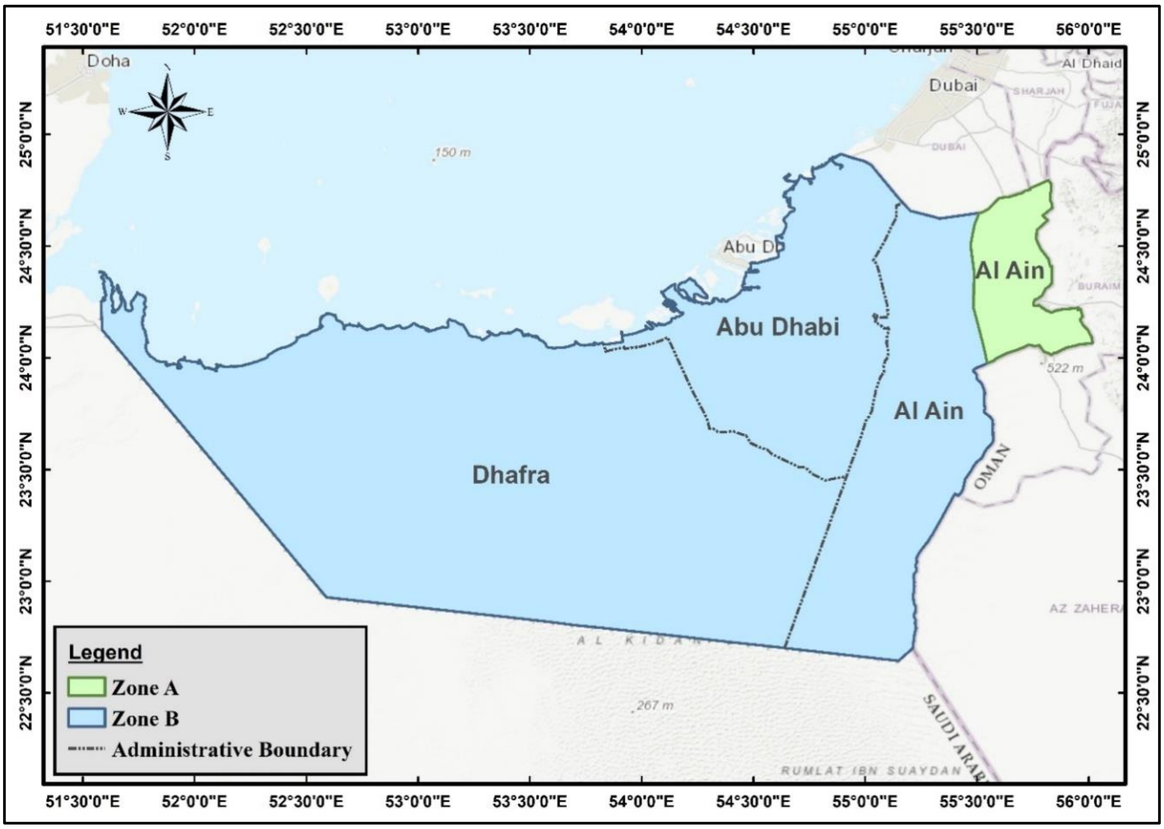


Figure 3-2: Two Zones Boundaries

The recommended IDF curves, IDF and DDF tables and IDF equations for ZoneAare shown below in Figure 3-3,Figure 3-4,

Return period (yrs)	Rainfall Depth (mm)									
100	35.72	42.07	46.04	53.90	63.10	73.88	81.00	94.86	111.00	130.08
50	30.48	35.90	39.29	46.00	53.84	63.04	69.12	80.94	94.68	110.88
25	17.85	22.11	24.91	30.66	37.74	46.46	52.44	64.56	79.44	97.68
10	11.24	14.34	16.44	20.85	26.44	33.54	38.55	48.90	61.92	78.48
5	9.96	12.69	14.55	18.42	23.32	29.54	33.90	42.96	54.36	68.88
2	6.18	7.41	8.20	9.78	11.65	13.90	15.39	18.36	21.84	26.16

Table 3-4 and Table 3-5. While, the recommended IDF curves, IDF and DDF tables and IDF equations for Zone B are shown below in Figure 3-3, Figure 3-6,

Return period (yrs)	Rainfall Depth (mm)									
100	35.72	42.07	46.04	53.90	63.10	73.88	81.00	94.86	111.00	130.08
50	30.48	35.90	39.29	46.00	53.84	63.04	69.12	80.94	94.68	110.88
25	17.85	22.11	24.91	30.66	37.74	46.46	52.44	64.56	79.44	97.68
10	11.24	14.34	16.44	20.85	26.44	33.54	38.55	48.90	61.92	78.48
5	9.96	12.69	14.55	18.42	23.32	29.54	33.90	42.96	54.36	68.88
2	6.18	7.41	8.20	9.78	11.65	13.90	15.39	18.36	21.84	26.16

Table 3-4 and Table 3-5.

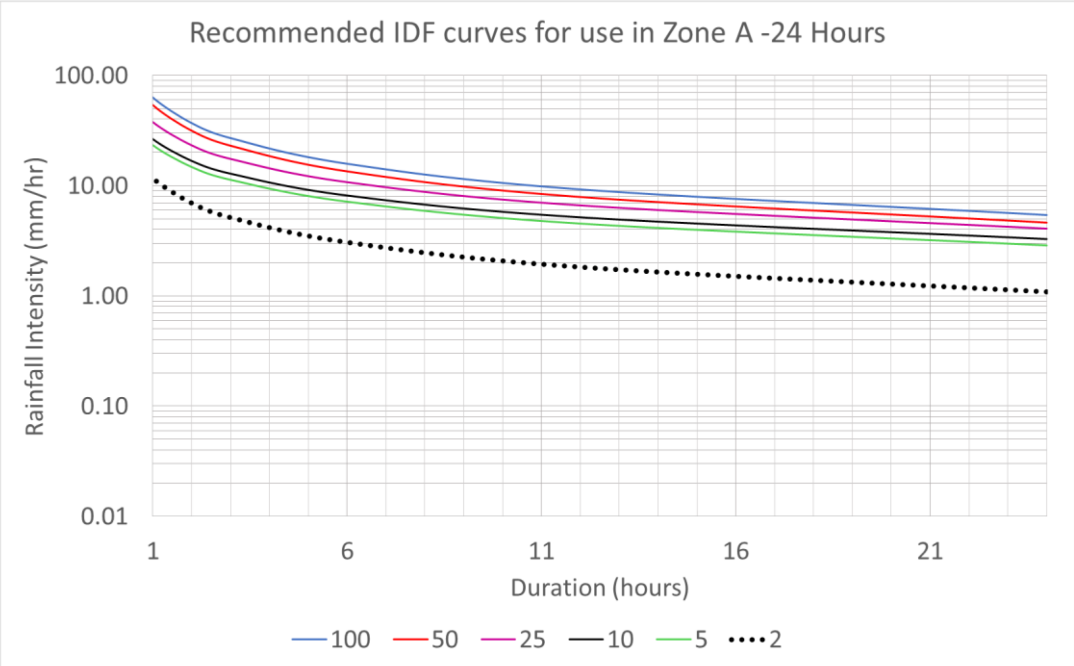


Figure 3-3 - Recommended IDF Curves for Zone A– 24 Hours

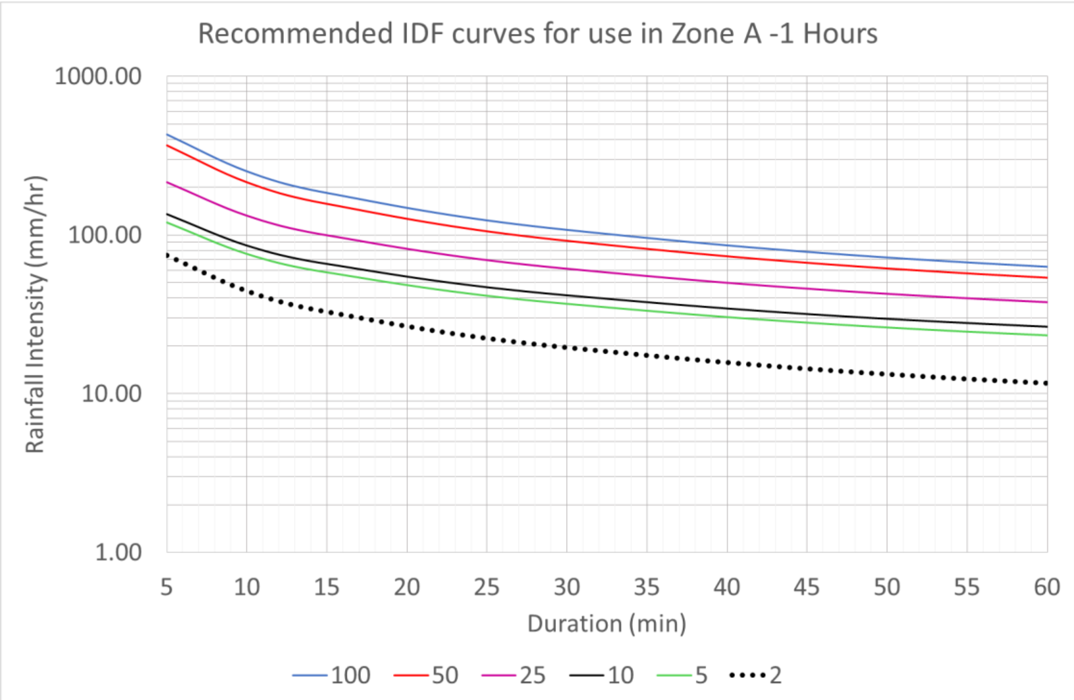


Figure 3-4 - Recommended IDF Curves for Zone A – 1 Hour

Duration (mins)	5	10	15	30	60	120	180	360	720	1440
Duration (hrs)	0.083	0.167	0.25	0.5	1	2	3	6	12	24
Return period (yrs)	Rainfall Intensity (mm/h)									
100	430.34	251.91	184.16	107.80	63.10	36.94	27.00	15.81	9.25	5.42
50	367.24	214.96	157.15	91.99	53.84	31.52	23.04	13.49	7.89	4.62
25	215.10	132.37	99.65	61.32	37.74	23.23	17.48	10.76	6.62	4.07
10	135.38	85.85	65.76	41.70	26.44	16.77	12.85	8.15	5.16	3.27
5	120.05	76.01	58.18	36.84	23.32	14.77	11.30	7.16	4.53	2.87
2	74.44	44.38	32.79	19.55	11.65	6.95	5.13	3.06	1.82	1.09
Return period (yrs)	Rainfall Depth (mm)									
100	35.72	42.07	46.04	53.90	63.10	73.88	81.00	94.86	111.00	130.08
50	30.48	35.90	39.29	46.00	53.84	63.04	69.12	80.94	94.68	110.88
25	17.85	22.11	24.91	30.66	37.74	46.46	52.44	64.56	79.44	97.68
10	11.24	14.34	16.44	20.85	26.44	33.54	38.55	48.90	61.92	78.48
5	9.96	12.69	14.55	18.42	23.32	29.54	33.90	42.96	54.36	68.88
2	6.18	7.41	8.20	9.78	11.65	13.90	15.39	18.36	21.84	26.16

Table 3-4 - Recommended IDF and DDF tables for use in Zone A

Return Period	α	Β
100 year	63.096	0.773
50 year	53.843	0.773
25 year	37.739	0.700
10 year	26.443	0.657
5 year	23.325	0.659
2 year	11.653	0.746
t = storm duration in hours (t_c) i= rainfall intensity in mm/hr M = multiplication factor	$i = \dfrac{\alpha}{(t * M)^{\beta}}$	

Table 3-5 – Recommended IDF equations for use in Zone A

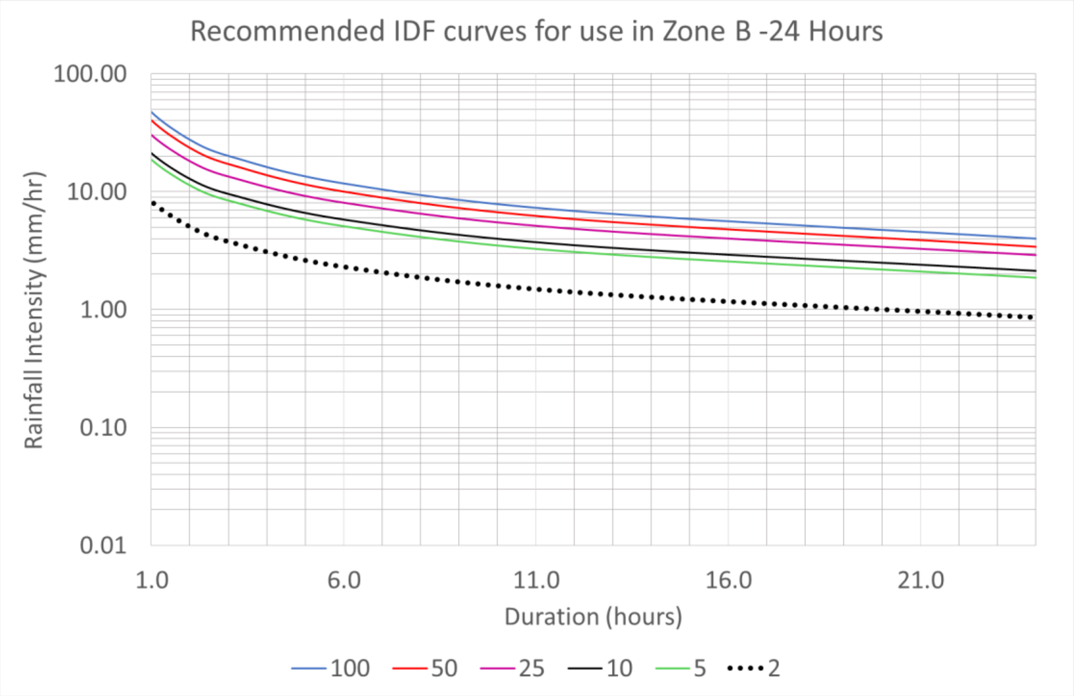


Figure 3-5 - Recommended IDF Curves for Zone B – 24 Hours

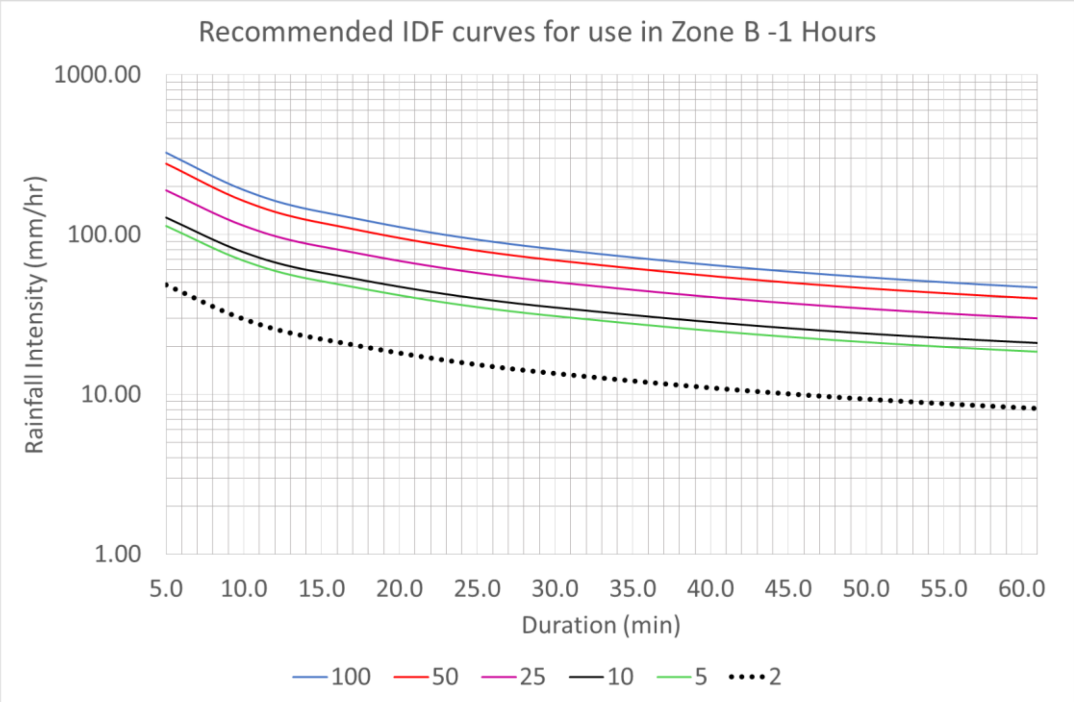


Figure 3-6 - Recommended IDF Curves for Zone B – 1 Hour

Duration (mins)	5	10	15	30	60	120	180	360	720	1440
Duration (hrs)	0.083	0.167	0.25	0.5	1	2	3	6	12	24
Return period (yrs)	Rainfall Intensity (mm/h)									
100	325.17	189.81	138.54	80.87	47.21	27.56	20.11	11.74	6.85	4.00
50	277.48	161.98	118.22	69.01	40.28	23.51	17.16	10.02	5.85	3.41
25	189.05	113.38	84.07	50.42	30.24	18.13	13.45	8.06	4.84	2.90
10	127.64	77.34	57.70	34.96	21.18	12.84	9.57	5.80	3.52	2.13
5	113.18	68.48	51.04	30.88	18.68	11.30	8.43	5.10	3.08	1.87
2	48.39	29.54	22.13	13.51	8.25	5.04	3.77	2.30	1.41	0.86
Return period (yrs)	Rainfall Depth (mm)									
100	26.99	31.70	34.64	40.44	47.21	55.12	60.33	70.44	82.20	96.00
50	23.03	27.05	29.56	34.51	40.28	47.02	51.48	60.12	70.20	81.84
25	15.69	18.93	21.02	25.21	30.24	36.26	40.35	48.36	58.08	69.60
10	10.59	12.92	14.43	17.48	21.18	25.68	28.71	34.80	42.24	51.12
5	9.39	11.44	12.76	15.44	18.68	22.60	25.29	30.60	36.96	44.88
2	4.02	4.93	5.53	6.76	8.25	10.08	11.31	13.80	16.92	20.64

Table 3-6 - Recommended IDF and DDF tables for use in Zone B

Return Period	α	β
100 year	47.206	0.777
50 year	40.284	0.777
25 year	30.237	0.738
15 year	22.327	0.695
10 year	21.183	0.723
5 year	18.685	0.725
2 year	8.248	0.712
t = storm duration in hours (t_c) i = rainfall intensity in mm/hr M = multiplication factor	$i = \frac{\alpha}{(t * M)^{\beta}}$	

Table 3-7 – Recommended IDF equations for use in Zone B

The 24 hours rainfall distribution curve for Zone A and B are shown in Figure 3-7, and Figure 3-8 respectively

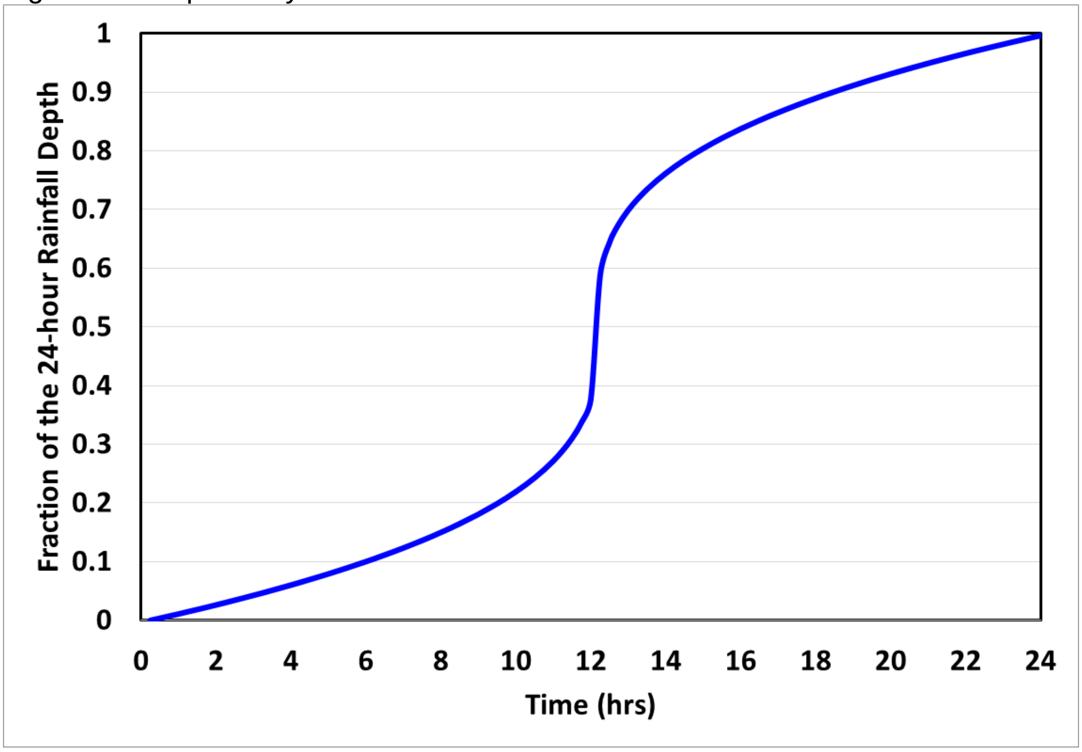


Figure 3-7–The 24 hours rainfall distribution curve for Zone A

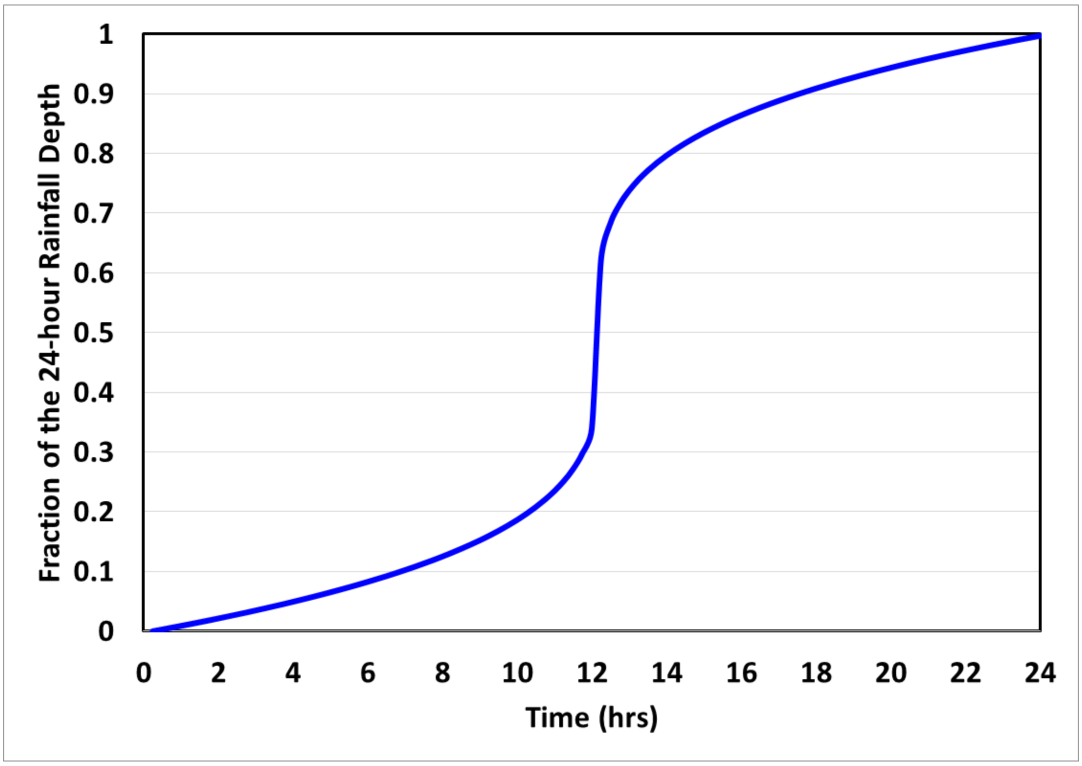


Figure 3-8 - The 24 hours rainfall distribution curve for Zone B

Confirm with DEPARTMENT as to which zone to use based on each project requirements. Some projects may also need individual analysis or equivalent factors to be used with the IDF data.
Consultants is responsible to ensure that using then given IDF data is appropriate and adequate for the specific projects in that give timeframe. Alternatively, individual IDF analysis may be required.

3.3. Rational Method

The Rational Method is based on assumptions that the rainfall intensity is uniform across the catchment and for the duration of the storm, and that the time of concentration is less than or equal to the storm duration. These assumptions mean that the catchment reaches a steady state toward the end of the storm, and therefore the flow out of the catchment depends only on the area, rainfall intensity and runoff coefficient.

The rational method should only be used when the catchment being modelled is less than 80 ha in area or the time of concentration is <30 minutes. Rational Method is not much suitable to simulate effect of external incoming flows, backwater, and routing through lagoons and soakaways, for this may result in uneconomical designs.

3.3.1. Runoff Coefficient

The runoff coefficient describes the proportion of rainfall forming rapid runoff and therefore contributing to peak flow. Calculation of the runoff coefficient is based on land use.

For urban areas, recommended values are shown in Table 3-8 (column 2).These values are applicable to the catchment range contributing to the network.

Type of Development / Surface (Column 1)	Recommended Runoff coefficient (Column 2)
> 70% irrigated grass & plantings	0.15
Poor ground cover	0.20
Asphaltic	0.90
Concrete	0.90
Interlocking	0.78
Roofs	0.85
Walled plot; not connected to network	0.20

Table 3-8 - Typical Urban Run-Off Coefficients

Consultants are required to confirm above coefficients based on land use / land cove in any specific project. Coefficient values need to be agreed with DEPARTMENT for each project.

Runoff coefficients for rural areas should be calculated as described in Table 3-9. This calculation is based on slope, soil cover types, vegetation and surface storage.

For catchments with a mixture of land cover types (both rural and non-rural) the runoff coefficient should be calculated as the area weighted average coefficients for the different land cover types (subject to the precautions concerning mixed catchments discussed in Section 3.5).

Type	Extreme	High	Normal	Low
Relief	0.35	0.28	0.20	0.14
	Steep, rugged terrain with average slopes above 30%.	Hilly, with average slopes of 10% to 30%.	Rolling, with average slopes of 5% to 10%.	Relatively flat land, with average slopes of 0 to 5%.
Soil infiltration	0.16	0.12	0.08	0.06
	No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity.	Clay or shallow loam soils of low infiltration capacity or poorly drained.	Normal; well- drained light or medium textured soils, sandy loams, silt and silt loams.	High; deep sand or other soil that takes up water readily, very light, well-drained soils.
Vegetal cover	0.16	0.12	0.08	0.06
	No effective plant cover, bare or very sparse cover.	Poor to fair; natural cover, with less than 20% of drainage area having been irrigated landscape.	Fair to good; about 50% of area in crop or other irrigated landscaping.	Good to excellent; about 90% of drainage area in crop or other irrigated landscaping.
Surface storage	0.12	0.10	0.08	0.06
	Negligible surface depression few and shallow; drainage ways steep and small.	Low; well- defined system of small drainage ways; no isolated low areas.	Normal; considerable surface depression storage.	High; surface storage, high; drainage system not sharply defined, typical for interior areas of Regions 1 and 3. Applies also to isolated sabkha areas that have no surface outlet*.
How to use this table: Select the appropriate coefficient value from each of the four relief categories and cumulate to find the composite ‘C’ value to use in the rational equation. For example: for a catchment area with rolling terrain with 8 percent slopes (0.20), with well-drained sandy loam soil (0.08), no plant cover (0.16), and normal surface depression storage (0.08), C = 0.20 + 0.08 + 0.16 + 0.08 = 0.52.				
*Special case ‘C’ for sabkha areas: Where sabkha areas have a direct surface flow connection (not isolated) within the drainage catchment area, use a C = 0.85 for the sabkha area				

Table 3-9 - Typical Rural Run-Off Coefficients

3.3.2. Flow calculation

To calculate peak flow using the rational method, the following equation should be used:

$$Q = 2.77 \times 10^{-3} CiA$$

Where:

Q = peak flow (m^3/s)

C = the runoff coefficient

i = rainfall intensity (mm/hr)

A = catchment area (hectares)

3.4. Soil Conservation Service (SCS) Method

This method was originally developed by the U.S. Department of Agriculture, Soil Conservation Service (now known as Natural Resources Conservation Service—NRCS) to predict direct runoff volumes for given rainfall events.
The SCS rainfall-runoff method is based on two components: a runoff equation and a unit hydrograph to route the runoff to the catchment outlet. The method should be used when the catchment area is greater than 80 ha.

3.4.1. Curve Number

The SCS method calculates runoff based on a series of runoff curves, which describe an initial storage followed by runoff increasing with rainfall. The initial infiltration and subsequent runoff is parameterised by a curve number. Runoff curves for different curve numbers are shown in Figure 3-9.

The initial storage in mm is given by:

$$I_a = 0.2 \times \left(\frac{1000}{CN} - 10 \right) \times 25.4$$

Once rainfall exceeds the initial storage, the depth of runoff in mm is given by:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Where P is the rainfall in mm and S is the potential maximum soil moisture retention given in mm by:

$$S = \frac{25400}{CN} - 254$$

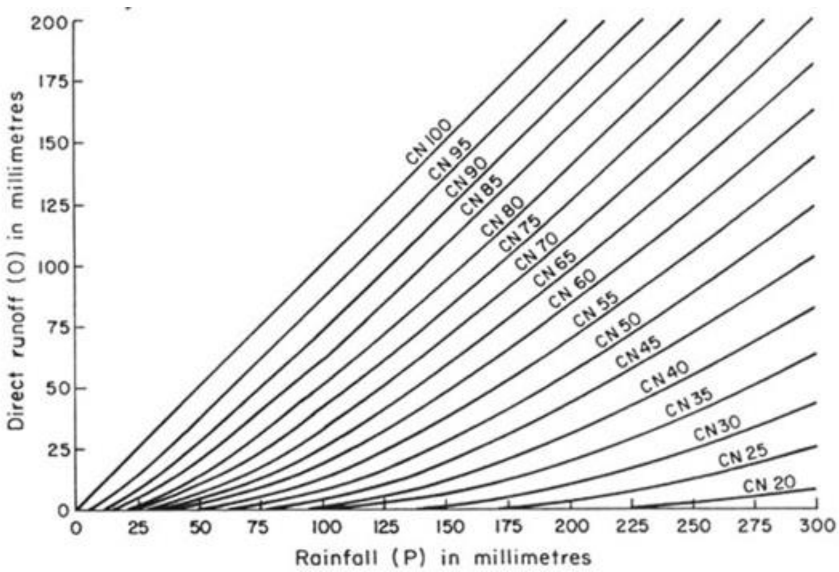


Figure 3-9 - Runoff curves for different SCS curve numbers

The curve number should be selected by firstly ascertaining the dominant soil type for the catchment and its hydrological soil group (Table 3-10). If no information on soils exists, then it should be assumed to be Group B. The most appropriate land cover type should be chosen from Table 3-11and used to find the curve number. Recommended type of curve to be used for hydraulic modelling should be SCS Type II Cumulative type curve, unless otherwise specified by DEPARTMENT.

General Soil Textures and their Approximate Hydrologic Grouping	
Soil Textures	**Hydrologic Soil Group**
Sand, loamy sand, and sandy loam	A
Silt loam and loam	B
Sandy clay loam	C
Clay loam, silty clay loam, sandy clay, silty clay, and clay. Undrained soils with high water table.	D

Table 3-10 - Soil characteristics and hydrological soil groups

Runoff Curve Numbers for Hydrologic Soil Cover Complexes (AMC II, I = 0.2S) (SCS, 1983)				
Land Use Description	**Curve Numbers for Hydrologic Soil Group**			
	A	**B**	**C**	**D**
Fully Developed Urban Area[1]				
Lawns, open spaces, parks, golf courses, cemeteries etc.:	39	61	74	80
good condition: grass cover on 75% or more of the area	49	69	79	84
fair condition: grass cover on 50% to 75% of the area	68	79	86	89
poor condition: grass cover on 50% or less of the area	98	98	98	98
Paved parking lots, roofs, driveways, etc. Streets and roads:				
paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89
paved with open ditches	83	89	92	93
Developing Urban Areas[2] (No Vegetation Established)				
Newly graded area or bare soil	77	86	91	94
Cultivated Agricultural Land				

Runoff Curve Numbers for Hydrologic Soil Cover Complexes (AMC II, I = 0.2S) (SCS, 1983)					
Land Use Description		**Curve Numbers for Hydrologic Soil Group**			
		A	**B**	**C**	**D**
	Hydrologic Condition[3]				
Fallow:					
Straight row		77	86	91	94
Overall tillage	poor	76	85	90	93
Overall tillage	good	74	83	88	90
Row Crops:					
Straight row	poor	72	81	88	91
Straight row	good	67	78	85	89
Contoured tillage	poor	71	80	87	90
Contoured tillage	good	64	75	82	85
Contoured terraces	good	62	71	78	81
	Hydrologic Condition[3]				
Small Grain:					
Straight row	poor	65	76	84	88
Straight row	good	63	75	83	87
Contoured tillage	poor	64	75	83	86
Contoured tillage	good	60	72	80	84
Close-seed Hay or Rotation Pasture[4]:					
Straight row	poor	66	77	85	89
Straight row	good	58	72	81	85
Contoured tillage	poor	64	75	83	85
Contoured tillage	good	55	69	78	83
Desert Range[5]:	poor	55	73	82	86
	fair	44	65	76	82
	good	32	58	72	79
Forest Land (irrigated by drip systems) (no sprinkler irrigation of secondary growth)[6]	poor	55	73	82	86
	fair	44	65	76	82
	good	32	58	72	79

Runoff Curve Numbers for Hydrologic Soil Cover Complexes (AMC II, I = 0.2S) (SCS, 1983)				
Land Use Description	**Curve Numbers for Hydrologic Soil Group**			
	A	**B**	**C**	**D**
(Table taken from the DEPARTMENT Road Drainage Manual) 1 For land uses with pervious areas, curve numbers are computed assuming that 100% of runoff from previous areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98. 2 Use for the design of temporary measures during grading and construction. 3 For contoured tillage poor hydrologic conditions, 5 to 20 percent of the surface is covered with residue. For contoured tillage good hydrologic condition, more than 20 percent of the surface is covered with residue. 4 Close-drilled or broadcast. 5 Poor hydrologic condition has less than 25 percent ground cover density. Fair hydrologic condition has between 25 and 50 percent ground cover density. Good hydrologic condition has more than 50 percent ground cover density. 6 Poor hydrologic condition has less than 30 percent ground cover density. Fair hydrologic condition has between 30 and 70 percent ground cover density. Good hydrologic condition has more than 70 percent ground cover density.				

Table 3-11 - Curve numbers for hydrological soil groups and cover complexes

If modelling an urban catchment, and the appropriate Average Percent Impervious area is not listed in Table 3-11, or the pervious areas are not best described as “lawns in good condition”, then an alternative approach should be used:

i. Estimate the curve number for the pervious area
ii. Estimate the curve number for the impervious area
iii. Estimate the runoff from each separately then summate for downstream runoff.

3.4.2. Flow calculation

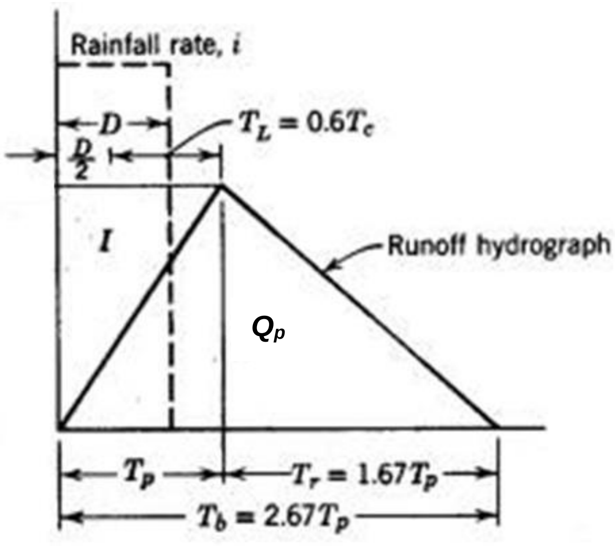


Figure 3-10: Rainfall and runoff with assumptions for the Soil Conservation Service triangular hydrograph method of rainfall estimation

The runoff calculated as described in the previous section should be routed to the outlet using the standard SCS unit hydrograph method (e.g., Figure 3-10).

Figure 3-10 above shows a dimensionless triangular unit hydrograph. To compute the peak discharge, the following equation is utilized:

$$Q_p = \frac{0.0021QA}{T_p}$$

Where

$$T_p = \frac{D}{2} + 0.6T_c$$

$$D = \frac{T_c}{7.5}$$

A is area (ha)

T_p is time to peak (hours)

T_c is time of concentration (hours)

D is duration (hours)

Q is the total runoff (mm)

Q_p is peak runoff (m^3/s)

There are a number of options for calculation of the outflow hydrograph once the curve number/runoff coefficient, unit hydrograph time to peak and storm profile have been determined:

- Hydraulic modelling software (hydrodynamic method if possible) is the preferred method of calculations. In hydraulic modelling software, the lag time T_L should be set at 0.6 Tc, the recession time to 1.67 x Tp and the default SCS peak rate factor should be used, with no base flow. Figure 3-10 above shows the relationships between all the parameters that the modelling software could prompt for. The rainfall profile should use a hyetograph for distribution.
- For small catchments (A<80 ha and t_c<30 min) an Excel spreadsheet system can be used (based on the rational equation)

Where the system contains the following, hydraulic modelling is to be used:

- Retention / detention tanks with significant volumes are used
- Retention in the network needs to be considered
- Backwater condition (e.g. from tide cycle) needs to be considered
- Flow restricting options need to be applied
- Incoming external flows
- Discharging to a surcharged storm system

Whichever method is used, the total runoff must be checked against a manual calculation to verify the magnitude of the runoff is correct.

The hydraulic model shall be compatible with CIVILSTORM (for ADM); and CIVILSTORM / SewerGEMS, and INFOWORKS (for AACM), or shape file exports are to be submitted to DEPARTMENT for input by DEPARTMENT into their hydraulic model to verify the results and extend the overall Abu Dhabi hydraulic models. Consultant shall integrate the project model within existing model if requested by DEPARTMENT (Existing hydraulic model shall be provided by DEPARTMENT upon submission of official request). Section 8 details DEPARTMENT's requirements for the submission of model files.

3.5. Special considerations for mixed catchments

The method described above assumes that the catchment behaves as a reasonably homogeneous hydrological unit. In this case, a single representative time of concentration and coefficient of runoff/curve number can be derived and will give reliable results.

Where one part of a catchment could dominate runoff, this may need to be treated independently. A typical example is where an urban sub-catchment, with high runoff coefficient and short time of concentration, dominates storm water runoff, and the rest of the catchment does not contribute significantly to runoff.

Where a catchment includes areas of very different hydrological characteristics, the analysis should be performed for the sub area or areas which may dominate the hydrological response. Where the flow from this sub catchment is greater than the flow predicted from the catchment when considered as a whole, the higher value from the sub-catchment should be used. Where the remainder of the catchment could contribute significant addition flow, this should also be modelled separately and the flows added together to give the total flow estimate. Section 8details DEPARTMENT's requirements for the submission of model files.

3.6. Flood volume calculations

Some applications will require calculation of flood volumes, as well as peak flows. Examples include the design of retention basins and lagoons, sub-surface storm water storage tanks etc. This requires an estimate of the volume of storage required to reduce the outflow downstream of the storage device to a manageable flow rate. Two methods are recommended: the Rational Method for flood volumes and the hydrodynamic / SCS method. The design standard should be as for flow calculations (see Section 3.2.1); no time of concentration multipliers should be applied.

The Rational Method for flood volumes should be used only for initial sizing of retention basins etc, for catchments where the rational method is valid for estimation of peak flows. The rational method for flood volumes estimates peak flow in exactly the same way as the rational method, but in addition assumes the catchment outflow hydrograph (i.e. the flow entering the storage structure) is trapezoidal and symmetrical in form (see Figure 3-11). The rise/fall time of the outflow hydrograph is taken as the catchment time of concentration. The procedure is illustrated in Figure 3-11.

For a trial storm duration (not necessarily equal to the time of concentration) and the design return period, the rational method is used to estimate peak flow, and the hydrograph determined. In the example below, the area between the hydrograph and the line representing the maximum design outflow (i.e., the design flow capacity of the storm water system downstream of the storage structure) is the retention volume required to store enough water to attenuate the hydrograph sufficiently. The calculation should be repeated for a range of storm durations and the maximum volume used as the retention volume.

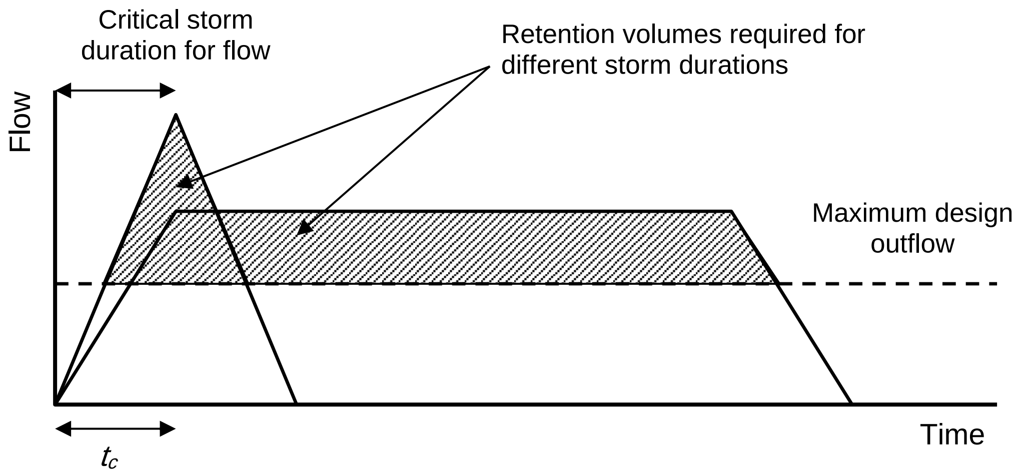


Figure 3-11 - Use of rational method for flood volumes for sizing of retention basins etc

For detailed design, and for catchments where the rational method is not appropriate, the SCS method should be used to estimate storage. The SCS method produces flood hydrographs as part of the calculation, and these should be used in a process similar to that for the rational method for flood volumes:

- For initial sizing, the retention volume represented as the area between the hydrograph and maximum design outflow should be calculated as for the rational method for flood volumes, but using the SCS hydrograph output (see Figure 3-12). This should be repeated for a range of storm durations, and the maximum volume required used as the design retention volume.
- For detailed design, the hydraulics of the retention structure, outflow control weirs etc should be represented in a modelling software package and the peak outflow and retention volume calculated for a range of storm durations for the design return period. Each potential set of design characteristics (retention volume, outflow weir design etc.) should be tested to verify that the storage volume is not exceeded and that the outflow from the storage structure is below the acceptable maximum.

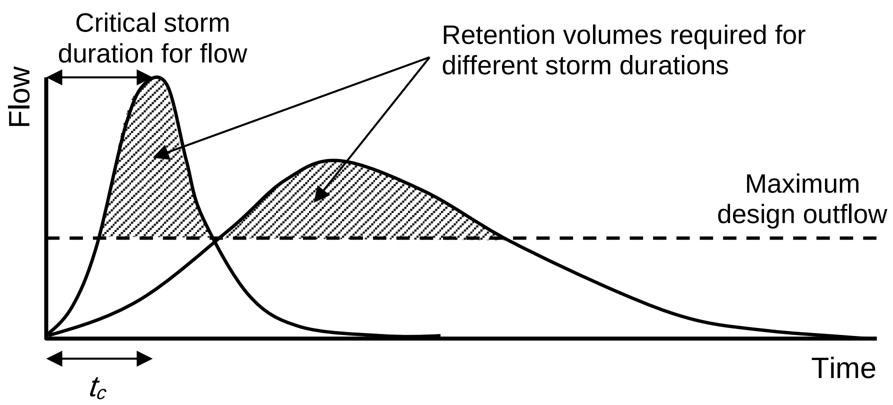


Figure 3-12 - Use of SCS method for sizing of retention basins

When designing a pond or lagoon for areas that are not connected to a stormwater network with a final outfall to sea (no downstream network), the minimum retention volume of the pond or lagoon will be represented by the entire area under the graph.

4. STORM WATER SYSTEM DESIGN

4.1. System Planning

Storm water drainage in municipal areas has the primary purpose of receiving and disposing of rainfall run-off in order to reduce the risk of surface water flooding. It also serves as a collection network for sub-soil drainage systems.

Storm water drainage systems in new development areas shall discharge to the existing storm water drainage system, to evaporation lagoons or direct to the sea. Uncontaminated rainfall run-off must only connect to the storm water system, and domestic sewage flow must only connect to the foul sewer system.

The design of a new storm water system shall take account of the hydraulic constraints imposed by the receiving network or the point of outfall.

DEPARTMENT will identify the constraints on the design of new connections to its storm water system. These will take the form of permitted points of connection, the maximum water levels in the existing system and the maximum permitted discharge rates from the new development.

For networks serving areas up to 80ha and / or ‘times of concentration’ up to 30 minutes the “Rational Method” shall be used for system design, supported by the use of proprietary network design software as appropriate. For larger catchments or where the ‘time of concentration’ is greater than 30 minutes the SCS method shall be used, again supported by proprietary network design software as appropriate. Descriptions of the Rational and SCS methods are given in Section 3.

Models must be submitted with the draft and final preliminary and detailed design reports. Section 8details DEPARTMENT’s requirements for the Database File (DBF) and Shapefile (SHP) data files exported from the modelling software.

4.1.1. Commencing Design

The design of a new network will progress in stages. Initially the pipeline gradient available for the longest branch should be estimated and initial inputs of flow provided along its length. If the pipe gradients available for gravity flow do not give the required minimum cleansing velocities given in Section 4.3.2 then they shall be steepened and intermediate pumping stations introduced where necessary, and will be subject to DEPARTMENT approval.

Once the profile of the longest branch has been established the initial design of the other branches can proceed.

Greater refinement is then progressively achieved by inputting the flows in more detail and by considering flooding constraints. The final sizes of pipes and their gradients may result from either flow velocity or flooding constraints. A minimum pipe diameter of 300mm is recommended for surface water pipes.

Service reservations are allocated by DPM and finally approved by DEPARTMENT, and details of these shall be obtained at the earliest time in the design process. The location of existing and planned utilities shall be obtained from the relevant utility provider.

Service reservation widths shall comply with the current requirements of the DPM’s Utility Corridor Design Manual (UCDM).

4.1.2. System Performance and Recommended Design Return Period

Refer to Section 3 for rainfall characteristics and run-off parameters.

The storm water system hydraulic performance shall relate to the importance of the catchment area and the consequences of flooding. Areas shall be classified into the groups in accordance with Table 3-2(Design storm standards for various facilities) to establish the level of performance required.

It may be necessary to test several storm types to arrive at the worst-case conditions to take account of the various types of contributing surfaces and area priorities. Large uncontrolled outgoing external flows are discouraged; these however may need to be estimated and their impact be evaluated.

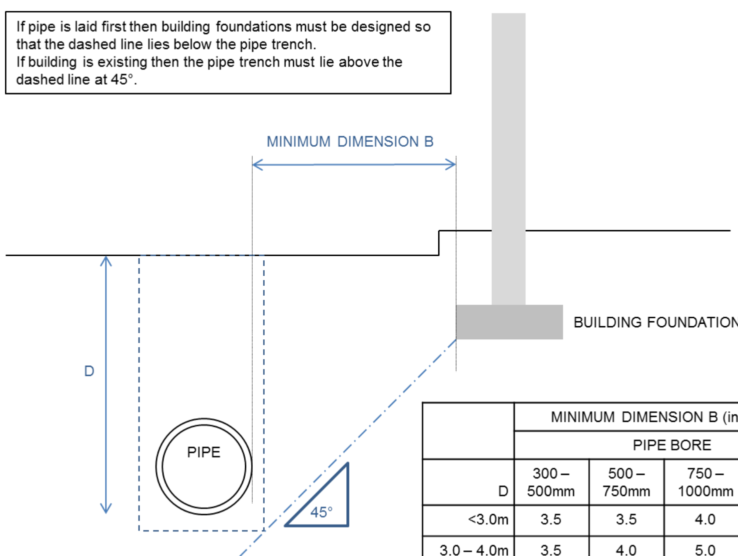
Lagoons (detention ponds) that do not have an outlet and where overtopping may lead to flooding of properties shall be designed to hold the run-off from a 1 in 25-year storm. Lagoons that have an outlet or overland flow track to sea that does not cause disruption to properties or road users, the lagoon is to be sized for the next higher event in Table 3-2(e.g., a lagoon serving an area which is generally a 1 in 5 year catchment type shall be sized for a 1 in 10 years storm before an overflow occurs).

4.1.3. Surface Flooding

No surface flooding will be allowed unless agreed with DEPARTMENT, along with the necessary checks at the appropriate design storms and return periods.

4.1.4. Pipes Located Near to Structures and Buildings

Pipelines shall be designed with a clear width on each side shown Figure 4-1 to allow for future maintenance/repair access and to avoid additional forces from structures and buildings.



	MINIMUM DIMENSION B (in metres)			
	PIPE BORE			
D	300 – 500mm	500 – 750mm	750 – 1000mm	>1000mm
<3.0m	3.5	3.5	4.0	5.0
3.0 – 4.0m	3.5	4.0	5.0	5.0
4.0 – 5.0m	4.0	5.0	5.0	6.0
5.0 – 6.0m	5.0	6.0	6.5	6.5
>6.0m	6.0	6.0	6.5	6.5

Figure 4-1 - Pipe Position Relative to Buildings

In sites where the clear width (B) needs to be less than the value given in Figure 4-1, a geotechnical investigation must be submitted and approved by the concerned authority.

Pipelines shall be positioned to give a minimum clear gap of 500mm in plan to any existing utility and shall comply with DEPARTMENT’s service corridor dimensions.

4.2. Flow Attenuation

Flow attenuation shall be provided to lower peak flow rate to produce more economic designs and to meet hydraulic constraints in the existing networks.

Attenuation may also be provided where a reduction in peak flow results in a reduction in overall costs. An example might be when attenuation results in a smaller pumping station when pumping is required. In all cases, Net Present Value (NPV) calculations shall be carried out to show that such a proposal would be cost-effective.

Attenuation shall be provided by either:

A. on-line storages (e.g., ponds or oversized pipes) with a flow control device at their downstream end to limit the pass-forward flow and to mobilise the storage,

B. off-line storages (e.g., concrete box)

   B.1 off-line storages with a small outlet flow control device and high-level overflow back to the main storm water line. A flushing system should be provided such as tipping buckets or high-pressure jetting.

   B.2 A geo-cellular storm water management system

- *Online storage is the preferred method of attenuation as it reduces the maintenance requirements.*

C. Lagoon or concrete boxes (detention pond) where levels and location permit. Forebay or other pre-treatment devices are recommended to be used with ponds to economise O&M costs. Open lagoons shall not be permitted within or close to developed areas. Permission to lay pipes and construct lagoons and other structures outside the developer’s area shall be obtained from DEPARTMENT and affected landowners.

Lagoons and off-line storages shall be sized to have a volume 5% larger than the required attenuation volume, to account for sedimentation of sand within the structure.

It may also be possible to provide flow attenuation by allowing some selected surface flooding, particularly during more severe storms, although the requirements in Section 4.1.3 shall be observed.

Where practical and cost-effective, flow attenuation measures shall be provided as close as possible to the source of flow.

Lagoons may be provided as a means of temporary flood storage (such as in recreation or playground areas), shaped and landscaped to be integrated with the surroundings. They shall be water-tight and designed to hold the volume derived in accordance with Section 4.1.2. The outflow may be by gravity return or pumping back to the storm water network. Lagoons shall start to empty automatically immediately after the storm event.

4.2.1. Geo-cellular stormwater management system

The system main components shall include but not limited to; geo-cellular blocks, connectors, manifold, outlet flow control devices, geo-textiles, geo-membranes and silt trap or sediment removal separator.

The anticipated total run-off volume from the site is estimated, and the allowable discharge rate from the site to an appropriate outfall is determined according to this manual. The volume to be stored underground in the system is then determined based on the site condition, and the number of units needed to contain these volumes calculated. Cleanouts and emptying arrangement as well as emergency escape to be provided. Refer to CIRIA C737 for further design guidance.

4.2.1.1. Connections

Connection is made to the units using a pre-formed socket and adaptor or flange adaptor. It is recommended that all connections into storage applications (using a geo-membrane) are made using a flange adaptor. Adhesive or double-sided tape should be used between the geo-membrane and flange adaptor to ensure a watertight seal.

4.2.1.2. Manifold Design

The units shall be manufactured to allow a connection to be formed by inserting pipes (to BS EN 1401-1: 2009) into the convenient knock-out incorporated in each cell. The flow may be split amongst a number of pipes connected to a manifold, to provide increased hydraulic capacity. The system designer should ensure the pipework connecting the units to the drainage system has sufficient capacity to cope with the design flow.

4.2.1.3. Outflow positioning and head calculations

The invert level of the outflow pipe should be flush with the bottom of the lowest unit to allow the system to drain. As the system fills, a depth of water develops on the upstream side of the outflow control device creating a driving head to push the flow through the control device. For design purposes, the head used in calculations is taken as that at the invert line of the outflow device.

4.3. Design Parameters for Gravity Pipelines

The required system design parameters are given below. Deviation from these is not permitted without the agreement of DEPARTMENT.

For Projects in Al Ain Region, connection details with the main pipeline and gullies details provided in Al Ain Municipality standard drawings and specification to be followed.

4.3.1. Pipe Hydraulic Formulae

The recommended hydraulic design formula for pipes is the 'Colebrook-White' formula, as this gives the greatest accuracy.

The Colebrook-White equation for full bore flow in circular pipes is given by:

$$V = -2\sqrt{2gdS}\log\left(\frac{k}{3.7d} + \frac{2.51\nu}{d\sqrt{2gdS}}\right)$$

Where:

V = velocity (m/s)

g = gravitational acceleration = 9.81 m/s^2

D = pipe bore (m)

S = hydraulic gradient (water surface slope where free surface) (m/m)

K_S = effective roughness (m)

ν= kinematic viscosity of fluid. Water = 0.727 x 10 $^{-6}$ m^2 s^{-1} (at 35^{o}C)

Where pipes are flowing part-full the following factors apply**:**

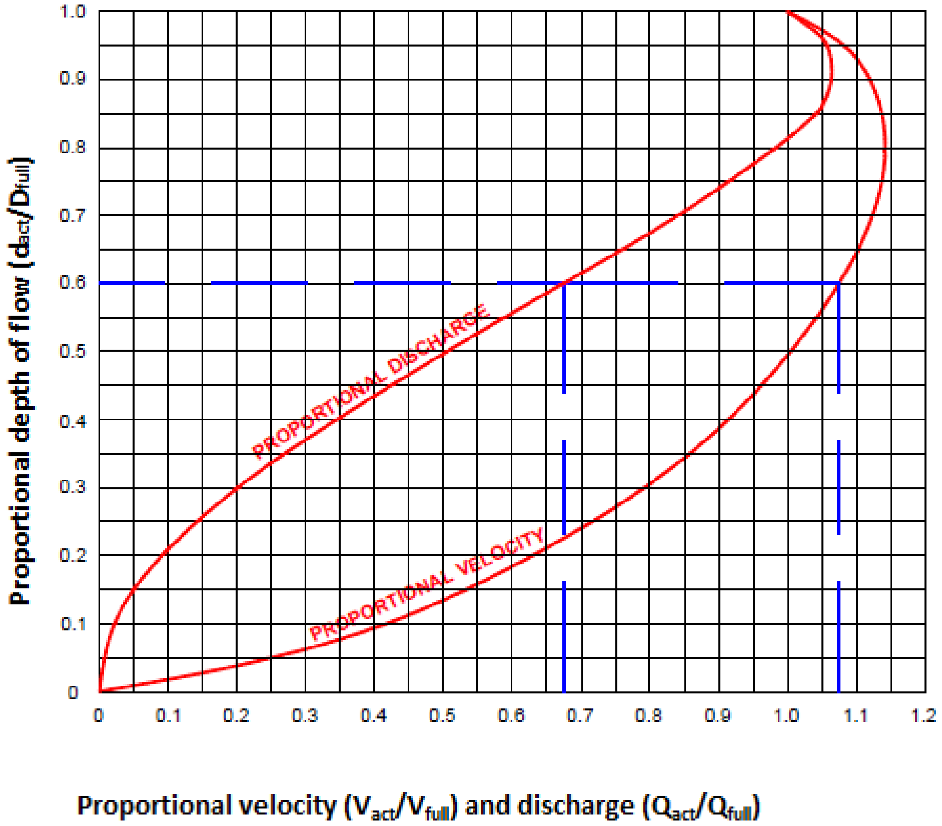


Figure 4-2 - Relative velocity and discharge in a circular pipe for any depth of flow Alternatively the Consultant may use the Manning formula

The Manning formula is normally used where there is ‘free surface’ flow and is given by:

$$V = 1/n\ (R^{2/3})\ (S^{1/2})$$

Where:

V = velocity (m/s)

n = Manning coefficient

R = hydraulic radius (area of flow ÷ wetted perimeter (A/P))

S = Pipeline gradient

The roughness values to be used for storm water design are tabled below:

Pipe Material	Colebrook-White, K (mm)			Manning’s Coefficient (n)
	Good	Normal	Poor	
uPVC Pipes	0.3	0.6	1.5	0.011
GRP Pipes	0.3	0.6	1.5	0.012
HDPE Pipes (plain and corrugated)	0.3	0.6	1.5	0.012
Lined Concrete Pipes	0.3	0.6	1.5	0.011
Un-lined Concrete Pipes*	*0.3	*0.6	*1.5	*0.013

Table 4-1 - Roughness Coefficients

*For Al Ain region only

Surcharge conditions of the flow are encouraged for all pipe systems, where applicable, if a clearance of 0.3 m (for the 1 in 5 – year storm event) is provided from manhole covers.

4.3.2. Pipe Hydraulic Design

To provide a self-cleansing regime within gravity storm waterpipes, the minimum velocity shall be above 0.75 m/s at peak flow. Less velocity may be accepted only at the early upstream of the storm water network after discussions with DEPARTMENT Engineers.

The maximum flow velocity shall not exceed 2.5m/sto minimiseenergy dissipation problems. Where system flows would result in higher velocities then the use of backdrop manholes for energy dissipation is generally not recommended. Specialist advice should be obtained if this occurs. Hydraulically critical flow and/ or acute bends are to be avoided as much as possible.

Where a scheme is to be developed in phases over several years, calculations shall be carried out to show the flow velocities at the end of each phase. Depending on the network and the nature of the development it may be necessary to increase the pipe gradients to give the required velocities from the lower flows during the early years of operation. This shall be by agreement with DEPARTMENT.

4.3.3. Minor Friction Losses

Calculations of the hydraulic grade line (HGL) for typical storm water systems usually ignore minor pipe losses. This can cause underestimation of flooding potential some cases. These include losses that occur in manholes (a summation of entrance, exit and bend losses), as well as losses that occur in pipes (at pipe to pipe junctions, bends, contractions, enlargement and transitions).

These minor losses, when cumulated, can cause substantial differences in the estimated HGL for the storm drain. In some cases (such as when profile slopes are minimal, and velocities are high, bends are acute, Mhdrops are large, etc), design calculations need to allow for the additional head losses that will occur in manholes, chambers and pipes. This is necessary to avoid restricting the capacity of the drain system by not accounting for them in design.

When using proprietary network design software these losses can be automatically calculated. For details on how to calculate minor losses manually, refer to the relevant literature.

4.3.4. Pipe Materials

The following gravity pipes materials are acceptable to DEPARTMENT:

Material	Relevant Standard for Pipe Manufacture	Comments
Concrete Pipes with Lining. Insitu concrete culverts with lining	BS5911, BS EN1916	Has the benefit of being available in rectangular culvert form
Unlined concrete pipe (For Al Ain city only)	ASTM C76M	
GRP	BS 5480	Cost-effective design possible due to choose of stiffness and optimisation of pipe strength/bedding. Minimum stiffness for GRP pipes is 5,000 N/m^2.
GRP (For Al Ain City Only)	BS EN 14364-2006 Al:2008 or ASTM 2996 or ASTM 3262	
uPVC	BS EN 1452	
uPVC (For Al Ain City Only)	BS EN 1452 or BS EN ISO 1452 part 1 to part 5 2009 or ISO 4422 or DI N 8061/62/63	Any stormwater pipe under the road shall comply with DIN 8061/8062 or ISO 4422 (2.5)
Structured Wall / Corrugated HDPE Pipes (SWPP)	ISO 4427	Cost-effective in larger sizes and for constructing underground storage tanks. HDPE corrugated or twin walled pipes commonly used elsewhere.

Table 4-2 - Approved Pipe Materials for Gravity Pipelines

As all these materials have acceptable properties, subject to correct design and allowable deflection and ovalization as per specifications, the choice will be determined by bedding and construction cost and size availability. Other pipe materials may be considered, subject to approval of DEPARTMENT. *All Materials bearing the AD QCC Trust Mark are preferable and recommended.* Consultants are referred to the schedule of standard reference documents in Section 9for more detailed information on the properties and applicability of different pipe materials.

4.3.5. Pipe Depth

The minimum cover to pipes without additional protection shall be 1200mm. In special conditions structural calculations in line with Appendix A2 may show that protection is required at greater depth for particular loading cases.

Where it is not cost-effective to have the head of the system starting with 1200mm cover then additional protection shall be provided to the pipes. Additional protection will usually take the form of concrete bed and surround or protection slabs.

When designing the pipeline, the corresponding cover depths, the pipe material and bedding, the Consultant shall consider if and by how much existing ground levels and imposed surface loads may change as a result of:

a) temporary conditions during construction
b) re-profiling of the ground as part of the site development
c) possible future changes to ground levels

Safety Note – Trenchless Solutions

Where the trench depth is greater than 10m or where difficult ground conditions exist or there is an obstacle to be passed, the Consultant shall consider whether a trenchless construction technique would be safer than open cut trenching. See also Section 4.10.

Safety Note – Pipe Flotation

During construction it may be possible for the pipe trench to contain fluid concrete or standing water. If the pipes are sealed and empty and have little or no backfill they can become buoyant with a risk of injury to personnel and damage to the pipeline and adjoining works. A similar risk is present after backfilling if the surrounding material can become fluid (as quicksand). If this can occur the buoyancy forces are greater due to the higher density of the material.

The Consultant shall carry out calculations to demonstrate that there is a Factor of Safety of at least 1.1 against pipe flotation under all practical circumstances. If the pipe self-weight is not sufficient then additional weight shall be provided by means of concrete saddles, or similar.

4.4. Inlet and Catch basins Structures

Surface water run-off areas will accumulate windblown sand and measures shall be taken to prevent or significantly reduce the amount of sand getting into the storm water network. Surface water collection inlets shall include a sump to trap sand as shown on the Standard Drawings. Each inlet shall be connected to a rider pipe of maximum length 50m which at its end shall connect to a catch basin. Each catch basin shall connect to a manhole by as short a pipe as possible. Catch basins shall be specified at junctions of two or more lateral pipes, and at the upstream of manholes.

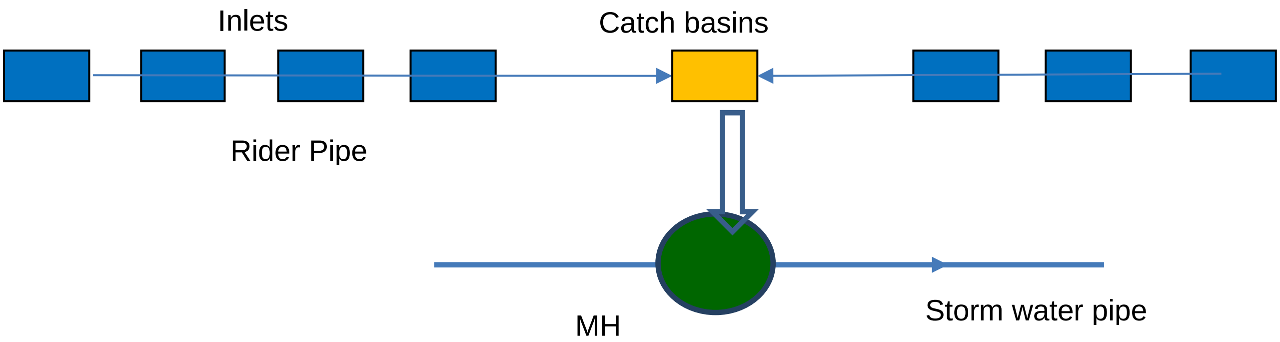


Figure 4-3 - Typical Inlet and Catch basin Layout

In addition to inlets for highway drainage the Consultant shall ensure that all other areas draining to the network have inlet arrangements that trap sand and connect to the network by pipes that are as short as possible.

Recommended spacing between inlets should be calculated as stated in Appendix A3, inlets/ gullies can generally be spaced between 20-50m intervals, as confirmed by the water spread calculations. At the pronounced low spots in a sagging road profile in urban areas, the system is to be designed for a single inlet; however, a double inlet is to be constructed to ensure that the system will operate in the event of the blockage of an inlet.

The methodology for the calculation of inlet spacing, spread and the hydraulic capacity of inlets is given in Appendix A3.

Additional risks from windblown sand and other surface material will occur during construction as a result of the ground being excavated or material stockpiled. Facilities shall be provided which minimise or prevent such material reaching the storm water network.

Inlets and catch basins shall be used where pipeline sizes are up to 500mm diameter. Depth to invert of inlets and catch basins shall not exceed 2.8m. Refer to below table and Standard Drawings Volume 2 for details.

For projects in Al Ain, refer to volume 2:N-series (: Al Ain Region) for Standard drawings : and details to be used.

Outlet pipe diameter up to (mm)	Maximum depth (D) to outlet pipe invert level (m)	Inlets internal dimensions mm	Catchbasins internal dimensions mm
300	D=<2.0	600 x 600	600 x 1600
500	2.0>D=<2.8	1000 x 1000	1000 x 1600

Table 4-3 - Use of Inlets and Catch basins

On special cases (such as areas subject to minimal or no sand storms), and depending on the project limitations, inlets could be arranged to discharge directly to the storm water manhole structure without catch basin considering Abu Dhabi Standard dimensions for inlets and Catch basins.

4.4.1. Surface Erosion Control

In addition to windblown sand there is also the possibility that surface material will be washed into inlets if overland flow occurs during times of storm.

The design and layout of the development shall incorporate features which prevent overland flow draining to inlets unless an intermediate settlement facility is provided, for example a shallow depression in the finished ground level. Such measures shall also be included during the construction period when the natural ground is disturbed or a system proposed by the contractor and agreed by the engineer for reducing the cost and increasing the effectivity of the whole installation.

4.4.2. Manhole Positioning and Access

The design of manholes shall conform to Volume 2: Standard Drawings.

Manholes shall normally be built at:

- every change of alignment or gradient

- the head of each branch of the network
- every pipe junction
- wherever there is a change of pipe size, the connection shall be 'soffit to soffit'.
- intervals not exceeding 100m for pipes ≤ 300mm diameter (perforated lines)
- intervals not exceeding 120m for pipes 300mm to 600mm diameter.
- intervals not exceeding 160m for pipes 700mm & 800mm diameter
- intervals not exceeding 200m for pipes from900mm to 1100mm diameter
- for pipe diameters ≥ 1200mm interval may reach 300m, subject to DEPARTMENT approval
- at any additional locations designated by DEPARTMENT

The above mentioned spacing values are based on typical installation conditions and shall only be taken as indicative. The Consultant shall undertake design calculations and site investigation to determine manholes spaces to suit the site conditions / layout.

Manholes shall be located to provide safe access for maintenance but also to minimise disruption to the general public and property owners during maintenance. The positioning of manholes should consider the size of the excavation required to construct the chamber and the impact this may have on adjacent utilities.

Particular features to note are:

- All manholes shall be provided with access ladders.
- Manhole covers shall be sized to allow for access for maintenance equipment, man-entry with breathing equipment and removal of a casualty by stretcher. In particular, means for direct access from the surface shall be provided for all installations requiring regular maintenance, such as penstocks.
- Blanked-off stub pipes shall be incorporated in manholes where necessary to allow system extension and future connections

Manhole cover levels shall conform to the cover levels in Table 4-4

Location	Cover Level
Paved areas	final paved level.
Landscaped areas	final ground level +0.1m
Open, unpaved areas	final ground level +0.25m.

Table 4-4 - Manhole Cover Levels

In exceptional cases and where agreed with DEPARTMENT the manhole may have a recessed cover, in which case the location shall be precisely identified in relation to adjoining permanent features and shown on the record drawings.

The spacing of shafts for trenchless construction shall be as agreed with DEPARTMENT.

Safety Note – Manholes

✓ Pipes of 900mm diameter or greater shall be fitted with stainless steel safety chains at the manhole upstream openings.

✓ In the layout and location of manholes, particular attention shall be paid to providing safe conditions for inspection and maintenance work and for dealing with emergencies and man-rescue.

✓ Where covers with a lift weight greater than 25kg are used then portable lifting mechanism for ease of lifting and no damages to the coating of covers shall be used.

- ✓ Where covers are in public areas or where the depth of the manhole or chamber is greater than 2m, provision shall be included for locking the covers. Provision for locking the covers should be considered for both vandalism and road safety, and that should in agreement with DEPARTMENT.

4.5. Contaminated Surface Run-off

Where there is the potential for surface water run-off to be contaminated then appropriate measures shall be included to prevent contamination reaching the network and the receiving body of water. Areas that might produce such run-off are car parks, oil station forecourts and leaking storage tanks, landfill sites, airports and overland run-off from industrial areas.

For car parks and oil station forecourts the risk arises from run-off being contaminated with hydrocarbons and oil interceptors shall be provided.

Interceptors shall be located to give good access for the removal of contaminated material. Interceptors shall be installed in areas with good natural ventilation and shall not be installed in enclosed areas.

Interceptors shall be of a multi-compartment design as shown in Volume 2: Standard Drawings or a proprietary multi-compartment design. They shall include the following features:

- ventilation
- Secure, non-flammable covers
- Uniform flow through the separation compartment
- The oil interceptor compartment shall be incorporated by a Hydrocarbon layer thickness sensor to detect when the maximum thickness is reached.
- The sludge trap compartment shall be incorporated with a Sludge layer sensor to alarm when maintenance is necessary. The sensor should be placed such that the end of the probe is 2 cm under the alarming level.

The capacity of oil interceptors shall be based on the following:

- Size of separator shall be based on double the maximum contaminated water flow.
- For light liquids the retention time shall be a minimum of 3 minutes up to a design flow of 20 l/s. For higher flows an additional minute shall be added per 10 l/s increase.
- The width to length ratio shall be 1:1.8
- Alternatively, in case of mechanical oil separators, the manufacturer’s recommended flow rates with a factor of safety of 2.

Domestic wastewater shall not be connected to an interceptor.

Where a pumping station is required in the vicinity it shall be located downstream of an interceptor.

Areas that regularly have contaminated surfaces or regularly produce contaminated run-off such as vehicle washing plants shall have site specific measures installed to limit the contamination entering the storm water network (e.g. oil separators shall be installed prior connection of the drainage lines from contaminated areas to the DEPARTMENT main stormwater network).

4.6. Hydrodynamic Separators

In addition to the sumps in inlets, catch basins and manholes, consideration should be given to the use of hydrodynamic separators to remove settleable solids and floating debris, particularly in networks that have pumped discharges or discharge to public amenity areas; or ponds and soakaways.

These are normally proprietary designs and are most effective where the materials to be removed from run-off are heavy particles, such as sand and gravel, which can be settled or floatable which can be captured rather than solids with poor settleability or dissolved pollutants. These can also be located in potential “hotspots” where there is a high likelihood of high sediment loads or debris entering the network.

The separators should be located in areas with safe access for the operators to “dip” the central shaft to measure the depth of accumulated sediment and safe off-road parking for the suction tanker for emptying.

The separators can also function as oil, grease and oil interceptors and this use should be discussed with the Manufacturer.

4.7. Crossing Culverts

The design of culverts under highways and other crossings, generally in rural areas, shall comply with the following:

- All culverts shall be designed to have sufficient hydraulic capacity to convey the design flood and to allow for greater floods without damage or flooding of adjacent structures or flooding of the highway. Refer to Section 3.2 for design storm return periods and durations for different kinds of culverts. See also Section 4.1 for stormwater system planning.
- Pipe and culvert foundations shall be designed to withstand the scour conditions arising from a 1 in 100-year frequency flood event and checked for 1:500-year storm when required by DEPARTMENT. Appropriate scour protection is needed using locally appropriate and well recognised methods, such as described in HEC-11, HEC-14, HEC18, HEC-23 and CIRIA C742, or equivalent.
- Maximum surcharging of 20% of depth may be permitted if agreed with DEPARTMENT/ AACM.

The culverts shall consist of pipes or rectangular reinforced concrete cells with a reinforced concrete or gabion headwall and discharge structure. Energy dissipation measures (e.g., refer HEC-14) on the discharge apron may be required. The culvert shall consist of a minimum of two pipes or concrete cells as they carry a high risk of blockage during a storm. Inlet bar screens shall not be provided unless there is a specific site safety or security requirement.

4.8. Tidal Discharge

Where the discharge from the stormwater system could be adversely affected by tide levels, the system design shall be carried out such that no flooding occurs at the return period stipulated in Table 3-2 at the time of Highest Astronomical Tide (HAT), and checked for a return period one category higher than the one used in the design. This will be modelled by setting the tail water at the outfall to the level of the HAT. Refer to Section 4.8.1 for details of tidal range and the HAT value recommended for use in design.

The design of outfalls is shown in Volume 2: Standard Drawings.

4.8.1. Tidal Range

Table 4-5 shows the existing, standard tidal data for Abu Dhabi relative to land datum.

Tide	Level to New Abu Dhabi Datum (m) (NADD)
Highest Astronomical Tide (HAT)	1.3
Mean Spring High Water (MHHW)	0.7

Tide	Level to New Abu Dhabi Datum (m) (NADD)
Mean Neap High Water (MLHW)	0.2
Mean Neap Low Water (MHLW)	-0.2
Mean Spring Low Water (MLLW)	-0.7
Lowest Astronomical Tide (LAT)	-1.1

Table 4-5 - Tide Levels

It should be noted that weather conditions and surge impact sea levels and that a maximum storm high tide of +1.3 m level to NADD has occurred. It is therefore recommended that 0.5m be added to the numbers in Table 4-5 to allow for long-term rising sea levels. In other words, the high tide value of +1.3 m level to NADD should be used for design.

4.8.2. Outfall Structures

The soffit of all outfall pipes shall lie below Mean Spring Low Water tide level.

The layout of outfall structures is shown in Volume 2: Standard Drawings.

A chamber shall be located on the pipe upstream of the outfall with cover level at a point above HAT (i.e., above 1.3m – Level to New Abu Dhabi Datum (m) NADD) and shall include non-return valve, such that the network or pumping station is protected by non-return valve against seawater ingress. Such valves shall be a double hinged flap valve or a flexible duck-bill or cone type or flap gate. The valves shall close under a minimum of head difference and shall prevent any backflow into the network. This chamber shall also include a trash screen with vertical bars at 100mm spacing. The chamber shall be located to give good access for maintenance purposes, including the removal of trash.

Outfall structure may also be provided with a manual or automatic trash screen for removal of debris. Trash bar screen shall be housed in a separate chamber

All materials that could come into contact with sea water or diluted sea water shall be specified for the appropriate corrosion resistance.

Safety Note – Outfall Safety Grill

Every outfall structure shall incorporate a safety grill to prevent unauthorised access into the outfall pipe. The grill shall be fabricated from 20mm diameter stainless steel bars and the maximum spacing of bars shall be 150mm. Refer to the Standard Drawings and Specifications for details.

For projects in Al Ain region, galvanized bars with the size 20mmx10mm or 15mmx10mm or 10mmx10mm to be used. Refer to the Al Ain Municipality Standard Drawings and Specifications for details.

4.9. Trenchless Technologies

The Consultant should be aware of the different technologies available for trenchless construction and the benefits these can bring to cost, safety and environmental impact. Such construction may be beneficial where:

- An open cut trench would be unacceptably deep for the particular location
- The ground conditions are unsuitable for open trenches
- A road, railway, powerlines or body of water crossing is required

The primary methods for trenchless construction are:

- Tunnel boring machine (TBM) with segmental rings for larger diameters.
- Pipe jacking - diameter 900mm to 2400mm and above
- Micro-tunnelling - diameter range 300mm to 900mm
- Auger boring – diameters up to 1000mm
- Directional drilling - diameter range 300mm to 1500mm

Note that significant areas are required for working shafts/pits at each end of a trenchless technique and possibly at intermediate points depending on length and conditions. The shafts/pits can be large in order to allow the insertion and removal of the tunnelling equipment.

Trenchless construction can provide safety, environmental and cost benefits where used appropriately. The successful use of trenchless construction relies heavily on:

- comprehensive knowledge of the ground conditions and buried obstacles from a detailed ground investigation
- the advice of an experienced geotechnical engineer
- the use of experienced, specialist contractors who can advise on the merits of the different techniques. It is important that construction advice is sought at an early stage in the design process

The assessment of preferred method shall consider:

- availability of contractor expertise
- construction safety
- ground conditions
- area of land required for the permanent works and construction working areas
- environmental impact, including impact on the public and property owners/occupiers
- construction cost
- construction duration

4.10. Hazardous Areas

Although a storm water network provides an inherently less dangerous environment than a foul sewer system, it is still important to consider the hazards that can arise and to address these in the design.

Hazard	***Possible Design Solution***
The accumulation of explosive or poisonous gases in the pipe network, chambers and pumping stations. Methane for example could arise from the decomposition of organic matter contained in run-off or by diffusion from the natural ground. Explosive gases are normally lighter than air and thus accumulate at high spots.	*Vent stacks to be provided.* *Avoid specifying materials that could generate sparks from static electricity.* *Check the geological reports for the presence of natural gases or organic material in the ground and identify this risk in the residual hazards document.*
The displacement of oxygen by other gases such that the air in the pipe network, chambers and pumping stations	*Ensure that there are no foul or contaminated area connections to the storm water network.*

Hazard	Possible Design Solution
becomes irrespirable for maintenance/repair workers.	*Check the geological reports for the presence of natural gases or organic material in the ground and identify this risk in the residual hazards document.*
The release of gases through disturbing pipe sediment during maintenance or repair activities	*Ensure that the minimum pipe gradients are provided to minimise sediment accumulation.*
The accidental discharge of toxic or explosive chemicals in a part of the catchment which then finds its way into the storm water network.	*Ensure that there are no foul or contaminated area connections to the storm water network.*
Deep manholes	*Provide intermediate landings or staircases* *Provide space for lift of unconscious person from base of manhole to ground level*

Table 4-6 - Potential Hazards in a Storm Water Network

In all cases careful consideration shall be given to the need for natural or forced ventilation of the network to remove any undesirable gases and before man-entry is made for maintenance purposes. In many instances residual hazards will remain and these shall be notified to DEPARTMENT through the Health and Safety records and on the as-constructed drawings.

4.11. Concrete Structures

All concrete structures shall be designed to comply with:

- DEPARTMENT publication “Building Regulations & Recommendations for Structural Design & Concrete Practices”
- User Guide for the International Building Codes in the Emirate of Abu Dhabi

4.12. Soakaways

Generally, the use of soakaways and soakaway trenches are not permitted in the design of new storm water systems. However, where the groundwater table is deep, the soil’s permeability is high, and the maintenance requirements are practical, infiltration systems can be considered as an alternative based on a study and on a case-by-case basis. Design criteria needs to be agreed with ADM. Refer to CIRIABRE365 or R156 for more design guidance. Pre-treatment devices upstream of soakaways are encouraged and reduces O&M costs.

In addition, the use of Geo-cellular units can be considered, the total areas of the base and sides of the blocks shall be designed to enable the rate of infiltration of the stored water through the geotextile soil interface to be developed.

Individual Soakaway systems shall be sized for infiltrating the runoff volume for paved/unpaved areas with a storage volume that accommodates the detention of runoff from design return period (i.e., 1:10-year, or other), 24-hour duration, stormwater flow to be infiltrated (volume emptied) within a 24-hour period after the end of the storm.

Generally, it is better to use multiple systems with smaller catchment areas to simplify the size and complexity of the Soakaway for both cost and maintenance purposes. It is also preferable, from a soil permeability and Soakaway interaction standpoint, to spread the individual infiltration areas apart, according to road structure, safety, and geotechnical requirements. Several types of soakaways, appropriate for use throughout the Emirate, are described below:

4.12.1. Soakaway vaults and chambers:

Structural-type chambers can be constructed of larger diameter pipe; precast perforated concrete ring units and rectangular section (Bricks), sometimes called an infiltration well; concrete vaults; and proprietary preformed chamber systems, such as plastic cells having a 90 percent void ratio to overall volume. In all cases, the structure will have a backfill surround of a uniformly graded aggregate wrapped in a Geotextile filter fabric.

Structure sizes are based on providing storage for the design storm runoff volume minus the volume lost due to infiltration during the storm period. Infiltration interface areas, or the outer areas of gravel backfill, must be sufficient to provide an infiltration flow that empties the storage volume within 24 hours after the storm event. Bottom surfaces are assumed to be blocked by sediment; thus, the infiltration interface surface is based on the sides only. Design can be based on the following procedure as taken from BRE365:

1. Preliminarily locate soakaways as to probable areas where suitably permeable soils exist with sufficient clearance of minimum 1.0 meter above the highest fluctuated groundwater table along period covers all range of seasons and tidal cycles in accordance to the geotechnical report. Further, define required runoff collection point locations, such as at low points in the cross sections and profiles, where sufficient utility and structure clearances exist to construct the Soakaway system. Where these conditions occur for larger areas of pavement, such as long lengths of roadway, establish trial Soakaway locations by providing a Soakaway for approximately each 500 m^2 of catchment area.
2. Perform field permeability testing for each Soakaway site. Where the soil types and groundwater conditions are consistent for large areas, testing may be reduced to two or three representative sites.
3. Determine the catchment area and calculate the volume of runoff for the design return period storm using either (i) the NRCS depth of runoff equation,(ii) the modified rational equation, or the Depth-Duration-Frequency (DDF) where available (refer to Equation A-4.1, and Equation 4.3, for additional information).
4. The effective storage depth is from the invert level of inlet connection to the Soakaway chamber bottom level.
5. Size the Soakaway such that the internal storage volume is the same as the design runoff volume, using the following formulas:
6.

$$V_{Soakaway} = V_R - V_I$$

Equation 4.1: Required Soakaway structure storage volume

$$\boldsymbol{V_{Soakaway} = V_{Structure} + vV_{Media}}$$

Equation 4.2: Total available Soakaway storage volume

- $V_{Soakaway}$ = total internal structure storage volume (m^3).
- V_R = volume of runoff for design storm (m^3).

- V_I = volume of infiltrated water during storm period (m^3). This can be calculated using the below formula:

$$VI = a_{50} * P * D$$

- $V_{Structure}$ = internal volume of the Soakaway storage structure, such as a concrete structure, pipe, or plastic box (m^3).
- V_{Media} = volume of the aggregate surround material (bedding and surround aggregate for the Soakaway structure) (m^3).
- v = void ratio of storage media percentage in decimal form. Determine void ratio, such as for aggregate used in Soakaway and infiltration trenches, by laboratory testing. The free volume in granular fill surrounding pipes in rectangular trench is based on percentage (30-40%) void space of the granular material.
- a_{50}= Internal surface area of Soakaway to 50% effective depth. This excludes the base area which is assumed to clog with fine particles and become ineffective in the long term

7. Check the total infiltration flow rate. Ensure that the conducting surface area of the infiltration structure in contact with the soil is sufficient to drain away the stored volume within a 24-hour period. Use the following formula to find the total infiltration flow rate:

$$Q1 = \frac{(Pa_{50})}{(24)}$$

Equation 4.3: Total infiltration flow rate

Where:

Q_1 = total infiltration flow rate for Soakaway (m^3/hour)

D = Storm Duration per hours

P = soil permeability (m/day) found from site testing based on average of at least three permeability tests performed at same excavated soil level as for the proposed Soakaway structure installation depth.

a_{50}= Internal surface area of Soakaway to 50% effective depth. This excludes the base area which is assumed to clog with fine particles and become ineffective in the long term.

D= Storm duration per hours.

Check time of emptying using the following formula:

$$T_D = \frac{V_{soakaway}}{Q_1}$$

Equation A-4.6: Formula for finding time for water to drain away

Where:

T_D = time to empty the Soakaway volume (hour)

If T_D is greater than 24 hours, reconfigure the Soakaway to reduce the catchment area, increase the size or adjust the internal configuration to increase the soil contact surface, then recalculate T_D.

8. Size of the Soakaway storage volume can usually be reduced by routing the runoff inflow and outflow rates to find the required maximum storage volume. This can be done using a simple spreadsheet procedure, as discussed in Volume

II, Section 3.4.1. This procedure cumulates the runoff volume calculated for short periods using the NRCS curve numbers and the design storm distributions (refer to Section 3.4.6 or additional information). Infiltrated outflow rate volumes for each period is subtracted from the cumulated volume. Required storage is that shown for the time period with the highest cumulated volume.

Infiltration rates can be improved in many cases by ripping hardpan, loosening the soil, or installing distributor pipes to infiltration trenches down in more pervious layers.

9. In some cases, to obtain the size of the soakaway, the dynamic simulation of the inflow hydrograph and the percolation rate maybe required.

4.12.2. Soakaways trenches:

Soakaway trenches are a modification of the Soakaway chamber. Soakaway trenches are placed on flat horizontal profiles consisting of trenches backfilled with permeable coarse aggregate, with a perforated longitudinal pipe, and wrapped in Geotextile filter fabric.

Perforated pipe improves the distribution of runoff flow along the Soakaway trench and helps to collect fine sediments before they enter the backfill and clog the infiltration interface surface. Perforated pipe can be located anywhere within the trench envelope, but shall have minimum bedding and cover thicknesses of at least 150 mm of aggregate material.

Lengths, widths, and depths of the trench are based on the storage volume required to detain the design storm frequency runoff volume. This design procedure is similar to that described for soakaways in Clause a, above. The effective depth of the trench Soakaway (H) is extended from the ground level to the bottom of the trench.

Storage volumes consist of the void space in the aggregate backfill and the perforated pipe. Aggregate void percentages must be determined by testing, although it typically ranges from 30 percent to 40 percent, depending on the size and gradation of the material.

Infiltration interface areas shall also be checked to ensure the total infiltration flow can sufficiently drain away the detained runoff within 24-hours after the storm.

Trench-type soakaways shall have at least two inspection and cleanout ports connected to the perforated pipe, one at each end of a straight trench, no more than 75 m apart. This consists of a catch pit, inlet, or manhole structure with a solid lid.

The free volume in granular fill surrounding pipes in a rectangular trench is based on a percentage of 30% void space of the granular fill material and can be calculated as in below formulas:

$$\textbf{Free Volume in granular fill} = \%_{free\ V} * \left(WH - \frac{\pi(OD)^2}{4}\right) * L$$

i. Where:

% free V = 30% void space of the granular fill material
W= Trench Width [m]
H = Trench Height [m]
0D = Outer pipe diameter [m]
L = Pipe length [m]

$$\textbf{Volume within pipes} = \left(\frac{\pi(ID)^2}{4}\right) * L$$

Where

ID = Inner pipe diameter [m]
L = pipe length [m]

Combining both equations, the Soakaway storage volume, S, is expressed as:

$$S = \left[\%_{free\,V} * \left(WH - \frac{\pi(OD)^2}{4}\right) + \frac{\pi(ID)^2}{4}\right] * L$$

4.13. Open Channels & Ditches

Constructed open channels and ditches in urban areas are not permitted in the design of new storm water systems within DEPARTMENT jurisdiction. However open channels and ditches are used for highways and within Al Ain City Municipality region, please refer to Appendix-A4 for further details.

4.14. Stormwater Ponds

Ponds can be classified as following:

Dry pond (detention pond) - A permanent pond that temporarily stores stormwater runoff to control the peak rate of discharge and also provide partial water quality treatment, primarily through the incorporation of extended detention. These ponds are normally dry between storm events.

Wet pond (retention pond) - A permanent pond that has a standing pool of water. These ponds can, through their normal storage of water, or in conjunction with extended detention, provide water quality treatment in addition to controlling peak rate of discharge. They can, also in conjunction with extended detention, provide protection of downstream areas from frequent storms.

The amenity value of dry ponds located in urban areas or areas close to residential developments is to be considered. In these areas consideration should be given to landscaping the area as park land and utilising the pond as a shallow open water feature, or wetland with reed beds or other plant life, to encourage wildlife. The increased depth following a storm should be lowered to the amenity level within one week. The presence of a mechanical aerator; such as a fountain in the middle of the lagoon, surface aerator, or submerged aerator (diffuser); deters the growth of unwanted vegetation and reduces the possibilities of mosquitoes using the lagoon as a breeding site.

Dry ponds in rural areas should be dry and facilities installed to empty the pond within one week installed unless DEPARTMENT approval is obtained to an evaporative lagoon.
Detailed modelling shall be carried out for the sizing of the ponds, in addition to environmental impact assessment. The design shall be approved by all concerned authorities prior to construction. For recreational ponds, water treatment (if required) shall be designed by the consultant based on project specifics and approved by the concerned authority.

4.14.1. Sizing of stormwater ponds

For the design of stormwater ponds, the following points shall be considered, and to be approved by the concerned authority:

a. Stormwater ponds must not be located within surface water bodies or any buffer zones required.
b. The elevation difference recommended at a site from the inflow to the outflow is 1.8 to 2.4 m, but lower heads will work at small sites.
c. Basin outlets may have energy dissipation, depending on the outlet speed and type of downstream soil type and material. It is Highly Recommended that a stilling basin or outlet protection be used to reduce flow velocities from the principal spillway to non-erosive velocities (1 to 1.5 meter per second).
d. Adequate maintenance access, typically with a minimum width of 5.0m, shall be provided, different values may be accepted based on site conditions and upon approval of the engineer.
e. Where a forebay is installed, direct vehicle/equipment access should be provided to the forebay for sediment removal and other maintenance activities. For ponds sites located far from the project, it may be more reasonable to reach the pond from a local road.
f. An emergency spillway must be provided to pass storms in excess of the pond hydraulic design. The spillway must be stabilized to prevent erosion and designed in accordance with applicable weir safety requirements.
g. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
h. If the spillway crosses the maintenance access, materials meeting the appropriate load requirements must be selected
i. Basin outlets must be designed to prevent discharge of floating debris
j. Inlet areas should be stabilized to ensure that non-erosive conditions exist during high-flow events
k. Public safety must be considered in every aspect of pond design
l. It is highly recommended that each pond have a sediment forebay or equivalent upstream pretreatment. The sediment forebay is a small pool, to be separated from the permanent pool by barriers such as earthen berms, concrete weirs, or gabion baskets, where initial settling of heavier particulates can occur.
m. It is highly recommended that flows from forebays enter the permanent pool area with non-erosive outlet conditions.
n. It is recommended that the forebay(s) be sized to contain 10 percent of the water quality volume.
o. It is recommended that a fixed vertical sediment depth marker be installed in the forebay to measure sediment deposition over time. The marker should be sturdy and placed deep enough into the bottom of the forebay so that ice movement does not affect its position.
p. In case of evaporative and/or percolation ponds, and due to the long period of emptying the pond, the issues of standing water and sedimentation in the pipe must be considered.Measurements may include one or all of the following: pipe invert must be at least 30 cm above the pond bottom level, regular maintenance of removing sediments from the pipe, and pumping water from the end manhole to the pond.

4.14.2. Ponds Water quality

The primary contaminant removal mechanism of all pond systems is settling or sedimentation. However, effectiveness may vary to some degree depending on the type of detention system (dry or wet).

One of the most important water quality parameters is the Dissolved Oxygen (DO) rate, which has to be kept above 5 ppm.

4.14.3. Dissolved Oxygen Measurements

Problems with DO levels can arise quickly and the response time for taking corrective measures can be very short. Consequently, aquaculturists need a rapid and reliable method of measuring DO concentrations to inform management actions.
The number of ponds or tanks to be measured, desired level of accuracy, and cost of the tool or technique are three of the most commonly considered factors.

One of two methods can be used based on the periodicity of the measurements.

A. The drop count method which is inexpensive, fairly rapid but is only appropriate if DO concentration is measured infrequently in a few ponds.
B. The dissolved oxygen meter is used where DO measurement of multiple ponds is routine.Where using the dissolved oxygen meter, proper calibration is essential to ensure accuracy, and must be appropriately maintained for reliable use.

4.14.4. Aeration Systems

Aeration is the process of adding oxygen to water. Maintaining the required concentration of oxygen dissolved in water, one of the most important, if not the most important indicator of the quality of water in the pond

Aeration efficiency is measured by “Standard Aeration Efficiency” or SAE. SAE is the value of the standard level of oxygen transfer divided by the power expended at the same time. Three different types are recommended, fountain aerator, surface aerator, or submerged aerator (diffuser). Details and specifications are given in volume 4.

4.14.4.1. Aeration System Design Considerations

1. **Size** – Aeration systems need to be capable of circulating the volume of water in the pond.

2. **Depth** –aeration systems feature diaphragm compressors that are energy-efficient but can wear prematurely if operated in depths greater than recommended. When selecting a system, make sure you know what your maximum depth is.

3. **Shape** –in case of multiple ponds or irregularly shaped ponds limit the circulation of oxygenated water can be limited. For ponds like this, aeration systems with multiple air stones or diffuser plates are preferred to ensure that your water is moving properly.

4. **Power source availability.**

5. **Aeration function**– the function of the aeration may vary between continuous aeration and emergency aeration (in some cases where accidents take place and aeration is required for a period of time). In all cases the aeration efficiency, the seasonal variability, power source availability, and fish harvesting (if any) must be considered.

4.14.4.2. Aeration System Placement

1. Locating aerators to take advantage of wind-driven currents in case of the long axis of a pond is oriented along the direction of prevailing winds.
2. Locating several aerators in a pond should be done according to site factors such as depth, direction of prevailing winds, proximity to electrical power sources, and accessibility for fuelling and maintaining the equipment.

3. Locating aerators to work together in producing current in case of using several aerators in a pond.
4. Typical PTO-powered aerators should be located in an area where the tractor can be safely positioned and there is suitable access to the pond.
5. PTO-powered units should be operated in designated areas within the pond where erosion-resistant materials have been incorporated into the pond bottom.
6. Floating aerators are often located in the middle of the pond.

The proper aeration system shall be chosen according to the site condition on a case-by-case basis. Mechanical, electrical, and instrumentation drawings for the aerator system shall be provided by the manufacturer and require the approval of the concerned municipality.

4.14.4.3. Side slope protection and bed lining

Lagoon side slope protection and bed lining shall be provided using grouted riprap, geocell panels or concrete geo-synthetic cementitious composite sheets. They shall be designed by a suitably qualified and experienced engineer considering the following concepts:

A. project specific details such as accommodating pipe penetrations, junctions and baffling.
    i. Stability of the lagoon embankments and natural slopes requires the following:
    ii. Soil boring(s) for embankment basin area and outlet structure.
    iii. Determination of groundwater elevation.
    iv. Embankment slopes stability.
    v. Grassed slopes (maintained).
    vi. Auxiliary spillway should not be constructed embankment.
B. overall stability of the structure to be lined
C. external stability of the structure to be lined
D. internal stability of the structure to be lined
E. substrate preparation to minimize voids underneath the lining
F. anticipated loading and abrasion conditions
G. the layup orientation of the product to provide shingling of overlap joints, enable practical installation and minimize material wastage
H. the specification of the overlap joints considering the impermeability and weed suppression requirements of the application

The key features of lagoon design are shown in Volume 2: Standard Drawings.

Safety Note – Minimum Safety Requirements for Lagoons

Particular attention shall be given to the safety of ponds and lagoons in areas where the public have access.

The following safety provisions shall be included:

- ✓ Protection in the form of fencing or railings

- ✓ Steps or ladders located at intervals not exceeding 100m to facilitate escape from the pond/lagoon, with at least one escape point per pond/lagoon.
- ✓ Life-saving devices (such as a lifebuoy) shall be positioned at intervals not exceeding 100m, with at least one device per pond/lagoon
- ✓ Appropriate warning signs at intervals not exceeding 50m, with at least two signs per pond/lagoon.

5. SUB-SOIL DRAINAGE DESIGN

5.1. Introduction

The purpose of subsurface drainage is to control groundwater level to a desired maximum level by removing excess subsurface water, where a desired maximum level is exceeded. Elevated groundwater may be caused by excess precipitation or irrigation, leakage from water and irrigation mains, high tides, trapped (perched) water due to impermeable soil and rock layers, or underground flows from upland areas. Under such situations groundwater control is required principally to protect underground structures and buried services, maintain plant growth (landscaping) conditions, to provide increased storage capacity during periods of excess surface free water (e.g. during rainfall events), and to prevent groundwater flooding.

Groundwater control systems are best when combined with the surface water system, so that combined flows are directed to common attenuation tanks, pumping stations and outfalls. These shared arrangements help minimise land use and environmental impacts of separate systems. A shared system can also operate all year round when taking groundwater flow contributions, thus maximizing reliability to deal with much larger flows from infrequent rainfall events.

The three most common techniques used to drain excess subsurface water are: surface drainage, subsurface drainage, and tube well drainage.

Surface drainage removes surplus water by means of open drains. They are typically adopted where the land is flat, the sub-soil is heavy, and infiltration rates are low, and used to eliminate ponding and to prevent prolonged saturation. They may be used alone or supplemented with subsurface drainage.

Subsurface drains are used to intercept and remove saturated water from natural and made ground. Subsurface drainage consists of perforated pipes and collector systems installed underground that allow flow under gravity to an open or closed collector drain. The marked advantage of subsurface drains over surface drains is that they can be placed where needed without sub-dividing land development plots.

Tube well drainage controls water table elevation by adoption of a group of wells to control the water table at all points in an area. These systems are best suited to soils of high transmissivity, reducing the number of wells required. Some of the advantages of tube well drainage include less earthwork and better handling of topographical and construction limitations. Similar to surface and subsurface drains, a collector system is required to convey water to a disposal/re-use site.

Depending on the location characteristics, it is recommended that subsurface drainage should be viewed as the ultimate solution in the planner’s arsenal when considering groundwater control. Ideally, planning should address the problem of excess subsurface free water by using designs and technologies that reduce the scale of the problem at source and restrict flows that closely mimic the natural environment.

5.2. Context of Subsurface Drainage in Abu Dhabi

The application of subsurface drainage is highly dependent on the unique set of variables relating to the physical and human environment of the Emirate of Abu Dhabi. A summary of the most important conditions includes:

- Topography – largely flat landscape with characteristic sand dunes rising gradually from the coastal plain inland, and with hilly/mountainous areas to the east and south of Al Ain;
- Climate – low annual rainfall of high intensity and high evaporation rates;
- Hydrogeology – shallow water table at the coast and more than 150 m below ground level far inland, low groundwater recharge rate and very poor groundwater quality; and
- Hydrology – generally there are few hydrological features, with exception around the Al Ain area.

Appropriate responses to the two general waterlogging situations in Abu Dhabi include:

shallow and saline water table renders the (Sabkha) soils unproductive as a result of deposition of migrating salts on the surface. Soils are heavy and flocculated as a result of the presence of Sodium.	**subsurface drainage**
soils of good quality overlay a heavy and confining layer that causes ponding of water in topographical lows.	**subsurface drainage**

Importantly though, each situation should be examined on a site-by-site basis, including existing utility services, and decisions made on the most appropriate drainage system for the local conditions encountered.

Where alternative responses to subsurface drainage are proposed, the Consultant shall discuss and agree with DEPARTMENT the justification.

5.3. Basic Principles of Subsurface Flow and Drainage Design

5.3.1. Subsurface Flow

Groundwater movement is governed by variations in piezometric head and soil hydraulic conductivity (permeability).

Indicative permeability data appropriate to soil conditions encountered in Abu Dhabi are provided in Table 5-1 and can be used as a useful starting point for drawdown estimations. The Consultant will need to demonstrate that data on actual groundwater levels and bulk permeability from site investigations has been obtained.

Permeability (m/s)	Soil Type	Drainage Characteristics
10^{0}	Clean Gravel	Good Drainage
10^{-1}		
10^{-2}		
10^{-3}	Clean Sands, Clean sand and gravel mixtures	Acceptable Drainage
10^{-4}		
10^{-5}		
10^{-6}		Acceptable Drainage
10^{-7}	Very fine sands, organic and inorganic silts,	
10^{-8}	clay, Sabkha, Stratified deposits	
10^{-9}		
10^{-10}	Homogeneous clays below zone of weathering	Practically Impervious

Table 5-1 – Typical Soil Permeability

NOTE: A wide range of values are likely to be encountered in the field, even across relatively small areas and with depth. As such, actual permeability testing or pumping testing may be essential.

5.3.2. Groundwater Levels

A critical parameter in designing a drainage system is theheight of the water table midway between two consecutive rows of pipes (refer to Figure 5-3). The final height of the water table at the radius of influence or the rate of fall or rise in the water table will be affected by the hydraulic conductivity of the ground,the storage capacity of the ground and the spacing between drains wherever more than one pipe is used.

It is necessary to achieve certain critical groundwater levels to ensure successful operation of urban infrastructure.

The Consultant shall demonstrate consideration for groundwater levels to rise in the future due to urban development effects, e.g., from new areas of irrigation. A proper network shall be provided to control the ground water rise if required based on the study with all supporting calculations

A general guide in designing for critical groundwater levels is shown in Table 5-2.

Facility	Depth of Formation Level (m bGL)	Minimum Depth to Groundwater (m)
Septic Tanks	3.0 - 4.0	0.5 m below formation level
Telephone Cables and Chambers	0.4 - 1.5	2.0 below Ground Level
Power Cables and Chambers	0.4 - 1.5	2.0 below Ground Level
Potable Water System; Pipes and Chambers	0.9	1.4 below Ground Level
TSE System; Pipes and Chambers	1.0	1.5 below Ground Level
Roads; Formation Level of Base Course	0.3 - 1.0	0.5 m below formation level
Buildings Foundation Level	1.0 - 1.5	0.5 m below formation level
Buildings Basement	4.0 - 4.5	0.5 m below formation level
Sewers *	1.2 cover min (highways) 0.9 cover min (private)	1.2 m below Ground Level

* for information only; to be verified by the ADSSC approved consultant prior to any construction activities

Table 5-2 - Guideline Depths of Infrastructure and Minimum Groundwater Levels

In urban landscaping requirements, experience in Abu Dhabi has shown that water table at the midway point must also be at least 50cm below the plantation root zone. For streetscape landscaping root zone is approximately 40 cm to 50 cm below soil surface, and for date palms in the street medians the water table must be at least 2.0 m below the surface.

Minimum depth for the ground water control system shall be 1.20 m from the finished road level.

Another parameter related to drainage design is the initial time required for the water table to drop from one level to another. Systems shall be sized based on the steady state outflow of drainage water not initial high discharge levels occurring immediately following installation. Designs causing very rapid water table drop will result in closely spaced drains with higher cost and present settlement risks.

5.4. Subsurface Drainage Planning

The main phases of planning a subsurface drainage project are shown in Table 5-3.

Reconnaissance study	Comprises of a desk and field research. Objective is to make an inventory of the problems (potential or actual), to determine whether a groundwater control system is needed and formulate possible solutions
Concept design	Involves setting out various alternatives and then subjecting them to an economic and technical feasibility analysis.
Detailed Design	The selected solutions are progressed to detailed design whereby final drawings / specifications are produced before implementation.

Table 5-3 – Phases of Subsurface Drainage Project Design

5.4.1. Reconnaissance Study

The Consultant shall collect relevant data as shown in Table 5-4.

Item	Description
Topography and Land Use	• topography and land use maps on a scale between 1: 50,000 and 1:100,000 showing contour lines of the land surface • alignment and slope of main sewer lines, laterals and drains • selection of the drainage outlet • the direction of natural drainage • the concentration points of flow • actual and proposed land use Where topography and land use maps are not readily available, a simple GPS survey coupled with field observations should be undertaken. Actual and proposed land use determines the degree of drainage required, the type of drainage system and individual drain alignment.
Hydrogeology	Groundwater and surface hydrology considered concurrently • rainfall • evapotranspiration • water elevation • water quality As a minimum sufficient data should be collected to derive piezometric contours maps and subsequently groundwater flow patterns at sufficient scale for the size of the project.
Geology	• geological map of the region • cross sections showing the lithological sequence • borehole data For reasons of economy and efficiency, bores of 5-10 meters generally provide adequate information on the soil profile and depth-to-water status. The actual depth of investigation will be dependent on the target depth of subsurface drainage requirements. For selected bores a pumping test will be required to measure soil permeability of the shallow substratum (1.5 to 5.0 m)
Soils	• soil map at a scale between 1:50,000 and 1:100,000 • data on soil stratification of the shallow substratum (1.5 to 5.0 m)

Table 5-4 – Reconnaissance Data

5.4.2. Concept Design

Data collected during the reconnaissance phase are analysed and the results used as background for the concept design.

The Consultant shall determine

- general alignment of field drains, collectors and main line from the prevailing topography.
- spacing of field drains in a trial and error procedure whereby the time required to control the water table is evaluated in relation to pipe spacing and depth.

- calculate system capacity and pipe discharge.
- number of checks to adjust calculated values and particularly pipe sizing and spacing to satisfy Abu Dhabi conditions

Topography and System Layout

Consultants shall evaluate the most appropriate layout based on specific drainage goals of the site under study in a broad, comprehensive manner, anticipating future needs where possible. Consultants will be required to demonstrate that system planning considers future requirements of the site. Example layouts are shown in Figure 5-1.

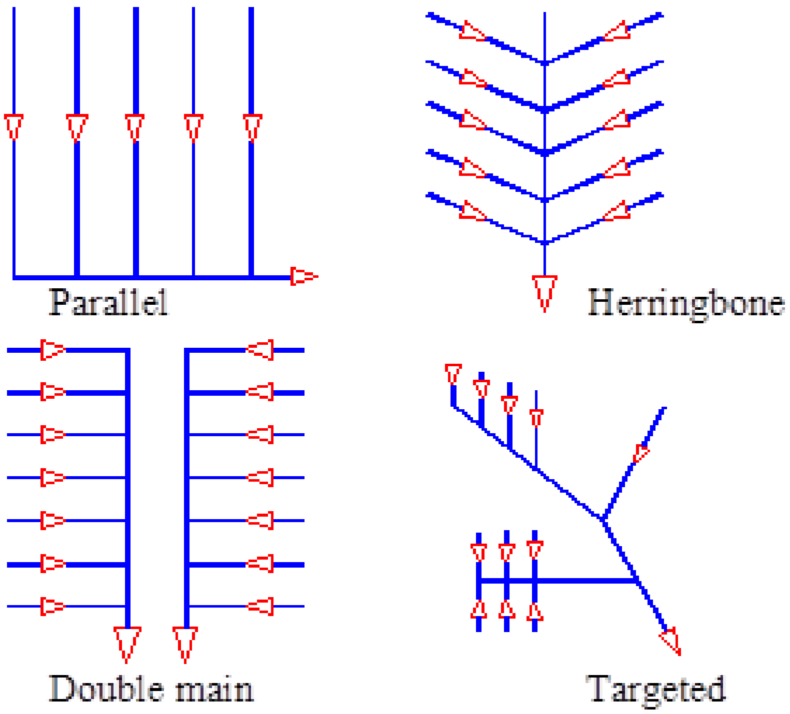
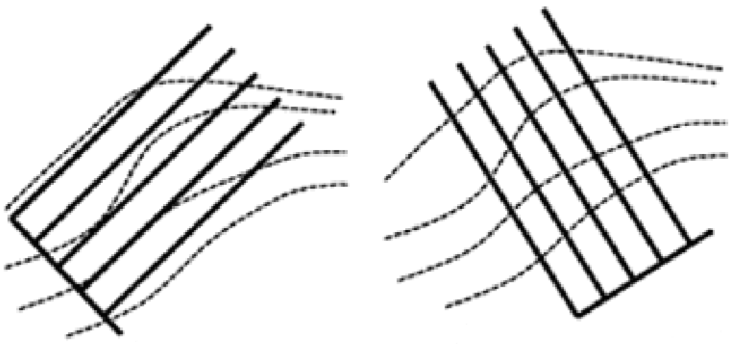


Figure 5-1 – Example Drainage System Layout Alternatives

The Consultant should select layout pattern where drains are oriented with the land contours as much as possible in order to intercept water as it flows down-slope. See Figure 5-2.



Desirable: laterals are aligned with piezometric contours

Undesirable: laterals cross piezometric contours

Figure 5-2 – Alignment of Field Laterals with Contours

Drain Spacing, Depth and Grading

Concept Design shall follow the method set out in "Computing Drain Spacings" (The International Institute for Land Reclamation and Improvement (ILRI), 1976, Bulletin 15). The work describes methods of computing drain spacings for a range of ground conditions. Average pipe spacing for soils of different textures are presented in below table:

Soil Texture	**Hydraulic Conductivity**		**Spacing**
	Class	(m/day)	(m)
Clay	Very slow	0.03	9 – 15
Clay loam	Slow	0.03 – 0.12	12 – 21
Average loam	Moderately slow	0.12 – 0.49	18 - 30
Fine sandy loam	Moderate	0.49 – 1.52	30 – 37
Sandy loam	Moderately rapid	1.52 – 3.05	30 – 60
Peat and muck	Rapid	3.05 – 6.10	30 – 90
Irrigated soils	Variable	Variable	45 - 200

Table 5-5 – Soil Properties and Pipe Spacing

Drainage pipes are recommended to be placed at depths varying between 1.8 and 2.7 meters below soil surface with a desirable starting depth at 2 m below soil surface. Only in rare instances, a depth of more than 2.7m below soil surface is justifiable.

For economy and efficiency, grades for field drains must be maintained between 0.1 percent (1 m per 1,000 m) and 0.3 percent (3 m per 1,000 m). The minimum acceptable flow velocity is 0.75 m/s as a lower flow velocity will cause silt and sand to settle and eventually lead to blockage of the drain pipes.

Determining Discharge from Subsurface Drains

With reference to Figure 5-3below, the steady state discharge of spaced drains can be computed using the following formulas:

drains placed above the impermeable layer	$$Q = CA\frac{2\pi KH(d + H/2)}{S^2}$$
drains placed on the impermeable layer	$$Q = CA\frac{4KH^2}{S^2}$$
Where *A* is the drained area and *C* is given by	$$C = \frac{1}{0.00054\sqrt{A} + 0.7795}$$

Where:

Q = the total discharge (Length³/Time (L^3/T))

K = the hydraulic conductivity weighted over the affected soil profile in (L/T)

d = the height above impermeable surface (L)

H = the initial height of water above the centreline of the drains (L)

S = spacing between consecutive drains (L)

Calculated discharge using any of the two formulas above can be checked using the product of the drainage coefficient and the area served by the respective field drain:

Q = L* S* drainage co-efficient (where L is the length of the field drain)

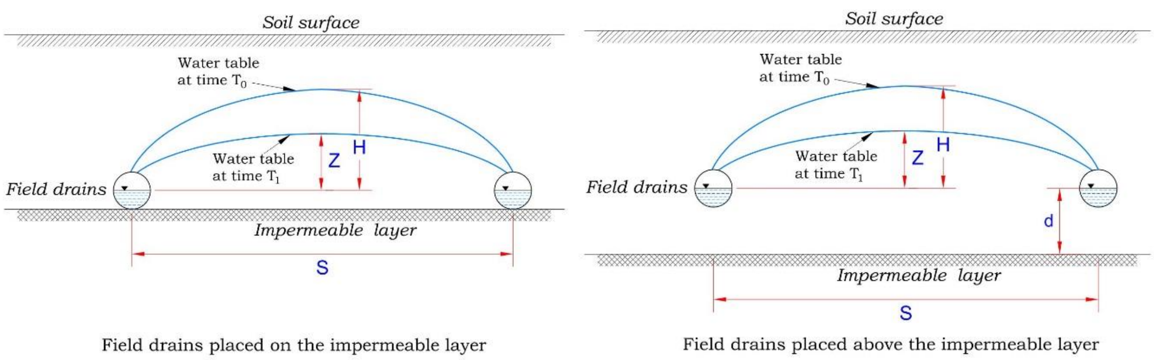


Figure 5-3 – Placement of Field Drains with Respect to the Impermeable Layer

For single-interceptor lines, it is possible to calculate the Radius of Influence using Sichardt's formula (published by CIRIA), as follows:

Radius of Influence	$R = 2000M\sqrt{K}$
Where: M = Drawdown (L) K = Coefficient of Permeability (L/T)	

With reference to Figure 5-4below, the steady state discharge of single line drains can be computed using the following formulas:

drains placed above the impermeable layer	$Q = CA\dfrac{2\pi KH(d + H/2)}{4R^2}$
drains placed on the impermeable layer	$Q = CA\dfrac{KH^2}{R^2}$

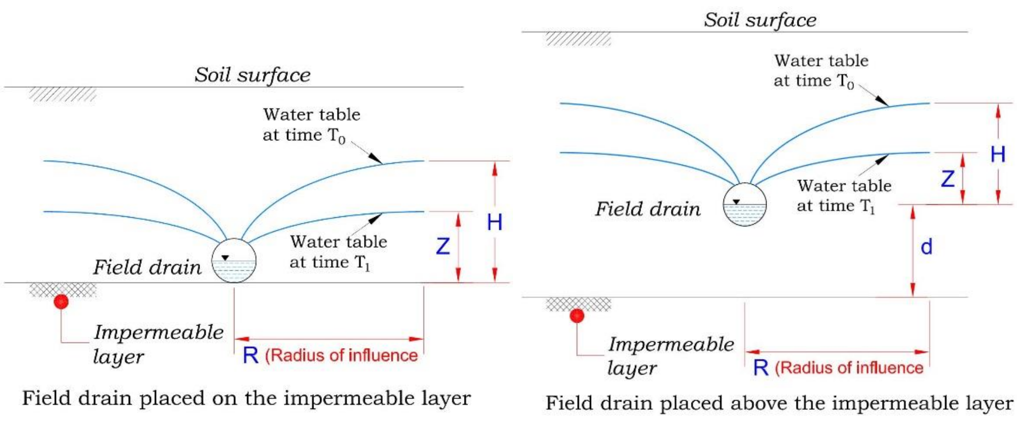


Figure 5-4 – Placement of Single Field Drain with Respect to the Impermeable Layer

To calculate the value of the drawdown (H-Z), the following two methods can be used:

- United States Bureau of Reclamation Charts; or

- Glover Dumm Equations

System Capacity and Drainage Coefficient

To protect plants, a subsurface drainage system must be able to remove excess water from the upper portion of the active root zone. System capacity shall provide the desired amount of water removal per day, commonly referred to as the "drainage coefficient."

Experience in drainage systems in Abu Dhabi has shown that this figure is often between 2 mm and 5 mm of water removal per day at steady-state operation of the drainage system. Initial drainage coefficient may reach up to 20 mm per day and must be adjusted during initial operation to prevent migration of fines and subsequent settlement.

Careful geotechnical and hydrogeological study shall be performed in order to clearly understand the water regime as well as the individual and the interface characteristics of the relevant site soil. Not only the soil hydraulic stability, solid mass transport, but also its physical and chemical stability shall be thoroughly studied.

Outflow from the drainage system is equal to the drainage coefficient multiplied by the area contributing to the drainage. Field drain pipe flow is equal to the drainage coefficient multiplied by the area served by the pipe. The length of this area is the length of the field drain whilst the width is the field drain spacing. Past experience has shown that maximum flow from a field drain ranged between 15 litres/hour/meter to 20 litres/hour/meter of field drain for spacing of 75 m – 80 m. These figures can be used to double-check the numbers calculated using the formulas presented in the previous paragraph.

Minimum size for perforated pipes is 160mm OD.

Pipe Materials

Approved pipe materials parameters are shown in Table 5-6.

Material	unplasticised polyvinyl chloride (uPVC) (plain or corrugated) – Class PN10 unless higher class is deemed necessary.
Rectangular perforations	0.6 to 2mm long 0.6 to 1mm wide
Circular perforations	The drain pipes shall be fully perforated in rectangular shapes around the pipes in an angle of 45 degrees measured from the vertical pipe axis. Perforation should be concentrated above the horizontal centre line of the pipe.

Table 5-6 - Approved Drainage Pipe parameters

Settlement Potential

Land drainage may result in changes in groundwater levels. The drawdown of groundwater below its normal seasonal variation may result in settlement of structures founded on or in the ground and/or collapse of voids that may be locally present within bedrock. The movement can result in damage to structures depending on the amount of settlement that is induced and how this changes beneath the structure, and the nature of the structure and its foundations.

Particular care shall also be taken with the design to ensure that ground material cannot wash into the sub-soil system and cause settlement of the surrounding ground.

The Consultant shall make an assessment of these impacts and report findings to DEPARTMENT for review. Design shall not proceed until the agreement of DEPARTMENT Dewatering Section has been given.

Water quality

The drainage assessment shall consider the chemical properties of water that will be drawn into the drainage system. This is because it may cause migration of existing contamination and/or affect the structural integrity of below ground infrastructure (e.g., foundations, piles, pipelines). In particular, where drainage systems are installed within or adjacent to areas of potential contamination, or have the potential to draw water from areas that may have different chemical properties to the native groundwater (e.g., close to the sea, natural and artificial wetlands), a risk assessment shall be provided and appropriate mitigation measures adopted by the Developer. The risk assessment shall consider the potential for all relevant risks including but not necessarily limited to:

- Changes to aquifer properties (e.g., dissolution or precipitation)
- Changes to groundwater chemistry
- Mobilisation of contamination
- Saline intrusion
- Detrimental effects on the drainage network (e.g., corrosion of pipes)
- Detrimental effects on infrastructure (e.g., corrosion of existing foundations / piles).

Pipe Bed and Surround

All pipes shall be laid with bed and surround in order to:

- prevent or restrict soil particles from entering the pipe
- provide good drainage permeability
- assist with accurate laying of the pipe
- provide structural support to the pipe

The pipe trench shall be 150mm wider than the pipe on each side for open trench installation with minimum pipe trench width of 600mm to allow sufficient bedding to support the pipe.

For Projects in Al Ain Region, pipe bedding and surround details provided in Al Ain Municipality standard drawings and specification to be used.

Gravel Bed and Surround

Where gravel bed and surround is provided, the depth beneath the pipe shall be a minimum of 100mm, and above the pipe shall be a minimum of 180mm. However, the depth of gravel above the pipe shall be increased where necessary to connect with permeable ground.

Where required, well-graded gravel shall be used for bed and surround. The measures of grading are given by the coefficients of uniformity and curvature from the following formulae:

Coefficient of uniformity $$C_u = \frac{D_{60}}{D_{10}}$$

Coefficient of curvature $$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$$

Where D60, D30 and D10 are the respective diameters corresponding to 60%, 30%, and 10 % finer particles in the particle-size distribution curve.

A well graded material shall have a coefficient of uniformity greater than 4 for gravel and greater than 6 for sands. In addition, the coefficient of curvature shall be between 1 and 3 for both gravel and sand.

Table 5-7 below shows the gradation relationship between the base material and gravel envelope for most soils. These relationships have been found to work satisfactorily under low-head conditions

Base soil limits for d_{60} (mm)	Lower limits (mm)						Upper limits (mm)					
	Percentage passing						**Percentage passing**					
	100	60	30	10	5	0	100	60	30	10	5	0
0.020-0.050	9.52	2.0	0.81	0.33	0.3	0.074	38.1	10.0	8.7	2.5	-	0.59
0.050-0.100	9.52	3.0	1.07	0.38	0.3	0.074	38.1	12.0	10.0	3.0	-	0.59
0.100-0.250	9.52	4.0	1.30	0.40	0.3	0.074	38.1	15.0	13.1	3.8	-	0.59
0.250-1.000	9.52	5.0	1.45	0.42	0.3	0.074	38.1	20.0	17.3	5.0	-	0.59

Table 5-7 – Design Criteria for Gravel Envelopes

Pipe Connections

Pipe connections will be achieved using manholes.

In urban areas, the groundwater lowering pipework will generally be laid in the same trench as the storm water pipes (as long as not under road asphalt) and connect to manholes for ease of cleaning.

5.5. Hydraulic Design

The flow from a sub-soil system shall be determined by using the highest value from the following methods:

- multiplying the drainage coefficient (ranging from 2 to 5 mm/day for Abu Dhabi), by the drainage area
- using the formulae in Section5.4.2, "Determining Discharge from Subsurface Drains"
- where irrigation is the only source of recharge to estimate the drainage rate as a fraction of the irrigation rate. This fraction is commonly taken as 10 percent.

The sizing of field drains shall be carried out using the modified version of the Manning equation:

$$d = 116 * \left(qn / S^{1/2}\right)^{3/8}$$

Where:

- d = pipe diameter in mm
- q = flow in litres per second
- S = pipe slope in m/m
- n = pipe roughness coefficient

For example: if a PVC field drain has a total length of 360 m, pipe roughness coefficient of 0.011, calculated flow of 1.7 l/s and a slope of 0.2 percent, the pipe internal diameter is calculated at 84 mm. Please note that minimum diameter for field drain shall be 150 mm (nominal diameter)

Pumping station design shall be carried out in accordance with Section 6.

5.6. Drainage Installation

Pipes shall be laid in accordance with Volume 4: Standard Specifications.

Manufacturer should be obliged to study the soil condition with the Contractor and the supervising Consultant in order to ensure the suitability of the pipe geometrical and material characteristics, including the joints, for installation in the given soil condition, may need to be added.

5.7. Construction near Utilities

Information shall be obtained by the Consultant on existing and proposed utilities.

As far as possible, drain lines shall not be installed across buried cables, pipelines and other facilities.

5.8. Maintenance

Maintenance of sub-soil drainage systems shall be carried out in accordance with the requirements of Volume 3: Operation and Maintenance.

5.9. Tubewell Drainage Design

Tube well drainage consists of pumping from a series of wells an amount of groundwater equal to the drainage surplus. The pumped water is then discharged into a pipe or open surface drain network.

Tube well drainage is not a substitute for subsurface drainage and shall be used only in combination with near surface drainage options to minimise the extent of subsurface drainage requirements over wider areas.

5.9.1. Patterns of Tubewell Drainage

Lowering of the water table sufficiently and uniformly requires considered placement of each well. Two regular configured tube well patterns are permitted:

Triangular pattern is hydraulically the most favourable well-field configuration, with a maximum area to be drained by one well and with no extra drawdown induced by neighbouring wells.

Rectangular pattern in which wells are placed along parallel collector drains. For this well field configuration, a reduced length of collector drains is required when compared to a triangular configuration.

Alternate patterns are permitted only upon the approval of DEPARTMENT.

5.9.2. Well Field in a Triangular Pattern

A triangular well field pattern is shown in Figure 5-5. Limited overlap of individual radii of influence occurs when the wells are placed in a triangular pattern. A simplifying assumption infers that drawdown and discharge of each well will not be affected by neighbouring wells, and therefore, the theory of a single well can be used.

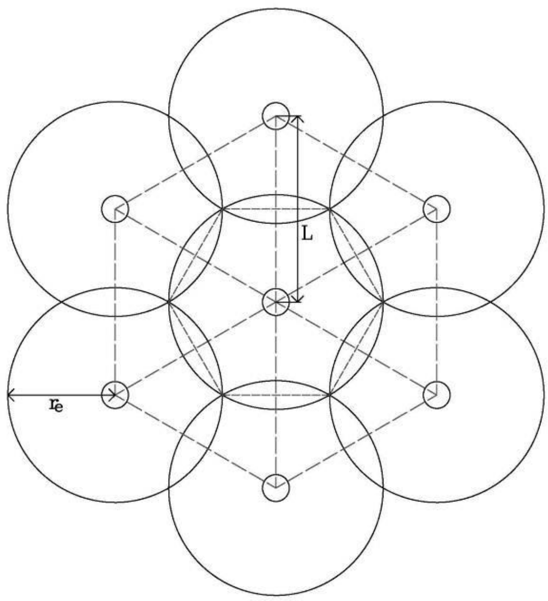


Figure 5-5 – Wells Located in a Pattern of Equilateral Triangles (Well Spacing *L=r*)

In a drainage well-field, there is a direct relationship between the discharge rate of the well, the recharge rate of the aquifer by percolation, and the area affected by pumping. The decline of the water level due to pumping is determined by the discharge rate of the well and the permeability and thickness of the aquifer. The discharge rate and the drawdown in the well are important factors in calculating the pumping costs of well drainage. In an unconfined aquifer, the steady-state flow through an arbitrary cylinder at a distance r (Q_r) from the well is given by:

$$Q_r = \pi(r_e^2 - r^2)R$$

Where:

- r_e: radius of influence of the well (m)
- r : distance r from the well (m)
- R : recharge rate of the aquifer per unit surface area (m/d)

According to Darcy's law, Qr equals to algebraic product of the cylindrical area of flow and the flow velocity. Hence, the discharge at distance r from the well can also be expressed by:

$$Q_r = 2\pi rhK\frac{\delta h}{\delta r}$$

Where:

- K : hydraulic conductivity of the aquifer (m/d)
- $\frac{\delta h}{\delta r}$: hydraulic gradient in the aquifer at distance r (-)
- r : distance r from the well (m)
- h : height of water above the well at distance r

To calculate the drawdown in a well field when the wells are placed in a triangular pattern following equation can be utilized. From the Figure 5.5, it can be seen that the distance L between the wells is equal to $r_e\sqrt{3}$.

$$\Delta h_r = \frac{2.3Q_r}{2\pi KH}\log\frac{r_e}{r_w}$$

Where:

- H = saturated thickness of the aquifer before pumping (m)
- Δh_r = drawdown due to radial flow towards the pumped well (m)
- r_w =radius of well (m)

The equation is applicable when r_e/r_w> 100 accepting an error of 10%. The radius of influence describes the distance at which infiltration recharge is sufficient to supply the yield from the well.

It can be derived from appropriately analysed pumping tests or estimated by the use of commonly used formulas, e.g. Sichardt's formula and Cooper and Jacob formula.

5.9.3. Well Field in a Rectangular Pattern

For rectangular configurations, which are formed in parallel lines at a distance B apart and the spacing of the wells along the lines is L, as presented in Figure 5-6.

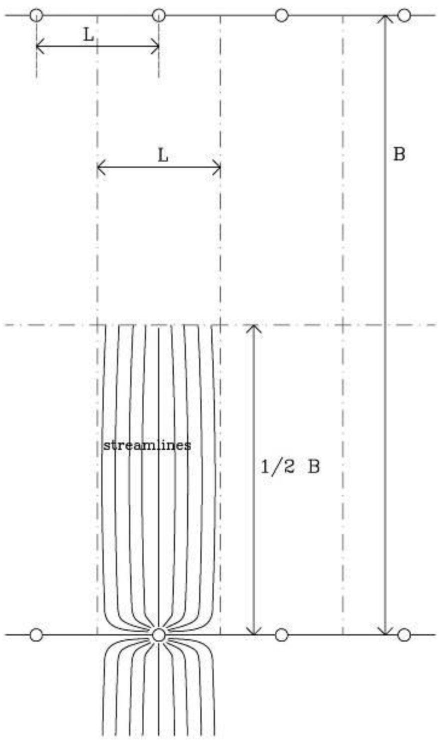


Figure 5-6 – Wells in Parallel Series with L<<B.

In such a situation, if the recharge on the land surface is uniform and flow towards the wells has attained a steady state, the discharge of each well can be written as:

$$Q = RBL$$

Where:

- Q : discharge rate of each well (m3/d)
- R : recharge rate of the aquifer per unit surface area (m/d)
- B :distance between the parallel lines (m)
- L : spacing of the wells along the lines (m)

For radial flow the drawdown is expressed as:

$$\Delta h = \frac{RB^2}{8KH} + \frac{2.3Q}{2\pi KH} log \frac{L}{\pi r_w}$$

Where Q,R,B and L are as above, and:

- K = hydraulic conductivity of the aquifer (m/d)
- H = saturated thickness of the aquifer before pumping (m)
- Δh = drawdown due to flow towards the pumped well (m)
- r_w = radius of well (m)

The equation can be used to calculate the head loss in a well field when the wells form a rectangular pattern. Such a pattern is recommended when surface drains running in parallel lines are proposed or already exist in the drainage area.

5.9.4. Semi-Confined Aquifers

A semi-confined aquifer, whose overlying layer is an aquitard, is replenished by percolating rain or excess irrigation water. Depending on the recharge rate and the hydraulic resistance of the aquitard, a difference in head between the free water table in the aquitard and the piezometric level of the aquifer will develop.

Equations for both triangular and rectangular pattern placement may be used to calculate the head loss in a well field.

Well Distance Calculation Procedure

An operating factor (the number of hours of tubewell operation in a 24 hour period) and the discharge rate determine how much water will be pumped by one tubewell. In combination with the drainable surplus (the annual discharge required to maintain the design water-level criteria), they determine the drainage area per tubewell and thus also the number of tube wells required for the total drainage area. This can be expressed in the following equation:

$$A_w = \frac{0.1Qt_w}{q}$$

Where:

A_w = drainage area per well (ha)

Q = discharge rate of the well (m^3/d)

q = drainable surplus (mm/d)

t_w = $\mathbf{t_w}$ = tubewell operating factor (-)

The total number of wells required can then be found by dividing the total drainage area per tubewell.

Well Field Configuration

For a triangular well field configuration, the distance between the wells for a selected discharge rate can be calculated by:

$$L = 100\sqrt{\frac{3A_w}{\pi}}$$

For a rectangular well field configuration with wells placed along the parallel main drains is:

$$L = 10{,}000\frac{A_w}{B}$$

Where;

L = the distance between the wells (m)

B = the distance between the lines of wells (m)

5.9.5. Well Design

The Consultant shall demonstrate how the following objectives are met:

- Pump water at the lowest cost
- Pump water that is free of sand
- Operate with minimum operation and maintenance costs
- Operate with a long and economic lifetime

Materials used in well construction shall be selected without compromising long-term performance. Established international standards and guides such as BS, ISO, ASTM, WIS, and WRc shall be followed in the selection of and specification of construction materials.

In particular, screen material shall be resistant to chemical and microbiological corrosion, and degradation in contaminated and uncontaminated waters. The screen shall be durable enough to withstand installation and well development and last for the entire designed operation period. The screen material shall conform to ASTM Standard D5092 or other similar international standard.

PVC and stainless steel are the most commonly used well screen materials. However, in some situations, other materials, such as Teflon® may meet project objectives.

PVC screens, casings, and fittings	Schedule 40 or 80 conform to ASTM Standards F480 or D1785 or other similar international standard
Stainless steel well screens	Type 316 flush threaded joints sealing “O” rings conform to ASTM Standard A312/A312M or other similar international standard
Teflon® well screen	flush threaded joints sealing Teflon® “O” rings conform to ASTM Standard D4894 or D4895 or other similar international standard chemically inert as technically practical with respect to the site environment

Table 5-8 – Well Screen Materials

5.9.6. Other Well Design Considerations

The Consultant shall demonstrate consideration of other well design factors:

Design Discharge Rate of Tubewells	The design discharge rate depends on autonomous and design factors shown in Table 5-10.
Tubewell Operating Factor	The tubewell operating factor is the number of actual operating hours of the well per 24 hours, expressed as a fraction. The tubewell operating factor largely depends on autonomous factors, but also on a design factor like the peak drainage requirement. It will not be possible to operate all wells continuously over an extended period. Time will be lost during maintenance, inspection, and repairs, stoppage due to power failures, etc. Social factors like the presence or absence of a pump operator will also influence the possible operating factor of the wells.
Annual Drainable Surplus	The annual drainable surplus of an area is the annual discharge, in mm/day, required to maintain the design water-level criteria. It depends on many factors and one of the essential factors is the depth at which the water table is to be controlled.
Peak Drainage Requirement	To maintain a stabilised water table in tube well drainage, the system should be based on the maximum expected recharge. This, however, would result in excessive investment costs. If the system were to be based on a continuous discharge to drain the annual drainable surplus at a constant rate, the water table would fluctuate throughout the year. This variation can be reduced by adjusting the monthly tubewell operating factor. This means higher operating factors during the periods with higher recharges and lower operating factors during the periods with lower recharges.

Table 5-9 – Other Well Design Considerations

Autonomous Factors	Design factors
Design should be based on the most economic pump capacity. If larger pumps are installed, fewer pumps will be required, which generally results in lower investment costs. On the other hand, larger capacity pumps result in higher drawdowns and thus higher energy costs. Determining pump capacities on a purely economic basis could lead to very high pumping rates. There are, however, several practical constraints to these high pump capacities.	The annual drainable surplus and the peak requirements. The maximum tubewell capacity will influence the distance between the wells or the maximum spacing in the well field. Hence, for a given operating factor, the drainable surplus would be the determining factor for the discharge rate of the well.
Selection of pumps and engines should be based on their availability on the local market; spare parts, especially, should be locally available.	The horizontal and vertical hydraulic conductivity and the thickness of the aquifer, and the vertical resistance of the aquitard, determine the drawdown for a given discharge rate and the expansion of the cone of depression.
A policy of reducing the number of different pump sizes may be another major constraint on the choice of the pump capacity.	Screen and casing specifications, together with the discharge rate, determine the entrance velocity of water flowing through the screen, which has a maximum value in order

Autonomous Factors	Design factors
	to ensure a maximum lifetime for the well.
A well with a very high pump capacity may serve a very large area that exceeds the spacing determined by other factors. If such a well were to be out of order for a prolonged period, the neighbouring wells would be overburdened, and proper drainage of the area would be impossible.	

Table 5-10 – Autonomous and Design Factors Affecting Design Discharge Rates

5.9.7. Environmental Design Considerations

This Section provides guidance on tubewell design and the potential for impact on the environment. Environmental considerations include:

- Aquifer pollution prevention – design and construction of suitable tubewell housing to prevent unauthorised access.
- Management of poor-quality discharge water – tubewell drainage may draw in contaminants from off-site or newly introduced on-site sources. Poor quality water may require treatment prior to its disposal into the reticulation network.

Environmental design considerations should be included as part of the reconnaissance and concept design stages of drainage design development. These considerations shall follow guidance provided in CIRIA document 'Control of water pollution from linear construction projects: technical guidance, Publication C648, 2006'.

5.9.8. Tubewell and Pump Maintenance

To determine any loss in performance some reference mark will be needed and an effective maintenance program established prior to implementation of the drainage design.

To evaluate the performance of a well, the following checklist shall be performed on commissioning each well:

- The static water level in the well.
- The pumping rate and the water level expressed as specific capacity and the ratio of the pumping rate and the drawdown after a specified period of pumping.
- The sand content of the pumped water.
- The total depth of the well.
- The original specific capacity of the well.
- The normal pumping rate and how many hours per day it is operated.

Significant changes in well yield, energy demand, or energy consumption indicate that the well, or the pump, requires maintenance measures.

It is important that the tubewell is designed to allow easy access for monitoring, and maintenance should it be required. Thus, design shall follow guidance provided in CIRIA document *'Monitoring, maintenance and rehabilitation of water supply boreholes, Report R 137, 2003'*.

6. STORM WATER PUMPING STATIONS

6.1. General

Pumping stations are required for the removal of the storm water from locations where gravity drainage is impossible or for maintenance purpose. However, pumping stations are expensive to construct, operate and maintain, contain additional health and safety risks over gravity systems and have number potential problems that must be considered. They also represent a weak-link in the operation of the system compared to gravity systems.

As such, pumping stations should only be incorporated in the system where they are essential for correct hydraulic functioning.

6.2. Pumping Station Types

6.2.1. Pumping Station Components

Following major pumping station components shall be considered as minimum during the design process:

- Inlet Chamber
- Pump house
- Discharge Chamber
- Inlet chamber shall be design to accommodate following items:
  - Penstock
  - Screening
  - Sand Trap
  - Oil separator
- Pump house arrangements shall mostly depend on the pumping station type. However, each pump house shall be designed to accommodate following items:
  - Suction piping (wet well installation)
  - Pumps
  - Discharge piping including required isolation valves, check valve, associated piping, etc.
- Discharge chamber shall be design to accommodate
  - Flowmeter
  - Line Valve
  - Surge vessel (if required)
- MCC room
- Lifting Equipment
- Ventilation/ HVAC system

6.2.2. Pumping Station Types

Commonly used types of pumping stations in DEPARTMENT jurisdiction are shown in the table below:

Pumping Station Type	Wet Well	Dry Well
outfall	✓	
pedestrian underpasses	✓	
traffic underpasses	✓	
sub-soil drainage	✓	✓
lifting stations	✓	✓
pumping station of the general purpose	✓	✓

Table 6-1– Commonly used types of pumping stations in DEPARTMENT jurisdiction

Wet Well Pumping Stations

Circular wet wells shall be used for pumps with individual flows up to 180 l/s. Rectangular wet wells shall be designed for large pumps with individual flows greater than 180 l/s.

The minimum required sump volume can be calculated, and it depends on the inflow to the pump station, the pump capacities, their allowed cycle time and their operating sequence.

It is Important to know the required active volume. It is the volume defined by the highest start level and lowest stop level in the pump sump.

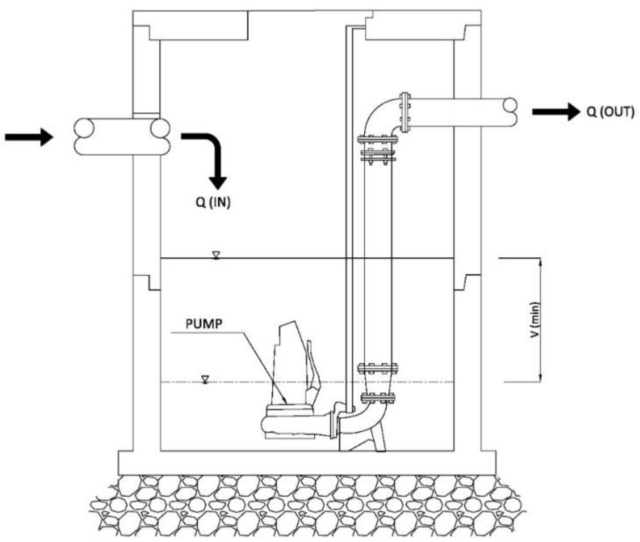


Figure 6-1 – Wet Well – Active Volume.

In general, the wet well should have a storage of a minimum cycle time of peak flow (6:10 starts per hour or as per the minimum allowable cycle time ‘t’ designated by the pump manufacturer)

For minimum active volume calculation following equation shall be used:

$V=0.25QT$

Q – Pump capacity

T – Minimum cycle time specified by pump manufacturer

The wet well volume when multiple pumps are installed is calculated as for a single pump where the minimum volume is the capacity between the start and stop level for each pump. However, additional capacity is required to allow a vertical distance of 150mm between the start or stop levels of consecutive pumps

For more details, refer to guidance provided in ANSI-HI_9.8.

Dry Well Pumping Stations

Comprising two main chambers: a dry well and a wet well. The minimum active volume of the wet well shall be calculated using equation in Wet Well Pumping Stations section.

For the dry well sizing following shall be considered as minimum:

- Equipment and piping requirement
- Sufficient space for installation of the equipment and piping
- Adequate access for personnel
- Civil Defence requirements

6.3. Mechanical Design

6.3.1. Pumps

Centrifugal pumps shall be used for all storm water pumping applications as follows:

Pump Type	Wet Well	Dry Well
Submersible direct coupled pumps	✓	✓
Submersible axial (propeller)	✓	×

Table 6-2 – Pump types and applications

6.3.2. Pump Performance Curve and Duty Point

The pump duty point shall be used when considering the suitability of alternative pumps for a particular duty by comparing the efficiency and power requirements for each pump at the duty point. Typical pump performance curves are illustrated in Figure 6-2.

In multiple pump installations, it is essential that the operating conditions of a single pump running are carefully checked to ensure that the pump will operate satisfactorily.

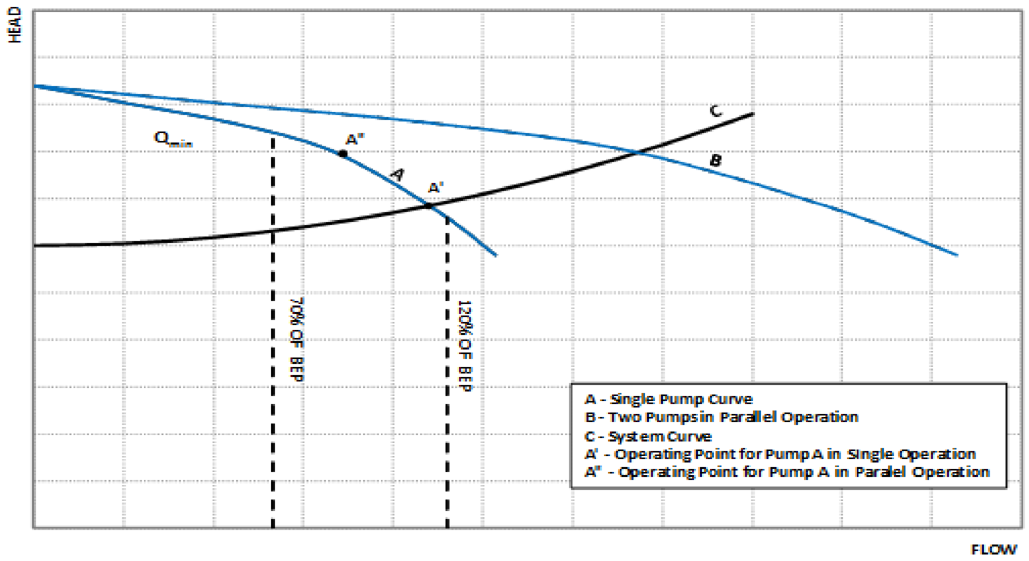


Figure 6-2 - Pumping System Characteristics

Pump design flow rates shall be determined from the range of working requirements with an additional allowance of 10% at maximum design flow.

The pump performance characteristic shall be:

- stable at all flow rates between closed valve and open valve
- steep enough to permit satisfactory operation in parallel with other pumps

6.3.3. Pump Selection

As part of the pump selection process the Consultant shall consider the following:

- Quality of the fluid to be pumped,
- Required design capacity and available storage (initial minimum, average and maximum flow rates), and
- Operating conditions (best/ worst case system head curves, maximum/ minimum flows, submergence, NPSH, etc.).
- Number of pumps
- Mode of operation,
- Inlet configuration,
- Type of driver (C/S or V/S),
- Location of the pumping station, configuration and constraints (pump number, parallel operation, etc.), and
- Miscellanies Considerations.

NOTE: Storm water may contain high levels of chloride, salinity, hydrocarbons, significant quantities of sand and plastic or rags accumulated over time. Pumps with high solid passing capacity, wear and corrosion resistance and lower speed drives shall be considered.

6.3.4. Design Capacity and Number of Pumps

The required design capacity (both initial and ultimate flows), including the maximum, normal, and minimum flows to be pumped, shall be established during pumping equipment selection.

For the pumping stations designed without special storage requirement (active volume only) the Consultant shall assume the following:

- The peak demand should be covered with two or three duty pumps.
- Normal demand should be covered with one duty pump.
- The expected sustained minimum flow is important as it may dictate installation of the low flow pumps to ensure energy saving and wear and tear reduction of the main pumps.

The minimum required number of pumps in each pumping station is two.

For two pump arrangements, pump oversizing to compensate possible pump failure is acceptable each designed to pump 66 to 100% of the required flow.

For three pump arrangements each designed to pump 50% of the design flow. Each main pump shall have equal capacity and be of the same make and type.

Low-capacity pump should be used as the first to switch on when using a small number of large capacity pumps. This pump is not part of the sequencing that follows.

6.3.5. Outfall Pumping Stations

Pumping stations shall be provided at outfalls where receiving water bodies are higher than the water level of the outfall.

The capacity of the proposed pump station varies depending upon the total runoff generated from the watershed catchment. The hydraulic modelling shall identify the maximum capacity of such pumping station during the design stage subject to approval by the concerned municipality.

The Pumping station should be designed based on 1 in 10-year return period.

The minimum capacity of the submerged outfall pumping stations working in parallel with gravity lines shall be 10% of estimated peak flow during the rain events in addition to the groundwater inflow if both networks is combined and directed to the same outfall.

6.3.6. Design Capacity of Storage Tank

The storage is needed to decrease the peak flow of the pumps to reduce the operation and initial costs (design that balances storage and pumping capacity provides the most economical design since storage permits use of smaller and/or fewer pumps).

In case attenuation in peak runoff of the catchment to downstream conveyance main or receiving water, bodies is required, the storage volume required to detain the surplus water in a storm water tank.

The Consultant shall calculate total storage capacity to be provided using inflow hydrograph and pump-system curves. An estimate of storage volume can be made by comparing the inflow hydrograph to the controlling pump discharge rate as illustrated in Figure 6-3.

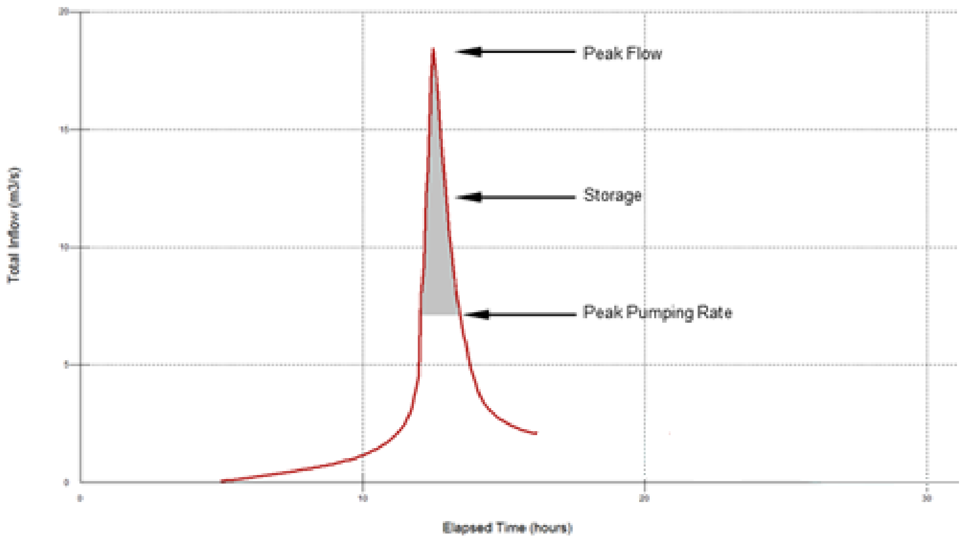


Figure 6-3 - Estimated required storage from inflow hydrograph

The shaded area represents the estimated volume required above the last pump turn-on point – this assumes that this volume of water is beyond the capacity of the pumps and must be stored.

The storage might also be required to reduce the discharge rate of the pump station and to store enough inflow (volume of water under the inflow hydrograph) to meet the specified limits and not exceed the maximum capacity of the outfall. There are three general approaches to pump system storage and pump sizing (HEC-24, item 6.3, storage concepts)

1. The collection system size for acceptable conveyance of the runoff using conventional storm drain design procedures (See FHWA's *Urban Drainage Design Manual*[10]*)*. Choose a wet well size, size the pumps and set the pump switching to maximize use of the provided storage.
2. The following approach may be used in areas where ordinances limit peak discharges to pre-developed rates, or where the outfall system has limited capacity. In this approach, choose pumps that have a total capacity of identified target outflow rate. Compute the required system storage in order to detain the design inflow hydrograph.
3. The third approach is to use iteration, vary the storage volume, pump sizes and number of pumps to determine a combination that minimizes total life-cycle cost. This is a more time-consuming approach but will most often produce the best overall design.

The first two approaches do not identify the combination of pump size and storage that minimizes ife-cycle costs. Hence, the third approach is preferred.

The system storage Volume is defined as the volume in the collection system, storage unit, and wet well between the lowest pump operation elevation and the maximum allowable high water serves to detain the inflow.

At any stage, H, the volume used for storage of the inflow is the volume between the water level and the lowest pump operation elevation (Stage = 0) as shown in Figure 6-4. The available storage is independent of inflow rate if the hydraulic grade line is ignored. The procedure to Estimate Total Available Storage Required

Step 1. Select a design inflow hydrograph

Step 2. Choose trial target total pumping rate.

Step 3. Using a plot of the inflow hydrograph, draw an estimated outflow hydrograph (based on the target pumping rate) from a tangent on the lower portion of rising limb to the falling limb at a flow equal to the target total pumping rate (Figure 6-5).

Step 4. The area between the estimated pumping rate line and hydrographer presents the estimated required volume.

For more details, refer to (HEC-24, Chapter 6)

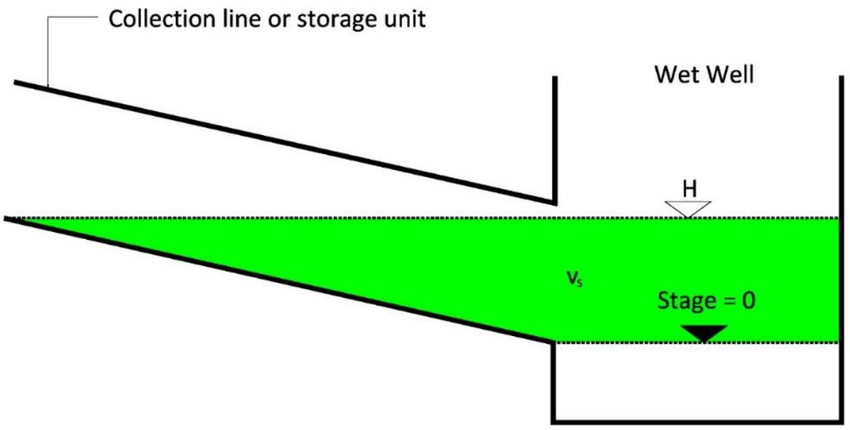


Figure 6-4 – volume in storage at any stage

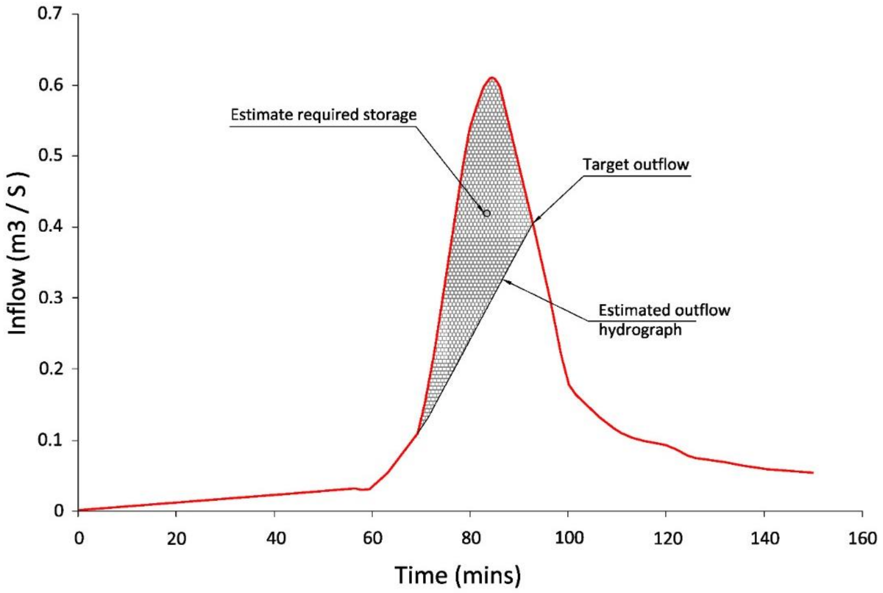


Figure 6-5–Estimate of storage required

6.3.7. Operating Conditions

Analysis of the pump operation shall be based on the following:

- The most frequent operating requirement
- The maximum operating requirement
- Net positive suction head available ($NPSH_a$) and NPSH margin

Assessment of the pumps against operational conditions shall follow ANSI/HI Pump Standards 9.6.3 and selection of the pump shall be based on the following:

- During normal operation or frequent operating conditions pump operation point should be located within Preferred Operating Region (POR) as close possible to Best Efficiency Point (BEP).

During the less frequent operating conditions (maximum operating requirement) pump operation point can be located outside POR but within Allowable Operating Region (AOR).. The POR for pumps with a specific speed less than 5200 (4500 U.S units) shall be from 70% to 120% of BEP. For higher specific speeds POR shall be from 75% to 115% of Best Efficiency Point (BEP).

Indicative Operating Regions are shown in Figure 6-6.

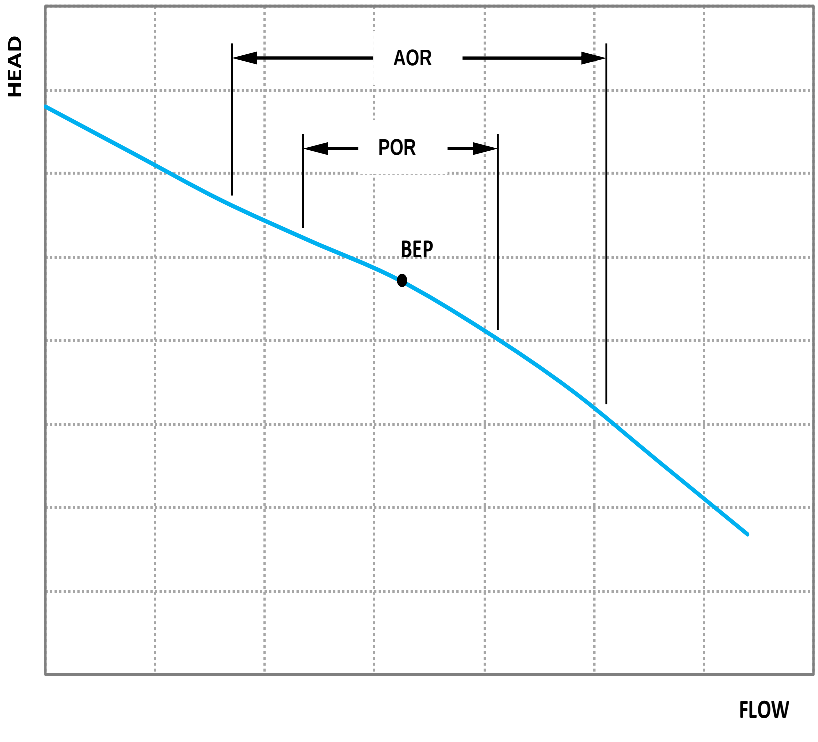


Figure 6-6 – Pump Operating Regions

6.3.8. Net Positive Suction Head NPSH

The Centrifugal pumps will only operate satisfactorily if there is no build-up of vapor (cavitation) within the pump. Therefore, the pressure head at the NPSH datum point must exceed the vapour pressure head of the medium handled.

The $NPSH_r$ is the value required by the pump and specified by the manufacturer.

$$NPSH_a \geq NPSHr + 1.5 \text{ m}$$

6.4. Pumping Station Arrangement

6.4.1. Suction and Discharge Pipework

Design of the suction piping shall follow ANSI/HI 9.8 Pump Intake Design standard and selected pump manufacturer recommendations.

In addition, the following suction piping recommendations shall be considered by the Consultant:

Design Element	Consideration
Direct connection of the pipe fittings to pump nozzle shall be avoided since.	Pipe fitting cause uneven flow patterns
The short radius elbows shall be avoided	Always use long radius elbows where possible
Suction piping approaching to pump should be straight with length from 5D to 10D	Ideally, length of the suction pipe should be 10D. (10D rule is not always applicable due to construction / economic limitations)
Pump inlet piping shall be designed and installed to provide smooth and even flow	Introduction of the suction bell is recommended. Suction bell shall reduce frictional losses and approach mean velocity
The eccentric reducer in flat to the top position should be used when a pipe size transition is required	Connection of the large pipe to pump suction
Installation of foot valves shall be avoided	
Gate valves shall be used as shut off valves	
Minimum pumping velocity shall be sufficient to avoid clogging of the suction line due to presence of the solids in the water	
Anchorage of the suction piping should be near to the pumps and in the same time piping arrangement should ensure easy removal of the pump	Misalignment of the suction piping can cause excessive loads at pump nozzle. The contraction/ expansion of the suction piping should be reduced on the acceptable level
The suction line size shall never be smaller than suction nozzle size	

Table 6-3 - Suction Piping Design Considerations

The Pumping Station discharge piping should be a combination of the following elements:

- Pipe
- Stop/check valves

- Flap gates/valves
- Air valves
- Elbows
- Manifolds
- Tee’s
- Reducers / Expanders
- Brackets, bolts and other fixtures

The following discharge line recommendations in Table 6-4shall be considered by the Consultant:

Discharge Line Design Considerations	
Discharge line should be kept as short and simple as possible	
Simplest configuration with pump has its own independent discharge line	
The elbow of the vertical riser from the pump should be set higher than the discharge line with a slope down to the discharge end	To minimise the volume of back flow when the pumps switch off
The centreline of each discharge pipe should be placed higher than the design backwater elevation	Where it is practicable
A flap gate shall be placed at the end of each discharge line	To prevent back flow if the centreline elevation at the end of the discharge pipe is below the design backwater elevation in the receiving structure
Connect the individual pump discharges into a common discharge header sized to direct the combined flow at an acceptable velocity in pumping main	Where excessive length and cost makes individual discharge lines impracticable
Each pump discharge line must include a check valve	to prevent recirculation of flow and applicable piping arrangement
Discharge pipe should be at least as large as the pump discharge diameter	discharge velocity range from 1.5 to 2.5 m/s
Piping Materials	See Section 7.2
Valve types	See Section 7.5 (Applicable for dry installed pumps)

Table 6-4 – Discharge Line Design Considerations

Example lifting pumping station pumping to discharge chamber is shown in Figure 6-7.

6.4.2. Intake Design

The intake structure shall be designed to supply an even distribution of flow to the pumps.

Since pump performance is dependent on conditions at the pump intake the following conditions should be avoided during design:

- Poor velocity distribution at the pump entrance
- Excessive swirling in the piping
- Air entrainment
- Inadequate $NPSH_A$
- Unstable approach conditions during multiple as well as single pump operation
- Vortices

The pump intake shall be designed according to the latest version Hydraulic Institute standard ANSI/HI 9.8 Pump Intake Design and as per manufacturer recommendations. Example configurations are shown in Figure 6-7.

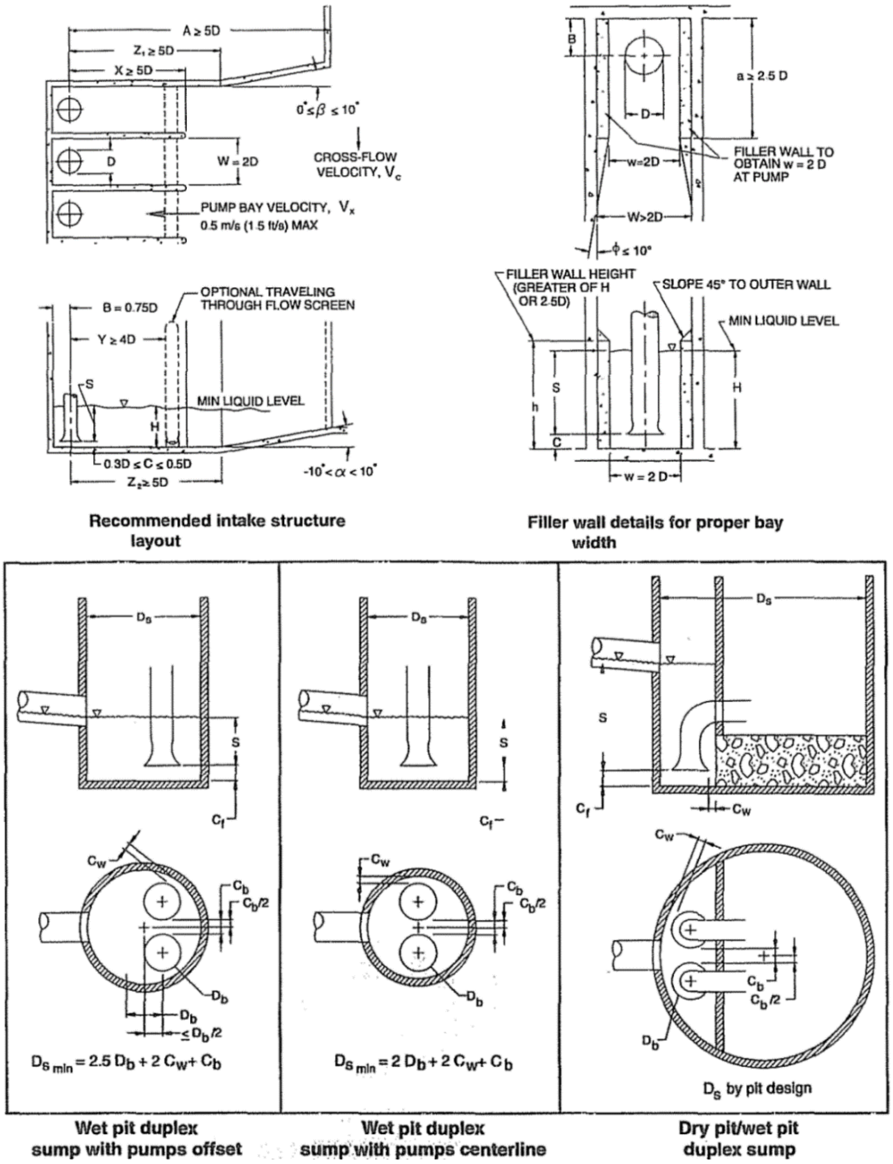


Figure 6-7 – Typical Pump station intake configurations

6.4.3. Pump Protection Screens

Removable bar screens handled through U- channel shall be provided for pump protection and shall be double layer type.

Screens shall be fitted in trash rack well or within chambers separate from wet well. In case of small wet well pumping stations with capacities in range from 10l/s to 30 l/s, screens can be installed within the wet well.

Maximum opening size of the bar screen shall be 50x50 mm. Final selection of the screens shall be based on the selected pump solid passing capacity.

6.4.4. Surge Suppression Systems

The Consultant shall determine whether surge suppression is required and where applicable employ one of the following permitted systems subject to approval of DEPARTMENT.

- Pressure vessel with bladder
- Dip-tube vessel
- Soft starters or variable frequency drives

Air valves as a method of surge control are not permitted.

6.5. Electric Design

The structure containing the electrical equipment shall be above ground and in a separate building. Due to client/special requirement, the structure can be below ground.

6.5.1. Electrical Room Air Conditioning

Minimum two split AC units working independently (mechanically and electrically) of each other should be used to air condition the room.

Each split AC should be rated at 50% above the required capacity (i.e. 150% total), so that should one unit fail, the other unit will provide 75% of the required air conditioning capacity.

6.5.2. Motors

The complete drive system must be matched to ensure compatibility.

The following items in Table 6-5shall be considered by the Consultant:

Motor type	Squirrel cage induction suitable for operation with a 400V, 3-phase, 50Hz, supply
Continuous maximum rating	At least 10% above the calculated maximum power requirements under all conditions of operation
Vertical installation	Specifically designed for vertical installation only with adequately rated end thrust bearings
Protection	IP68 class F, as a minimum
Cooling systems	Designed to withstand the maximum load at most severe climatic conditions
Variable frequency drives	Designed to be applicable for particular requirement

Table 6-5 – Motor Specification

6.5.3. Starters

The use of soft starters and VFD's are recommended to avoid surge pressures in scheduled start/stop of pumps. The following items must be considered in the design:

- Direct on-line starter is allowed for motors up to 2 kW or as per ADDC requirement.

- Soft starters are mandatory for motors above 4 kW.

6.5.4. Variable Frequency Drives

Where Variable Frequency Drives (VFD) are used the Consultant shall demonstrate the consideration of the following factors in sizing the VFD:

- Operating voltage
- Motor peak current
- Ambient temperature (site temperature)

For pumping applications the most suitable type of VFD shall give both variable torque and variable speed.

6.5.5. Motor Control Centre

Motor Control Centre (MCC) panel forms the link between the electrical loads such as motors and actuator valves, and the power generation source. The design of the MCC must take into consideration the following points in Table 6-6:

Total Connected Load	The control panel size and design needs to cover the demand of the total load connected including the standby load as well.
Short Circuit Level	The short circuit level calculation shall be carried out according to the total connected load and power source from local authority electricity network. Care must be taken in the design stage to control the fault level. If the total connected load is too high then the load to the switchgear can be split into two or more assemblies to reduce the fault level. The short circuit capacity shall be 50kA/sec or as per ADDC requirement.
Type of Coordination	Type of coordination shall be Type-2. Manufacturer tests components such as contactor, circuit breaker, all together must confirm what will happen under short circuit conditions.
Form of Internal Separation	The form of separation must be according to BS EN 61439-1 or suitable equivalent. If the MCC is located above ground, Form-2B should be considered. If the MCC is located below ground, due to client/special requirement, Form-4B,with anti-flood detector and above-ground power cut-off including shunt trip, should be considered for ease of carrying out maintenance without interruption to other equipment, in case of isolation of a particular feeder.
Bus Bar Rating	The bus bar rating must be suitable to carry the total connected load as mentioned previously, consider any future loads by increasing the size of the bus bars and suitability of extension at both ends
Type of Starter	The Consultant must consider the appropriate starter as mentioned in Section 6.5.3 and 6.5.4

Table 6-6 – Motor Control Centre Specification

The MCC, Main Distribution Board (MDB) and Distribution Board (DB) shall comply with the latest ADDC requirements.

6.5.6. Protection Device

The Consultant shall categorise the entire load connected to the switchgear according to its critical status in the process and its effect on operator safety.

Permitted protection types are as follows:

- Short circuit protection
- Overload protection
- Under/over voltage protection
- Phase losses/phase miss reversal protection
- Earth leakage protection
- Motor protection relay (electronic relay)
- Interlocking facility where required

6.5.7. Power Factor Correction Capacitors

The Consultant shall demonstrate consideration of following items in the design of Power Factor Correction Capacitors:

- PFCC shall improve the overall power factor of the plant/equipment to 0.93 lagging (or better)
- PFCC shall be designed for automatic centralised operation for site-wide Power Factor compensation by employing multiple steps
- The enclosure shall be of equal height of the associated MCC and located adjacent to it or at other suitable location within the MCC room
- The PFCC enclosure shall be fitted with forced ventilation fan and louvers as necessary. The IP rating when fitted with forced ventilation must be at least IP43
- The PFCC enclosure shall be sized to accommodate an additional spare step of equal rating for future use
- The design of the PFCC shall take into account any harmonic filter installations connected to the same power distribution system so as to avoid any LC resonance with these and any upstream transformer reactance.

6.5.8. Earthing

All the metallic parts shall be earthed.

The Consultant shall specify appropriate earth connection between metallic parts separated by rubber joints.

Earthing shall be designed in accordance with ADDC regulations and achieve the following objectives:

- To maintain high operational availability of electrical installation
- To eliminate the problems often caused by undesired potential difference between different parts of an installation leading to malfunctioning, damage to installation and human lives
- To provide segregated earthing system as follows:
    - Electronic earth "Clean Earth"
    - Electrical safety earth and panels steelwork earth "Dirty Earth"
    - Telephone system earth
    - Lightning protection system earth
- The earthing system must be designed to include the following as a minimum:

- Preventing voltage discharge to earth
- Protecting persons and property
- Protection against occurrence of over voltages due to transients and spikes
- Protection against discharge of static electricity
- Protection against lightning
- Protection against stray currents
- Facilitating supervision and measurement

6.5.9. Lightning Protection

The Consultant shall demonstrate consideration of the following items in lightning protection:

- The protector shall be designed that it must neither interfere nor restrict the systems normal operation.
- Protection shall be rated for a peak discharge current of no less than 10kA (8/20-microsecond waveform)
- The protector shall limit the transient voltage to below equipment susceptibility levels. In general, the peak transients let-through voltage must not exceed 600V for protectors with a nominal working voltage of 230V
- The peak transient let-through voltage shall not be exceeded for all combinations of conductors (Phase to Neutral / Phase to Earth / Neutral to Earth)
- Lightning system shall consist of Air terminal system, Downstream conductors with test links, and Earth terminal system

6.5.10. Lighting

Lighting installations shall be designed to provide the illumination levels to suit the site orientation are shown in Table 6-7.

General lighting	in accordance CIBSE or other national approved standard measured at a plane 1,000mm above finished floor level
Emergency lighting	in accordance with BS5266 or equivalent At least 30% of all lighting fittings shall be emergency light fittings Wall-mounted twin lamp units (with battery back-up) arranged to give at least 4 hours illumination in the event of mains power supply failure
Switching	Two-way switching must be provided for areas where access may be gained via two physically separate doors

Table 6-7 – Lighting Design

6.5.11. Uninterruptible Power Supply

The Consultant shall demonstrate consideration of the following in the design of Uninterruptible Power Supply (UPS):

- The UPS shall be designed and manufactured “standalone” for automatic operation
- The UPS shall be designed to supply clean, uninterrupted power to the loads
- The design of UPS shall take account of continuous load capacity, de-rating for non-linear loads and ambient temperature conditions

- The UPS shall be sized to supply all instrumentation and control equipment, including but not limited to, control and computer room hardware, mimic, annunciation, telecommunication systems and emergency lighting etc., where applicable.

6.5.12. Cables

The Consultant shall demonstrate consideration of the following items in the design of cables:

- All LV cables shall be 600/1000V-grade copper single/multi cores
- Where required a neutral conductor shall be in the form of a core of the same section as the other cores - separate neutral cables are not permitted
- Each cable is of sufficient rating for its normal and fault conditions
- Cables shall be sized considering following:
    - Laid in ground/air/duct
    - Depth of laying for cables laid direct in the ground/duct
    - Temperature of the ground/air
    - Group rating factors
    - Thermal resistivity of soil (for cable laid direct in ground/duct)
    - Allowable voltage drops
- Earthing conductor shall be of adequate cross-sectional area and be either one core of a multicore cable or a separately run single-core cable. The use of conduit and water pipes in any part of the earth continuity conductor is not permitted.
- General routing of cables shall be indicated on the drawings and the final routes and duct locations agreed with DEPARTMENT.

6.5.13. Mobile Generator Junction Box / Stand-by Generator with ATS

A mobile generator junction box shall be provided for mobile generator connection in case of mains power supply failure.

A stationary generator shall be provided for tunnel lifting pumping station.

6.6. Instrumentation

6.6.1. Level Measurement Sensors

Summary of permitted level sensor applications is shown in Table 6-8.

Instrument Type	Wet Well	Closed or Pressure Vessel	Field Indication	Remote Indication	Point Level	Continuous Measurement	Anti-flood Detector
Float	✓			✓	✓		
Conductive Electrode				✓	✓		✓
Vibrating Reed				✓	✓		✓
Magnetic Float		✓		✓		✓	
Ultrasonic	✓	✓	✓	✓		✓	
Radar time of reflection		✓	✓	✓		✓	
Hydrostatic pressure	✓	✓	✓	✓		✓	
Site glass with magnetic flaps		✓	✓	Possible if fitted with transmitter		✓	
Counter weight & float mechanical site gauge		✓	✓	Possible if fitted with transmitter		✓	

Table 6-8 - Level Sensor Application

6.6.2. Methods of Flow Measurement

Summary of permitted methods of flow measurement is shown in Table 6-9. Consultant is responsible for correct application of the method to the site conditions.

Instrument type	Open Channel	Closed Conduit (gravity/non-full bore)	Closed Conduit (pressurized)	Field Indication	Remote Indication	Point Level	Continuous Measurement
			✓	✓	✓		✓
Ultrasonic Doppler Effect (continuous/ pulse)	✓	✓	✓	✓	✓		✓
Ultrasonic Transit Effect	✓		✓	✓	✓		✓
Electromagnetic	✓	✓	✓	✓	✓		✓
Cross Correlation	✓	✓	✓	✓			✓
Paddle Switch			✓	✓	✓	✓	
Radar Doppler Effect	✓	✓		✓	✓		✓
Weir	✓			✓	✓		✓
Flume	✓			✓	✓		✓

Table 6-9 – Methods of Flow Measurement

Pressure Measurement

Summary of permitted pressure measurement applications is shown in Table 6-10.

Instrument type	Field Indication	Remote Indication	Point Level	Continuous Measurement
Pressure Gauge	✓			
Pressure Switch	✓	✓	✓	
Pressure Transmitter	✓	✓		✓

Table 6-10 - Pressure Sensor Application

6.6.3. Compatibility to SCADA

All instruments providing remote indication, i.e. instruments with transmitters shall be able to communicate with a central control system via one of the following communication protocols:

- HART
- Profibus
- Foundation Fieldbus
- Modbus

The appropriate protocol shall be determined by the Consultant based on the existing control system communications capability for refurbishment or extension projects or suitable interface with DEPARTMENT existing SCADA systems for new projects.

6.6.4. Control Systems

Refer to the Electro-mechanical Specification Section 15000, Part 8 (Control Philosophy for Pumping Stations).

6.6.5. Pump Control Philosophy

The Consultant shall establish the type of control required for the pumping station. Pump control shall be based on level in the wet well. Where deviation from this philosophy is deemed necessary, prior approval shall be obtained from DEPARTMENT.

The following minimum interlocks are necessary for safe operation of the pump is shown in Table 6-11:

Protection	Detection Method
Dry run	Wet well low level, via level instrument in the suction Wet well Suction pressure low level. Non-return valve not opened, via proximity or limit switch No discharge flow zero, via flow meter or switch on pump discharge pipe
Blocked discharge or closed discharge valve	Non-return valve not opened, via proximity or limit switch No discharge flow zero, via flow meter or switch on pump discharge pipe
Pump/Pump Motor Bearing Temperature	Temperature sensors such as PT 100 embedded in the bearing housing
Motor Winding Temperature	Temperature sensors such as PT 100 embedded in the winding
Pump/Pump Motor vibration	Vibration sensors such as accelerometers embedded in the pump/motor casing or bearing housing, depending on where the measurement is to be taken or noise sensor fixed at the pump house.

Table 6-11 – Pump Protection Measures:

6.6.6. Fieldbuses

Fieldbuses shall be designed in accordance with IEC 61158 Industrial communication networks - Fieldbus specifications.

6.6.7. SCADA System

Each pumping station location shall be provided with a Programmable Logic Controller (PLC) and Remote Terminal unit (RTU). Both PLC and RTU shall be installed within the same cubicle. The cubicle door shall have a door mounted HMI which shall be connected to the PLC and RTU via Ethernet switches.

The PLC and RTU shall be programmed through HMI. PLC, RTU Instrumentation shall be connected with battery back-up.PLC and RTU shall be programed with the following functions:

1 PLC back-up to RTU
2 Float switches back-up to level transmitter
3 Level/float control selection
4 Pump duty selection
5 Dry run float switch prevent pumps to start and shall reset control cycle
6 Power supply OFF/ON shall not trip pumps
7 Pumps start/stop delay shall be applicable
8 Duty pump shall be changed to other if pump running time exceeds set point

In normal operation the RTU shall control the station. Where the RTU fails the PLC shall take over control seamlessly.

The cubicle shall be fully assembled by one system integrator.

The IO signals to/from the PLC and RTU must be connected to the field equipment via signal splitters.

The PLC and RTU design shall be of modular construction. Single board PLC and RTUs are permitted where the IO counts are low.

The signal splitters, PLC and RTU IO terminal blocksand wiring shall be provided with 25% spare IO capacity above the design requirement.

Minimum requirements of the PLC and RTU are shown in Table 6-12.

Minimum PLC and RTU Requirements	
1	Data acquisition from field equipment (digital and analogue)
2	Data recording and tagging (time and date)
3	Sequence of events recording
4	Alarm management (prioritise) including reporting by exception
5	Handling data received from control centres
6	Data processing (including math outputs and software routines functions)
7	Initiating commands (digital and/or analogue outputs) and software routines (pump sequence control)
8	Self-diagnostics
9	Communications and control including route selection
10	Programming and diagnostics via control centre, and RTU configuration and maintenance units
11	An intelligent, rule-based adaptive system is required to minimise false alarms and nuisance alarms
12	Compliant and supporting with DNP3 communication protocol

Table 6-12 – Minimum PLC and RTU Requirements

Functional design specification shall be submitted with the following:

i. System architecture
ii. P & ID drawing
iii. Inputs/outputs list, physical and internal signals
iv. Alarms priority, triggered / buffered alarms
v. Operation philosophy
vi. Control set points
vii. Test procedure, materials check list

The facility for communication between the pumping station and SCADA system shall be by GPRS/3G telephone line or fibre optic cable, based on the availability of service in the area

SCADA hardware and software requirements are set out in detail in General Specifications.

6.7. Structural Design

All structures shall be designed to the required standards of ACI 318 – “Building Code Requirements for Structural Concrete” or equivalent.

7. PUMPING MAIN DESIGN

7.1. Introduction

The required layout of all pressure mains will depend on local circumstances. The Consultant shall demonstrate due consideration of the factors in Table 7-1 in the design of pumping main.

Pumping Main Design Factors	
1	Reliability of supply
2	Good access for maintenance
3	Provision and location of line valves, air valves, washouts and flow meters
4	Adverse ground conditions and difficult terrain, road levels
5	Risk of damage to and from trees, tree roots and other utilities.
6	Pipe materials and corrosion protection systems in aggressive / contaminated soils and other utilities
7	Minimum gradient is 1:500 as per the recommendation of BS EN 805
8	Utility corridors
9	Crossing of roads and railways to be 90 degrees
10	Adoption of shortest practical route
11	Location of other services, buildings and structures
12	Telemetry, control and metering
13	All design pressures
14	Earth loads
15	Traffic loads
16	Ease of operation and maintenance
17	National and local planning, environmental protection
18	Risk of damage to and from other utilities, works and apparatus
19	For buried pipes the minimum depth of cover
20	The maximum depth of cover for ease of repair

Table 7-1 - Pumping Main Design Factors

7.2. Pipe Material Selection

Piping for storm water pumping mains shall be ductile iron, GRP or HDPE and the pipework within valve chambers and pumping stations is to be ductile iron. The selection of the material is to be based on consideration of the:

- operating pressure
- surge pressure alleviation
- economics
- delivery time
- resilience

In addition to above for Projects in Al Ain Region, uPVC pipe materials can also be considered as an alternate material.

7.3. Hydraulic Design of Pumping Mains

7.3.1. Design Considerations

The Consultant shall perform hydraulic calculations in order to demonstrate that the system will:

- Deliver the required flow
- Operate at the accepted velocities defined below
- Operate within the required pressure range of the pipes and fittings

In addition, the design pressure and the maximum design pressure must be established at appropriate points in the system.

Pipe diameter shall be such that velocity will be in the range of 1 m/s at minimum flow and 2.0 m/s under peak flow. A maximum velocity of 2.5 m/s in emergency scenarios, for example, to avoid flooding where additional standby pumps are brought online.

Single pump discharge pipes shall be sized such that the discharge velocities in these pipes will be in the range of 1.5 to 2.5 m/s and suction headers shall be sized to achieve a velocity in the range of 0.8 to 1.5 m/s.

The use of twin mains should be considered on a case-by-case basis. The main factors for consideration include design elements, the risk assessment and cost benefit analysis.

Consideration for design elements comprise the:

- rate of build-up of flow
- range of flow conditions
- range of velocities in mains
- availability of land for twin mains and associated chambers
- added complexity in pumping operation
- O&M requirements and confirmation.

A thorough risk assessment should be carried out, which should include the likelihood of a main bursting, the consequences of failure, areas affected, sensitive receivers affected and the feasibility of temporary diversions.

The cost benefit analysis should include all tangible factors (such as cost of pipework, land costs, energy cost) and intangible factors (such as nuisance).

Where twin mains are found to be required, which may not necessarily be the same diameter; it is advisable to use both mains as duty rather than one as duty and the other as standby from an economical and operational point of view.

7.3.2. Friction Head Losses

Friction head loss is the loss of pressure caused by water flowing through the pipe in a system. Flow in pipes are usually turbulent and the roughness of the inside walls of pipes have a direct effect on the amount of friction loss. Turbulence increases and consequently friction loss increases with the degree of roughness. Friction losses are dependent on the flow rate, the pipeline diameter, roughness, and length. The commonly used formulas for computation of head loss due to friction losses are:

- Hazen-William's formula
- Darcy-Weisbach formula

Hazen-Williams and Darcy-Weisbach are most commonly used and result in a good degree of accuracy. In metric terms, Hazen Williams' formula is given below:

$$h_f = 10.67 \text{ x } L \text{ x} \frac{Q^{1.85}}{C^{1.85} \cdot D^{4.87}}$$

Where:

hf = Friction head losses of the pipe in m

L = Length of the pipe in m

Q = flow through the pipe in m^3/s

D = Inside diameter of the pipe in m

C = Coefficient depending on the pipe diameter, dimensionless as shown in Table 7-2below.

Pipe diameters	**Hazen Williams "C" Coefficient**
Pipe diameters > 500mm	140
Pipe diameters < 500mm	135

Table 7-2 – Hazen Williams C Coefficients for Types of Pipe

In metric terms, the Darcy-Weisbach formula is:

$$\boldsymbol{h_f} = \frac{8\, f\, L\, Q^2}{g\, \pi^2 D^5}$$

Where:

hf: Friction head losses of the pipe in m

f = Friction factor, dimensionless

L = Length of the pipe in m

Q = flow through the pipe in m^3/s

G = gravity acceleration = 9.81 m/s^2

D = Inside diameter of the pipe in m

The friction factor, in its more general expression, is calculated from the Colebrook-White equation:

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left(\frac{K_s}{3.7D_h} + \frac{2.51}{R_e\sqrt{f}}\right)$$

Where:

R_e = Reynolds Number, dimensionless

K_s = Roughness coefficient in mm, as shown below in Table 7-3.

Flow Velocity (m/sec)	**Colebrook-White, K_s (mm)**
From 1.1 m/sec to 1.8 m/sec	0.15mm
Less than 1.1 m/sec	0.3mm

Table 7-3 –K_s Coefficients for velocity of flow

7.3.3. Localised Head Loses

Head losses also occur at valves, tees, bends, and other appurtenances within the piping system. These losses, called localised head losses or minor head losses, are calculated using the following equation:

$$h_L = K\frac{V^2}{2g} = K\frac{8\,Q^2}{g\,\pi^2 D^4}$$

Where:
h_L = Localised head losses of the pipe in m
K = Resistance coefficient, dimensionless, shown in Table 7-4below
V = Velocity in the pipe in m/s
Q = flow through the pipe in m^3/s
g = gravity acceleration = 9,81 m/s^2
D = Inside diameter of the pipe in m

Type of Fitting	**K Value**
Entrances	
Standard bell mouth	0.1
Pipe flush with entrance	1
Pipe protruding	1.5
Sluice gated or square entrance	1.5
Bends 90°	
Medium radius (R/D = 2 or 3)	0.5
Medium radius (mitred)	0.8
Elbow or sharp angled	1.5
Bends 45°	
Medium radius (R/D = 2 or 3)	0.25
Medium radius (mitred)	0.4
Elbow or sharp angled	0.75
Tee 90°	
In line flow	0.4
Branch to line or reverse	1.5
Contraction - Sudden	
D2/D1 = 0.8	0.18
D2/D1 = 0.5	0.37
D2/D1 = 0.2	0.49
Contraction - Conical	
D2/D1 = 0.8	0.05

Type of Fitting	K Value
D2/D1 = 0.5	0.07
D2/D1 = 0.2	0.08
Expansion - Sudden	
D2/D1 = 0.8	0.16
D2/D1 = 0.5	0.57
D2/D1 = 0.2	0.92
Expansion - Conical	
D2/D1 = 0.8	0.03
D2/D1 = 0.5	0.08
D2/D1 = 0.2	0.13
Valves	
Gate valve fully open	0.25
Gate valve ¾ open	1
Gate valve ½ open	5.6
Gate valve ¼ open	24
Check valve fully open	4.3

Note that for valves it is advisable to obtain manufacturers data on head losses. System head calculations would normally be carried out using valve open figures

Table 7-4–K Values for Type of Fittings

7.4. Structural Design

7.4.1. Internal Forces

In accordance to BS EN 805 standard, pipelines shall be designed to withstand a transient pressure of 80 kPa below atmospheric pressure (approximately 20 kPa absolute pressure).

7.4.2. Temperature Range

Pressure mains shall be designed for continuous operation over the anticipated temperature range of 10 – 50°C. Note that de-rating factors will need to be applied to flexible pipes for the high temperature.

7.4.3. Unbalanced Thrust

Unbalanced forces (forces at valves, changes in direction, etc.) shall be compensated by an adequate number of restrained joints, thrust blocks or other anchorages. Where thrust blocks are to bear against the soil the safe bearing pressure shall be determined.

Blocks shall take the form of a cradle wedged against the undisturbed trench side and design based on the safe bearing pressure of the ground. Minimum safety factors are shown in Table 7-5.

minimum friction safety factor	1.5
minimum sliding safety factor	2.0
minimum overturning safety factor	2.0

Table 7-5 – Minimum Thrust Safety Factors

7.5. Valves

7.5.1. Valve Selection

Manufacturer data must be considered when selecting and sizing valves. The following general items must be considered in selecting valves:

- Temperature: The valve bodies, trim, and operating parts must be capable of withstanding the highest temperature expected during sustained normal and transient operating conditions
- Pressure: The valve must be rated for the highest transient pressure that might be expected
- Shutoff: The degree of allowable shutoff must be known. No leakage must be allowed.
- Valve operation: It must be determined whether the valve be used only for ON/OFF use or for throttling
- Pressure drop: Allowable pressure drop must be established and the size selected
- Velocity: The velocity of the fluid through the valve must be considered to avoid cavitation in any operating condition

7.5.2. Valve Losses

There may be occasions where precise determination of the pressure drop through valve (check valve or flap valve) shall be required. This is done by using the standard measure of valve flow, the coefficients Kv or Cv. These coefficients are determined by the valve manufacturer using actual flow tests. With the Kv or Cv known, the pressure differential can be found.

Kv value is defined as the rate of flow of water in m^3/hr in temperature range from 5°C to 30°C at a pressure drop of 1 bar across the valve. Thus for a given Kv and Q the pressure drop across a valve is calculated as:

$$\Delta P = \left(\frac{Q}{K_v}\right)^2$$

Where:

Kv = valve coefficient

Q = flow through the valve in m3/h

ΔP = pressure drop in bar

Cv value is defined as the rate of flow of water in US gallons per Minute at 15°C at a pressure drop of 1 psi across the equipment. Thus, for a given C_v, the K_v value is calculated as:

Kv = 0.86488 Cv

7.5.3. Flushing Flow

The flushing flow (qf) for scouring and cleaning can be calculated by the following formula:

$$q_f = 0.012524 \; x \; d^2 \sqrt{\frac{10.195 \; x \; MDP}{K}}$$

Where:

qf = flushing flow, in m^3/h

d = inside diameter of drainage pipe, in mm

MDP = maximum design pressure, in bar

K = Sum of individual K-factors for the components of the drainage pipework, dimensionless

7.5.4. Maximum Draining Flow

The maximum draining flow (qdr), which must be used to size air valves for pipeline draining, can be calculated by the following formula:

$$q_{dr} = 0.012524 \; x \; d^2 \sqrt{\frac{\boldsymbol{h_1} - \boldsymbol{h_2} - 10.195 \; x \; \boldsymbol{\Delta}P}{K}}$$

Where:

qdr : maximum draining flow, in m^3/h

d = inside diameter of drainage pipe, in mm

h_1 = head of water above drain pipe at start of discharge, in m

h_2 = head of water above drain pipe at end of discharge, in m

ΔP = differential pressure, in bar

K = Sum of individual K-factors for the components of the drainage pipework, dimensionless

7.5.5. Draining Time

Sufficient washouts must be provided to limit the draining time of the section affected to a maximum of 4 h (14,400 s). The draining time can be obtained applying the Torricelli's formula as follows=

$$t = \frac{2 \; x \; A \; x \; \sqrt{K}}{a \; x \; \sqrt{2g}} \; x \; \left(\sqrt{\boldsymbol{h_1} - 10.195 \; x \; \boldsymbol{\Delta}P} - \sqrt{\boldsymbol{h_2}}\right)$$

Where:

t = draining time, in s

A = Surface area of the water in the pipeline, in m^2

K = Sum of individual K-factors for the components of the pipework, dimensionless

a = Cross sectional area of the drainage pipework, in m^2

g = gravity constant = 9.81 m/s^2

h_1 = head of water above drain pipe at start of discharge, in m

h_2 = head of water above drain pipe at end of discharge, in m

ΔP = differential pressure, in bar

The water surface takes the form of an ellipse during draining, which minor axis is equal to pipe diameter. The pipe slope affects the major axis. The surface area (A) is computed as

$$A = \pi \frac{D^2}{4 \; x \sin \delta}$$

Where:

A = Surface area of the water in the pipeline, in m^2

D = inside diameter of main pipeline, in m

δ = angle of slope, in degrees

7.5.6. Actuators

There are three operating methods for valve actuators: multi-turn (used for gate, globe, and diaphragm valves), quarter-turn (used for ball, and butterfly valves), and linear (used for gate, diaphragm, and globe valves).

Manual or power actuated valve decision criteria are shown in Table 7-6.

Manual Operation	Where valve is easily accessible does not require automatic operation operated infrequently valves with a size ≥ DN 350 shall be provided with a gearbox
Power Actuators	where valves are remotely located frequent operation is required automatic operation is necessary due to system considerations power actuators shall be provided for isolation valves in pumping stations power source must be capable of exceeding the torque required by the actuator by an adequate safety factor

Table 7-6 – Manual vs Power Operated Valves

7.5.7. Equipment Isolation Valves

Gate valves shall be specified for equipment isolation. In line equipment shall have isolating valves upstream and downstream. Off-line equipment require only one isolating valve located upstream.

For small diameter pipes, e.g. on instrumentation, ball valves are permitted, but are not intended for controlling the rate of flow or pressure of water.

Wet well pumps, tubewell pumps and self-priming pumps shall have isolating valves at the discharge side.

7.5.8. Check and Flap Valves

- Check valves shall be specified at the discharge of each pump and in surge vessels fitted with bypass.
- Flap valves, flexible duckbill check valve or cone type check valveshall be provided at discharge chambers and outfalls.
- Check valves:
  - Are recommended at outlet (partially buried) where sediment may prevent Flap Gates from functioning, as they will operate with higher efficiency.
  - Are recommended to be used at the pipe outlet inside a manhole with limited space for flap gates operation

- May be used in line, end of pipe and or pipe outlet subject to backflow
- Could be installed on vertical pipes
- Operation must satisfy the maximum opening pressure, minimum closing pressure, and maximum back pressure on adequate to the design of the network.
- Must have a wide range of opening pressure (recommended 200 to 450 mm), Closing pressure (recommended (60mm to 250 mm), and Maximum Back pressure (recommended 5 m to 8 m of water)
- Pressure drop (losses), and the velocity must be checked/adequate to meet the design requirements
- Must be Completely tight to prevent backflow
- Must have durable membrane
- Must have minimum mechanical and moving parts
- Must be easy installation and maintenance
- Must be Self-cleansing
- Must be wear resistant
- Must have a low operation and maintenance cost.
- Check valves shall avoid slamming after flow stoppage
- Installation of check valves in vertical position is not permitted.

7.5.9. Air Release and Vacuum Valves

Air valves shall be specified at 1.0km intervals and the following locations:

- At any pipeline high point
- in pump discharge lines

Permitted air valve sizes are shown in Table 7-7.

Main line size (DN)(mm)	Air valve size (DN)(mm)
150 – 250	80
300 – 500	100
600 – 900	150
1000 – 1200	200
1200 - 1800	250 or 2 x 200

Table 7-7 - Air Valve Nominal Sizes v Main (DN)

Air valves shall be specified to release the air at maximum design pressure.

7.5.10. Washout Valves

Washout valves shall be specified at 1.0km intervals and the following locations:

- at pipeline low points to provide complete draining of the pipeline
- at other locations for maintenance purposes
- between 2 air valves

Washouts shall be specified together with air valves, according the following parameters:

- In general, washout valves must be gate valves
- Maximum velocity in the drain pipe = 4.5 m/s. Wear valves or orifices plates must be provided to limit the velocity in the pipework, if necessary
- Washouts must be designed to achieve a minimum velocity of 0.75 m/s in the main pipe during flushing

- Maximum time to drain the section of the pipeline affected by the washout must be 4 h (14,400 s)
- Air valves must be designed for pipeline draining, according to the maximum draining flow of the washout

Permitted washout valve sizes are shown in Table 7-8.

Main line size (DN)(mm)	Washout valve size (DN)(mm)
150 – 200	80
250 – 300	100
400 – 500	150
600 – 1000	200
1000 – 1200	250
1400 - 1800	300

Table 7-8 – Washout Nominal Sizes vs Main (DN)

7.6. Valve Chambers

Valve chambers shall be designed in accordance with Volume 2: Standard Drawings.

Design criteria are shown in Table 7-9.

Valve Chamber Design Criteria	
1	Siting of chambers in carriage ways shall be avoided; preferably in the road verge or in the footway, utility corridors must be respected
2	All chambers shall be designed of reinforced concrete
3	The cover level of the valve chambers shall be flushed with surrounding finished ground level unless otherwise specified.
4	The bottom of the chamber shall have a minimum slope of 2%
5	The bottom of the chamber shall have suitable sump holes covered by GRP grating cover.
6	A GRP ladder for access shall be provided in chambers of more than 3.0 m of height. In chambers <3.0m deep a GRP ladder or step irons can be provided.
7	Two flexible joints, with a "rocker pipe" shall be provided on either side of the chamber to avoid damaging pipework in case of differential settlement. Rocker pipes are not required for HDPE solid wall piping.
8	Sufficient working space and clearances inside valve chambers, proper access arrangements and gravity ventilation by employing vent pipes shall be specified
9	A minimum clearance of 500 mm from the walls of the chamber to the equipment shall be provided
10	A minimum clearance of 500 mm from the walls of the chamber to the joints shall be provided.
11	A minimum clearance of 400 mm from the floor of the chamber to the invert level of the pipe shall be provided.
12	All chambers must have a removable cover with lifting hooks for easy installation and repair. The chamber construction and cover shall facilitate the lifting of equipment.
13	In cases where non-restrained pipe systems are used the chamber shall be designed to take the full thrust when the valves are closed. In such cases pipework must be fixed to the chamber walls by means of puddle flanges.
14	All the equipment shall be easily dismantled
15	All the equipment must be properly fixed and supported
16	Flood detector and gas monitors shall be installed as necessary

Table 7-9 – Valve Chamber Design Criteria

7.6.1. Flow Meter Chamber

Equipment that may create flow disturbances, such as isolation valves or reducers, shall not be placed next to flow meters.

Sufficient strait run downstream and upstream of flowmeter shall be designed in accordance to manufacturer recommendations. For typical piping arrangement and details refer to flow meter chamber standard drawings.

7.6.2. Air Valve Chamber

Air valve chambers shall be composed by a gate valve double function air valve with automatic air release and pipework.

The gate valve shall incorporate mitre gearing and vertical extension to suit below cover level so as to allow operation from ground level. For typical piping arrangement and details refer to air valve chamber standard drawings.

7.6.3. Washout chamber

The washout concrete chamber shall consist of dry and wet chamber.

The drain pipe with blind flange shall terminate in the wet chamber. For typical piping arrangement and details refer to washout valve chamber standard drawings.

8. DESIGN SUBMISSION REQUIREMENTS

8.1. General

These submission requirements are related to Stormwater and Subsurface Drainage Designs and it also covers the connection to existing DEPARTMENT stormwater network submission. **This document supersedes “Appendix-C” in the “Consultant Procedure Manual (Version 2.0 April 2014)** Consultants are required to follow these requirements for any design approval required from DEPARTMENT. For Stormwater and subsurface drainage projects owned by developers, DEPARTMENT design approval does not guarantee the acceptance of the DEPARTMENT for taking over the Stormwater and Subsurface Drainage assets for Operation and Maintenance after construction. This will also require a MOU between the developer and Management of Emirate of Abu Dhabi.

8.2. Required Standards

All stormwater and subsurface drainage designs must comply with the latest Abu Dhabi Emirate Drainage Design Manual and Standards, Technical Specifications and Standard Drawings. Attention is drawn to the following documents in particular and it is up to the consultant to ensure that they understand the requirements of these documents:

“STORMWATER & SUB-SOIL DRAINAGE SYSTEMS DESIGN Manual, - Emirate of Abu Dhabi

It should be noted that the above mentioned Manual is under continuous updating to ensure their compatibility with the latest UAE and international standards. It is the responsibility of the Design Consultant to confirm obtaining the latest version of the Manual prior to start any design works.

All stormwater and subsurface drainage designs and connection to existing submittals must be addressed with an Arabic cover letter to:

السيد المهندس/ .... ‘‘

مدير إدارة الدعم الفني للبنية التحتية

دائرة البلديات و النقل

بلدية مدينة أبوظبي

ص.ب 263

أبو ظبي ، دولة الامارات العربية المتحدة

8.3. Submission Requirements

The following requirements are applicable to the preliminary design and the detailed design submission:

1. Check list as per section 7 below must be submitted. Any submittal not containing the relevant check list will be returned to the consultant.
2. One complete set – Hard Copies (If requested only) - of the design package produced in sufficient details and readable scale to facilitate a comprehensive and prompt review (A4 for Report and A3 size for drawings is preferred).
3. One soft copy of:
   - Report and Drawings. MS word and PDF/CAD (if requested) format or compatible.
   - Technical design calculation sheets with visible formulas. MS Excel format or compatible.
   - Hydraulic modelling files and Shape files as per checklist including input and output data sheets and drawings presenting water surface and HGL profile for all pipelines. These shall be submitted with a format that is stated in Section - 3.4.2.
4. A presentation to DEPARTMENT might be required during the review process to address particular issues. MS PowerPoint format or compatible to it.

8.4. Design Review Stages and Requirements

8.4.1. General Notes

1. Submissions can only be accepted through qualified Stormwater drainage Consultant. No design submittals by Contractors or non-qualified Stormwater drainage Consultant can be acceptable.
2. Master Plans and/or Concept Designs approval shall be obtained and reviewed by DEPARTMENT.
3. Environment Agency - Abu Dhabi (EAD) approval shall be obtained to confirm that designs meet the allowable discharge limits as set out in the EAD “Technical Guidance Document”. For the discharge to Marine Environment. A specific approval related to outfall discharging to marine water (Sea/Canal) is required from EAD clearing mentioning outfall location during the design stage (Not to be confused with Environmental Permit Application).
4. The applicant shall not incorporate in his designs any technology which has not been approved in advance by DEPARTMENT.
5. All Drawings must be in PDF or CAD (if requested) -produced at readable scale with the Key Plan and Legend.
6. Approvals of Preliminary Designs cannot be granted on a conditional basis pending the fulfillment of certain requirements.
7. All submittals must be complete and self-explanatory and must not be based on a submittal previously commented and/or rejected by DEPARTMENT. Approvals of Detailed Designs are always conditional to the implementation of complete DEPARTMENT specifications and approved materials list during execution.
8. DEPARTMENT shall be contacted before making any changes to the approved designs. An approval of these changes must be obtained from DEPARTMENT.

8.4.2. Preliminary Design

The Preliminary Design Package shall include but not limited to the following:

a. Copies of:
   i. Assignment Letter of the Client.
   ii. Approval of DPM of the Master Plans and Concept Design.
   iii. Environment Agency - Abu Dhabi (EAD) approval (Environmental Permit for overall project and Specific approval for outfall to discharge to marine environment)
   iv. Approval of connection from DEPARTMENTADM Design Section and Operation and Maintenance Section refer to section 5 of this document.
b. Project Vision Statement.
c. Project site location plan showing existing site boundary, access roads and primary infrastructure facilities serving the site.
d. Site survey and site context including existing utilities / services, access and connectivity, surrounding land use.
e. Topography survey to be submitted in digital (X, Y, Z) AutoCAD and Excel format.
f. Geotechnical investigations report/results (soil, rock, water tests and groundwater conditions, infiltration rates etc.).
g. Site analysis to identify opportunities as well as – technical and design constraints.
h. Full and detailed description of design criteria adopted for all project components including pumping stations.
i. Designs including the following:
   a. Proposed site plans and grading plans.
   b. Proposed network layouts. Layouts containing stormwater and subsoil drainage network should be presented separately.
   c. Proposed catchment area plan highlighting sub catchments and any catchments contributing or expected to contribute to the proposed network.
   d. Proposed infrastructure layouts plan for gravity and pressure pipelines showing pump stations, outfalls, storage ponds., etc. (where applicable).
   e. Electromechanical concept design including pump curves, pump selection and details of any ancillary works (if applicable).
   f. Hydraulic model and other design calculations and technical calculation sheets (MS Excel format.
   g. Must contain as a minimum pipe diameters, invert and cover levels, slopes, and catchment area with respective runoff coefficients or Curve Number, time of concentration, rainfall intensities, full and partial pipe flows and velocities. Hydraulic calculations shall be prepared based on Storm events as specified in the Design Manual (refer to section 2 above).
   h. Electromechanical Design to include System Curve for Parallel Operation Scenarios and Sump operation levels as well. and surge analysis for pumps operation
   i. Hydraulic modelling for the proposed network as specified using the mentioned software (refer to section 3.0 above).
   j. Hydrology report shall be prepared and submitted for the for urban and rural highways/expressways. A watershed model shall be prepared and submitted with all detailed calculations. A layout shall be submitted showing the delineated catchment areas, location of stream, location of culverts, preliminary sizing of

culverts. Culverts shall be analyzed using international software such as HY-8 or compatible including the culver report.

k. Subsoil drainage calculation shall be based on soil parameters and groundwater conditions. The calculations must contain as a minimum pipe diameters, levels, slopes, and radius of influence (if applicable), drainage coefficients, pipe flows and velocities. Hydraulic calculations shall be prepared with reference to the Design Manual (refer to section 2 above).These calculations shall be based on actual soil parameters obtained through geotechnical investigations.
l. Full and detailed design calculation of each element of the works.
m. Sizing design calculations for any proposed structures.

j. Sections showing ROW with all details of all services corridor. Project implementation plan.
k. Operations and Maintenance strategy.
l. Cost assumption breakdown for Operation and Maintenance.
m. Construction cost estimation breakdown.
n. Detailed calculation of Net Present Value, NPV, for all proposed Design alternatives and/ or Options.
o. Statement of status of all Authority Approvals including any pending NOI / NOC.
p. Compliance Form to be signed and stamped by the Consultant stating full compliance in his submittal to the latest DEPARTMENT manuals, specifications and standards and noting any non-conforming elements with justification.(refer to section 6.0 of this document).
q. Other details as applicable and requested by DEPARTMENT.

8.4.3. Detailed Design

The submitted Detailed Design Package shall include the following:

a. Copies of:
   a. Approvals of all authorities related to the project scope including DEPARTMENT Preliminary Design approval.
   b. Statements: the status of all pending NOI / NOC (if any).
   c. Approval of Service Reservation for Stormwater pipeline from the Town Planning Department.
b. Detailed Design Report with full and detailed design updated calculations for all products, materials, plant, and equipment in accordance with the scope of work. Final hydraulic model shall also be included. To be submitted as described in section 3 above.
c. All applicable design drawings including:
   i. Drawings list.
   ii. Project location Plan.
   iii. General site layout.
   iv. Detailed layout Plans showing all stormwater and subsurface drainage systems.
   v. Detailed profile drawings showing all other utilities crossing the storm water.
   vi. Structural layouts and details for the stormwater pumping station including site layout, approved site plan from DPM and Town Planning department.
   vii. Structural design of manholes (where applicable), outfalls, culverts with final detailed calculations,
   viii. Electro-mechanical layouts and details.
   ix. Surge analysis report and calculations,

x. Standard drawings.

d. All applicable standard specifications.
e. All particular specifications.
f. Bill of Quantities for the works.
g. Detailed breakdown of the estimated construction cost.
h. Detailed Operation & Maintenance strategy with associated programme of activities / tasks.
i. Operations and Maintenance budget estimate and breakdown covering 10-year period from final handover (FAC / Final TOC).
j. All other necessary details as applicable.
k. Compliance Form to be signed and stamped by the Consultant stating full compliance in his submittal to the latest DEPARTMENT manuals, specifications and standards and noting any non-conforming elements with justification.(refer to section 6.0 of this document).

8.5. Connection to Existing Approval

The following requirements are applicable to the connection to DEPARTMENT existing network submission:

a. Two complete sets of the concept design package (as approved by DPM) produced in sufficient details and readable scale to check the possibility to connect to the proposed connection point (exiting DEPARTMENT Manhole). This package shall include:
   i. General catchment area with the proposed layouts.
   ii. Description of criteria adopted to calculate flows produced from the project catchments.
   iii. Calculated flows at the connection point/(s). Flow calculation should be based on methods provided in Design Manual, and it should also show peak flow for desired return period.
   iv. General hydraulic calculation to confirm the possibility of the gravity network to connect at the invert level of the existing manholes.
   v. Any other details can facilitate the review and approval of the connection to existing.
   vi. Shape file for the catchment
b. A presentation to DEPARTMENT might be required as well as coordination with DEPARTMENT Operation and Maintenance section during the review process to address particular issues.
c. DEPARTMENT approval for the connection to existing is always conditional to the submission of the preliminary and detailed design and obtains DEPARTMENT approval on the submittals.

8.6. Compliance form

DETAILS OF THE CONSUTANT MAKING THE APPLICATION:

NAME OF CONSULTANT: ……………..……..…...……………………………………………………..…

FULL ADRESS: ……………...………..…...……………………………………………….……………

TELEPHONE NO: ……………………………………………………………………………………

DETAILS OF THE PROJECT:

PROJECT TITLE: ……………………………………….………………………………………………

PROJECT OWNER……………………………………………………………………..……………………

SUBMISSION STAGE: ……………………………………………………………………………….

WE HEREBY CERTIFY THAT THE STATEMENTS AND INFORMATION IN ALL FORMS, REPORTS, DOCUMENTS, AND ATTACHMENTS SUBMITTED TO DEPARTMENT ARE TRUE, ACCURATE AND COMPLETE. WE ALSO CERTIFY THAT ALL SUBMITTED DESIGNS ARE FULLY COMPLYING WITH DEPARTMENTMANUALS, SPECIFICATIONS AND STANDARDS.

WE NOTE THAT THE FOLLOWING LISTED ELEMENTS AND ITEMS ARE NON-CONFORMING DEPARTMENTMANUALS, SPECIFICATIONS AND STANDARDS WITH JUSTIFICATION:

………
………
………
………

(IF SPACE NOT SUITABLE CAN ATTACH AUTHENTICATED DOCUMENT)

NAME OF REPRESENTATIVE:

……………………………………………………………………………………….…….

POSITION IN THE COMPANY: ……………………………………………………….………………

MOBILE NO: ………………………………………………………………………………………..

SIGNATURE: ………………………………………………………………………………………

CONSULTANT STAMP:

8.7. Check list

8.7.1 PRELIMINARY DESIGN CHECK LIST					
#	Description	Yes	No	NA	Clarifications
1	One complete set – Hard Copy (If Requested) - of the full design package.				
2	One soft copy of the Preliminary Design Report and Drawings.				
3	One soft copy of the Technical calculation sheets with visible formulas. MS Excel format or compatible to it.				
4	One soft copy of the Hydraulic model / Shape files that is compatible with ADM software and format as stated in Section - 3.4.2.				
5	Copy of Assignment Letter of the Client.				
6	Copy of DPM approval on the Master Plans and Concept Design.				
7	Copy of Environment Agency - Abu Dhabi (EAD) approval				
8	Copy of connection approval from DEPARTMENT Operation and Maintenance Section.				
9	Project Vision Statement and full and detailed description of design criteria adopted.				
10	Project site location plan.				
11	Site survey and site context including existing utilities / services.				
12	Digital topography survey (X, Y, Z).				
13	Geotechnical investigations report/results.				
14	Proposed site plans and grading plans.				
15	Proposed network layouts.				
16	Proposed catchment area plan				
17	Proposed infrastructure layouts showing pump stations, storage ponds (if applicable).				
18	Hydrology Report, watershed model, calculations, watershed catchment area, location plan showing culverts and other hydraulic structures				
19	Electromechanical concept design (if applicable).				
20	Sizing design calculations for any proposed structures.				
21	Sections showing ROW with all details of all services corridor marked on the layout plans.				
22	Project implementation plan.				
23	Operations and Maintenance strategy.				
24	Cost assumption breakdown for Operation and Maintenance.				
25	Construction cost estimation breakdown.				
26	Statement of status of all Authority Approvals including any pending NOI / NOC.				
27	Compliance Form.				

8.7.2 DETAILED DESIGN CHECK LIST					
#	Description	Yes	No	NA	Clarifications
1	Copy of Approval of the concerned authorities.				
2	Status of all pending NOI / NOC (if any).				
3	Approval of Service Reservation for Stormwater pipeline from the Town Planning Department.				
4	Detailed Design Report.				
5	All detail design drawings.				
6	Particular specifications. (if any)				
7	Bill of Quantities for the works.				
8	Detailed breakdown of the estimated construction cost.				
9	Detailed Operation & Maintenance strategy with associated programme of activities / tasks.				
10	Operations and Maintenance budget estimate and breakdown covering 10-year period.				
11	Compliance Form to be signed and stamped by the Consultant stating full compliance in his submittal to the latest DEPARTMENT manuals, specifications and standards and noting any non-conforming elements with justification.(refer to section 6 of this document).				

8.7.3 CONNECTION TO EXISTING CHECK LIST					
#	Description	Yes	No	NA	Clarifications
1	One complete set – Hard Copy (If Requested) - of the concept design package (as approved by DPM).				
2	General catchment area with the proposed layouts.				
3	Description of criteria adopted to calculate flows produced from the project catchments.				
4	Calculated flows at the connection point/(s).				
5	General hydraulic calculation to confirm the possibility of the gravity network to connect at the invert level of the existing manholes.				

8.8. Hydraulic Modelling Shape File Data Requirements

Layer	Feature class	No.	Parameter name	Field name	Unit	Method	Decimal Places (Field	Field type	Data type	Apply to Subsoil	Remarks
NODES	Points	1	Label	Label	text		-	string	I/O	Yes	
		2	Diameter	Dia	m		3				
		3	B size (if rectangular)	B	m		4				
		4	W size (if rectangular)	W	m		5				
		5	Elevation (Ground)	elev_grd	m	RM&HD			I		
		6	Elevation (Invert)	elev_inv	m		3				
		7	Elevation (Rim)	elev_rim	m			double		Yes	
		8	External inflow (e.g. from exist. net.)	inflow	m3/s		4				
		9	Hydraulic Grade Line (In)	HGL_in	m	RM	3		O		
		10	Hydraulic Grade Line (Out)	HGL out	m						
		11	Type (manhole/catch basin/inlet…)	Type	text	RM&HD	10	string	I		
		12	Is Flooded?	Is_flood	0,1		1	integer			0=No, 1=Yes
		13	Flood volume	flood_vl	m3	HD		double	O		
		14	Head(HGL)	Head	m		3				
		15	Existing?	Existing	0,1						Does it exist? 0=No, 1=Yes
		16	Upgrade?	Upgrade	0,1	RM&HD	1	integer	I	Yes	Is it upgraded? 0=No, 1=Yes
		17	Phase	Phase	-		-	string			
PIPES	Lines	1	Label	Label	text		-		I/O		
		2	Start Node (Label)	Up_nod	text		-	string		Yes	
		3	Stop Node (Label)	Dn_nod	text		-				
		4	Diameter (Internal)	Dia	m	RM&HD	3	double	I		
		5	cross-section shape	Shape	text		10	string			
		6	Cross-section height	CH	m		3	double			optional
		7	Cross-section width	CW	m						

Layer	Feature class	No.	Parameter name	Field name	Unit	Method	Decimal Places (Field	Field type	Data type	Apply to Subsoil	Remarks
		8	Number of barrels	barrel	-		1	integer		Yes	
		9	Invert (Upstream)	Up_inv	m		3	double			
		10	Invert (Downstream)	Dn_inv	m						
		11	Ground elev. (Upstream)	Grnd_up	m	RM					
PIPES	Lines	12	Ground elev. (Downstream)	Grnd_dn	m		3	double	I		
		13	Cover (Start)	Cov_s	m						
		14	Cover (End)	Cov_e	m						
		15	Manning's n	Manning	-	RM&HD	4	double	I	Yes	
		16	Material	Material	text		-	string			
		17	Entry losses coef. (ζ)	los_ent	-		2	double			Should contain also minor losses coef. for nodes
		18	Exit loss coef. (ζ)	los_ext	-						
		19	Length	Len	m					Yes	
		20	Slope (Calculated)	S	-		6				
		21	Hydraulic Grade Line (In)	HGL_in	m	RM	2		O		
		22	Hydraulic Grade Line (Out)	HGL_out	m						
		23	Upstream C*A	CA_up	ha		4				
		24	System Time of concentration	Tc	min		1				
		25	Time of peak	T_peak	min	HD					analogical to TC in RM is related to time of peak
		26	System Intensity	Intens	mm/hr	RM	3				
		27	Capacity (Full Flow)	Q_full	m3/s	RM&HD	5			Yes	
		28	Flow(Maximum)	Q_max	m3/s						
		29	Velocity (Maximum)	v	m/s		2				
		30	D/d	D_d	%	RM					

Layer	Feature class	No.	Parameter name	Field name	Unit	Method	Decimal Places (Field	Field type	Data type	Apply to Subsoil	Remarks
		31	Flow max/ Capacity	Qm_Qf	%						
		32	Existing?	Existing	0,1		1	integer			Does it exist? 0=No, 1=Yes
		33	Upgrade?	Upgrade	0,1	RM&HD		integer	I	Yes	Is it upgraded? 0=No, 1=Yes
		34	Phase	Phase	-		-	String			
SUBCATCHEMENTS	Polygons	1	Label	Label	text		-	string	I/O	Yes	
		2	Outlet node Label	out_Lab	-	RM&HD	-			Yes	
		3	Area	A_ha	ha		4			Yes	
		4	C	C	-	RM					
		5	C*A	CA	ha			double	I		
		6	CN	CN	-	HD	2				
		7	Depression storage	Dp	mm						
		8	Time of concentration	Tc	min	RM&HD	1	string			
		9	Rainfall Intensity	Intens	mm/hr	RM	3		O		
		10	Q max	Q_max	m3/s	RM&HD	5	double			
		11	Other used rainfall-runoff calculation parameters depends on used methods			HD					
		12	Existing?	Existing	0,1		1	integer	I	Yes	Does it exist? 0=No, 1=Yes
		13	Upgrade?	Upgrade	0,1	RM&HD				Yes	Is it upgraded? 0=No, 1=Yes
		14	Phase	Phase	-		-	string		Yes	
		15	C_1	C1	-						pavement categories component
		16	A_1	A1	ha	RM	4	double			
		17	…	A…	-						

Layer	Feature class	No.	Parameter name	Field name	Unit	Method	Decimal Places (Field	Field type	Data type	Apply to Subsoil	Remarks
		18	…	C…	ha						(pavement categories need to be explained in attached file e.g. txt/docx)
		19	C_n	Cn	-						
		20	A_n	An	ha						
OUTFALLS	Points	1	Label	Label	text	RM&HD	-	string	I/O	Yes	
		2	Diameter	Dia	m		3	double	I		
		3	Fixed Tide Elevation	Stage	m						
		4	Flap gate	Fl_gate	0,1		1	integer			
		5	Elevation (Ground)	elev_grd	m		3	double		Yes	
		6	Elevation (Invert)	elev_inv	m						
		7	Existing?	Existing	0,1		1	integer		Yes	
		8	Upgrade?	Upgrade	0,1						
		9	Phase	Phase	-		-	string			
PUMPING STATIONS	Points	1	Label	Label	text	RM&HD	-	string	I/O	Yes	
		2	Diameter	Dia	m		3	double	I		
		3	Elevation (Ground)	elev_grd	m						
		4	Elevation (inlet Invert)	e_in_inv	m						
		5	Wet well invert elevation	elev_inv	m		4				
		6	Wet well shape	Shape	text		-	string			
		7	Wet well size W	W	m		2	double			
		8	Wet well size B	B	m						
		9	MAX level	max_lev	m						
		10	MIN level	min_lev	m						
		11	max capacity	Q_max	m3/s		5				
		12	Existing?	Existing	0,1		1	integer			Does it exist? 0=No, 1=Yes
		13	Upgrade?	Upgrade	0,1						Is it upgraded? 0=No, 1=Yes
		14	Phase	Phase	-		-	string			
Storage	Points/Polygons	1	Label	Label	text	RM&HD	-	string	I/O	Yes	
		2	Maximum Volume	Vol	m3		4		I		
		3	Elevation (Ground)	elev_grd	m		3	double			
		4	Elevation (Invert)	elev_inv	m						

Layer	Feature class	No.	Parameter name	Field name	Unit	Method	Decimal Places (Field	Field type	Data type	Apply to Subsoil	Remarks
		5	Existing?	Existing	0,1		1	integer			Does it exist? 0=No, 1=Yes
		6	Upgrade?	Upgrade	0,1						Is it upgraded? 0=No, 1=Yes
		7	Phase	Phase	-		-	string			

Remarks:

RM - rational method

HD - hydrodynamic method

Please apply following rules for importing

1) Use same field names and order as in table above!
2) Use for Labels only numbers, Latin letters and underscore, avoid spaces or e.g. "\," Labels duplications for full project (have to be unique for full project area) are not accepted.
3) Create separate SHP files for each analyzed scenario.
4) Combination of SHP/DBF and CSV is allowed. The General rule is: SHP/DBF should contain Labels and model network input data and CSV should contain model output data, please produce separate CSV for each analyzed scenario. Please follow I/O column.
5) If input differs in different scenarios please keep in SHP/DBF only Label and rest data in CSV
6) Merge all type of nodes (manhole, catch basin, inlet ...) into one layer
7) Please include existing elements to be demolished or abandoned as separate layers. In this case only id field is needed. Please apply fields existing for existing elements and upgrade for existing to be upgraded elements.
8) Additional fields (not listed above) with information needed for modeling should be explained in attached readme.txt file. Other data needed for model provide in EXCEL/CSV format (e.g. pump/storage curves, input hydrograph).
9) Sub catchment geometry should be precise and refer to outlet node, sub catchment geometry simplification is not allowed. Sub catchments aggregation is not allowed - number of sub catchments should reflect number of nodes.
10) Please add scenario name/number to layer name. Please do not apply for different phases separate layers please fill field phase.
11) Use coordinate system WGS 84 / UTM zone 40N.
12) Please apply unique layer/file names.
13) Please submit original model files.

9. REFERENCES

The following Standards and other reference documentation are referred to and/or provide additional guidance:

1. BS EN 752 – 2008 Drain and Storm water networks Systems Outside Buildings
2. BS EN 1295:2010 Structural design of buried pipelines under various conditions of loading
3. BS 9295 – The Structural Design of Pipelines
4. BS 1916 and 5911 – Concrete Pipes
5. ISO4427– Polyethylene Pipes
6. BS EN 295 – Vitrified Clay Pipes
7. BS EN 545 – Ductile Iron Pipes and Fittings
8. BS 8004 – Code of Practice for Foundations
9. BS 8110 – Structural Use of Concrete
10. BS8007 – Design of Concrete Water Retaining Structures
11. BS EN 13480 – Metallic Industrial Piping
12. CIRIA Report R128 – Design of Thrust Blocks
13. CIRIA Report C737- structural and geotechnical design of modular geocellular drainage systems
14. H R Wallingford - Air Problems in Pipelines
15. ANSI/HI – Pump Standards
16. CIRIA & BHRA The Hydraulic Design of Pump Sumps and Intakes
17. IEC 61158 Industrial communication networks - Fieldbus specifications.
18. AWWA M11 Steel Pipe: A Guide for Design and Installation
19. AWWA M41 Ductile-Iron Pipe and Fittings
20. AWWA M45 Fiberglass Pipe Design
21. ANSI 31.4 Pipeline Transportation Systems for Liquids and Slurries
22. ANSI 31.3 Process Piping
23. CIRIA Monitoring, maintenance and rehabilitation of water supply boreholes, Report R 137
24. Water Resources Engineering”, 2010, Larry W. Mays, John Wiley & Sons, 890 pp
25. Div-15 Section 15000 – Electro-mechanical specifications
26. The US Natural Resources Conservation Service (NRCS) National Engineering Handbook; Part 630: Hydrology
27. Stormwater Conveyance Modelling and Design; First Edition; 2003 (by Haestead Methods, Inc)
28. Department of Municipal Affairs and Transport (DEPARTMENT)Storm Water and Subsurface Drainage Systems Manuals; First Edition; 2016
29. Abu Dhabi Sewerage Services Company (ADSSC) Design Guidelines, Specification & Drawings
30. Dubai Sewerage and Drainage Masterplan; February 2000
31. Department of Urban Planning and Development (DPM) Design Manual
32. Abu Dhabi Utility Corridor Design Manual (UCDM) issued by the Urban Planning Council

10. APPENDIX

Appendix A1– Prevailing Environmental Legislation

Appendix A2 – Pipe Structural Design

Appendix A3 – Road Drainage – Gutter and Inlet Design

Appendix A4 – Al Ain Municipality Requirements

Appendix A5 –Main Roads Requirements

AppendixA1– Prevailing Environmental Legislation

A1. Prevailing Environmental Legislation

A1.1 Local Environmental Legislation

Name of Local Legislation	Date
Local Law No. 1 of 1997 of 27th April 2000, Order No. 31 Session 23/2000 declared EWRDA (now EAD) as the Competent Authority for all issues relating to the environmental and wildlife management in Abu Dhabi Emirate.	1997
Law No 16 of 2005 concerning the Re-organization of the Environment Agency Abu Dhabi (replaced Law No. (4) of 1996), as amended.	2005
Law No. (21) of 2005 for Waste Management in the Emirate of Abu Dhabi.	2005
Local Law No. 22 of 2005 concerning terrestrial hunting with the Abu Dhabi Emirate.	2005
Decree No. 17 of 2008 establishes the Centre of Waste Management (CWM) by the Government of Abu Dhabi.	2008
Executive Council Decree No. 24 of 2009, Tariff System on waste produced.	2009
Executive Council Decree No. 21 of 2010 enforced the Waste Tracking System throughout Abu Dhabi.	2010
Decision No. 3 of 2022 by the Chairman of the Board of Directors of EAD, regarding the executive regulations for the integrated management of waste in the Emirate of Abu Dhabi. Issued By EAD	2022
Source: https://www.ead.ae/en/portal/environmental.laws.aspx (Environment Agency Abu Dhabi, 2012) and various other sources.	

Table A1- 1 - Local Environmental Legislation

A1.2 Federal Environmental Legislation and Executive Orders relevant to land and marine projects

Name of National Legislation / Executive Order	Date
Federal Law No. 1 Concerning the Organization of Industrial Affairs	1979
Federal Law No. 27 Concerning Control of Communicable Diseases	1981
Federal Law No. 35 Pertaining to the Issuing of the Penal Code Procedures Law	1992
Federal Law No. 7 Pertaining to the Setting up the Federal Environment Agency	1993
Federal Law No. 4 of 1996 for the Establishment of the Environmental research and Wildlife Development Agency (ERWDA)	1996
Federal Law No. 23 Concerning Exploitation, Conservation, and Development of Living Aquatic Resources in the United Arab Emirates and its Executive Order issued by Ministerial Decree No. (302) of 2001.	2001
Federal Law No. 24 for the Protection and Development of the Environment	1999
Executive Orders to Law No. 24 on Regulations concerning Environmental Impact Assessments of Projects	2001
Executive Orders to Law No. 24 on Regulations concerning handling of Hazardous Substances, Hazardous Wastes and Medical Wastes	2001
Executive Orders to Law No. 24 on Regulation concerning the Protection of Air from Pollution	2001
Executive Orders to Law No. 24 on Regulation concerning Agriculture Pesticides and Fertilisers	2001
Executive Orders to Law No. 24 on Regulations Concerning Protected Areas	2001
Federal Law No. 1 Concerning Regulation and Control of the Use of Sources of Radiation and Protection against its Hazards	2002
Federal Law No. 11 Concerning Regulating and Controlling the International Trade in Endangered Species of Wild Fauna & Flora	2002
Executive Order of Federal Law No. 11 on the Regulatory Procedures for the Implementation of the Law	2003
Executive Order of Law No. 1 Concerning the Approval of the Basis Regulatory Rules for Protection Against Radiation	2003
Federal Law No. 28 of The Establishment of the Abu Dhabi Authority for Culture and Heritage	2005
Ministry of Cabinet No. 12 of 2006 Regarding Protection of Air Against Pollution	2006
Source: https://www.ead.ae/en/portal/environmental.laws.aspx (Environment Agency Abu Dhabi, 2012) and various other sources.	

Table A1- 2 - Federal Environmental Legislation

A1.3 International and Regional Environmental Protocols applicable in Abu Dhabi

Name of International and Regional Protocol/Convention	Date of Ratification
Convention on the Prevention of Marine Pollution by Dumping of Wastes and Other Matter (LDC),1972.	1974
Kuwait Regional Convention for cooperation on the protection of the marine environment from pollution, 1978.	1979
International Convention for the Safety of Life at Sea (SOLAS), 1974.	1983
International Convention for the Prevention of Pollution of the Sea by Oil, 1954 and its amendments.	1983
International Convention Relating to Intervention on the High Seas in Cases of Oil Pollution Casualties (INTERVENTION), 1969.	1983
International Convention on Civil Liability for Oil Pollution Damage (CLC), 1969.	1983
Vienna Convention for the Protection of the Ozone Layer of 1985.	1989
Montreal Protocol on Substances that Deplete the Ozone Layer of 1987.	1989
Protocol concerning Marine Pollution resulting from Exploration and Exploitation of the Continental Shelf, 1989.	1990
Convention on International Trade in Endangered Species of Wild Fauna and Flora (CITES) 1973.	1990
Basel Convention on the Control of Transboundary Movements of Hazardous Wastes and their disposal, 1989.	1990
UN Framework Convention on Climate Change for the year 1992.	1995
1992 Protocol Concerning Amendments on International Convention on Civil Liability for Oil Pollution Damage (CLC), 1969 and International Convention on the Establishment of an International Fund for Compensation for Oil Pollution Damage, 1971.	1997
Convention on Limitation of Liability for Maritime Claims (LLMC), 1976.	1997
United Nations Convention to Combat Desertification for the Year 1994.	1998
Convention on Biological Diversity for the Year 1992.	1999
Stockholm Convention on Persistent Organic Pollutants (POPS), 2001.	2002
Rotterdam Convention on Prior Informed Consent Procedure for Certain Hazardous Chemicals and Pesticides in International Trade (PIC Convention),1998.	2002
Convention on Conservation of Wildlife and its Natural Habitats in the GCC Countries, 2001.	2003
Montreal Amendments (London 1990, Copenhagen 1992, Montreal 1997, Beijing 1999).	2005
Kyoto Protocol, 1997.	2005
Protocol on the Control of Marine Transboundary Movements and Disposal of Hazardous Wastes and Other Wastes, 1998.	2005
Source: https://www.ead.ae/en/portal/environmental.laws.aspx (Environment Agency Abu Dhabi, 2012)	

Table A1- 3 - International and Regional Environmental Protocols and Conventions

A1.4 Environmental Permitting Requirements

In accordance with Article 4 of Federal Law No. 24, '…No project or establishment is permitted to commence works prior to securing the appropriate license in accordance with the regulations determined by the Competent Authority...'

In accordance with Article 1 of Federal Law No. 24 the Competent Authority is the 'Local Authorities in each Emirate of the United Arab Emirates'. Local Law No 16 identifies the Environment Agency Abu Dhabi as, '…the agency concerned with environmental affairs in the Emirate of Abu Dhabi...'

In accordance with Federal Law No. 24, industrial, commercial facilities and development projects require an environmental permit or No Objection Certificate prior to the commencement of site activities. In addition, certain environmental studies may be required by the Environment Agency Abu Dhabi in order to process the permit.

It is the responsibility of the Design Team / Contractor to ensure that all the Environment Agency Abu Dhabi requirements related to storm water and sub-surface drainage projects are fulfilled either as part of their own storm water and sub-surface drainage design or as part of a larger project. All current and applicable standard operating procedures and technical guidance should be complied with during all stages of the project execution. Standard Operating Procedures and Technical Guidelines are produced by the Environment Agency Abu Dhabi (Environment Agency Abu Dhabi, 2012).

The EAD Standard Operating Procedures are defined in the following:

- Standard Operating Procedures for Permitting of Development and Infrastructure Projects in Abu Dhabi.
- Standard Operating Procedures for Permitting of Industrial, Commercial, and Light Industrial Projects in Abu Dhabi.
- Standard Operating Procedures for Permitting of Chemicals and Hazardous Materials in Abu Dhabi.
- Standard Operating Procedures for Registration of Environmental Consultancy Offices in Abu Dhabi.

Referring to the latest waste management chapter approved by QCC. The EAD Technical Guidelines are defined as listed below and as in the following link:

https://jawdah.qcc.abudhabi.ae/en/Registration/QCCServices/Services/STD/ISGL-List.pdf

- Technical Guidance Document for Preliminary Environmental Review (PER).
- Technical Guidance Document for Environmental Impact Assessment (EIA).
- Technical Guidance Document for Strategic Environmental Assessment (SEA).
- Technical Guidance Document for Terms of Reference (TOR).
- Technical Guidance Document for Construction Environmental Management Plan (CEMP).
- Technical Guidance Document for Operation Environmental Management Plan (OEMP).
- Technical Guidance Document for Decommissioning Environmental Management Plan (DEMP).
- Technical Guidance Document for Environmental Action Plan (EAP).
- Technical Guidance Document for Submission of Environmental Permit Applications and Environmental Studies.
- Standard Operating Procedure (SOP) for Compliance Monitoring Using Continuous Emissions Monitoring Systems (CEMS).
- Air Quality Modelling Guidance.
- Technical Guidance Document for Environmental Audit Reports.
- Technical Guidance Document for Wastewater and Marine Water Quality Monitoring.

Appendix A2 – Pipe Structural Design

A2. Pipe Structural Design

A2.1 Introduction

Pipes shall be designed to withstand applied loads without risk of collapse or undue deformation.

Loads on pipes arise from the following:

Surface loading, e.g. traffic, including construction traffic

- Earth loads
- The weight of the pipe contents

These loads are resisted by the pipe barrel together with its bed and surround.

Pipes are classified into three structural categories, as shown in the table below:

Type	Material	Structural Properties of pipes made of various materials	Design Limiting Condition
Flexible	HDPE, uPVC, GRP	Pipe strength is largely provided by the bed and surround and the surrounding ground	Pipe deformation
Semi-rigid	Ductile iron	Pipe strength is provided by a combination of the bed, surround and surrounding ground and the pipe barrel	Pipe deformation, except for small sizes
Rigid	Concrete	Pipe strength is largely provided by the pipe barrel	Pipe collapse

Table A2- 1 - Structural Properties of Pipes of Various Materials

The section that follows presents the calculations relating to the structural design of pipes, for the reference of the Consultant.

Where the applied loads on a pipe do not give the required structural factor of safety as required by the appropriate standard then an alternative pipe material and/or bed and surround detail shall be used.

The design of pipes shall take into account the loading from the passage of construction plants and other temporary conditions, as well as the final design loading.

In deep pipelines and where the system can surcharge, an internal pressure will arise in what are otherwise gravity pipes. The Consultant shall check that the internal pressure arising from a surcharge to ground level will not overstress the pipes.

The Consultant's attention is drawn to the considerable cost of constructing long lengths of large, deep pipelines. Under these circumstances the Consultant shall carry out calculations to verify that the pipe manufacturers' standard designs are structurally sound and cost-effective. The cost effectiveness of open cut excavation and micro-tunnel construction also has to be studied in these circumstances.

A2.2 Calculations for Pipe Structural Design

In many cases pipe manufacturer's technical catalogues will give tables of permissible cover for different bedding types and surface loading conditions. However, information is given in this Appendix to aid understanding of the reference information in order to enable:

a) calculations to be done where conditions occur outside the range of manufacturers' catalogues
b) the Consultant to verify manufacturers' pipe designs
c) the Consultant to carry out specific calculations where it may be possible to devise alternative, more cost-effective designs

The primary references for the calculations are BS EN 1295:2010 and BS 9295: 2010.

A2.2.1. Flexible Pipes (MDPE, HDPE, GRP, uPVC)

The primary source of strength for flexible pipes is the backfill at the sides of the pipe. It is very important that the material surrounding flexible pipes is carefully specified and properly compacted on site. Greater care is required in constructing flexible pipes than rigid or semi-rigid pipes.

It must be noted that GRP is a composite material which is manufactured in a range of stiffnesses. Additional factors must be taken into account in the pipe design for which reference to BS EN 1295 must be made.

The design method for flexible pipes is based around not exceeding the permissible ovalisation of the pipe, but with checks for other parameters.

The ovalisation formula below is based on Eqn. 23 in BS EN 1295:

$$\Delta = 100K_X (D_L P_e + P_s) / (8S + 0.061 E')$$

Where:

Δ = the pipe ovalisation (percent)

K_x = bedding factor or deflection coefficient

D_L = deflection lag factor

Pe = pressure from soil loading (kN/m^2)

Ps = surcharge pressure (kN/m^2)

S = pipe diametral stiffness (kN/m^2)

E' = modulus of soil reaction (kN/m^2)

Permissible Pipe Ovalisation (Δ)

Flexible pipes should be designed so that the ovalisation does not exceed the values below:

Permissible Pipe Ovalisation % for Different Pipe Materials		
GRP	**HDPE**	**uPVC**
3 (initial) 5 (long term)	6	5

Table A2- 2 – Permissible Ovalisation

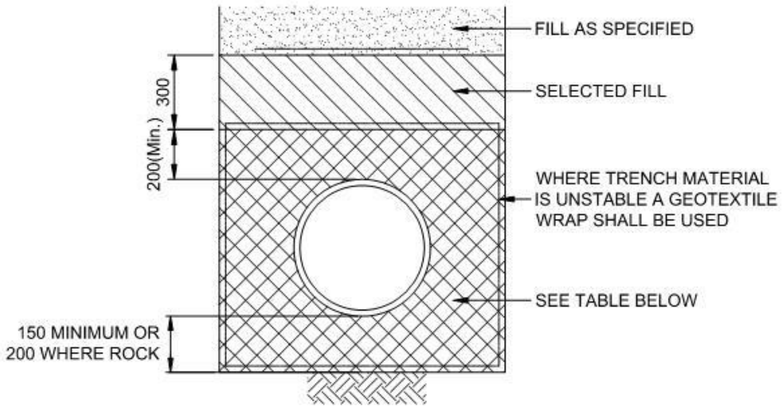
Bedding Factor or Deflection Coefficient (Kx)

The bedding factor depends on the soil pressure distribution at the top and bottom of the pipe.

Figure A2-1 shows the different bed and surround details and the corresponding K_x values. In summary these are:

Bedding Type	Bedding Factor K_x
S1, S2	0.083
S3 – S5	0.10
B1, B2	0.083

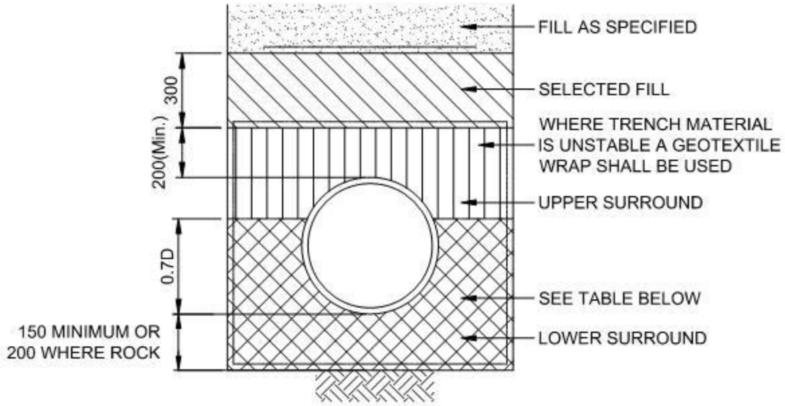
Table A2- 3 – Bedding Factors for Different Bedding Types



FLEXIBLE AND SEMI-RIGID PIPE BEDDING - CLASS 'S'

BEDDING CLASSES AND FACTORS		
BEDDING CLASS	DEFLECTION COEFFICIENT Kx	BED AND SURROUND MATERIAL
S1	0.083	SINGLE SIZE PROCESSED GRAVEL 10/14/20mm
S2	0.083	GRADED GRAVEL 5-14 OR 5-20mm
S3	0.10	SAND AND COARSE GRAINED AS-DUG SOIL WITH MORE THAN 12% FINES
S4	0.10	COARSE GRAINED AS-DUG SOIL WITH MORE THAN 12% FINES OR FINE GRAINED SOIL, LL<50%, MEDIUM TO NO PLASTICITY AND MORE THAN 25% COARSE GRAINED MATERIAL
S5*	0.10	FINE GRAINED SOIL, LL<50%, MEDIUM TO NO PLASTICITY AND LESS THAN 25% COARSE MATERIAL

SEE BS EN 1295 : 1997 TABLE NA.6 FOR FACTORS GIVEN BY DIFFERENT DEGREES OF COMPACTION
* ONLY SUITABLE FOR SEMI-RIGID PIPES



FLEXIBLE AND SEMI-RIGID PIPE BEDDING - CLASS 'B'

BEDDING CLASSES AND FACTORS		
BEDDING CLASS	DEFLECTION COEFFICIENT Kx	BED AND SURROUND MATERIAL
B1	0.083	UPPER SURROUND AS FOR S3 OR S4 LOWER SURROUND AS FOR S1 OR S2
B2	0.083	UPPER SURROUND AS FOR S5 LOWER SURROUND AS FOR S1 OR S2

NOTE : CLASS 'B' BEDDING IS NOT RECOMMENDED FOR PIPES WITH DIAMETRAL STIFFNESS LESS THAN 10 KN/M²

Figure A2- 1 - Pipe Bedding Classes S and B for Flexible and Semi-Rigid Pipe Bedding

Deflection Lag Factor (DL)

This represents the relaxation of the pressure from side fill over time resulting in further deflection (ovalisation) of the pipe. The values are given in BS EN 1295 Table NA.6 reproduced in Table A2-4.

Flexible and semi-rigid pipe embedment properties								
Embedment class as Figure A2-1 and deflection coefficient K_x	Compaction M_p	Deflection lag factor D_L[2]	Strain factor Df for various pipe stiffness[1]					
	%		KN/m^2					
			1.25	2.5	5.0	10	15	30 or more
Class S1 K_x = 0.083	Uncompacted	1.5	4.7	4.5	4.3	4.0	3.75	3.0
	80	1.25	4.7	4.5	4.3	4.0	3.75	3.0
	85	1.0	4.7	4.5	4.3	4.0	3.75	3.25
	90	1.0	4.7	4.5	4.3	4.0	3.75	3.5
	95	1.0	-	-	-	-	3.75	3.5
Class S2 K_x = 0.083	Uncompacted	1.5	4.7	4.5	4.3	4.0	3.75	3.0
	80	1.25	4.7	4.5	4.3	4.0	3.75	3.0
	85	1.0	4.7	4.5	4.3	4.0	3.75	3.25
	90	1.0	4.7	4.5	4.3	4.0	3.75	3.5
	95	1.0	-	-	-	-	3.75	3.5
Class S3 K_x = 0.100	85	1.5	6.2	5.5	4.75	4.25	4.0	3.25
	90	1.25	7.75	6.6	5.5	4.7	4.25	3.5
	95	1.0	-	-	-	-	4.75	3.5
Class S4 K_x = 0.100	85	1.5	6.2	5.5	4.75	4.25	4.0	3.5
	90	1.25	7.75	6.6	5.5	4.7	4.25	3.5
	95	1.0	-	-	-	-	4.75	3.5
Class S5 K_x = 0.100	85	3.0	-	-	-	-	4.0	3.5
	90	2.0	-	-	-	-	4.25	3.5
	95	1.25	-	-	-	-	4.5	3.5
Class B1 K_x = 0.083	85	1.5	-	-	-	5.0	4.0	3.5
	90	1.25	-	-	-	5.5	4.25	3.5
Class B2 K_x = 0.083	85	2.0	-	-	-	5.5	4.25	3.5
	90	1.75	-	-	-	6.0	5.0	3.5

1) Pipe stiffnesses referred to in this table are initial values
2) Where the Consultant can be certain that initial pressurization will take place within one year of backfilling, a value of 1.0 may be taken for the deflection lag factor.

NOTE 1. For construction details of embedment classes see Figure A2-1.
NOTE 2. M_P indicates modified Proctor density and corresponds to the heavy compaction test in BS 1377.

Table A2- 4 – Flexible and Semi-rigid Pipe Embedment Properties

Pressure from Earth Loading

This is given by:

$$\mathbf{Pe = \gamma\ H}$$

Where:

γ = density of backfill (kN/m^3), normally taken as 20kN/m^3

H = cover depth (m).

Pressure from Surcharge (Traffic) Loading

This is derived from the Boussinesq equation. A range of values is given in the following table, taken from BS 9295:

	Surcharge Pressure from Vehicles Ps (kN/m2) *Note: axle loads and configurations are those in Figure 4 of BS 9295*		
Cover Depth H (m)	**Main Roads**	**Light Roads**	**Fields**
1	74	63.2	36.1
2	40.3	21.5	12.3
3	26.7	10.3	5.9
4	18.5	6	3.4
5	13.3	.9	2.2
6	9.9	2.7	1.6
7	7.6	2	1.1
8	6.0	1.5	0.8
9	4.9	1.2	0.7
10	4	1	0.6

Table A2- 5 – Surcharge Pressure from Vehicles at Different Cover Depths

Pipe Diametral Stiffness

Pipe diametral stiffness = EI/D^3.

Where:

E = Young's modulus (kN/m2)

$I = t^3/12$ (t = pipe wall thickness (m))

D = average pipe diameter (m)

Typical values of E are:

Pipe Material	**Young's Modulus E (kN/m2)**	
GRP	20,000,000	check with supplier
HDPE (PE100) Long term	160,000	check with supplier
uPVC	3,000,000	check with supplier

Note: GRP is a composite material whose properties cover a wide range.

Table A2- 6 – Young's Moduli for Flexible Pipe Materials

Example values of pipe diametral stiffness are:

Pipe Material	**Pipe Diametral Stiffness (kN/m2) for all pipe sizes**	
GRP 5,000	5	
GRP 10,000	10	
HDPE (PE100) SDR 21 (long term)	1.5	

Structured Wall Plastic Pipes (SWPP)	2 – 8	wall section can be a bespoke design
uPVC SDR 41	3.6	

Table A2- 7 – Example Pipe Diametral Stiffnesses

Modulus of Soil Reaction

This is an empirical factor related to the degree of compaction applied to the pipe surround material on installation.

	E' kN/m2			
Nature of Soil in Pipe Zone	**Uncompacted**	**Light Compaction >85% Sp**	**Medium Compaction >90% Sp**	**High Compaction >95% Sp**
Single size gravel	5000	7000	10,000	14,000
Graded gravel	3000	7000	10,000	20,000
Sand	1000	5000	7000	14,000
Coarse soil with >12% fines Fine soil LL<50% and containing > 25% coarse particles	0	3000	5000	10,000
Fine grained soil LL<50% and containing <25% coarse particles	0	1000	3000	7000

Source: Saint Gobain Pipe & Fittings Water & Sewer Design Guide, Table A2- 7

Table A2- 8 - Modulus of Soil Reaction

A2.2.2. Semi-rigid Pipes (Ductile Iron)

Semi-rigid pipes exhibit similar structural properties to flexible pipes and the structural design procedure is similar to that for flexible pipes, i.e., based around ovalisation with checks for other factors. However, there are additional factors to be taken into account and reference should be made to BS EN 1295 for the details.

The maximum recommended ovalisation is 3%.

For pipes supplied to BS 598 the pipe stiffnesses are as follows:

Pipe Nominal Bore (mm)	Pipe Diametral Stiffness (kN/m2) DI to BS EN 598
500	22
600	18
700	24
800	20
900	18

Pipe Nominal Bore (mm)	Pipe Diametral Stiffness (kN/m2) DI to BS EN 598
1000	16
1200	20
1400	18
1600	17
1800	16
2000	16

Table A2- 9 - Ductile Iron Pipe Diameteral Stiffness

Simplified Table of Bedding and Depth

The table below based on:

a) trench width = pipe external diameter plus 600mm

b) soil compaction of 85%

For other conditions refer to BS 9295.

Pipe Bore	Bedding Class	Permissible Cover Main Roads m.	Permissible Cover Fields m.
300	S1 & S2	0.6 – 6.0+	0.5 – 6.0+
	S3	0.6 – 6.0+	0.5 – 6.0+
	S4	0.6 – 6.0+	0.5 – 6.0+
	S5	0.9 – 2.7	0.5 – 3.4
450	S1 & S2	0.7 – 6.0+	0.5 – 6.0+
	S3	0.6 – 6.0+	0.5 – 6.0+
	S4	0.9 – 6.0+	0.5 – 6.0+
	S5	---	0.7 – 1.9
600	S1 & S2	1.0 – 6.0+	0.5 – 6.0+
	S3	0.6 – 6.0+	0.5 – 6.0+
	S4	1.0 – 4.9	0.5 – 6.0+
	S5	• ---	----
• 600	Carry out calculations or refer to manufacturers' literature		

Table A2- 10 - Simplified Table of Bedding and Permissible Cover for DI Pipe

A2.2.3. Rigid Pipes (Concrete)

The load carrying capacity of a rigid pipe is its crushing strength multiplied by the pipe bedding factor. The design is complex and for a full description reference should be made to BS9295 and BS EN 1295. However, using the assumptions of a ‘wide trench’ and ‘incomplete projection’ the required bedding factor is given by:

$$\mathbf{F_m \geq W_e F_{se} / W_t}$$

Where:

F_m = required minimum bedding factor (refer to the Standard Drawings for options)

W_e = total external load

W_t = crushing strength of pipe (from BS 5911 /ASTM C76M or manufacturer)

F_{se} = minimum factor of safety (1.25 except for reinforced concrete pipes where it is 1.5)

Figure A2-2 shows the different bed and surround details and the corresponding K_x values. In summary these are:

Bedding Class	Bedding Factor (K_x) Provided
F	1.5
B	1.9
S	2.2
A	2.6 Unreinforced Concrete 3.4 Reinforced Concrete

Table A2- 11 - Bedding Classes for Rigid Pipes

The total external load is given by:

$$\mathbf{W_e = W'_c + W_{csu} + W_W}$$

$W'_c = C_c\, \gamma\, B_c^2$ (where C_c is from Table NA.2 in BS EN 1295; γ = soil density (kN/m3); B_c= pipe outside diameter (m))

$W_{csu} = P_s B_c$ (where Ps is from Table A2-5 above)

W_W= equivalent weight of water in the pipe per metre length (kN/m), taken as 75% of the pipe full weight

The crushing strengths of standard Class 120 pipes to BS 5911 are:

DN	Crushing Strength kN/m
300	36
450	54
600	72
750	90
900	108
1200	144
1500	180
1800	216
2100	252
2400	288

Table A2- 12 - Crushing Strength of Class 120 Concrete Pipes

Simplified Table of Bedding and Depth

The following table gives a simplified summary of bedding and cover depth:

1. Concrete Class 120

Pipe Nominal Bore (mm)	Bedding Class and Factor (see Standard Drawings)	Permissible Cover Main Roads m.	Permissible Cover Fields m.
600	F (1.5)	1.0 – 3.0	0.6 - 4.6
	B (1.9)	0.9 – 6.0+	0.6 – 6.0+
	S (2.2)	6.0+	6.0+
750	F (1.5)	-	0.6 – 3.6
	B (1.9)	0.9 – 4.6	0.6 – 5.6
	S (2.2)	0.9 – 6.0+	0.6 – 6.0+
900	F (1.5)	-	0.6 – 3.2
	B (1.9)	0.9 – 3.6	0.6 – 4.6
	S (2.2)	0.9 – 5.2	0.6 – 5.8
1200	F (1.5)	1.4 – 1.6	0.6 – 3.4
	B (1.9)	0.9 – 4.0	0.6 – 5.0
	S (2.2)	0.9 – 5.6	0.6 – 6.0+
>1200	Carry out calculations or refer to manufacturers' literature		

Source: BS 9295:2010 Table B.2

Table A2- 13 - Simplified Table of Bedding and Permissible Cover for Concrete Pipes

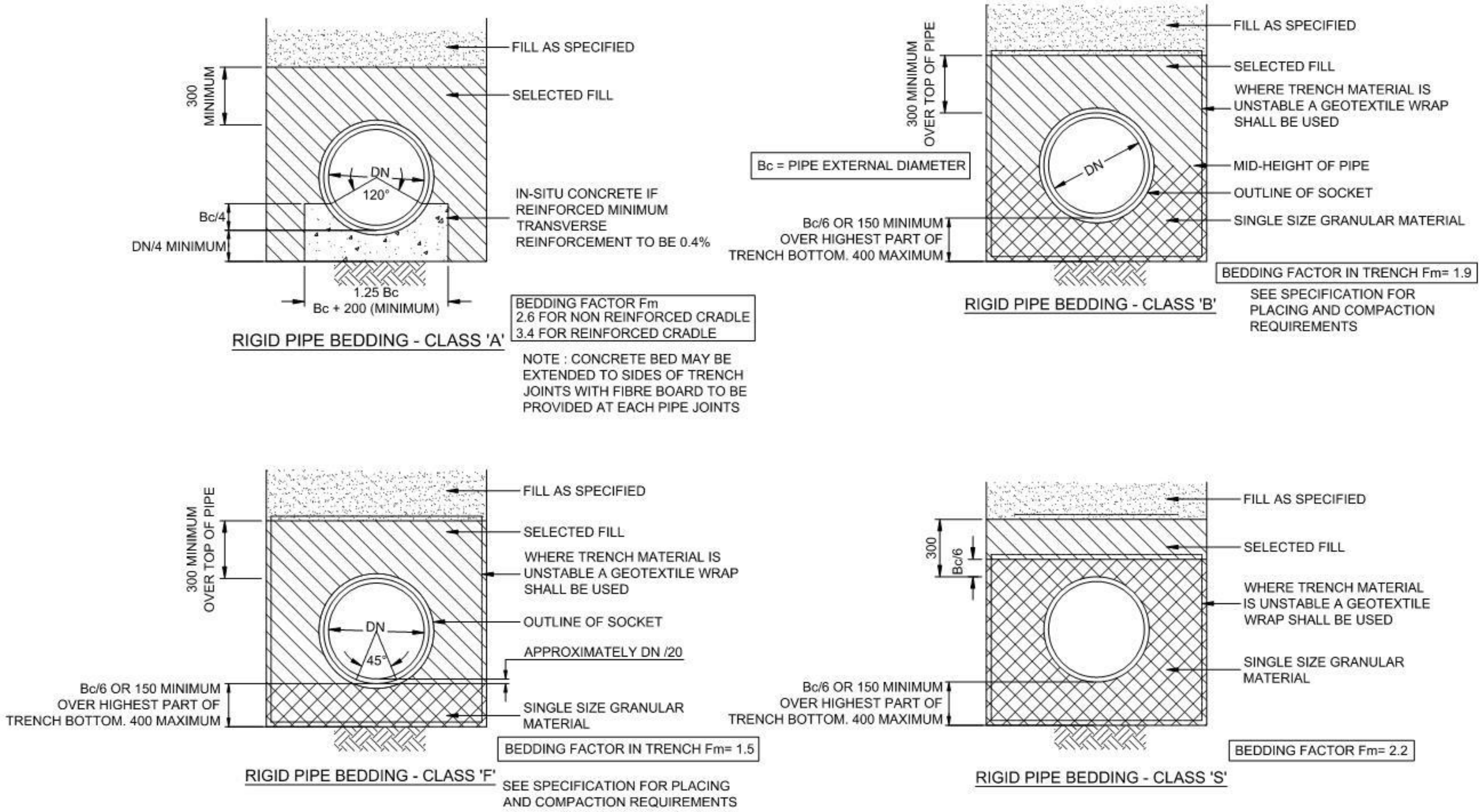


Figure A2- 2 - Pipe Bedding Classes F, B, S and A for Rigid Pipes

Appendix A3– Road Drainage – Gutter and Inlet Design

A3. Road Drainage-Gutter & Inlet Design

Note: This appendix was reproduced from the Roadway Design Manual – Drainage, Section 400: Roadway Drainage (Gutter, Inlet, and Pavement Hydraulics), unless stated otherwise.

A3.1 Introduction

Variables of major concern for pavement drainage evaluations include depth of gutter flow and pavement spread. These variables and roadway features such as cross slope, longitudinal slope, and gutter sections can affect the size, type, and spacing of inlets. This section provides fundamentals of gutter flow, inlet interception capacity, and bridge deck drainage, and an evaluation of the potential for hydroplaning.

The level of service of facilities that provide drainage of roadway surfaces should be consistent with the level of service being provided by the roadway. Guidelines are given for evaluating roadway features as they relate to pavement drainage, and for selecting an appropriate design frequency. The potential for hydroplaning, which can occur at design speeds greater than 75 km per hour when there is water on the pavement, is discussed. Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for kerb opening inlets, grated gutter inlets, combination inlets, slotted pipe and trench drain inlets, and bridge deck drainage features are discussed.

A3.2 Roadway Features

Roadway features considered during gutter, inlet, and pavement drainage calculations include:

- Longitudinal slope
- Cross slope
- kerb and gutter sections
- Ditches
- Bridge decks
- Shoulder gutters

A3.2.1 Longitudinal Slope

A minimum longitudinal gradient is important for a kerbed pavement, since it is susceptible to stormwater spread. Flat gradients on unkerbed 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 0.3 percent for kerbed pavements, and not less than 0.2 percent in very flat terrain. Minimum grades can be maintained in very flat terrain by use of a saw-tooth or rolling profile. To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent should be maintained within 50 feet of the level point in the curve. This is accomplished where the length of the curve (L) divided by the algebraic difference in grades (A) is equal to or less than 167 ($L/A < 167$). Special gutter profiles should be developed to maintain a minimum slope of 0.2 percent up to the inlet. Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

A3.2.2 Cross Slope

The AASHTO policy on geometric design (Green Book, 1984) is standard practice and should be consulted for more details than those presented in this section.

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. The USDOT, FHWA (FHWA-RD-79-30, 31, 1979) reports that cross slopes of 2 percent have little effect on driver effort in steering, especially with power steering, or on friction demand for vehicle stability. Use of a cross slope steeper than 2 percent on pavements with a central crown line is not desirable. In areas of longitudinal flat slopes and slower traffic speeds (less than 75 kph), a somewhat steeper cross slope may be necessary to facilitate drainage. In such areas, the cross slope may be increased to 2.5 percent. When three or more lanes are inclined in the same direction on multi-lane pavements, it is desirable that each successive pair of lanes, or the portion thereof outward from the first two lanes from the crown line, have an increased slope. The two lanes adjacent to the crown line should be pitched at the normal slope, and successive outward lane pairs, or portions thereof, should be increased by about 0.5 percent. Where three or more lanes are provided in each direction, the maximum outside lane pavement cross slope should be limited to 3 percent.

It is desirable to provide a break in cross slope at two lanes, with three lanes the upper limit. Although not widely encouraged, inside lanes can be sloped toward the median. This should not be used unless four continuous lanes or some physical constraint on the roadway elevations occurs, since inside lanes are used for high speed traffic and the allowable water depth is lower. Median areas should not be drained across travelled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas, and consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades. Where kerbs are used, depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement.

Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians.

A3.2.3 Kerb and Gutter

Kerbing at the outside edge of pavements is normal practice for low-speed, urban roadway facilities. Where formed gutters are used, they are generally 0.5 to 1 metre wide. Formed gutters (either asphalt or Portland cement concrete) are on the same cross slope as the pavement on the high side and depressed with a steeper cross slope on the low side, usually at 8%. Typical practice is to place kerbs at the outside edge of the travelway, the shoulders, or the parking lanes on low speed facilities. The gutter width is included as a part of the pavement width.

A3.2.4 Roadside and Median Ditches

Roadside ditches are commonly used with unkerbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to access and right-of-way limitations, roadside ditches cannot be used on most urban areas. 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.

Kerbed highway sections are relatively inefficient at conveying water, and the area tributary to the gutter section should be kept to a minimum to reduce the hazard of water on the pavement. Where practicable, the flow from major areas draining toward kerbed highway pavements should be intercepted by ditches or other pipe/inlet systems, as appropriate.

It is preferable to slope median areas and inside shoulders to a centre swale, to prevent drainage from the median area running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction. In some cases, detention can be included in shallow medians to reduce the size of runoff facilities, but this must be carefully engineered to handle higher intensity rainfall.

A3.2.5 Bridge Decks

Drainage of bridge decks is similar to other kerbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for becoming clogged by debris. Bridge deck constructability usually requires a constant cross slope, so the guidelines in Section A3.2.2 do not apply. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.2 percent. When bridges are placed on a vertical curve and the longitudinal slope is less than 0.2 percent, the gutter spread should be checked to ensure a safe, reasonable design.

Piped and grated scupper type inlets are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns, and scuppers should always be piped in down drains to stable and erosion protected runoff points. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains, although paved flumes may be used for extremely minor flows in some areas.

A3.2.6 Shoulder Gutters

Shoulder gutters are used to protect fill slopes from erosion caused by water from the roadway pavement. Shoulder gutters are required on all fill slopes higher than 3 metres in areas where permanent vegetation cannot be established. An inspection of the existing/proposed site conditions and contact with the Agriculture Section shall be made by the Consultant to determine the likelihood that vegetation will survive.

Shoulder gutters are also required at bridge ends where concentrated flow from the bridge deck would otherwise run down the slope or over a retaining wall/abutment. This section of gutter should be long enough to include the transitions necessary. Shoulder gutters are not required on the high side of super-elevated sections or adjacent to barrier walls on high fills.

Shoulder gutter ditch bottom type inlet spacing shall be based on consideration of future additional median lanes. Bypass should not exceed 25 percent for intermediate inlets.

The terminal inlet must be assured of intercepting 100 percent of the flow coming to it, including bypass from intermediate inlets. The maximum flow allowed at the terminal inlet should be 0.043 cubic metres per second for grades steeper than 1.4 percent. For grades flatter than 1.4 percent, the flow shall not exceed 80% of the maximum 100% intercept width of the gutter inlet. In any case, the distance from the last intermediate inlet to the terminal inlet should not exceed 70 metres.

A3.3 Hydroplaning

Hydroplaning occurs when a tire is separated from the road surface by a layer of fluid. Major variables influencing hydroplaning conditions include:

- Pavement texture depth (TXD), in millimetres
- Cross slope of pavement (Sx), in m/m
- Pavement width (W), in m
- Rainfall intensity (i), in mm/hour
- Tire tread depth (TD), in mm
- Tire pressure (P), in psi
- Vehicle speeds, in kph
- Water depth (WD), in millimetres

Only the first three variables are design parameters that can be controlled by the engineer, with typical values as follows:

- Pavement texture depth, 0.25 to 1.5 millimetres
- Cross slope of pavement, 0.01 to 0.03 m/m (can be higher for super-elevated sections)
- Pavement width, 3 to 15 metres

An evaluation of the potential for hydroplaning requires selection of design variables that minimize the pavement water depth. An empirical equation for computing water film depth, based on experimental studies conducted at Texas A&M University for the USDOT, FHWA, and presented in the two volume report FHWA-RD-79-30 and 31 (1979), can be used for this evaluation. Knowing the water film depth and various operational parameters, the vehicle speed at which hydroplaning occurs can be estimated using a second empirical equation. Both equations are presented below:

The equation for estimating the vehicle speed at which hydroplaning occurs is expressed as:

$$\mathbf{v = 1.609[SD^{0.04}P^{0.3}\,(.039T_D + 1)^{\,0.06}A]} \qquad \textbf{(Eq. A3-1)}$$

where:

v = vehicle speed at which hydroplaning occurs, in kph

SD = $100\,[(W_d - W_W) / W_d]$ = spindown percent (10 percent spindown is used as an indicator of hydroplaning)

W_d = rotational velocity of a rolling wheel on a dry surface

W_W = rotational velocity of a wheel after spinning down due to contact with a flooded pavement

P = tire pressure, in pounds per square inch (use 24 psi for design)

T_D = tire tread depth, in millimetres (use 1.59 mm for design)

A = the greater of: $(10.409 /.039WD) + 3.507$
or $((28.952/(.039WD^{0.06})) -7.817)(.039TXD^{0.14})$

WD = water depth, in mm (see Equation A3-2)

TXD = pavement texture depth in millimetres (use 0.51 mm for design)

An equation for evaluating the depth of stormwater on pavement is expressed as:

WD = 25.4 {0.00338 [(0.039TXD0.110.305L0.430.039i0.59) / Sx0.42] - 0.039TXD}
(Eq. A3-2)

where:

L = pavement width, in metres
I = rainfall intensity, in millimetres/hr
S_x = pavement cross slope, in m/m

The following guidelines for pavement drainage can help reduce the potential for hydroplaning problems:

- A permeable surface course or a high macrotexture surface course appears to have the highest potential for reducing hydroplaning accidents. This has been accomplished using friction courses.
- Pavement cross slope is the dominant factor in removing water from the pavement surface. A minimum cross slope of 1.5 percent is recommended.
- As a guideline, a wheel path depression in excess of 5 millimetres should be considered as a threshold to indicate the need for resurfacing to reduce the potential for pavement drainage problems on dense asphaltic concrete or Portland cement concrete pavements. The potential for hydroplaning is greater from wheel path settlement and wear depressions than from sheet flow depth. This is also true for most multi-lane facilities.
- Surface drains located parallel to the lane lines will probably not solve potential drainage problems caused by the creation of wheel path depressions.
- Transverse surface drains located on the pavement surface would probably result in a rough pavement, increase maintenance costs, and increase potential for ponding water, and are not recommended for general use.
- Grooving may be considered as a corrective measure for severe localized hydroplaning problems.

A3.4 Spread

The design storm frequency for pavement drainage should be consistent with the frequency selected for other components of the drainage system.

A3.4.1 Selection Considerations

The major considerations for selecting a design frequency and spread are:

- Highway classification, because it defines and reflects public expectations for finding water on the pavement surface. Ponding should be minimized on the traffic lanes of high-speed, high-volume highways, where it is not expected to occur.
- Highway speed, because at speeds greater than 75 kilometres per hour, even a shallow depth of water on the pavement can cause hydroplaning. Design speed is recommended for use in evaluating hydroplaning potential. When the design speed is selected, consideration should be given to the likelihood that legal posted speeds may be exceeded. It is clearly unreasonable to provide the same level of pavement drainage for low-speed facilities as for high-speed facilities.

- Rainfall events intensity may significantly affect the selection of design frequency and spread.

Other considerations include inconvenience, hazards, and nuisances to pedestrian traffic and buildings adjacent to roadways which are located within the splash zone. These considerations should not be minimised and, in some locations such as commercial areas, may assume major importance.

For urban multi-lane kerbed roadways designed for less than 80 kph with no parking and no shoulders, it is not practical to avoid travel lane flooding when longitudinal grades are flat or less than 1.0 percent. However, flooding shall never exceed the lane adjacent to the kerb or shoulder for design conditions. Bridges with kerbs and no shoulders should also use this criterion. For single-lane roadways, such as ramps at interchanges, at least 2.5 metres of roadway shall remain unflooded for design conditions. For design speeds of 80 kph and greater on roads with shoulders, the design storm runoff shall not impinge on the travel way.

A3.4.2 Gutter Fundamentals

A pavement gutter is the section of a roadway normally located at its outer edge to convey stormwater runoff. It may include a portion of a travel lane or be a separate section at the edge of the travel way or at the edge of the shoulder, but it usually has a triangular shape defined by the cross slope and kerb. In lieu of kerbs, it is possible to use a v-shaped monolithic pavement section, but only when traffic control is unnecessary, such as on low volume residential streets.

Major components of a typical gutter section include:

- Pavement cross slope (Sx)
- Longitudinal slope (S)
- Width of flow or spread (T)
- Width of depressed gutter flow (W)
- Depth of gutter flow (d)
- Cross slope of depressed gutter (Sw)

Sketches showing the relationship of these components for three typical gutter sections are presented in Figure A3-1. The first sketch shows a kerb and gutter section with a straight cross slope; the second a V-shaped section without a kerb; and the third a depressed kerb and gutter section of width (W) and cross slope (S_w). These sketches are provided to define fundamental parameters and are not intended to be used as standard details of the Department.

Gutter flow is a form of open channel flow that can be analysed using a modified form of Manning's Equation. The modification is necessary because gutter flow typically has a water surface width of more than 40 times the depth of flow. Under such conditions, the hydraulic radius does not properly describe the cross section of flow. The modified form of Manning's Equation is presented in HEC-22 (USDOT, FHWA, 1984) as follows:

$$\mathbf{Q = (0.0159 / n)\, S_x^{5/3}\, S^{1/2}\, (3.28T)^{8/3}} \qquad \textbf{(Eq. A3-3)}$$

where:

Q = gutter flow rate, in m^3/sec
N = Manning's roughness coefficient
Sx = pavement cross slope, in m/m
S = longitudinal slope, in m/m

T = width of flow or spread, in metres

Depth of gutter flow and pavement spread are parameters which must be evaluated to properly establish the size, type, bypass, and spacing of inlets. The depth of flow and pavement spread are related as follows:

$$d = TS_x \qquad \textbf{(Eq. A3-4)}$$

where:

d = depth of gutter flow, in metres
T = width of spread, in metres
S_x = pavement cross slope, in m/m

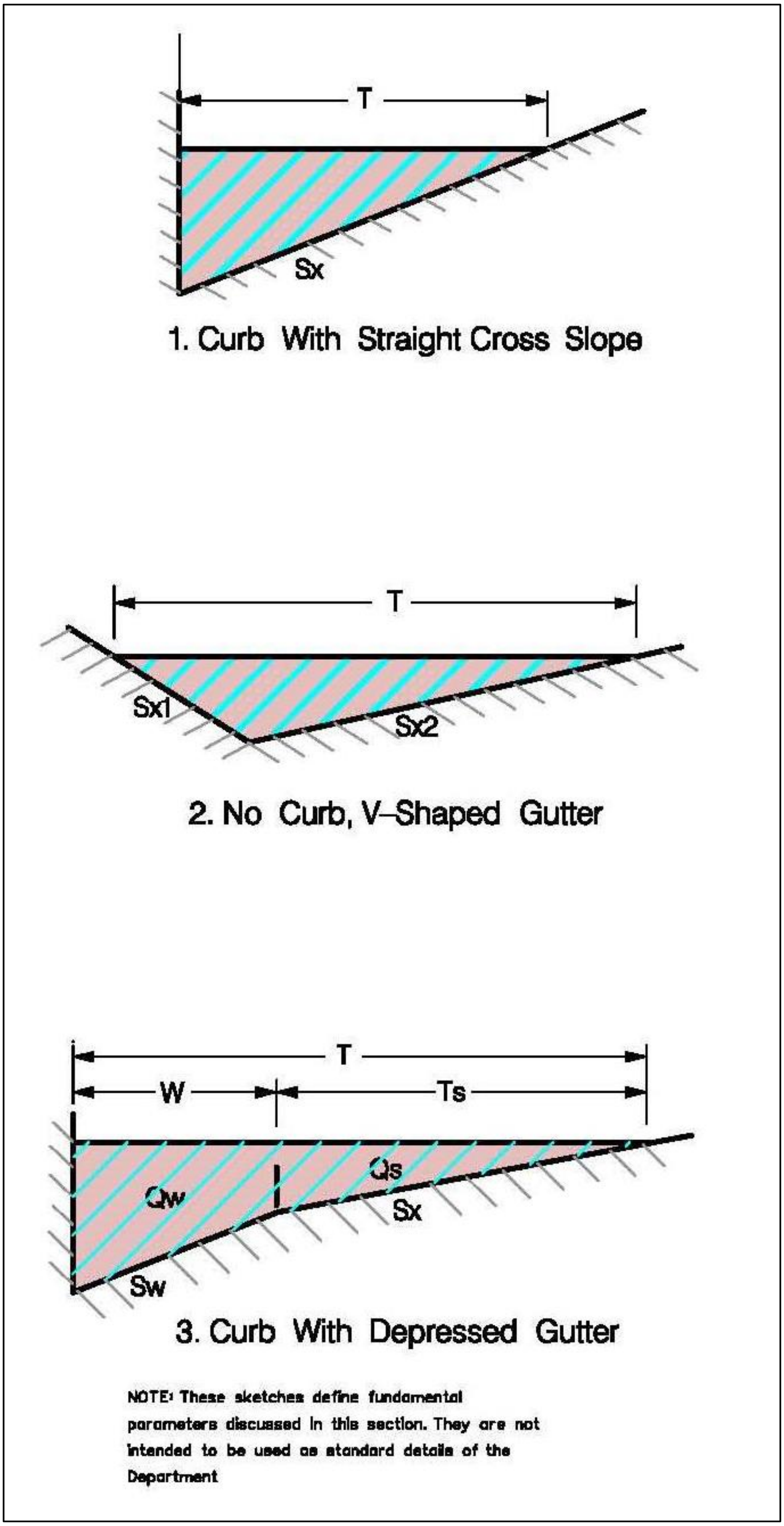


Figure A3- 1 - Typical Gutter Section

Manning's "n" Values for Street and Pavement Gutters	
Type of Gutter or Pavement	**Range of Manning's "n"**
Concrete gutter, trowelled finish	0.012
Asphalt pavement: Smooth texture Rough texture	 0.013 0.016
Concrete gutter with asphalt pavement: Smooth Rough	 0.013 0.015
Concrete pavement: Float finish Broom finish	 0.014 0.016
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

Table A3- 1 - Manning's "n" Values for Street and Pavement Gutters

For conditions where the pavement cross slope is curved or parabolic instead of straight, a special adaptation of Equation A3-3 is required to evaluate gutter capacity. Additional information for these conditions is presented in HEC-12 (USDOT, FHWA, 1984).

The relative effects of spread, cross slope, and longitudinal slope on the capacity of a gutter with a straight cross slope are presented in Figure A3-2. Each of the lines is based on the relationship between these variables, as expressed by Equation A3-3. Gutter spread is shown to have the greatest impact on gutter capacity, followed by cross-slope and, to an even lesser degree, by longitudinal slope. For example, doubling the spread would increase gutter capacity by 6 times, while doubling cross slope or longitudinal slope would result in increases of only about 3 and 1.4 times, respectively.

A nomograph for solving Equation A3-3 is presented in Figure A3-3. Manning's "n" values for various pavement surfaces are presented in Table A3-1.

The nomograph in Figure A3-3 is used with the following procedures to find gutter capacity for *uniform cross slopes:*

Condition 1: Find spread, given gutter flow.

a. Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's "n."
b. Draw a line between the S and S_x scales and note where it intersects the turning line.
c. Draw a line between the intersection point from Step b and the appropriate gutter flow value on the capacity scale. If Manning's "n" is 0.016, use Q from Step a; if not, use the product of Q and n.
d. Read the value of the spread (T) at the intersection of the line from Step c and the spread scale.

Condition 2: Find gutter flow, given spread.

a. Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's "n."
b. Draw a line between the S and S_x scales and note where it intersects the turning line.
c. Draw a line between the intersection point from Step b and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.
d. For Manning's "n" values of 0.016, the gutter capacity (Q) from Step c is selected. For other Manning's "n" values, the gutter capacity times n (Qn) is selected from Step c and divided by the appropriate n value to give the gutter capacity.

Figure A3-3 can be used to find the flow in a gutter of width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets. The following steps are used to evaluate *composite gutter sections*:

Condition 1: Find spread, given gutter flow.

a. Determine input parameters, including longitudinal slope (S), cross slope (Sx), depressed section slope (Sw), depressed section width (W), Manning's "n," gutter flow (Q), and a trial value of the gutter capacity above the depressed section (QS).
b. Calculate the gutter flow in W (Qw), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(Eq. A3-5)}$$

c. Calculate the ratios Q_w/Q and S_w/S_x and use Figure A3-4 to find an appropriate value of W/T.
d. Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step c.
e. Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step d.
f. Use the value of T_s from Step e along with Manning's "n," S, and S_x to find the actual value of Q_s from Figure A3-3.
g. Compare the value of Q_s from Step f to the trial value from Step a. If values are not comparable, select a new value of Q_s and return to Step a.

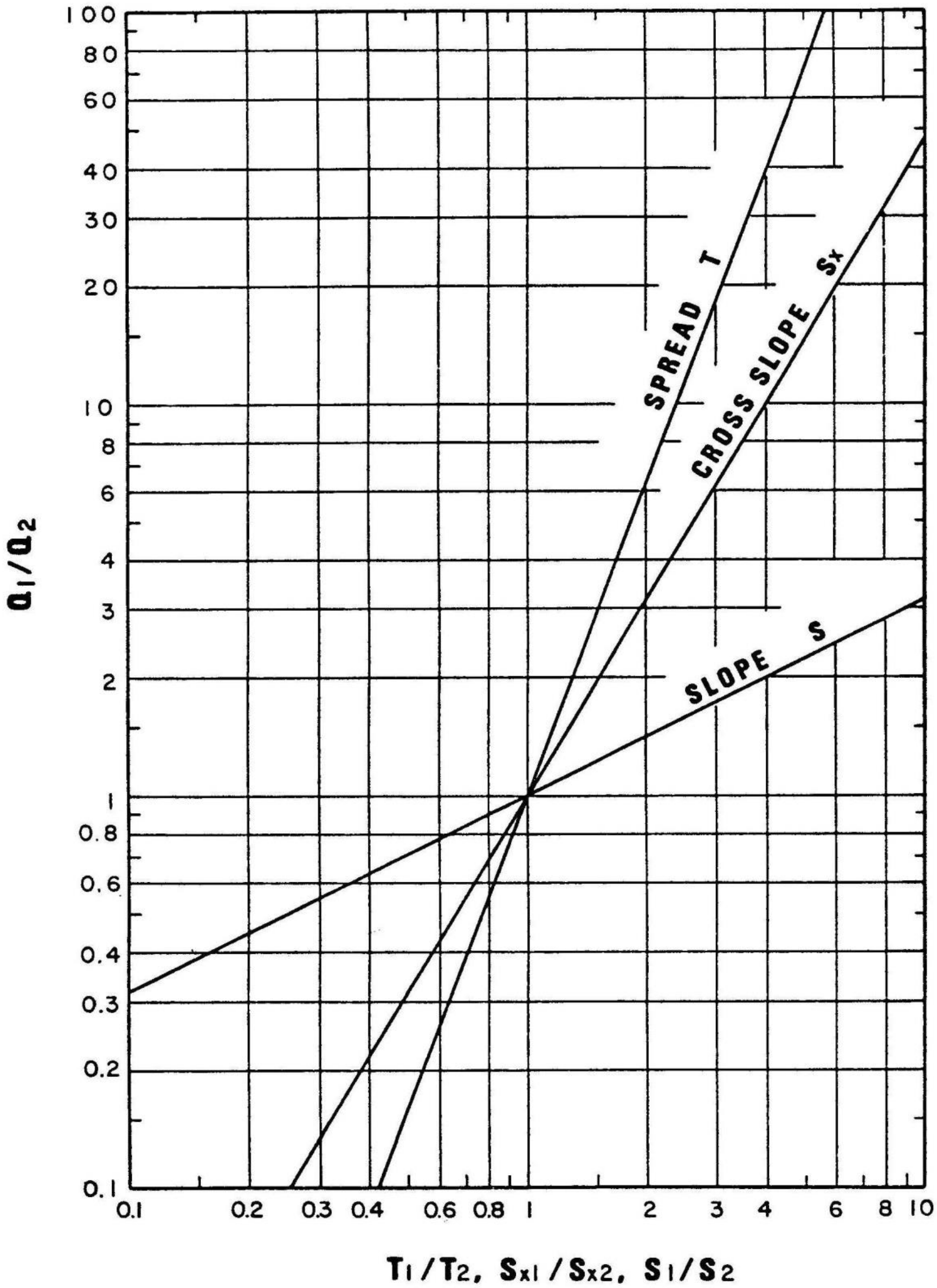
Condition 2: Find gutter flow, given spread.

a. Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's "n," and depth of gutter flow (d).
b. Use Figure A3-3 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for non-composite gutter sections, Condition 2, substituting T_s for T.
c. Calculate the ratios W/T and S_w/S_x, and, from Figure A3-4, find the appropriate ratio of E_o (the ratio of Q_w/Q).
d. Calculate the total gutter flow using the equation:

$$\mathbf{Q = Qs\ (1 - Eo)} \qquad \textbf{(Eq. A3-6)}$$

where:
Q - gutter flow rate, in m^3/sec
Q_s = flow capacity of the gutter section above the depressed section, in m^3/sec
E_o - ratio of frontal flow to total gutter flow, Q_w/Q

e. Calculate the gutter flow in width (W), using Equation A3-5.



NOTE: A straight cross slope is assumed and Equation 403 provides the basis for these relationships.

Figure A3- 2 - Relative Effects of Spread, Cross Slope and Longitudinal Slope on Gutter Flow Capacity

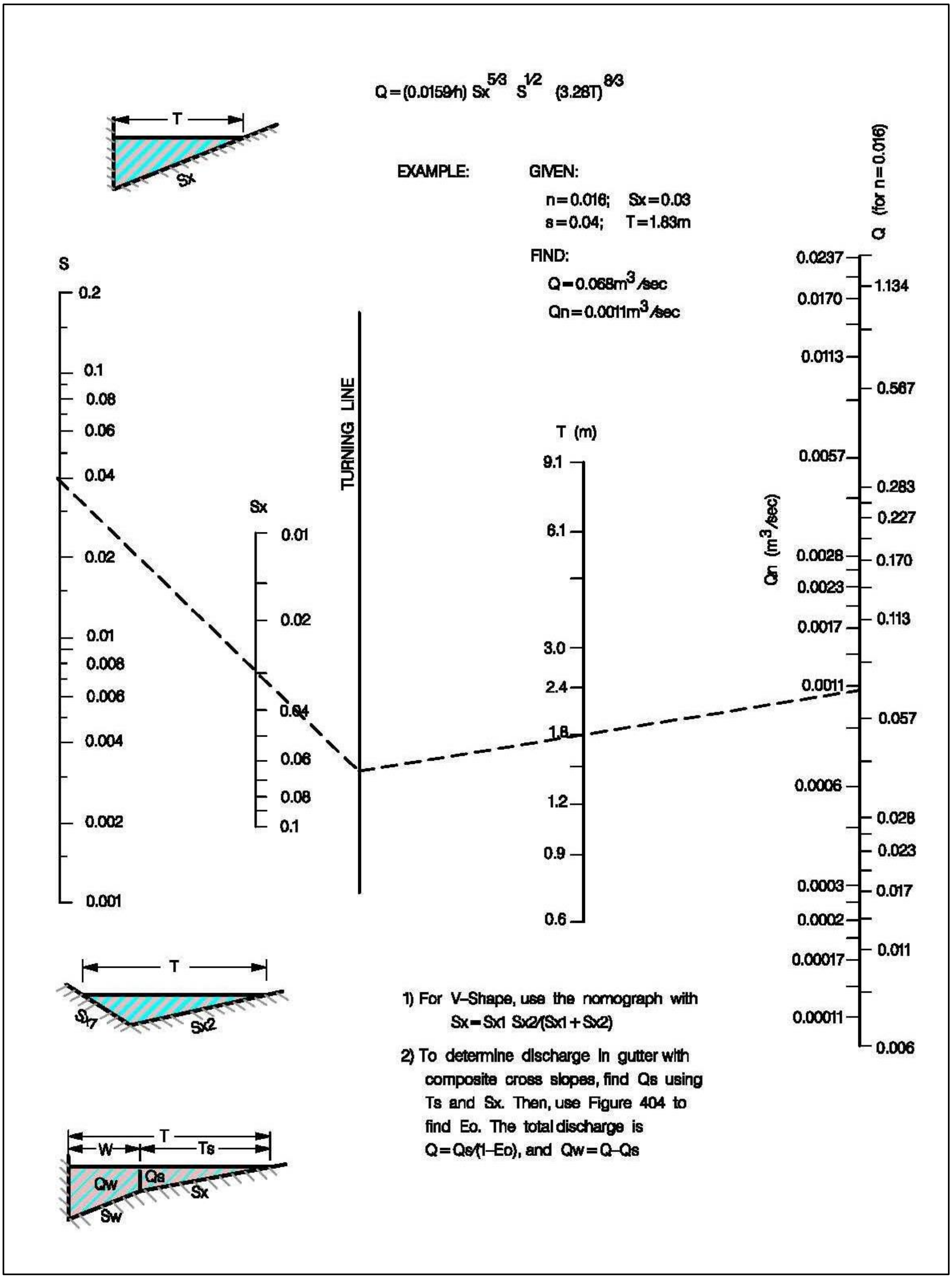


Figure A3- 3 - Flow in Triangular Gutter Sections

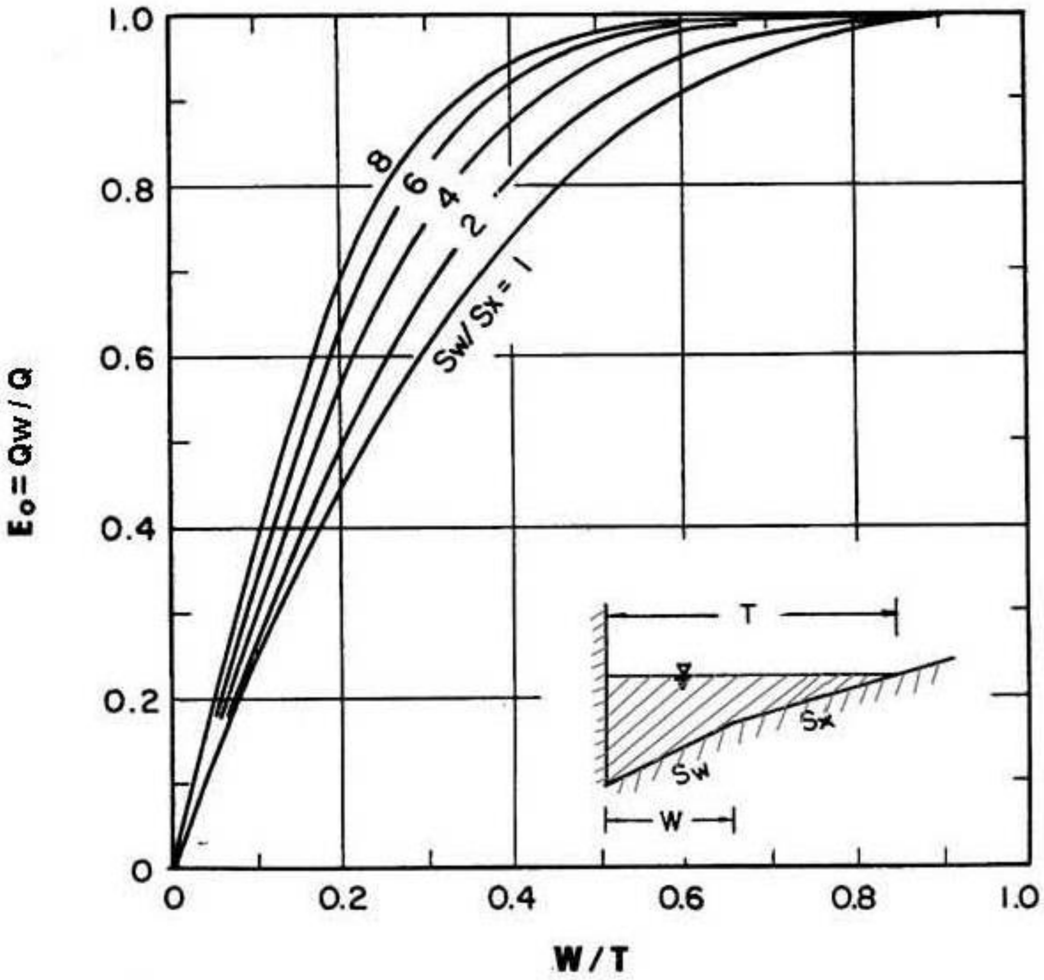


Figure A3- 4 - Ratio of Frontal Flow to Total Gutter Flow

A3.5 Inlet Fundamentals

Inlets for collecting pavement drainage can be divided into three major categories:

- kerb-opening (can be grated or un-grated)
- Grated gutter
- Combination

kerb-opening inlets are openings in the kerb face which are generally placed in a depressed gutter section. *Gutter inlets* consist of a metal grate or grates placed over an opening in the gutter. A modification of the gutter inlet is a slotted pipe or slotted trench drain that allows pavement drainage to enter continuously along its longitudinal axis. *Combination inlets* are composed of both a kerb-opening and a gutter type. Perspective drawings of these four types of inlets are presented in Figures A3-5 and A3-6. The sketches define fundamental parameters discussed in this section. They are not intended to be used as standard details of the Department.

Pavement inlets can be placed either on a *continuous grade* or in a *sump* or *sag condition*. If pavement drainage can enter an inlet from only one longitudinal direction, a continuous grade condition exists. On the other hand, if the inlet is located at a point where flow enters it from two directions, a sump condition exists.

The *interception capacity* of an inlet is the gutter flow that enters an inlet under a given set of conditions. The capacity changes as those conditions change. Factors affecting the interception capacity of kerb-opening and gutter inlets are briefly discussed in Sections A3.5.1 and A3.5.2.

The *efficiency* of an inlet is the percent of total gutter flow that the inlet will intercept for a given set of operating conditions. In mathematical form, efficiency is defined as:

$$\mathbf{E = Q_i/Q\ (100)} \qquad \textbf{(Eq. A3-7)}$$

where:

E = efficiency of an inlet, in percent
Q_i = intercepted flow, in m^3/sec
Q = total gutter flow, in m^3/sec

Flow that is not intercepted by an inlet is called *bypass* or *carryover*, and is expressed mathematically as:

$$\mathbf{Q_b = Q - Q_i} \qquad \textbf{(Eq. A3-8)}$$

where:

Q_b = bypass flow, in m^3/sec
Q = total gutter flow, in m^3/sec
Q_i = intercepted flow, in m^3/sec

In most cases, an increase in total gutter flow causes an increase in the interception capacity of an inlet and a decrease in its efficiency.

Pavement inlets do not provide an efficient method for collecting large quantities of stormwater. Therefore, non-pavement drainage should be collected upstream of the pavement where possible.

The use of capacity charts as presented by specific inlet grate manufactures and for the more general case in USDOT, FHWA, HEC-12 (1984) should be utilized to evaluate interception capacity of inlets, evaluated empirically by physical testing.

A3.5.1 Kerb-Opening Inlets

The advantages of un-grated kerb-opening inlets are that they are less susceptible to clogging and less hazardous to pedestrians and bicyclists than grated gutter inlets. However, they are usually not as efficient. Grated kerb-opening inlets are subject to the same clogging inefficiencies.

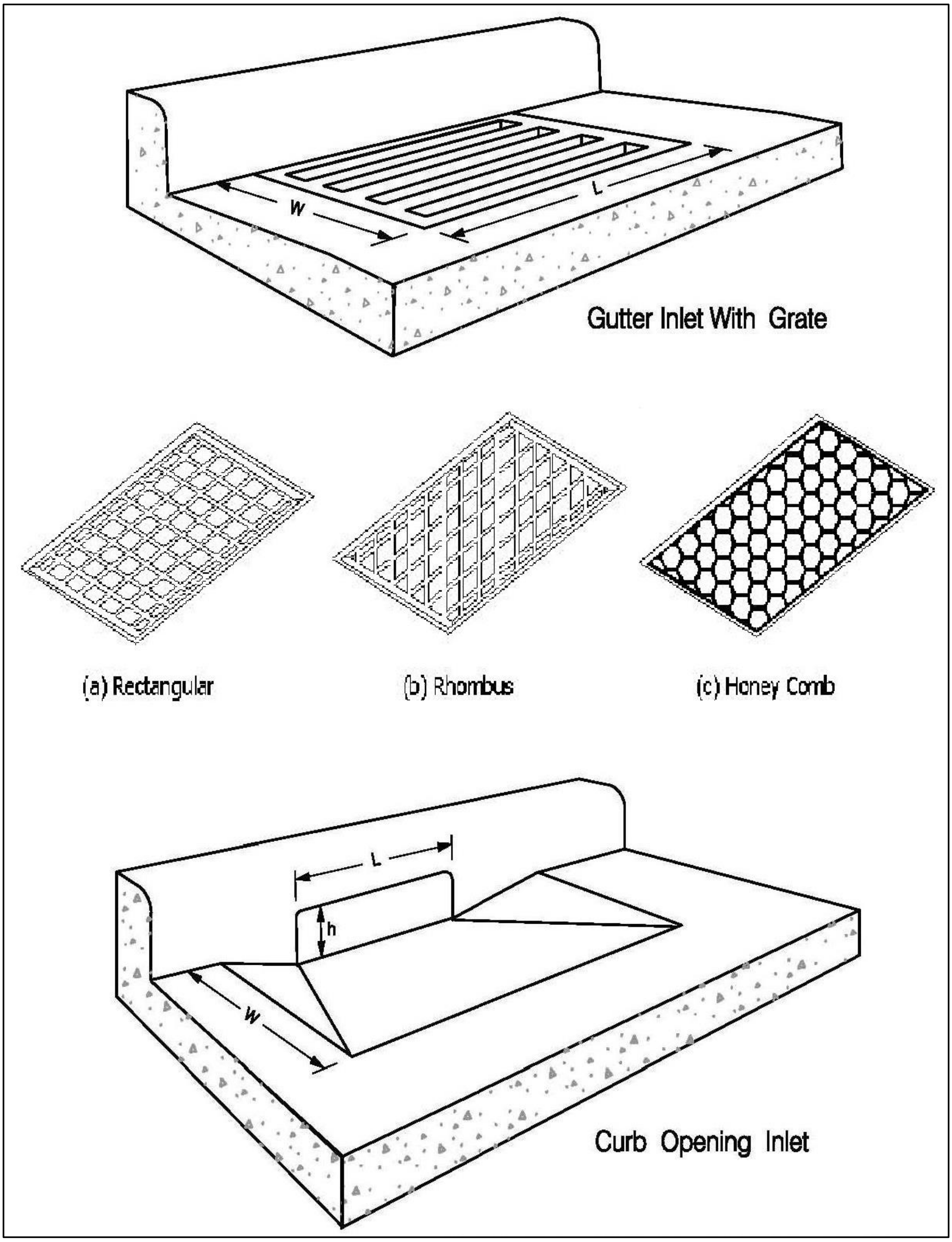


Figure A3- 5 - Perspective Views of Gutter and Curb-opening Inlets*

* This Figure is a modified version for the Stormwater and Subsoil Drainage Systems Design Manual

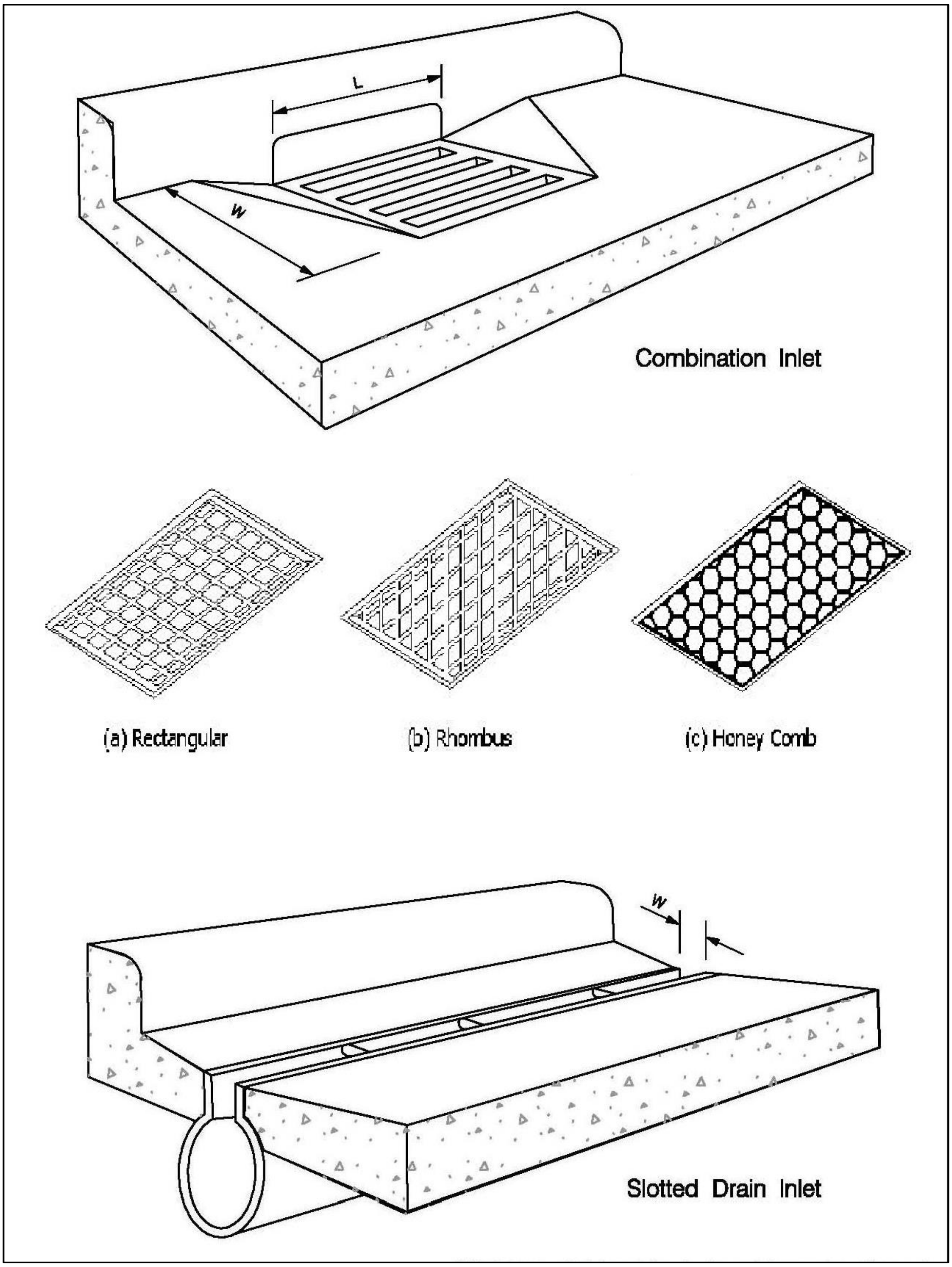


Figure A3- 6 – Perspective Views of Combination and Slotted Drain Inlets*

* This Figure is a modified version for the Stormwater and Subsoil Drainage Systems Design Manual

Continuous Grade

A kerb-opening inlet located on a continuous grade function as a falling head weir, the efficiency of which is affected by cross slope, longitudinal slope, total gutter flow, and weir length. The interception capacity of the inlet depends primarily on the flow depth at the kerb and the kerb-opening geometry. If the kerb opening can be depressed several inches below the gutter elevation, the interception capacity of the inlet can be increased. This can be done as part of a continuous gutter depression or as a local depression at the inlet. kerb inlets with grates are subject to clogging and the allowable design capacity should never be more than 50% - 60 % of the capacity calculated by the following formula (Various storm drainage grate manufacturers have published capacity charts based on actual testing under various conditions. These references should be obtained and compared to the theoretical capacities calculated via the formulae contained herein):

$$\mathbf{Q = 0.0015\ d\ L\ (10.76g\ d)^{1/2}} \qquad \textbf{(Eq. A3-9)}$$

or

$$\mathbf{Q = .093L\ \{1.87\ i^{0.579}\ [35.29Q_o\ /\ (s\ /\ n)^{1/2}]^{0.563}\}} \qquad \textbf{(Eq. A3-10)}$$

Assuming a gutter of wedge-shaped cross-section, where:

Qo = flow in the gutter in cubic metres per second
I = pavement or gutter cross-slope, m/m
S = hydraulic gradient of gutter, m/m
N = Manning's coefficient of roughness of pavement or gutter
Q = discharge into inlets in cubic metres per second
L = length of inlet opening in metres
D = depth of flow in gutter in metres

Sump Locations

kerb-opening inlets in sump locations operate as weirs up to a depth equal to the opening height. At depths above 1.4 times the opening height, the inlet operates as an orifice, and between these depths, a transition from weir to orifice flow occurs. The weir flow equation for a depressed kerb-opening inlet is expressed as:

$$\mathbf{Q_i = 0.065\ (3.28L + 5.904W)(3.28d)^{1.5}} \qquad \textbf{(Eq. A3-11)}$$

where:

Q_i = interception capacity of a depressed kerb-opening inlet operating as a weir and located at a sump, in m^3/sec
L = length of kerb-opening, in m
W = lateral width of depression, in m
D = depth of flow at kerb, measured from the normal cross slope, in metres

Equation A3-11 is applicable for flow depths less than or equal to the kerb-opening height plus the depth of the depression. This limitation is expressed mathematically as:

$d \leq h + a$ **(Eq. A3-12)**

where:

d = depth of flow at kerb, measured from the normal cross-slope, in metres
h = height of kerb-opening inlet, in metres
a = depth of depression, in metres

Since Equation A3-11 is based on a local depression, it will give conservative capacity estimates for inlets with a continuously depressed gutter.

The weir flow equation for kerb-opening inlets without a depressed gutter is expressed as:

$Qi = 0.214\ L\ (3.28d)^{1.5}$ **(Eq. A3-13)**

where:

Q_i = interception capacity of a non-depressed kerb-opening inlet operating as a weir and located at a sump, in m^3/sec
L = length of kerb-opening, in m
D = depth at kerb, measured from the normal cross-slope, in m

The depth limitation for weir flow represented by Equation A3-13 is expressed as:

$d < h$ **(Eq. A3-14)**

where:

d = depth at kerb, measured from the normal cross slope, in m
h = height of kerb-opening, in m

The orifice flow equation for evaluating the capacity of a submerged kerb-opening inlet is expressed as:

Qi = 0.204 A [6.56 g (3.28di - 1.64h)]0.5 **(Eq. A3-15)**

where:

Qi = interception capacity of a kerb-opening inlet operating as an orifice and located at a sump, in m3/sec
A = clear area of the inlet opening, in m2
G = acceleration due to gravity, 9.806 metres/sec2
H = height of kerb opening, including the depression height (a) if appropriate, in m
D = depth of flow at lip of kerb opening, in m

Equation A3-15 is applicable to depressed and undepressed kerb-opening inlets, and the height of the kerb opening includes the depression height, if appropriate.

A3.5.2 Grated Gutter Inlets

Gutter inlets usually have one or more metal grates covering an opening in the gutter. Grates come in many different shapes these shapes include but are not limited rectangular, rhomboidal, and honey comb (refer to Figures A 3-5 and A 3-6). Detailed hydraulic design should be carried out by an experienced engineer and approved by the ADM [added text for stormwater and subsoil drainage systems design manual]. Since debris carried by stormwater can clog the grates, effective operation may not be possible where the potential for debris transport exists. In any case, the efficiency used should be only 50% - 60% of the calculated capacity. The Consultant should also be aware of what happens to the bypass flows, and take care of them accordingly.

Continuous Grades

The water flowing in the section of a gutter inlet occupied by the grate is called *frontal flow*. When the gutter flow velocity is low enough, the grate inlet intercepts all of the frontal flow and a small portion of the *side flow*, which occurs along the length of the grate. As the gutter flow velocity increases, water may begin to skip, or *splash over*, the grate and the efficiency of the inlet may be reduced. If splash-over does not occur, the capacity and efficiency of a gutter inlet will increase with an increase of the longitudinal slope (the reverse of the effect on kerb-opening and slotted inlets).

The ratio of frontal flow to total gutter flow for a straight cross slope (see Figure A3-1) is expressed as:

$$\mathbf{E_o = Q_w / Q = 1 - (1 - W_{net}/T)^{2.67}} \qquad \textbf{(Eq. A3-16)}$$

where:
E_o = ratio of frontal flow to total gutter flow
Q_w = frontal flow in width (W), in m^3/sec
Q = total gutter flow, in m^3/sec
W_{net} = net width of gutter inlet or grate (see Figure A3-5), in m
= W - W_{bars}[modified for storwater and subsoil drainage systems design manual]
Where, W is the overall width of the grate and W_{bars} is the total width of thebars.
T = total spread of water in the gutter (see Figure A3-1) in m

The ratio of side flow to total gutter flow is expressed as:

$$\mathbf{Q_s/Q = 1 - Q_w/Q = 1 - Eo} \qquad \textbf{(Eq. A3-17)}$$

where:
Q_s = side flow intercepted by gutter inlet, in m^3/sec
Q = total gutter flow, in m^3/sec
Q_w = frontal flow in width (W), in m^3/sec
E_o = ratio of frontal flow to total gutter flow (see Equation A3-16)

The ratio of intercepted frontal flow to total frontal flow is expressed as:

$$\mathbf{R_f = 1 - 0.09\ (v - v_o)} \qquad \textbf{(Eq. A3-18)}$$

where:
R_f = ratio of intercepted frontal flow to total frontal flow
v = average velocity of gutter flow, in m/sec
v_o = average gutter velocity where splash-over first occurs, in m/sec

The ratio of intercepted side flow to total side flow is expressed as:

$$\mathbf{R_s = 1 / \{1 + [(0.15\ (3.28v)^{1.8}) / (S_x\ (3.28L_{net})^{2.3})]\}} \qquad \textbf{(Eq. A3-19)}$$

where:
R_s = ratio of intercepted side flow to total side flow
v = average velocity of gutter flow, in m/sec
S_x = cross slope of gutter, in m/m
L_{net} = net length of grate, in m

$= L - L_{bars}$

Where, L is the overall length of the grate and L_{bars} is the total length of the bars.

[modified for stormwater and subsoil drainage systems design manual]

The operating efficiency of a gutter inlet with a grate located on a continuous grade is expressed as:

$$\mathbf{E = R_f E_o + R_s (1 - E_o)} \qquad \textbf{(Eq. A3-20)}$$

where:
E = efficiency of the gutter inlet on a continuous grade
R_f = ratio of intercepted frontal flow to total frontal flow (see Equation A3-18)
E_o = ratio of frontal flow to total gutter flow (see Equation A3-16)
R_s = ratio of intercepted side flow to total side flow (see Equation A3-19)

The interception capacity of a gutter inlet with a grate located on a continuous grade is equal to the efficiency of the inlet multiplied by the total gutter flow, which can be expressed as:

$$\mathbf{Q_i = EQ = Q [R_f E_o + R_s (1 - E^o)]} \qquad \textbf{(Eq. A3-21)}$$

where:
Q_i = interception capacity of the gutter inlet on a continuous grade, in m^3/sec
E = efficiency of the gutter inlet on a continuous grade (see Equation A3-20)

Sump Locations

Gutter inlets in sump locations operate as weirs up to depths that are dependent on grate size and configuration, and as orifices at greater depths. A transition occurs between weir and orifice flow depths. In this transition, the capacity is ill-defined and may fluctuate between weir and orifice control.

The efficiency of inlets in passing debris is critical in sump locations. If the inlet plugs, hazardous ponding conditions can result. Since gutter inlets with grates tend to clog, it is generally beneficial to place an ungrated kerb opening inlet behind each grate. This is known as a *combination inlet*. However, because of the potential for clogging, it is usually appropriate to assume not more than 50% - 60% efficiency for the grated inlet portion.

The interception capacity of a gutter inlet with a grate operating as a weir in a sump location is expressed as:

$$\mathbf{Qi = 0.085 (3.28P) (3.28d)^{1.5}} \qquad \textbf{(Eq. A3-22)}$$

where:
Q_i = interception capacity of a sump gutter inlet operating as a weir, in m^3/sec
P = perimeter of the grate, disregarding bars and the kerb side, in m
D = depth of water above the top of the grate, in m

The interception capacity of a gutter inlet with a grate operating as an orifice in a sump location is expressed as:

$$\mathbf{Q_i = 0.204 A (21.52g d)^{0.5}} \qquad \textbf{(Eq. A3-23)}$$

where:
Q_i = interception capacity of a sump grated gutter inlet operating as an orifice, in m^3/sec
A = clear opening area of the grate, in m^2
G = acceleration due to gravity, 9.806 metres/sec^2
D = depth of water above the top of the grate, in m

The transition zone from weir flow to orifice flow for gutter inlets in sump locations is typically between a depth of about 120mm to 430mm. However, this transition zone is known to vary depending on perimeter and clear opening area. This transition zone should be evaluated by empirical methods and further information can be found from specific storm drainage grate manufacturer’s test data.

A3.5.3 Combination Inlets

Continuous Grade

On a continuous grade, the capacity of an unclogged combination inlet with the kerb opening located adjacent to the grate is approximately equal to the capacity of the grated gutter inlet alone due to the inefficiencies of the kerb inlet. The appropriate efficiency level is a function of the location and associated debris potential.

Sump Locations

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Since this will increase the probability of clogging for grated inlets, it is appropriate to delete the grate from the kerb inlet and assume a combination inlet at a sump will be limited by the kerb-opening capacity.

A3.5.4 Slotted Trench Drains and Slotted Pipe Inlets

A slotted trench drain is a version of the grated gutter inlet but is more susceptible to clogging. This inlet should not be used for main highway drainage but as an interceptor of sheet surface flow or for isolated low areas that are on flat grades and need an exceptionally long intercept length. The capacity for slotted trench drains can be calculated the same as for grated gutter inlets.

Slotted pipe is a version of the gutter inlet that allows pavement drainage to enter the pipe continuously along its longitudinal axis. Slotted pipes can be used on kerbed or un-kerbed pavement and present minimal interference to traffic and pedestrians; however, they are susceptible to clogging. The interception capacity of slotted pipe inlets on a continuous grade has been found to be hydraulically similar to that of a kerb-opening inlet. Both inlets function as falling head weirs, with the flow subjected to lateral acceleration caused by the pavement cross slope. The analysis of test data collected by the USDOT, FHWA for slot widths greater than 45 millimetres indicates that the length of inlet required for complete interception of gutter flow can be computed using an equation developed for kerb-opening inlets. This equation is expressed as:

$$\mathbf{L_T = 6.46\ Q^{0.42}\ s^{0.3}\ (1 / (n\ S_x))^{0.6}} \qquad \textbf{(Eq. A3-24)}$$

where:

L_T = length required to intercept 100 percent of gutter flow on a continuous grade, in m
Q = total gutter flow, in m^3/sec
S = longitudinal slope of gutter, in m/m
N = Manning’s roughness coefficient for gutter pavement
S_x = cross slope of pavement, in m/m

The efficiency of a slotted inlet on a continuous grade with a length shorter than that required for total interception can be computed using another equation developed for kerb-opening inlets, expressed as:

$$\mathbf{E = 1 - (1 - L/L_T)^{1.8}} \qquad \textbf{(Eq. A3-25)}$$

where:

E = efficiency of the inlet on a continuous grade, as a decimal

L = slotted inlet, or kerb-opening length, in m
L_T = length required to intercept 100 percent of gutter flow on a continuous grade (see Equation A3-24), in metres

Appendix A4 – Al Ain Municipality Requirements

A4. AL AIN MUNICIPALITY REQUIREMENTS

A4.1 Modified rational equation

When determining the total amount of runoff volume, the rational equation can be modified as follows:

$$\mathrm{R_V} = \mathrm{CPA}$$

Equation A4.1: Modified rational equation for runoff volume
Where:
RV = total volume of runoff (m3)
C = runoff coefficient refer to Table 3.6 and Table 3-7 (DEPARTMENT Drainage design manual).
P = precipitation depth (m) (precipitation depth from Figure A-1)

A4.2 Cost effective inlet and mainline layout and connection configuration:

Figure A-4.3 shows the most cost effective inlet collector and main drain layout and connection configuration for both urban and rural divided dual-lane kerbed roadways where sand storm have minimal impacts as per new Islands or areas which are not subject to sand storm. Inlets shall be arranged to discharge directly to the stormwater main drain by direct connection through a manhole structure. For this configuration, the following items apply:

a. Inlet connection shall be installed in straight alignment on a minimum uniform slope of 1 percent. Minimum sizes of pipes for expressways, freeways, and motorways shall measure 300 mm in diameter.
b. Maximum number of inlets connect to a manhole is four.
c. Maximum length of inlet connection drains is 35 m.
d. Drainage structure grates shall never be placed directly in the wheel path.
e. Locating grate inlets within pedestrian paths, or areas subject to bicycle traffic, shall also be avoided. If grate inlets must be located in roadway areas, where cyclists may be expected to travel, bicycle-proof grates are to be specified.
f. Structure offsets, shown in the plans, shall be to the centre of grate or cover, not to the centre of structure, to ensure the grate is located along the kerb face.
g. Proposed Stormwater drainage pipe lines and manholes shall follow the proposed by DPM – Abu Dhabi Infrastructure Framework plan and Utility Corridors location Design Manual and Design or the proposed Utility distribution Manual of DoT.
h. Basket inlets equipped with sand traps may be used as inlet structures, being smaller and less expensive to install then the traditional precast or cast-in situ concrete inlets.
i. Criteria for the location and spacing of manholes shall meet the requirements in DEPARTMENT design manual.

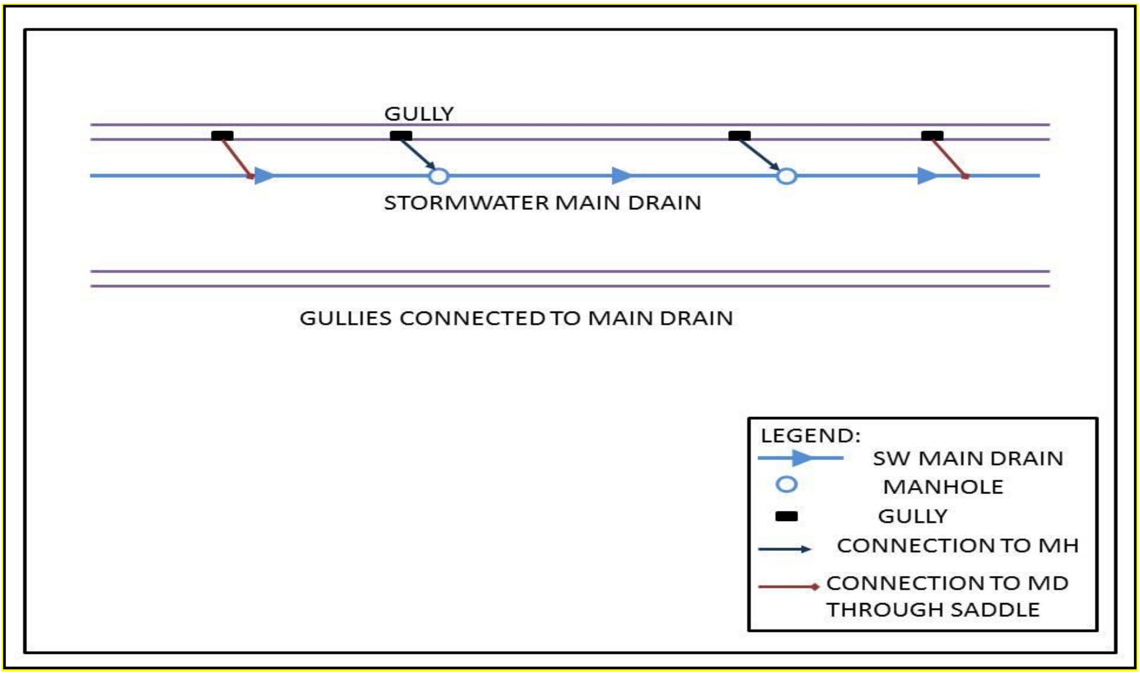


Figure A4- 1 - Inlet to Main Drain Connection Configuration – Preferred Method

This inlet layout method is preferable from a construction cost and hydraulic performance point of view; however, the selected arrangement and configuration shall be approved by the Al Ain City Municipality during early design phases, according to Project conditions.

On special cases (such as areas subject to minimal or no sand storms), and depending on the project limitations, inlets could be arranged to discharge directly to the storm water manhole structure without catch basin considering Abu Dhabi Standard dimensions for inlets and Catch basins.

Appendix A5 –Main Roads Requirements

A5. MAIN ROADS REQUIREMENTS

A5.1 Open channels and ditches

Open channels are a drainage feature in which water flows with a free surface. Open channels are generally classified as those that occur naturally, are manmade, or improved natural channels. Manmade channels, or artificial channels, used on most roadway projects, include Interceptor ditch, diversion channel, wadi channelization, roadside or median ditch, outfall ditch or canal and Low-flow ditch

This section provides specific requirements and criteria related to channel design analysis procedures and documentation. For a more complete discussion of practical channel hydraulics for road projects, refer to Chapter 4 of the FHWA publication HDS 4. For designs of larger outfall channels, the Design Engineer shall refer to the FHWA publication HDS No. 6or the American Association of State Highway Transportation Officials (AASHTO) Highway Drainage Guidelines.

A5.1.1 Catchment Delineation

Determining the size of the drainage area that contributes to the flow at the site of the drainage structure is a basic step in a hydrologic analysis, regardless of the method used to evaluate design flows. Drainage area, expressed in hectares or square kilometers, is frequently determined from field surveys, topographic maps, or aerial photographs. Catchment areas include all of the area that will contribute runoff to the point of interest such as open channel or cross drains and are delineated on a drainage map to support the design calculations as part of a design review submittal for all development phases of a project. Drainage maps shall include the following items:

- Drainage basin and sub-basin (catchment areas) identification;
- Discharge area delineations indicating flow paths and outfalls to receiving waters;
- Metrologic features of the catchments;
- Basin and catchment area size determinations;
- Existing drainage facilities, such as storm drains, inlets, cross culverts, and outfalls;
- Environmentally sensitive areas;
- Proposed solutions and alternatives layout plans; and
- Tentative details for main drain facilities.

A5.1.1.1 Areal-reduction factor

Over a drainage catchment area, the point rainfall will vary and the magnitude of this variation depends on the size of the catchment area due to variations in the storm as it moves through the area. Predicting this average rainfall depth for the whole area from the point rainfall is done by means of applying an A_{rf}. This factor is multiplied by the point rainfall depth or intensity when determining the runoff volume or flow rate.

$A_{rf}$are determined using a statistical analysis of long-term precipitation records and comparing results between the various recording stations within the region. When the Engineer lacks longer term precipitation data for the emirate’s interior recording stations to make this determination, an estimated A_{rf} based on storm rainfall records and statistical calculations for East Africa (refer to Transport and Road Research Laboratory Report 623, Prediction of storm rainfall in East Africa) can be used as follows:

$$Arf = 1 - (0.044A^{0.275})$$

Equation 1.1: Estimated A_{rf} based on storm rainfall records and statistical calculations for East Africa

Where: A = area in km^2

The Engineer shall only apply the Art to catchments greater than 25 km^2.

A5.1.1.2 Design Flow Calculation

For the hydrological parameters and peak design calculations refer to chapter 3 of this manual.

A5.1.1.3 Channel design criteria

Open channel design criteria are as follows:

Preferred cross section of a ditch is trapezoidal; however, a V-ditch may be used where ROW is limited and/or the design requirements can still be met. Note that channel depth shall be sufficient to remove the water whilst maintaining the required design water surface clearance below the pavement base courses.

1. Channels shall normally be designed for smooth, laminar subcritical flows as the normal flow depth is greater than critical flow.
2. Channel side slopes shall be stable throughout the entire length and side slopes shall depend on the channel material. Generally for all ditches, the maximum side slope of earth channels shall be 3 horizontal to 1 vertical (3h: 1v), depending on the soil type and geotechnical study requirements. Otherwise the ditch shall be lined using riprap or concrete.
3. Larger diversion or main outfall channels shall not be located parallel to the roadway, unless it is offset by at least the clear zone distance, as required by the Road Geometric Design Manual. Otherwise, the traffic shall be protected by use of guardrail or traffic barrier.
4. Roadside ditches are part of the roadway cross section, which typically have a maximum side slope of 4h: 1v for the side adjacent to the traffic or as otherwise required by the Road Geometric Design Manual.
5. Refer to Table A5.1 for drainage feature design values for maintenance consideration. Ditches and outfalls must be provided with beams and other physical access devices that facilitate maintenance activities.

Feature	Absolute minimum	Standard	Desirable
Ditch geometry			
Ditch side slopes	1.5h:1v (Rigid linings)[a]	4h:1v[b]	6h:1v[b]
	2h:1v (gravel and vegetative linings)[a], 3h:1v minimum for natural non-cohesive soils		
Ditch bottom width	1.5 m or 0.6 m greater than the culvert	Required by flow	Min. width to drive in for cleaning
Berms (top width)	3.5 m[c]	3.5 m[c]	6 m[d]
Change in alignment			

Feature	Absolute minimum	Standard	Desirable
Less than 45 degrees	Radius = 8 m	Radius = 15 m	Radius = 15 m
Greater than 45 degrees	Radius = 15 m	Radius = 15 m	Radius = 23 m
Ponds/retention area			
Fenced	2h:1v for vegetated or gravelled slopes, 1.5h:1v for paved or riprap slopes	3h:1v	3h:1v
Not fenced	3h:1vg	4h:1vf	6h:1ve

[a]Side slopes not suitable for roadside ditches within the clear zone, unless otherwise protected from traffic by guardrail or barrier.; however, the 1.5h:1v paved/rocked or a 2h:1v natural/vegetated side slope is often used for the ditch back slope.
[b]Side slope acceptable for recovery area adjacent to traffic areas shall meet the criteria in the DoT Road Geometric Design Manual(1), but is generally constructed on a 4h:1v, or flatter slope.
[c]Berm widths shall be suitable for maintenance equipment access. For roadside ditches with standard to desirable side slopes, maintenance access is provided by the roadway shoulder from within the ditch itself.
[d]This top of berm width is usually associated with main ditch or flood protection barriers, which are located away from the roadside.
[e]Use 6h:1v for vegetated landscape areas with public access, such as in parks or playgrounds.
[f]Vegetated slopes shall be easily maintained at this slope, especially if irrigated grass requiring mowing.
[g]Safety provisions for open channels in public areas, not otherwise fenced, require side slopes of not steeper then 3h:1v (so people can easily climb out of), design velocities within the allowable velocities for bare-earth channels, and where in the vehicle clear zone along highways, design depths no deeper than 1.3 m.

Table A5- 1 - Drainage features design value for maintenance consideration

6. Velocity of flow in open channels shall not exceed the maximum permissible velocities given inTable A5.2.
7. Ideally, a drainage channel shall have flow velocities that neither erode nor cause deposition in the channel.

Type of channels	Maximum permissible velocity (m/second)
Bare-earth channels:	
Fine sand and silts (non-colloidal)	0.8
Coarse sand	1.2
With channel linings:	
Gravel, 13-mm to 19-mm size	2.2
Coarse gravel, crushed > 19 mm	2.4
Coarse gravel with rock riprap (under 150-mm size)	2.7
Loose riprap with gravel bedding**	3.0
Grass with mulch	Less than 1.2
Sod; grass; or well-rooted, low-growing ground cover (irrigated and well maintained)	1.4
Asphalt	2.4
Geo-textile grid	1.2 to 2.4*
Concrete and grouted riprap linings	> 3.0**

Table A5- 2 - Maximum permissible velocities in channels

* Varies with grid type.
** Higher velocities acceptable with energy dissipaters and additional lining bedding and filter fabrics.

8. Minimum channel slopes are typically 0.3 percent for roadside collectors, medians, and main outfall channels. The Design Engineer shall maintain a flushing velocity for the design storm. This may require that larger channels contain a separate low-flow area located within the channel bottom. Minimum flushing velocity for lined channels is 0.6 m per second. Minimum velocity for bare earth channels shall not be less than 0.4 m per second.
9. Roadside ditch longitudinal slope usually follows the profile slope of the road for a uniform depth below the pavement level. The Drainage Design Engineer shall provide separate profile details for roadside ditches that require a different longitudinal slope (depth below pavement level varies) either to steepen the grade or for a reverse flow direction. Minimum flushing velocity shall be maintained.
10. Channel stabilisation (rock riprap or other lining) shall be provided where there is any flow disturbance, such as at contractions, drops, junctions, expansions, or for concentrated flows entering the channel from the side. Concentrated side flows may originate from storm drain outlets, concentrated over-the-bank flows, or side channels. Bank stabilisation shall cover the local side, bottom, and side slopes, including short sections up- and downstream of the affected flow disturbance area. Length of protection shall extend through the velocity transition, from uniform flow upstream to uniform flow downstream channel sections. Typically this will be at least 2.5 times the channel bottom width up- and downstream of the actual channel transition area.

    Longitudinal slopes of channels can be stepped such that non-erosive velocities are maintained in areas with steeply sloping terrain. Step or drop points between the grades shall be designed using energy dissipation, drop structure techniques. The Design Engineer shall refer to a speciality technical publication on this subject, such as the FHWA publication, HEC-14.
11. Open-channel drainage systems shall be sized to handle design storms, as mentioned in Table 1.3 with a 0.3-m freeboard. Channel depths shall allow the sufficient removal of water whilst maintaining the required high-water surface clearance below the pavement sub-base.

Type of Ditch	Design storm return period (years)*
Ditches – roadside, median, and interceptor	Freeways, expressways, and arterials: 10 years Collector and local roads: 5 years
Ditches – outfall	Freeways, expressways, and arterials: 25 years Collector and local roads: 10 years

Table A5- 3 - Design recurrence interval for Ditches

12. Design Engineer shall check the 50-year design storm hydraulics for main diversion and outfall channels. Routing analysis through the channel system shall determine if the 50-year frequency storm flow levels impacts any adjacent structures, causing excessive channel flows or overtopping that can erode and damage the roadway facilities.
13. Whilst not required for roadside ditches, larger diversion and outfall channels shall have their bends designed with sufficient radius to avoid additional freeboard requirements due to superelevation. Minimum radius (for channel velocities within the minimum and maximum allowable in accordance to natural or lining type) for bends, measured along the channel centreline, is two times the top width for design flow or a minimum radius of 7.5 m for

roadside collector ditches, 15 m for low-velocity outfall ditches, and 30 m for high-velocity outfall channels, whichever is greater. For other radius or for higher velocities, the flow depth superelevation for determining the freeboard elevation can be calculated using Equation 1.2.

$$\Delta d = (v^2 T)/(g R_c)$$

Equation 1.2: Flow superelevation at channel bends

Where:

Δd = superelevation of the water surface profile due to the bend (depth above the water surface on the tangent section) (m)

v = average channel velocity in tangent section (m/second)

T = top width of flow (m)

g = acceleration of gravity (9.81 m/second^2)

R_c= centreline radius of the bend (m)

14. Bends cause additional flow turbulence and eddies that increase shear forces along the outside of the bend. For bends with radiuses greater than the minimum discussed in 5, no additional erosion protection is required, other than that required for the design tangent sections. If the radius is smaller than the minimum, or velocities exceed that allowable for the natural or lining type, then rock riprap revetment protection shall be installed. Recommended longitudinal extent of protection is 1.5 times the width of channel bottom from both ends of the bend.
15. If maximum bend flow velocity is required, the calculation involves adjustment factors based on empirical data for various channel conditions and radiuses. Use the procedure in Chapter 8 of Part 654 of the NRCS National Engineering Handbook.
16. Safety provisions for open diversion and outfall channels in public areas, not otherwise fenced, require side slopes no steeper then 3h:1v (easy for people to climb out of), design velocities within the allowable velocities for bare earth channels, and where in the vehicle clear zone along highways, design depths no deeper than 1.3 m.
17. Open channels in mountainous areas may require the following additional design considerations:

    a. Steep longitudinal slopes in bedrock. Due to high velocities with supercritical flows, provide extra SF freeboard at bends, transitions, and locations of hydraulic jumps. Develop designs based on water surface profiles starting from a definable control point upstream of the design section and calculating the energy loss/surface elevations in a downstream direction.
    b. Open channels in mountainous terrain usually have high bed loads in the flow, with the higher velocities moving large quantities of gravel, cobbles, and boulders (depending on velocity) along the bottom. Estimates of the bed load types and movements need to be made, and allowances in size, inverts, and scour protection provided at structures, in particular for channels crossing the roadway through culverts and bridges.

       Culverts require specific protection, larger sizes, or thicker walls to help withstand the abrasion effects of these bed loads. If the high velocity abruptly slows at a transitional drop, widening or outlet of a structure, these bed loads will accumulate and reduce the capacity of the channel. Provide maintenance access to the culvert inlet to assist with removal of this material by maintenance crews.
    c. Mountainous roads in the cut section, with roadside waterfalls, need to be provided with large inlet structures designed with barriers, such that high-flow volumes, debris, and

bed loads will not affect the traffic area, but be contained by the inlet, to flush through the culvert to the downstream side of the roadway.

d. Energy dissipaters may be required to slow high-velocity flows to acceptable velocities before transitioning to a side of roadway channel, or where the channel is discharging from a rock-lined channel to an earth channel.
e. The Design Engineer shall refer to a speciality reference for additional design details and considerations, such as the FHWA publications HEC-15and HEC-20.

18. For evaluating the capacity of natural channels and wadis, geometric elements are usually not well defined. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so flow can be computed separately from the main channel and for overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients.

    a. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as a wetted perimeter.
    b. Use of surveyed cross sections and a channel modelling programme, such as the U.S. Army Corps of Engineers' (USACE) hydrologic engineering centre's River Analysis System Programme (HEC-RAS), will help to facilitate the analysis.

A5.1.1.4 Design method

Channel design is a trial-and-error process using the following general steps:

1. Determine the catchment area and the existing ground level/ topography conditions and calculate the design storm peak runoff flow rate.
2. Select a trial channel cross section and longitudinal slope. Typically these will be assumptions based on the Design Engineer's experience that best fits the existing or proposed terrain and the required roadway cross-sections and ROW.
3. Calculate the flow capacity of the trial channel using Manning's equation 1.3and compare calculated capacity against the design flows. If the channel capacity is less than the required design flows, increase the bottom slope or increase the cross-section dimensions and recalculate.
4. Calculate the design velocity. Compare to the allowable velocities in Table A5-2 Adjust channel geometry and longitudinal slopes as necessary to reduce velocity to less than allowable. If this is not possible, then provide the necessary channel erosion protection at the bottom and side slope armouring or lining.
5. Check and provide calculations for additional criteria items, such as freeboard, clearance of design water surface below pavement, and critical depth.

Before finalising a channel design, the Design Engineer must verify that the normal depth of a channel is either greater or less than the critical depth. Generally, open channels shall be designed with greater than critical depths for laminar, uniform flows. Designs within this flow regime will help assure that the channels and ditches are stable and non-erosive.

Hydraulic principles of open-channel flow are based on steady state uniform flow conditions. Though these conditions are rarely achieved in the field, generally the variation in channel properties is sufficiently small enough that the use of the uniform flow theory will yield sufficiently accurate results.

Manning's equation is the fundamental formula for performing open-channel capacity calculations.

$$Q = A\left(\frac{R^{2/3}S^{1/2}}{n}\right)$$

Equation 1.3: Manning's equation

Where:

Q = (V)(A) = flow rate (m^3/second)
V = average flow velocity (m/second)
A = cross sectional; areas of flow (m^2)
R= hydraulic radius = cross sectional area of flow divided by the wetted perimeter (m)
S =slope of energy line (m/m)
n = Manning's roughness coefficient (refer to **Error! Reference source not found.** for more a dditional information.

Manning's equation is used to determine the velocity of flow at a specific point in the channel therefore the variables in the equation must be representative of the point being assessed. Choice of an appropriate value for Manning's roughness coefficient for the design of an open channel is critical and requires a considerable degree of judgement. Commonly accepted values for Manning's roughness coefficient (**n**) are provided in Table A5-4.

Type of Channels	'n' value
Unlined channels	
Clay or silt	0.023
Sand	0.020
Gravel	0.030
Rock	0.040
Lined channels	
Rock riprap (loose, graded with tamped	0.035
Portland cement concrete	0.014
Air blown mortar (trowelled)	0.012
Air blown mortar (untrowelled)	0.016
Grouted rock (smooth surface)	0.014
Grouted rock (rough surface)	
Asphalt concrete	0.015
Pavement and gutters	
Portland cement concrete	0.015
Asphalt concrete	0.016

Table A5- 4 - Average values of manning's roughness coefficient (n) for channels

For additional design considerations and procedures, the Design Engineer shall refer to the FHWA publication HDS 4.

A5.1.1.5 Chutes

Chute is usually referring to steep open channel which used to convey water flow down steep slopes. Chutes may be lined or stepped depending on flow velocity. Chute lining may be grouted riprap or concrete. The outlet velocity of the chute must be considered in relation to the Downstream material type. The limits of the velocity as function of the soil type is presented in Table A5-2. Moreover, energy dissipation structure (e.g., stilling basin) may be added at chute outlet due to high exist velocity (more than 4.5 m/s). For outlet flow velocity less than 4.5 m/s;

loose riprap, grouted riprap, gravel, or stones may be used based on flow velocity values. For more details, regarding the velocity and the type and design of the stilling basin, refer to FHWA's HEC-11, HEC-14, and HEC-15. Typical details of the chute are given in volume 2: standard Drawings: Main Roads: DWG No. 236

If the height of road embankment exceeds 3.0 m, chutes should be used to protect embankments in fill section. Higher road embankment without chutes could be accepted upon receiving the related calculations that prove the embankment safety and stability. Chutes spacing should be properly designed to avoid water spread that exceeds the allowable extents. Chute flow depth may be greater than those predicted using Manning's equation due to entrained air. This can be reimbursed using a larger "n" value.

A5.2 Cross Drainage

Cross drainage involves the conveyance of surface water and stream flow across or from the highway ROW. This is accomplished by providing one of the following:

- Culvert; or
- Bridge.

Crossings will normally be done using a bridge for higher flows and deeper crossings or using a culvert. Culverts are discussed in Clause A5.2.1. Bridge crossings are described in Clause A5.2.2.

Culverts are hydraulically short conduits that convey runoff flow through a roadway embankment. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

Culverts and multiple barrel culverts shall meet the requirements listed in SectionA5.2.1.1 and A5.2.1.2, regarding placement and sizing.

Decisions whether to use a bridge or a culvert, or multiple barrel culverts, shall be based on the best-value assessment when considering the impacts of the crossing opening on the channel flow and flood plain, scour effects, construction methods, public safety, and maintenance concerns.

Sizing of culvert and bridge openings is dependent on the waterway hydraulics regarding high water clearance, allowable upstream flooding depths, submergence of inlets, and scour effects.

This section provides a description of the constraints and criteria for crossing designs with a general discussion on the design methodology. It is recommended that the Design Engineer refer to specific references on bridge and culvert hydraulic design for a more detailed discussion on the engineering principles. Suggested references include the following:

- AustRoad publication Waterway Design: A Guide to the Hydraulic Design of Bridges, Culverts and Floodways (35); and
- FHWA publication HDS 5 Hydraulic Design of Highway Culverts (21)

A5.2.1 Culverts

A5.2.1.1 Culvert type

When selecting a culver type, the following factors shall be considered:

- Application;
- Shape;
- Material; and

- End treatment.

These factors are discussed in detail below:

I. Application:

- For purposes of application, the following definitions apply:

c. Cross drain: Concrete box or pipe culvert placed transversely under the highway section, with end walls or some other end treatment.
d. Side drain: Culvert pipes that are used longitudinally to connect roadway ditches under driveways, ramps, or intersections. Same principles apply to side drains as for cross drains.
e. Median drain: Culvert pipe used to drain median ditches to the outside of roadway. Median end may be open or connected to a flush inlet structure.

II. Shape and material:

- Cross drain pipes used throughout Abu Dhabi shall be circular pipes and shall meet the material selection limitations.
- Concrete box culverts shall be structurally designed to withstand all live and dead loads for both the construction and permanent conditions in accordance to the requirements within the DoT Road Structures Design Manual (30).

III. Concrete box culverts shall also meet the pipe durability, service life, corrosion protection, and abrasion resistance for concrete pipe as per manufacturers requirement.

IV. Size:

- Minimum culvert sizes shall meet the requirements of Table A5-5.

Culvert type	Minimum size (mm diameter)
Cross drain**	600
Median drain	500*
Side drain	500*
Box culvert	900 by 900 precast 1,200 by 1,200 cast-in situ
Drains from inlets on high fills (e.g., gutter drain)	300*

Table A5- 5 - Minimum culvert sizes

*When debris control is provided by grates and sand trap facilities use a 300-mm-dia. minimum.
**Size of cross drains shall also be agreed with the O&M department, according to site conditions.

V. End treatment:

Selection of end treatment facilities must be consistent with hydraulic requirements, maintenance concerns, and provide vehicle collision safety where culvert ends terminate within the roadway clear zone.

A5.2.1.2 Culvert Fundamentals

1. Flow control:

Culverts can operate either under inlet control, where the barrel has a greater hydraulic capacity than the inlet; or, under outlet control, where the inlet has a greater hydraulic capacity than the barrel.

When a culvert is operating under inlet control, the barrel will flow partially full (i.e., the culvert is capable of carrying more flow than the inlet will accept). Culverts flowing in inlet control have a shallow, high-velocity flow categorized as supercritical. For supercritical flow, the control section is at the upstream end of the barrel, or the inlet. When an outlet is submerged under inlet control, a hydraulic jump will occur inside the barrel.

When a culvert is operating under outlet control, the barrel is intended to flow full for design conditions (i.e., the culvert barrel is not capable of conveying as much flow as the inlet will accept). Culverts flowing in outlet control will have relatively deep, lower velocity flow — termed subcritical flow.

For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross sectional area of the culvert, inlet geometry, and HW elevations at the entrance are of primary importance. Outlet control involves the additional consideration of the outlet channel TW elevations and the slope, roughness, and the length of the culvert barrel.

Discussions of these two types of control, and charts for selecting a culvert size for a given set of conditions, are included in the FHWA publication HDS 5.

2. Headwater:

HW is the depth of the upstream water surface measured from the invert of the culvert entrance.

It is not always economical or practical to use all of the available HW, especially where debris must pass through the culvert or where the natural gradient is steep and high outlet velocities are objectionable.

3. Tailwater:

Tailwater (TW) depth is either critical depth at the culvert outlet or the downstream channel flow depth, whichever is higher.

A5.2.1.3 Culvert design procedures

General procedure to follow when designing a culvert is summarised in the steps below. Design Engineer shall refer to a detailed reference on culvert theory and design methods, such as the FHWA publication HDS 5. Culvert spans more than 6 m, or a width paralleling the roadway centreline, may be subject to additional performance measures, as described in SectionA5.2.1.

a) Calculate the culvert design flows:

- The crossing culvert are designed for 50 years storm return intervals for freeways, expressways, and arterials and check the 100-year design for upstream flooding or overtopping. For collector and local roads, 25-year design storm return intervals is recommended for design and 50 – year for check the upstream flooding or overtopping.

b) Determine the allowable headwater (HW) elevation:

- Circular and box culverts shall be designed such that the ratio of the HW to diameter (D) during the design flow event is less than or equal to 1.25 ($HW_i/D \leq 1.25$). HW_i/D ratios larger than 1.25 are permitted, provided that existing site conditions, such as available ponding areas on deeper fills warrant a larger ratio. The maximum allowable HW_i/D ratio shall never exceed 3.

c) Determine the TWL elevation at the design flow:

- Hydraulic conditions downstream of the culvert site shall be evaluated to determine a TWL depth for the design discharge. For culvert outlets operating under free-fall conditions, such as a cantilevered pipe; critical depth and equivalent hydraulic grade line are determined using procedures as discussed below.
- For culverts discharging to an open channel, normal depth of flow in the channel shall be evaluated using procedures presented in Section A5.1.1.3
- For culverts discharging to a reservoir, wadi, tidal bay, or other major water body; the expected high water elevation of the particular water body or wadi for the same size or frequency storm may establish the culvert TWL.
- Where tidal conditions occur at the outlet, the mean high water shall be used.

d) Determine the type of control that exists at the design flows, either inlet or outlet control:

- There are two basic types of flow control, i.e., inlet control and outlet control. Two different methods are used to determine the HW, one for inlet and one for outlet control.

Design of the culvert is made using a trial-and-error process, wherein an initial size is selected and HW depths are calculated for both flow control conditions. If the higher HW elevation control exceeds the allowable level, then another culvert size is selected and the calculations repeated until an acceptable HW depth is obtained.

- For inlet control, the HW to diameter ratio is readily obtained from the appropriate nomographs, as contained in the FHWA publication HDS-5(21). Otherwise, HW can be solved for using a tube-type orifice equation, adjusted for different inlet conditions, as described in most hydraulic engineering texts.
- For outlet control, the total amount of head loss in the barrel of the pipe, including the minor losses at the entrance and the exit of the pipe. Head loss can be determined by using Equation .

$$H = \left[1 + K_e + \frac{19.63 n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Equation 2.1 Head loss equation

Where:

H = Head loss (m)

K_e= entrance loss coefficient (Refer to additional information in **Error! Reference source not found.**).

n = Manning's equation roughness value 'n' (Refer to **Error! Reference source not found.** for more information)

L = length of the culvert barrel (m)

R = hydraulic radius (m/sec.), or cross-sectional flow area (m^2) divided by the wetted perimeter (m)

V = velocity of flow (m/second), or flow rate (m^3/second) divided by the cross sectional flow area (m^2)

g = acceleration of gravity, or 9.81 m/second^2

- The head loss (H) may also be determined by the outlet control nomographs shown in the FHWA publication HDS 5and as presented in Vol-II of this manual Appendix C. Both the nomographs and the equation are based on the assumption that the barrel is flowing completely full or nearly full.
- Depth of HW above the culvert invert resulting from outlet control is determined by Equation 2.2: HW equation 2.2.

$$HW_o = H + h_o$$

Equation 2.2: HW equation

Where:

ho = actual TWL depth above the culvert invert or the term (dc + D)/2, whichever is greater.

Culvert Material	"n" value
Concrete pipe or box (non-lined)	0.015
Concrete pipe (PVC-lined)	0.011
Vitrified clay pipe	0.013
Corrugated metal pipe	0.022 – 0.027*
68 mm by 13 mm – annular or helical	0.025
76 mm by 25 mm – annular or helical	0.028
125 mm by 25 mm – annular or helical	0.026
152 mm by 51 mm – annular or helical	0.035
229 mm by 64 mm – annular or helical	0.035
Spiral rib metal pipe:	
19 mm (W) x 25 mm (D) @ 292 mm o/c	0.013
19 mm (W) x 19 mm (D) @ 191 mm o/c	0.013
19 mm (W) x 25 mm (D) @ 213 mm o/c	0.013
Corrugated polyethylene – smooth	0.009 – 0.015*
Corrugated polyethylene – corrugated	0.018 – 0.025*
PVC – smooth	0.009 – 0.011*
Composite steel spiral rib pipe	0.012

Table A5- 6 - Manning's equation roughness coefficient (n) for culverts

*Refer to pipe manufacture's data sheets

Culvert end treatment	Ke
Pipe concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5

Culvert end treatment	Ke
Headwall or headwall and wing walls	
Socket end of pipe (groove-end	0.2
Square-edge	0.5
Rounded (radius = D/12	0.2
Mitred to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Bevelled edges, 33.7°or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe or pipe-arch corrugated metal	
Projecting from fill (no headwall)	0.9
Mitred to conform to fill slope, paved or unpaved slope	0.7
Box, reinforced concrete Headwall parallel to embankment (no wing walls)	
Square-edged on three edges	0.5
Rounded on three edges to radius of D/12 or B/12, or bevelled edges on three sides	0.2
Wing walls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or bevelled top edge	0.2
Wing walls at 10°to 25° to barrel	
Square-edged at crown	0.5
Wing walls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2
*Note: "End sections conforming to fill slope," made of either metal or concrete, are the sections commonly available. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance. These latter sections can be designed using the information given for the bevelled inlet.	

Table A5- 7 - Culvert entrance loss coefficients

e) Calculate outlet velocities:

a. If the controlling HW is based on inlet control, determine the normal depth and velocity in the culvert barrel. Velocities at normal depths are assumed to be the outlet velocity. This can be calculated by using Manning's equation.

b. If the controlling HW is in outlet control, the outlet velocity is the flow rate divided by the cross-sectional flow area. Determine the area of flow at the outlet based on the barrel geometry and the following:

   i. Critical depth, if the TWL is below critical depth;
   ii. TWL depth, if the TWL is between critical depth and the top of the barrel; and
   iii. Height of the barrel, if the TWL is above the top of the barrel.

c. Outlet flows shall closely match the velocity of the existing channel. If they do not, an alternate culvert design that reduces the outlet velocity or uses energy dissipaters shall be considered

f) Provide embankment slope protection at inlet and outlet of culvert:

a. Install rock riprap, geo-cell panels, or concrete geo-synthetic cementicious composite sheets on inlet slope to a height equal to the design HW and width of 2D on each side of the culvert opening.
b. Install rock riprap, geo-cell panels, or concrete geo-synthetic cementicious composite sheets on outlet slope to a height equal to 5D above the top of the culvert and width of 1D on each side of the culver opening.

g) Provide channel erosion protection at the inlet and outlet of culvert, if required.

A5.2.2 Wadi bridges

Bridge opening hydraulics will be used to design larger box culverts and bridges across Wadi's in (Al Ain). Bridges shall be considered when design flow is large and using multiple barrel culverts is not effective, or where located in channels with high-velocity flows with debris or heavy bed loads. Hydraulic designs that size bridge or large culvert openings shall meet the following hydraulic criteria:

I. Bridges and large culverts shall be designed to have sufficient hydraulic capacity to convey the required design storm frequency channel flows without damage to the structure and to approach embankments.
II. Design of all structure openings shall be analysed for the peak design storm frequency flow of 100 years design return period maintaining 1 m clear between high water and bridge deck for freeways, expressways, and arterials and 1:50 years design return period for collector and local roads.
III. Backwater Effects: bridges have an effect on the flow of the watercourse over which they are located. This is because part of the bridge structure is usually located within the channel of the river or stream, and causes an obstruction of water flow. Bridges which cross a river in more than one span have piers located in the watercourse, which force water to flow around them. The abutments of bridges also generally protrude into the watercourse, causing obstacles at either side of the channel. Even if under normal flow conditions no part of the bridge is obstructing flow, in flood conditions, where the water level is significantly raised, parts of the bridge superstructure may cause an obstruction to flow.
IV. As water flowing in the channel approaches a bridge structure that restricts its flow area, the flow is forced to contract, in order to pass through the bridge, before expanding once again to the full channel width. As the constricted flow passes through the obstruction of the bridge, it accelerates, causing a depression in level of water surface. As the flow expands once more to the full channel width, so the water level recovers, to its downstream boundary condition level. The successive contraction and expansion results in a local head loss, which is compensated by an increase in water level upstream of the bridge. This phenomenon is known as afflux, or the backwater effect. Water levels upstream are raised by Δy with respect to downstream levels, (refer to**Error! Reference source not found.**), where Δy is equal to the head loss caused by t he contraction and expansion of flow at the bridge.
V. Design flows shall be determined in accordance to the procedures in Chapter 3. Hydraulic conditions for the design and overtopping flows shall be determined using an open channel-type modelling program, Modelling of backwater flow effects with HEC-RAS HEC-RAS can be used to model the effects of a bridge crossing a watercourse, and to examine backwater effects). In particular, modelling shall be done for both the existing channel and the proposed crossing conditions and differences compared.

VI. Bridge crossings of sea channels, inlets, and bays with tidal flows shall be analysed using an engineering approach suitable for the given site and shall consider flow due to normal tidal exchange, storm surge, and wave attack.

VII. Design of the bridge flow openings shall be such that the proposed water surface is not increased more than 0.3m at a distance of 50 m upstream of the bridge — above the existing water surface in the channel during the peak design storm frequency flow.

VIII. Low chords of the structure shall provide at least a 1-m clearance above the peak design storm frequency water surface or, in areas with tidal influence, of 1-m above the mean high tide level.

IX. Economical approach for crossings of wide wadi floodplains will usually require a bridge structure over the main flow channel. This bridge opening is sized for the maximum flow occurring within the wadi channel banks, but does not overtop the channel banks.

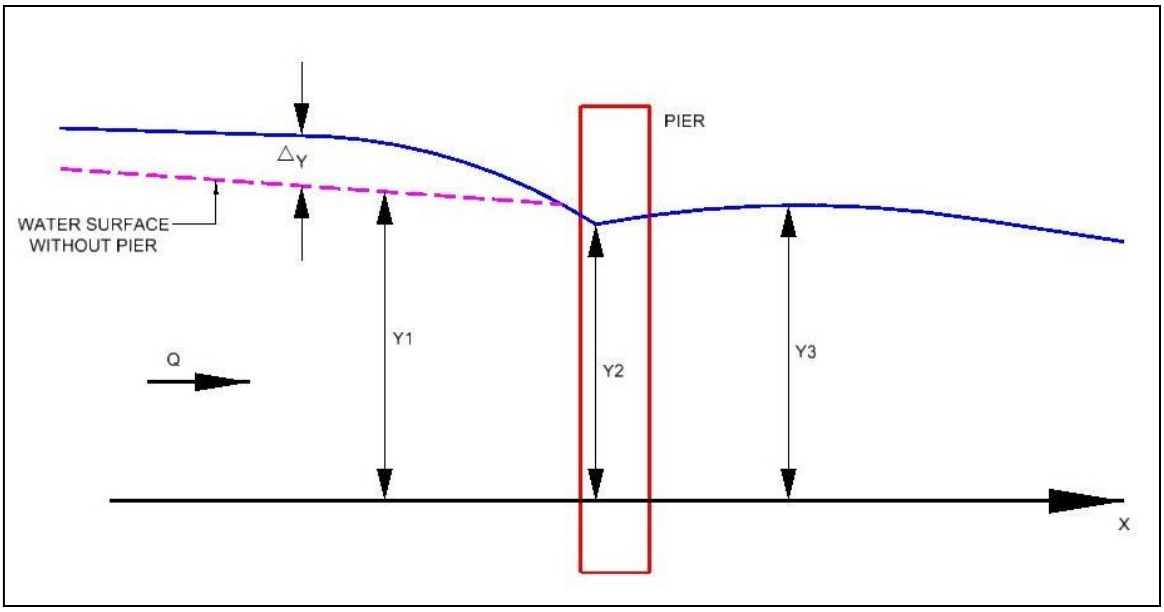


Figure A5- 1 - Schematic of backwater effect cause by bridge pier

Larger storms, that overtop the banks and spread out across the floodplain, require additional openings under the roadway. These are called relief crossings and can consist of additional bridge spans or large culverts and shall be placed at locations that match the low points in the floodplain terrain. Relief crossings will generally be at higher levels than the main channel.
An example of this is shown in Figure A5-2, where the bridge's main span is sized for the main channel area, with additional spans acting as flood relief. Although this picture shows the relief crossings using additional spans, they can also be provided through the roadway fill using large culverts. Bridge hydraulic design programs, such as HEC-RAS, will perform modelling of the design flows through both the main and secondary relief openings. Total opening areas are sized to meet the design requirements noted above.

Figure A5- 2 - Wadi bridge crossing with floodplain relief spans

X. For crossings subject to small boat traffic, a minimum horizontal clearance of 5 m and a vertical clearance of 3 m shall be provided. For boat access under bridges that cross shipping channels, the minimum clearances shall be in accordance with the requirements of the Port Authority. Where no boat traffic is anticipated, horizontal clearances shall be consistent with hydraulic requirements and structure economy. Horizontal clearances are defined as the unobstructed clear distance between piers, fender systems, and culvert walls, which is projected by the bridge or box culvert normal to the flow.

XI. Bridge foundations and abutments shall be analysed for scour depth for the design storm frequency and the overtopping storm frequency flows. Scour analysis will discuss recommended foundation depths and scour protection measures. Effect of debris may need to included in the hydraulics and scour calculations.

XII. To facilitate maintenance and provide for abutment stability, a minimum berm width of 3 m shall be provided between the top edge of the main channel and the toe of bridge abutments.

XIII. Special consideration shall be given to roadway locations across desert wadis, washes, and similar natural geographical features. Where roadways are located across a succession of outwash areas, discharge is typically infrequent, wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridging the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

XIV. Final bridge opening requirements and calculation should be summarised in a Bridge Recommendation report. These reports will define the recommended option for both the hydraulic and road geometric requirements for the most cost-effective design.

A5.2.2.1 Scour

Scour estimates shall be developed using a multidisciplinary approach involving the Hydraulics, the Geotechnical, and the Structures engineers.

Bridges and bridge culverts shall be designed to withstand the design flood without damage and shall withstand the overtopping storm frequency flow without failure. Scour estimates for these events shall consist of the total scour (refer to **Error! Reference source not found.**for a n example resulting from the followings.

1. **Natural or manmade channel aggradations and degradation:**

- Aggradations and degradation are long-term streambed elevation changes due to natural or manmade causes that can affect the reach of the river on which the bridge is located. Aggradations deposits material eroded from the channel or watershed upstream of the bridge; whereas, degradation lowers or scours the streambed due to a deficit in sedimentsupply from upstream.

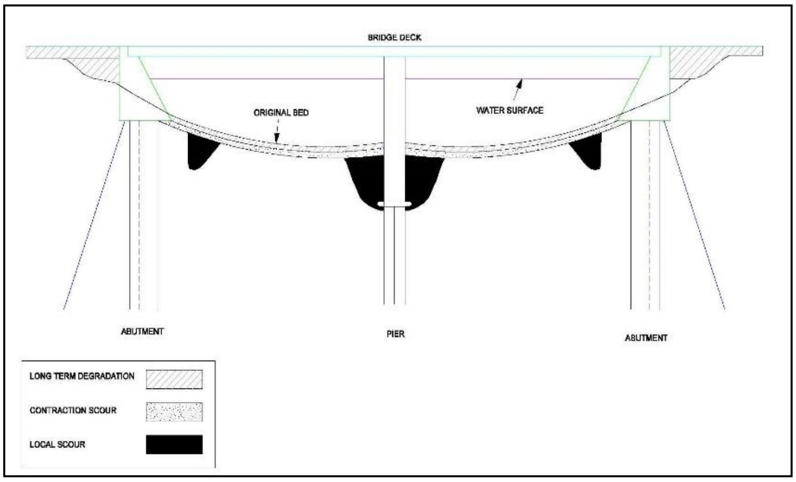


Figure A5- 3 - Schematic of typical bridge scour

2. **Channel migration anticipated during the life of the structure:**

- Naturally occurring lateral migration of the main channel of a stream within a floodplain may affect the stability of the piers, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers and abutments. Factors that affect lateral stream movement also affect the stability of a bridge foundation. These factors are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials.

3. **General scour, including contraction scour:**

- General scour is the general decrease in the elevation of the bed across the bridge opening. It does not include localised scour at the foundations (local scour) or the long-term changes in the stream bed elevation (aggradations or degradation). General scour may not have a uniform depth across the bridge opening. General scour can be cyclic, that is, there can be an increase and decrease of the stream bed elevation (cutting and filling) during the passage of a flood.
- Contraction scour is a common general scour. There are several cases and flow conditions for contraction scour. Typically, contraction scour occurs where the bridge opening is smaller than the flow area of the upstream channel or the floodplain. Additional general scour conditions can result from erosion related to flow characteristics of the stream, flow around a bend, variable downstream control, or additional changes that decrease the bed elevation at the bridge.

4. **Local scour, including pier and abutment scours:**

- Basic mechanisms that cause local scour at piers or abutments include the formation of vortices (known as the horseshoe vortex) at their base (refer to **Error! Reference source not found.**for an example). Horseshoe vortices results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment.

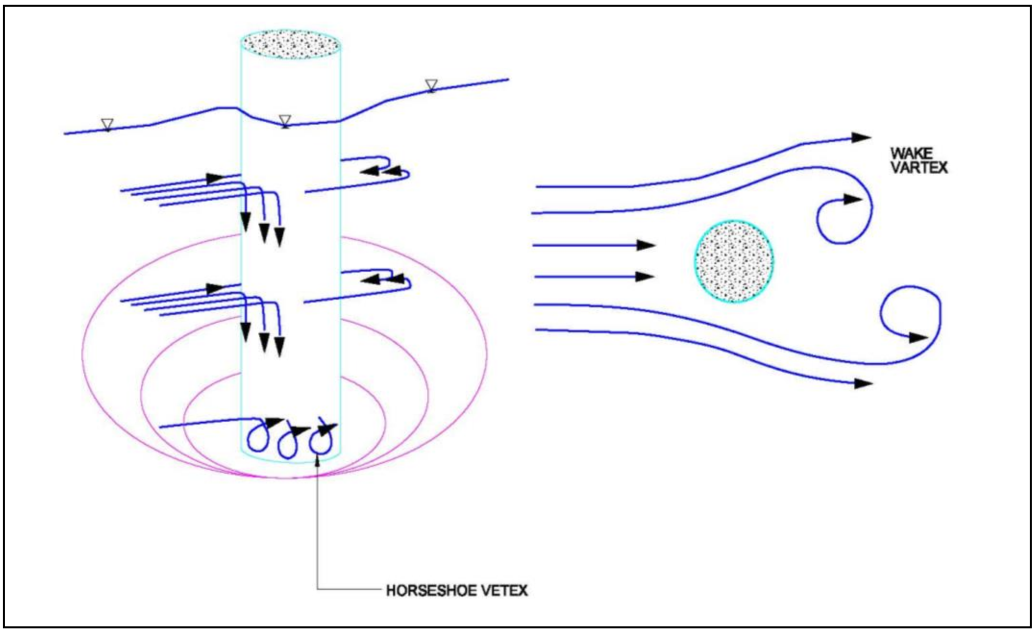


Figure A5- 4 - Schematic of local scour at circular pier

Action of the vortex removes bed material from around the base of the obstruction. Transport rates of sediment away from the base region are greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is re-established between bed material inflow and outflow and scouring ceases.
Scour depth is a basis for design of the abutment and pier protection. For design of piers and supporting piles, the total of the predicted scour depths is used as the maximum vertical unsupported pier length for reinforcement design. Pier support pile caps will typically be placed at this elevation. Even where piles require less depth for structural loading, the depth of the piling must extend beyond this depth to act as a SF against undermining by the flow.
Scour predictions will also have a large effect on the type of abutment support, whether pile-supported or using spread footings. Use of spread footings require suitable erosion protection, such as heavy riprap with concrete wing walls or flow deflection dikes to protect both the supports and the backfill from flow erosion. It may be more economical to provide a positive scour protection, such as concrete slab on the channel bottom, than construct the additional pier and abutment protection structures that will otherwise be required.

Scour and revetment protection design to be performed for the 1:100-yearflood event. These may need to be checked at the 1:500-yearor other extreme flood events, as agreed with AACM.

Appropriate scour protection is needed using locally appropriate and well recognized methods, such as described in HEC-11, HEC-14, HEC18, HEC-23 and CIRIA C742, or equivalent, as agreed with AACM/ DEPARTMENT.

A5.2.2.2 Deck drainage

Design of deck drainage shall meet the minimum spread and inlet spacing requirements as stipulated in Section A3.2.5, for kerbed roadways.

A5.2.2.3 Bridge hydraulic design documentation

Bridge hydraulic designs shall be documented in a Bridge Hydraulic Report. Documentation shall be provided in detail, commensurate with the complexity of the Project. Documentation shall be sufficient enough so that an independent Engineer with expertise in bridge hydraulics, but not involved with the Project design, can fully interpret, follow, and understand the logic, methods, computations, analysis, and considerations used to develop the final design.

1. Documentation for bridge designs shall include, at a minimum, the following:
   a. Hydrologic analysis, including sources of data and methodology.
   b. Alternative analysis or evaluation of structure sizes, including length and vertical height or clearance. This evaluation shall include the following considerations:
      i. Cost;
      ii. Design standards;
      iii. Structure hydraulic performance, including backwater, velocity, and scour; and
      iv. Impacts of the structure on the adjacent property.
2. Alternative analysis shall include the reasons for selecting the recommended structure and a clear explanation as to why it is the most economical structure for the site in question. At a minimum, the following structure sizes shall be evaluated:
   a. Minimum structure size required to meet hydraulic standards for vertical and horizontal clearance, scour, and backwater.
   b. Existing structure size, if applicable.
   c. Recommended structure size, if different from the above.
3. Design recommendations for bridges shall include the following:
   a. Bridge length and justification for the length, including locations (stations) of abutments;
   b. Channel excavation requirements;
   c. Minimum vertical clearance;
   d. Minimum horizontal clearance;
   e. Abutment type and orientation;
   f. Pier orientation;
   g. Scour depths for the design storm return period flows, 100-year flows, and maximum overtopping flows — if needed for main or major roadways;
   h. Scour protection requirements for abutments, piers, and channels;
   i. Deck drainage.
4. Documentation of hydraulic designs of culverts, including larger culverts with bridge design characteristics, shall include recommendations for the following:
   a. Culvert size and justification for the size, barrel length, and location;
   b. Up- and downstream invert elevations;
   c. End wall type for entrance and outlet, including the need for an improved inlet;
   d. Skew;
   e. Inlet and outlet ends scour protection requirements; and,

f. Final Project plans shall show the peak stages, peak discharges, peak velocities, and peak scour predictions for the design flows, and the greatest flow or overtopping flow that can be expected at the structure.

g. Many computer programs are available for computation of backwater curves. The most general and widely used program, HEC-RAS was developed by the U.S Army corps of Engineers and it’s recommended for floodwater profile computations. This program can be used to computer water surfaces profiles for both natural and artificial channels and culverts.