حكومـــة دبــــي
GOVERNMENT OF DUBAI

بـــلـــديـــة دبـــي
Dubai Municipality

STORMWATER DESIGN GUIDELINES

SEWERAGE AND RECYCLED WATER PROJECTS DEPARTMENT

WASTE AND SEWERAGE AGENCY

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Version 02

STORMWATER DESIGN GUIDELINES

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LIST OF ABBREVIATIONS

AMC	Antecedent Moisture Content
ANSI-HI	American National Standards Institute - Hydraulic Institute
AOS	Apparent Opening Size
ARI	Average Recurrence Interval
ASCE	American Society of Civil Engineers
BMP	Best Management Practice
CAPEX	Capital Expenditure
CIRIA	Construction Industry Research and Information Association of the UK
DEFRA	Department of Environment, Food and Rural Affairs of the UK
DEWA	Dubai Electricity and Water Authority
DI	Ductile Iron
DM	Dubai Municipality Sewerage and Recycled water Projects Department
DUSUP	Dubai Supply Authority
DXB	Dubai International Airport
FRC	Fibre-Reinforced Concrete
GPT	Gross Pollutant Trap
GRP	Glass-Reinforced Plastic
HAT	Highest Astronomical Tide
HDPE	High Density Poly-Ethylene
HGL	Hydraulic Grade Line
HSE	Health and Safety Executive of the UK
IDF	Intensity-Duration-Frequency
IPCC	Intergovernmental Panel on Climate Change
IUD	Integrated Urban Drainage

IWRM	Integrated Water Resources Management
MHWN	Mean High Water Neap
MHWS	Mean High Water Spring
MUSIC	Model for Urban Storm water Improvement Conceptualisation
MWL	Mean Water Level
NPSH	Net Positive Suction Head
NRCS	Natural Resources Conservation Service of the US
O&M	Operation and Maintenance
OL	Obvert Level
OPEX	Operational Expenditure
PE	Polyethylene
POA	Percent Open Area
QA/QC	Quality Assurance and Quality Control
RCP	Reinforced Concrete Pipe
RoW	Right-of-Way
RTA	Roads and Transport Authority of Dubai
SRPD	Sewerage and Recycled Water Projects Department
SS	Suspended Sediment
SuDs	Sustainable Drainage Systems
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
TWL	Tail water Level
UNEP	United Nations Environment Programme
UPS	Uninterruptible Power Supply

uPVC	Unplasticised Poly-Vinyl Chloride
WSA	Waste and Sewerage Agency
WSUD	Water Sensitive Urban Design

OBJECTIVES

1.1 Introduction

This guideline is for use by design consultants and developers when planning, designing and constructing storm water and subsurface drainage system in Emirate of Dubai. There shall be no deviation from these guidelines except where formally confirmed by Dubai Municipality (DM) in writing; such deviation from guidelines being technically justified or representing advances in knowledge or technology.

DM is committed for using new and innovative technologies where they, in DM's opinion, represent the best technical solution, provide low life cycle costs and value for money. All technologies will be considered for use by DM providing they have been proven in terms of performance, quality and cost.

All design shall be based on the guideline and DM reserve the right not to approve the connection to DM network or adopt any system that fails to meet the minimum standards of these guidelines.

Engineers and other disciplines using this Design Guidelines must be experienced and appropriately qualified professionals who are familiar with the planning, design, construction, operation and maintenance of drainage networks. The Design Guidelines is to be utilised as guide to good practice and compliance with the guidelines does not absolve users of their professional and contractual responsibilities. The Design Guidelines is not exhaustive in its coverage and is not intended to replace the proven theory listed elsewhere.

1.2 Approval Process

The Consultant shall submit the design document to DM for review and approval. The design stage requirements will be collected from DM before submission of the documents.

1.3 Permanent Works

This guidelines applies only to the design of the ‘permanent works’. This 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 Innovation

The Consultants and Developers shall encourage the parties involved in the planning, design and construction of a storm water and subsurface drainage system to devise innovative solutions and challenge conventional thinking where this could be beneficial to the project and of course to the Emirate of Dubai.

Required documentation and sufficient detail must be submitted to DM to allow the proposal to be appraised.

1.5 References

Within the formulation of this guideline, many existing regional as well as international publications were studied, with necessary changes implemented to adapt to the climate and strategic vision of Dubai. Most notably, the following publications effectively aided in the benchmarking and building of this guideline:

ADM, 2016. *Stormwater & Subsoil Drainage System,* Abu Dhabi City: s.n.

‘DS185 SEWERAGE, DRAINAGE & IRRIGATION MASTER PLAN FOR EMIRATE OF DUBAI,’ Dubai Municipality, Dubai, Jun. 2016.

1.6 Copyright

Copyright of the Design Guidelines is the property of DM.

1.7 Updates

This guidelines will be revised by DM from time to time to keep up to date with technical developments and improved practices. It is the responsibility of the users to ensure that they are working to the latest issue. The DM can be contacted for information on revisions. Any errors that are found or recommendations for improvement shall be notified to DM.

1.8 Inquiries

All inquiries regarding the DM Sewerage Guidelines shall be sent to DM's official incoming email (dm@dm.gov.ae) and copying the SRPD Director and the Head of Projects Planning and Development Section.

PRINCIPLES AND OBJECTIVES OF URBAN DRAINAGE MANAGEMENT

The primary aim of an urban drainage management system is to ensure storm water generated from developed catchments causes minimal nuisance, danger and damage to people, property and the environment. This requires the adoption of a multiple objective approach, broadly considering issues such as (NRW, 2007):

- Flooding and drainage control
- Ecosystem health, both aquatic and terrestrial
- Public health and safety
- Economic considerations
- Recreational opportunities
- Social considerations
- Aesthetic values

Hence, the principles and objectives of urban drainage management can be listed as below. All of the objectives presented below may not be relevant in all circumstances and individual objectives may be expanded to highlight site-specific issues. The objectives are (NRW, 2007):

- Protect and/or enhance downstream environments, including recognised social, environmental and economic values, by appropriately managing the quality and quantity of storm water runoff
    - ➢ Minimise changes to the quality and quantity of the natural urban drainage regime
    - ➢ Identify and control the primary sources of storm water pollution
    - ➢ Develop drainage systems based on a preferred management hierarchy. The preferred hierarchy is:

- Retain and restore valuable elements of the natural drainage system
- Implement source control measures using non-structural techniques
- Implement source control measures using structural techniques
- Install in-system constructed management techniques within the drainage system to manage storm water quality and quantity prior to discharging to receiving waters. To achieve the best results, storm water quality treatment systems shall be part of a comprehensive approach to controlling storm water pollution. Such an approach would include regulations and enhanced community awareness, as well as structural controls.

➢ Develop robust storm water treatment systems that do not rely on a single treatment system of focus on a single target pollutant

- Limit flooding of public and private property to acceptable or designated levels. The alignment and capacity of major drainage corridors such as waterways and major overland flow paths shall be preserved. Drainage corridors must be recognised as a legitimate land use and during the planning of new urban developments and re-development of existing areas.

- Ensure storm water and its associated drainage systems are planned, designed and managed with appropriate consideration and protection of community health and safety standards, including potential impacts on pedestrian and vehicular traffic. A safe, affordable and socially equitable and acceptable level of urban drainage and flood control shall be established and maintained.

- Adopt and promote sustainable drainage principles, including appropriately managing storm water as an integral part of the total water cycle, protecting natural features and ecological processes within urban waterways, and optimising opportunities to use rainwater/storm water as a resource

- Minimise the quantity of directly connected impervious surface area. This will help to reduce changes to the natural water cycle, pollutant runoff rates and the cost of providing storm water management systems.

- Identify and optimise opportunities for storm water to be valued and used as a resource. Where circumstances allow, urban storm water can be used to recharge aquifers provided groundwater quality is protected. This requires very careful management as potential issues include rising water tables, salinity problems and disputes over groundwater extraction rights.

- Maintain and protect natural drainage systems and their ecological health. It is noted that the control of building/construction site soil erosion and sediment runoff is essential for the sustainable management of most natural drainage systems. Sediment runoff from building and construction sites must be actively controlled.

- Appropriately integrate storm water systems into the natural and built environments while optimising the potential uses of drainage corridors

    - Ensure adopted storm water management systems are appropriate for the site constraints, land use and catchment conditions. Storm water management practices should reflect proposed land use practices, climatic conditions, soil properties, site constraints, identified environmental values, and the type of receiving waters. Certain land uses produce concentrations of specific storm water pollutants, thus requiring the adaptation of specialist storm water treatment systems that may not be as effective in other areas of the catchment. Certain receiving waters may also be sensitive to certain pollutant inflows, thus requiring a further refinement to the list of preferred storm water management systems. As a general guide, large receiving water bodies, such as lakes, rivers and bays, benefit from any and all measures that reduce total pollutant loads, independent of when the pollutant runoff occurs.

  - Appropriately integrate both wildlife and community land use activities within urban waterway and drainage corridors

- Ensure storm water is managed at a social, environmental and economic cost that is acceptable to the community as a whole and that the levels of service and the contributions to costs are equitable

  - Assess the economics of storm water management systems on the basis of their full lifecycle costs, i.e. capital and operational costs. Storm water management systems shall be based on solutions that are economically sustainable. Developers of new urban communities must give appropriate consideration to the anticipated on-going maintenance (operational) costs of storm water management systems even if they are not required to furnish such maintenance costs.

  - Ensure adopted storm water management systems are sustainable. Storm water designers have a responsibility, within reason, to ensure that their design can function effectively throughout their specified design life based on the financial and technical abilities of the proposed asset manager.

  - Ensure appropriate protection of storm water treatment measures during the construction phase. Storm water treatment measures, especially filtration and infiltration systems, need to be isolated or otherwise protected during the construction phase of urban development so that their ultimate function is not compromised by sediment or construction damage.

- Enhance community awareness of, and participation in, the appropriate management of storm water.

  - Engage the community in the development and evaluation of urban drainage management strategies/solutions. Community participation helps to:

    - Identify strategies which are responsive to community concerns

- Explore problems, issues, community values and alternative strategies
- Increase public ownership and acceptance of proposed solutions
- Generate broader decision making perspectives not limited to past practices or interests
- Reflect the community's life style values and priorities

Urban drainage management plans should consider several key parameters in order to achieve the objectives as outlined in **Table 0-1** (NRW, 2007).

Table 0-1— Key Urban Drainage Management Parameters and Desired Outcomes

Parameter	Desired outcomes
Drainage	Public health Pedestrian and vehicular safety Minimisation of storm-related nuisance to public
Infiltration	Runoff volume control Delivery of high quality, dry weather inflows to urban drainage system through maintenance of groundwater levels
Runoff volume	Control of bed and bank erosion in waterways Reduction of annual pollutant load to water bodies Optimum use of storm water as a resources Protection of aquatic ecosystems within receiving water bodies
Peak discharge	Flood control Minimisation of legal disputes between neighbouring land owners and communities Control of bed and bank erosion in waterways
Flow velocity	Pedestrian and vehicular safety

Parameter	Desired outcomes
	Control of bed and bank erosion in waterways Protection of aquatic ecosystems within receiving water bodies
Flow depth	Flood control Pedestrian and vehicular safety Minimisation of storm-related nuisance to public
Water quality	Protection of aquatic ecosystems and public health Optimum use of storm water as a resources Integrity of urban waterways/water bodies through control of sediment inflow
Aesthetics	Appropriate integration of storm water systems into the natural and built environments, including retention of natural drainage systems Protection/restoration of environmental values
Infrastructure & maintenance cost	Acceptable financial cost Sustainable operational and maintenance requirements

DESIGN CONSIDERATIONS

3.1 Network Integration

Dubai Municipality will share the existing and future land use for the area where the Developer shall identify his proposals. The storm water and subsurface drainage systems in Dubai will need to accommodate flows from the following contributing sources.

- The existing networks being operated by DM.
- The existing networks being operated by Developers.
- Proposed future planning in vicinity of the project area:
- New developments with their associated sub-catchment drainage
- The extensions to the trunk main network being planned and/or constructed by DM to accommodate surface water from new developments and changes in subsurface 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,
- Storm water drainage for new highways or major roads being planned and/or constructed by the Road and Transport Authority (RTA) considered overland run-off from open areas if graded toward roads, and
- Assessment of free discharge from the surrounding existing roads/junctions (if applicable) to be considered while designing network.

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

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

- Gravity solutions are always recommended for storm water system. However, if depth of the proposed system get increase or intention is to connect to the existing manhole or discharge to outfall at shallow depth then electromechanical solutions can be permitted.

- 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 and also smaller pump/lift station if necessarily required. A detailed appraisal, including NPV calculations, shall be carried out to justify the solution adopted.

- Optimise the number of pumping stations when pumping is 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.

3.2 Design Life

The minimum periods of time for new assets to last before replacement are shown below.

Table 0-1— Design Life for DM Assets (ADM, 1998)

Sr. No.	Asset type	Minimum Design Life (Years)
1	Pipelines	60
2	Structures	30
3	Mechanical and Electrical Equipment	15
4	Instrumentation, Computer Hardware and Sensors.	5

The maintenance of all civil, mechanical, electrical and instrumentational equipment will be required during the above periods. A plan maintenance schedule and spare parts list shall be submitted as part of the design submission. During NPV analysis, the cost for maintenance and replacement of equipment shall be considered.

3.3 Investigations

The Consultants may require additional investigations include but not limited to below mentioned list, during the design process.

- Environmental studies, e.g. hydrodynamic modelling
- Marine studies
- Ecological studies
- Geotechnical and groundwater investigations
- Geotechnical investigations
- Topography survey
- Bathymetric survey
- Salinity monitoring
- Flow monitoring

3.4 Environmental Legislation

The Consultant's proposals shall comply with all relevant local, regional and international legislation. The Consultant shall comply with the requirements of Dubai Municipality, Environment Department and other authority in UAE, if required. In case of any conflict between the legislation and standards, DM have all the rights to take decision.

3.5 Health and Safety in Design

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

3.6 Value Management and Value Engineering

Value engineering is mandatory to enhance the value of a project by structurally examining the decisions about benefits, risks and costs. The value engineering workshops with DM shall be arrange throughout the project duration from concept to detail design stages or even afterwards. Each individual project has different value engineering requirements which shall be confirmed with DM.

Consultant is obliged to prepare a report for each individual value engineering workshop, covering, and DM requirements and submit for review and approval.

3.7 Options Appraisal

For selection of best possible option, consultant is to provide more than one option for storm water and subsurface drainage design with layouts. The consultant also checked the following before reaching to the conclusion.

- Capital Cost (CAPEX)
- Operational and Maintenance Cost (OPEX)
- Economic Appraisal of Option
- Evaluation of each option

3.7.1 Capital Cost (CAPEX)

The project estimate shall be based on the latest market rates, preferably obtained from DM Tender/Contract documents. The accuracy of estimates will vary with the stage of the project. The unit rates can be used for master plan and strategic studies but for detail design the estimate shall be based on the detailed bill of quantities. The format and method for estimation shall be provided by DM on the request before commencing the design stage.

3.7.2 Operational and Maintenance Cost (OPEX)

The Operation and Maintenance Cost (OPEX) for all the options shall include but not limited to the following:

- Cost for labour (per hour/day/month/year)
- Vehicle and equipment cost (per hour/day/month/year)
- Power (DEWA latest tariff)
- Parts and consumables
- Chemicals (if any)
- Operation and Maintenance

The Consultant shall provide the OPEX for each individual option and submit to DM for review and used further to recommend the best possible options.

3.7.3 Economic Evaluation for Each Option

The proposed competing options shall be evaluated economically by comparing the associated life cycle cost. The life cycle cost includes the CAPEX and OPEX (calculated from start of construction) where the costs are to be set at the current base price. Consultant will communicate with DM to finalize the discount rate and period to be used for life cycle cost.

3.7.4 Evaluation of Each Option

For selecting the best possible option for the individual project that proposed within jurisdiction of Emirates of Dubai, consultant shall critically analyse the proposed options. The analysis shall cover but not limited to the following:

- Sustainability:
- Adaptability and Resilience:

- Feasibility:
- Operability:
- Constructability
- Financial
- Environment

DESIGN RAINFALLS AND RUNOFF

4.1 Design Storms

4.1.1 Design Return Period

The design rainfall event shall be based on the below mentioned table.

Table 0-1— Design Rainfall

Sr.	Event	Area
1	1 in 5 Years Storm	Housing developments, local roads - All storm water network will be designed to serve the 1 in 5 year with exception highlighted below. The network will be checked for 1 in 10 years storm;
2	1 in 10 Years Storm	Highways, Freeways, Major roads (as per classification of RTA), Seaports, Palaces, Government buildings, basements and other critical areas, as approved by DM.
3	1 in 25 Years Storm	Airports, Culverts on minor roads (check for 1 in 50 years storm);
4	1 in 50 Year Storm	Underpasses and underground car parks (Ramps); Culverts on major roads, lagoons where overtopping may lead to flooding of properties* (check for 1 in 100 years storm)
5	1 in 100 Year Storm	The buffer area (protection zone) along the open natural wadis.

4.1.2 Design Storm Duration

For storm water networks and drainage design the time of concentration, shall be taken as the design storm duration. The time of concentration is the time at which the entire watershed begins to contribute to runoff. This is calculated as the time taken for runoff to flow from the most hydrologically remote point of the drainage area to the point under investigation.

4.2 Intensity-Duration-Frequency (IDF)

The rainfall IDF, provide average intensity of rainfall during storm event with specified duration and frequency of occurrence.

Table 0-2 present the calculated IDF values for Dubai for the design storm. The table showcases the intensity values for each duration and return period:

Table 0-2— IDF values for Dubai including the Estimated values

Return Period (years)	Duration (min)														
	5	10	15	20	30	45	60	90	120	180	240	300	360	720	1440
2	59.90	45.22	37.15	31.10	23.57	17.44	13.87	10.22	8.18	5.92	4.73	3.96	3.38	1.93	1.00
5	97.41	76.98	63.34	53.54	41.68	31.78	25.35	18.66	15.11	10.81	8.62	7.21	6.22	3.68	1.93
10	118.00	97.61	80.41	68.58	54.72	42.82	34.22	25.18	20.57	14.60	11.61	9.73	8.44	5.13	2.71
20	131.23	115.25	95.09	82.01	67.36	54.26	43.46	31.95	26.37	18.54	14.70	12.34	10.79	6.75	3.59
25	133.66	120.18	99.22	85.92	71.32	58.03	46.52	34.20	28.31	19.84	15.72	13.20	11.58	7.31	3.90
50	169.22	152.15	125.61	108.78	90.30	73.47	58.89	43.29	35.85	25.12	19.91	16.71	14.65	9.25	4.94
100	209.20	188.11	155.29	134.49	111.63	90.83	72.81	53.52	44.32	31.06	24.61	20.66	18.12	11.44	6.10

The IDF curves for estimated values for Dubai are shown below in **Figure 0-1** and **Figure 0-2.**

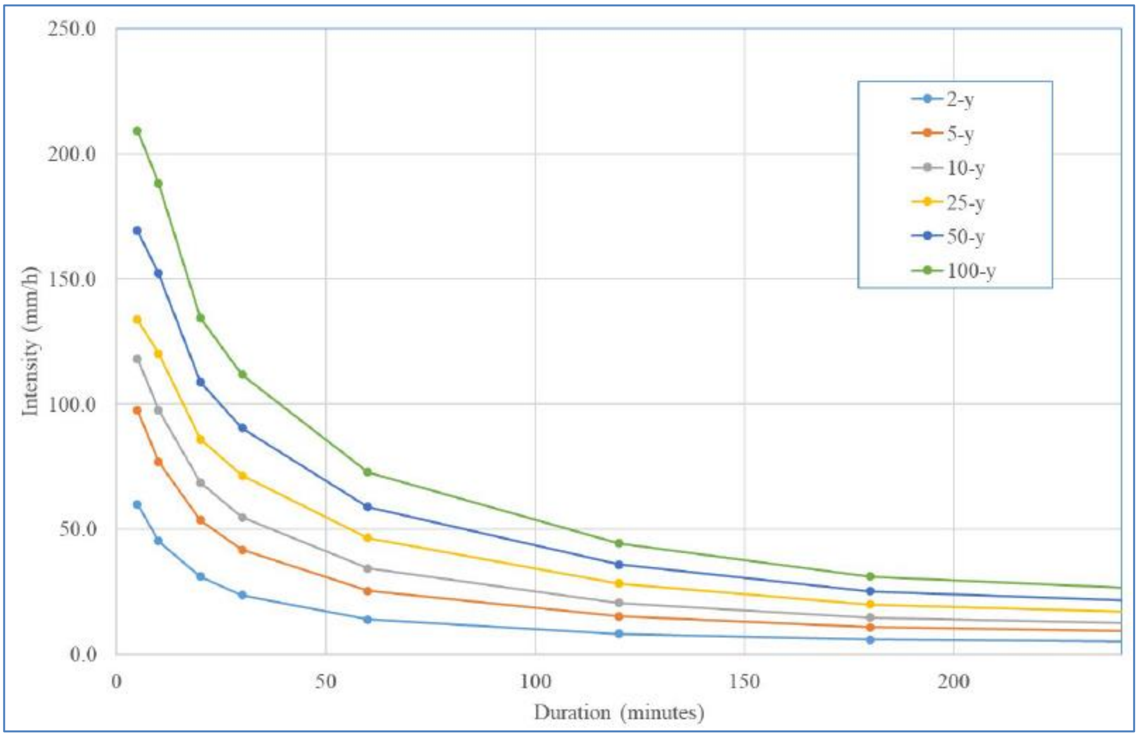


Figure 0-1— Intensity-Duration-Frequency Curves for Dubai from 1 hrs to 4 hrs.

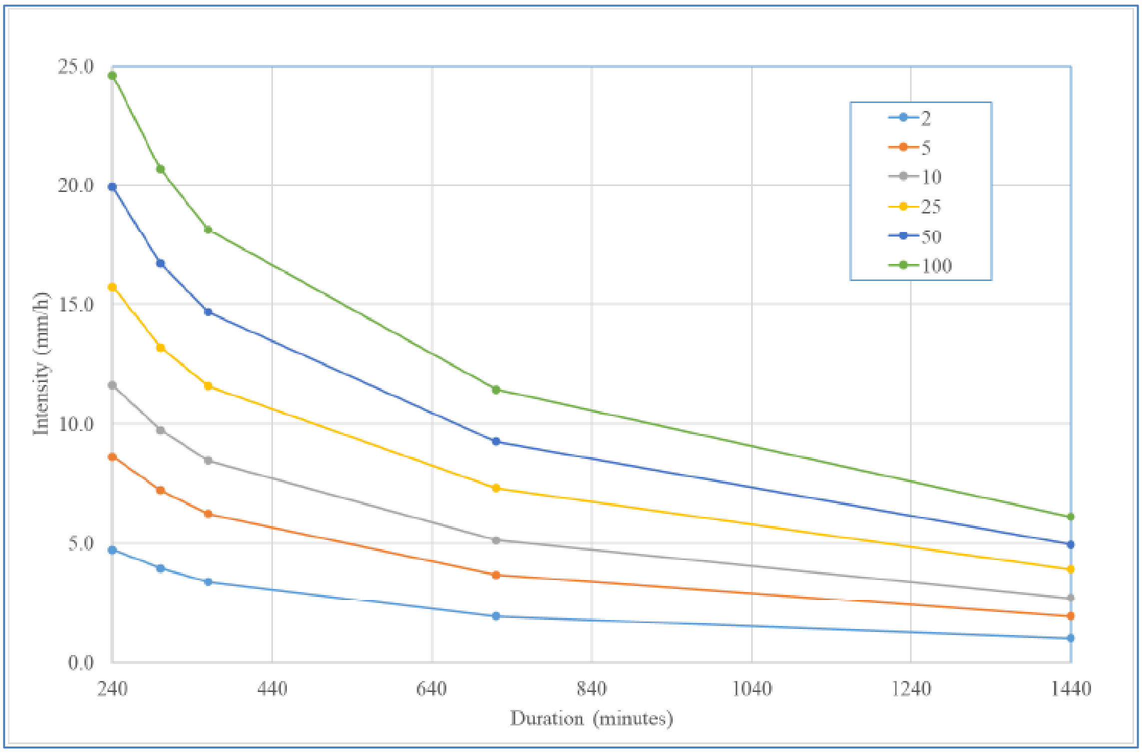


Figure 0-2— Intensity-Duration-Frequency Curves for Dubai from 4 hrs to 24 hrs.

Table 0-3— IDF values for Hatta including the Estimated values

Return Period (years)	Duration (min)														
	5	10	15	20	30	45	60	90	120	180	240	300	360	720	1440
2	104.1	73.8	56.9	48.6	38.2	30.1	23.7	17.0	13.6	9.8	7.8	6.5	5.6	3.1	1.78
5	205.3	145.5	112.3	95.9	76.0	59.2	46.3	33.4	26.9	19.6	15.7	13.3	11.3	6.3	3.6
10	272.3	193.0	148.9	127.2	101.0	78.4	61.3	44.3	35.7	26.1	21.0	17.7	15.1	8.4	4.81
25	357.0	253.0	195.2	166.8	132.7	102.7	80.2	58.0	46.8	34.3	27.7	23.4	19.9	11.1	6.33
50	419.8	297.5	229.5	196.1	156.1	120.7	94.3	68.2	55.0	40.3	32.6	27.5	23.4	13.1	7.46
100	482.1	341.6	263.6	225.2	179.4	138.6	108.2	78.3	63.2	46.4	37.5	31.7	27.0	15.1	8.58
200	544.2	385.7	297.5	254.3	202.6	156.4	122.1	88.4	71.4	52.4	42.4	35.8	30.5	17.0	9.70
400	606.2	429.6	331.4	283.2	225.7	174.2	135.9	98.4	79.5	58.4	47.3	40.0	34.0	19.0	10.81
1000	688.1	487.6	376.2	321.5	256.3	197.7	154.3	111.7	90.3	66.3	53.8	45.4	38.6	21.6	12.28

The IDF curves for estimated values for Hatta are shown below in **Figure 0-1** and **Figure 0-2.**

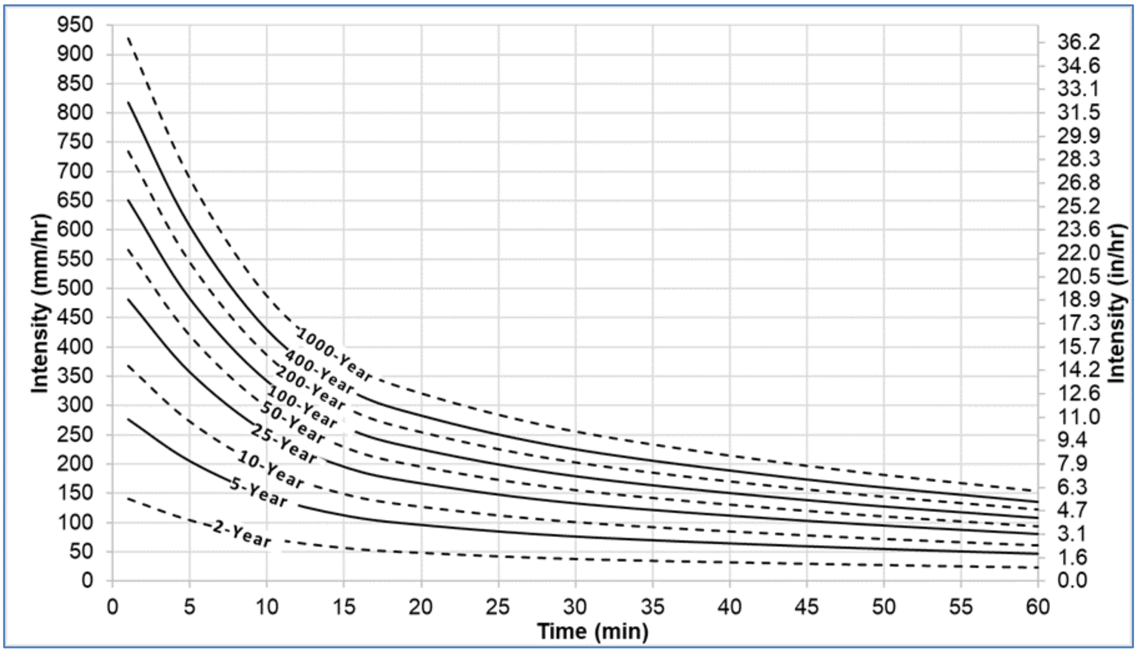


Figure 0-3— Intensity-Duration-Frequency Curves for Hatta from 1 min to 1 hr.

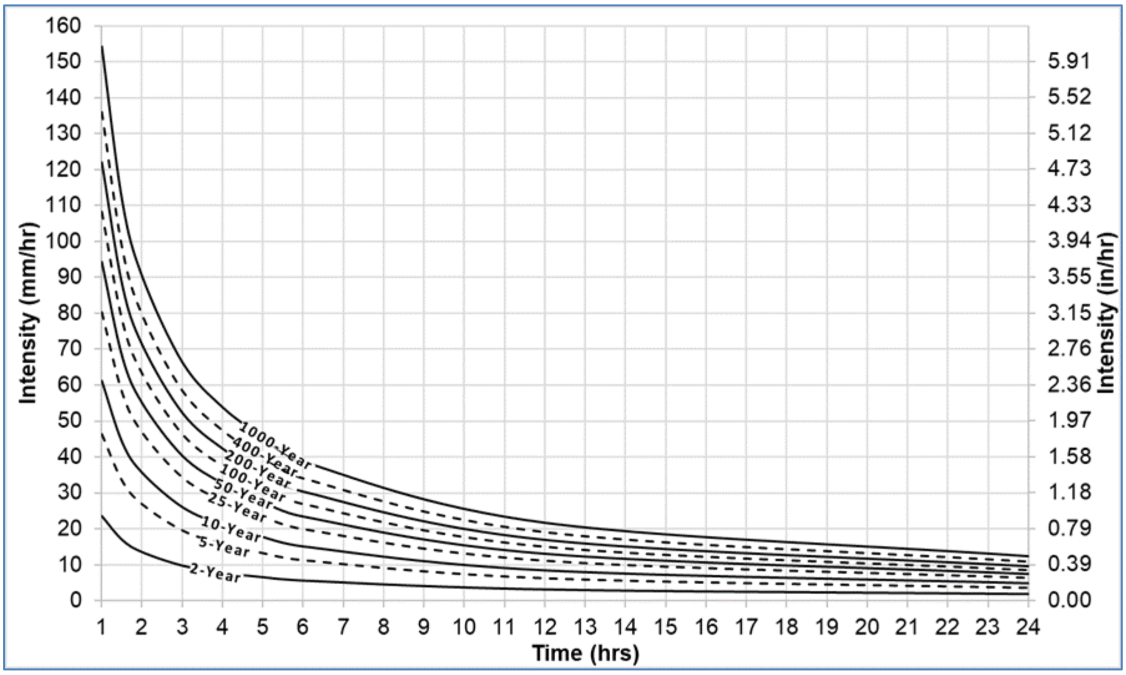


Figure 0-4— Intensity-Duration-Frequency Curves for Hatta from 1 hr to 24 hrs.

4.3 Design Storm Profile (Design Hyetograph)

Based on the recent rainfall analysis, a modified storm profile is recommended.

Table 0-4 provides the standard dimensionless modified median storm profile for Dubai. Synthetic storm profiles for different return periods and durations can be generated based on this standard shape and the IDF values. **Figure 0-5** also shows the graphical representation of this standard shape. The profile is symmetrical around its mid-point (50% duration).

Table 0-4— Dimensionless modified median storm profile for Dubai

% duration	% of mean intensity	% of cumulative	% duration	% of mean intensity	% of cumulative depth
1	28	0.28	26	61	10.74
2	29	0.57	27	63	11.37
3	29	0.86	28	66	12.03
4	30	1.16	29	69	12.72
5	31	1.47	30	72	13.44
6	32	1.79	31	75	14.19
7	33	2.12	32	79	14.98
8	34	2.45	33	83	15.81
9	35	2.80	34	87	16.68

% duration	% of mean intensity	% of cumulative	% duration	% of mean intensity	% of cumulative depth
10	36	3.16	35	91	17.59
11	37	3.53	36	96	18.55
12	38	3.90	37	102	19.57
13	39	4.29	38	108	20.66
14	40	4.70	39	115	21.81
15	42	5.11	40	123	23.04
16	43	5.54	41	132	24.36
17	44	5.98	42	143	25.79
18	46	6.44	43	155	27.34
19	47	6.91	44	170	29.05
20	49	7.40	45	189	30.93
21	51	7.91	46	213	33.06
22	52	8.43	47	245	35.51
23	54	8.98	48	294	38.45
24	56	9.54	49	381	42.26
25	59	10.13	50	774	50.00

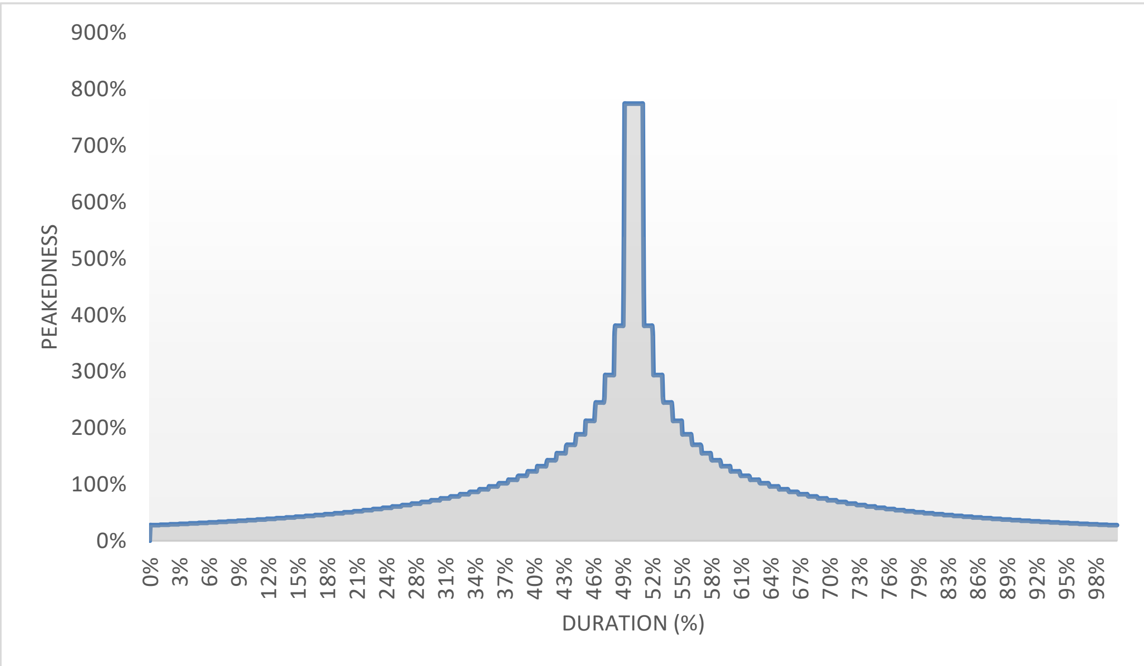


Figure 0-5— Standard dimensionless modified median storm profile Shape

The storm profiles can be generated for other return periods and storm durations.

4.4 Catchment Hydrology and Rainfall-Runoff Modelling

Understanding the hydrologic processes of a catchment is essential for the estimation of design flows. Estimation of peak flows is generally adequate for design and analysis of conveyance systems such as storm drains or open channels. However, if the design or analysis must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is required.

Estimation of peak flows and flood hydrographs can be achieved through two main groups of methods, i.e. statistical/stochastic methods and rainfall-runoff modelling methods.

Stochastic methods, or frequency analysis, can be used to evaluate peak flows where adequate gauged streamflow data exist, which is usually not the case in urban overland flow (pluvial) conditions, but more common in river and floodplain (fluvial) hydrology.

Rainfall-runoff modelling is the common practice in urban hydrology (pluvial flooding) and for the estimation of peak flows and flow hydrographs in different location of an urban storm water drainage system.

The rate of runoff resulting from any constant rainfall intensity is maximum when the duration of rainfall equals the Time of Concentration (tc). That means if the rainfall intensity is constant, the entire drainage area contributes to the peak discharge when the time of concentration has elapsed. This assumption becomes less valid as the drainage area increases. For large drainage areas, the time of concentration can be so large that the assumption of constant rainfall intensities for such long periods is not valid, and shorter more intense rainfalls can produce larger peak flows. Additionally, rainfall intensities usually vary during a storm. In semi-arid and arid regions, storm cells are relatively small with extreme intensity variations. These characteristics shall be considered by applying storm profiles and more advanced rainfall-runoff modelling methods.

The choice of rainfall-runoff modelling method must be appropriate to the type of catchment and the required degree of accuracy. Simplified hydrologic methods such as the Rational Method should not be used whenever a full design hydrograph is required, i.e. in design of complex networks and volume-dependent system components such as storage (detention and retention) basins.

Different rainfall-runoff modelling methods are described below.

4.4.1 Rational Method

4.4.1.1 Peak flow

The use of Rational Method as a rainfall-runoff model shall be limited to initial sizing of small and simple drainage systems with catchments smaller than 80 hectares.

Use of the rational method includes the following assumptions and limitations:

- Peak flow occurs when the entire watershed is contributing to the flow. Hence, the method is applicable if the selected duration of rainfall is equal to or greater than tc for the drainage area.
- Rainfall intensity is uniform throughout the duration of the storm.
- Rainfall is distributed uniformly over the drainage area.
- The frequency of occurrence for the peak discharge is the same as the frequency of the rainfall producing that event.
- The minimum duration to be used for computation of rainfall intensity is 10 minutes. If the time of concentration computed for the drainage area is less than 10 minutes, then 10 minutes shall be adopted for rainfall intensity computations.
- The rational method does not account for storage in the drainage area. Available storage is assumed to be filled.
- Runoff coefficient is the same for all storms of all recurrence probabilities.

The rational method represents a steady inflow-outflow condition of the watershed during the peak intensity of the design storm. Any storage features having sufficient volume that they do not completely fill and reach a steady inflow-outflow condition during the duration of the design storm cannot be properly represented with the rational method. Such features include detention ponds, channels with significant volume, and floodplain storage. When these features are present, an alternate rainfall-runoff method is required that accounts for the time-varying nature of the design storm and/or filling/emptying of floodplain storage. In these cases, the hydrograph method is recommended (TxDOT, 2011).

Rational Method estimates the peak rate of runoff at any location in a catchment as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analysed). The rational method formula is expressed below:

$$Q = \frac{C\,I\,A}{360} \qquad \textbf{(Equation 1)}$$

where:

Q = maximum rate of runoff (m^3/s)

C = runoff coefficient (refer to **Table 0-5**)

I = rainfall intensity with a duration equal to the time of concentration (mm/hr)

A = drainage area (ha)

4.4.1.2 Runoff Coefficient (C)

In selecting the runoff coefficient, the future characteristics of the catchment shall be considered. If land-use varies within a catchment, segments shall be considered individually, and a weighted runoff coefficient value shall be calculated. **Table 0-5** suggests ranges of C values for various categories of ground cover.

Table 0-5— Rational Method Runoff Coefficients for Urban Watersheds (FHWA, 2001)

Type of Drainage Area	Runoff Coefficient (C)*
Business	

Type of Drainage Area	Runoff Coefficient (C)*
Downtown areas	0.70 - 0.95
Neighbourhood areas	0.50 - 0.70
Residential	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.40
Railroad yards	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Lawns	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35

Type of Drainage Area	Runoff Coefficient (C)*
Streets	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs	0.75 - 0.95
***** Higher values are usually appropriate for steeply sloped areas and longer return periods because infiltration and other losses have a proportionally smaller effect on runoff in these cases.	

Runoff coefficients listed in **Table 0-5** apply to storms with ARIs of up to 10 years. Storms with higher return periods require modifying the runoff coefficient because in storms with higher return periods, infiltration and other abstractions have a proportionally smaller effect on runoff. In such cases, the runoff coefficient (C) shall be multiplied by the factor C_f as indicated in **Table 0-6.** The product of C and C_f should not exceed 1.0.

Table 0-6— Rational Method Runoff Coefficient Adjustment Factors (TxDOT, 2011)

Design storm ARI (years)	C_f
25	1.10
50	1.20
100	1.25

4.4.1.3 Time of Concentration

To estimate t_c, the flow path along which the longest travel time is likely to occur shall be identified. Generally, it is reasonable to consider three following components of flow that can characterise the progression of runoff along a travel path. These three components are as below:

1. sheet flow (overland flow),
2. shallow concentrated flow,
3. open channel and pipe flow (concentrated channel flow).

Sheet Flow Travel Time

Sheet flow is the shallow mass of runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs over relatively short distances, rarely more than about 130 m. Sheet flow is commonly estimated with a version of the Kinematic Wave equation, a derivative of Manning's equation, as follows (FHWA, 2001):

$$\mathbf{T_{ti}} = \frac{6.92}{\mathbf{I}^{0.4}}\left(\frac{\mathbf{n\,L}}{\sqrt{\mathbf{S}}}\right)^{0.6} \qquad \textbf{(Equation 2)}$$

where:

T_{ti} = sheet flow travel time (h)

n = Manning's roughness coefficient for overland flow (see **Table 0-7**)

L = flow length (m)

I = rainfall intensity (mm/h)

S = surface slope (m/m)

Since rainfall intensity (I) depends on t_c which is not initially known, computation of t_c is an iterative process.

Table 0-7— Manning's Roughness Coefficient (n) for Sheet Flow (FHWA, 2001)

Surface Description	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013

Wood	0.014
Brick with cement mortar	0.014
Cast iron	0.015
Vitrified clay	0.015
Corrugated metal pipe	0.024
Cement rubble	0.024
Fallow (no residue)	0.050
Cultivated soils	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda grass	0.41

Shallow Concentrated Flow Velocity

After a short distance of at most 130 m, sheet flow tends to concentrate in rills. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using a relationship between velocity and slope as follows (FHWA, 2001):

$$\mathbf{V = k\, S_p^{0.5}} \quad \textbf{(Equation 3)}$$

where:

V = velocity (m/s)

k = intercept coefficient (see Table below)

S_p = slope (%)

Table 0-8— Intercept coefficients for velocity vs. slope relationship (FHWA, 2001)

Land Cover/Flow Regime	k
Short grass pasture	0.213
Nearly bare and untilled	0.305
Grassed waterway	0.457
Unpaved	0.491
Paved	0.619

Open Channel & Pipe Flow Velocity

Open channel and pipe flow velocity can be estimated from the hydraulic properties of the pipe or channel using Manning's equation.

$$\mathbf{V} = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \textbf{(Equation 4)}$$

where:

n = Manning's roughness coefficient (see Table)

V = velocity (m/s)

R = hydraulic radius (m)

S = slope (m/m)

The time of concentration (t_c) is then calculated by adding the flow travel times in different segments which are calculated as:

$$\mathbf{T_{ti}} = \frac{L}{60\,V} \qquad \textbf{(Equation 5)}$$

where:

T_{ti} = flow travel time for segment (min)

L = length of the segment (m)

V = flow velocity in segment (m/s)

It is recommended to use a minimum t_c of 10 minutes where the estimated t_c is less than 10 minutes.

Table 0-9— Manning's Roughness Coefficient (n) for Channels and Pipes (FHWA, 2001)

Conduit material	n
Closed conduits	
Asbestos-cement pipe	0.011 - 0.015
Cast Iron pipe (cement-lined)	0.011 - 0.015
Concrete pipe	0.011 - 0.015
Corrugated metal pipe	
Plain	0.022 - 0.026
Paved invert	0.018 - 0.022
Spun asphalt lined	0.011 - 0.015
Plastic pipe (uPVC, GRP, PE)	0.011 - 0.015
Open channels	
Lined channels	
Asphalt	0.013 - 0.017
Brick	0.012 - 0.018
Concrete	0.011 - 0.020
Rubble or riprap	0.020 - 0.035
Vegetal	0.030 - 0.400
Excavated or dredged	
Earth, straight and uniform	0.020 - 0.030
Earth, winding, fairly uniform	0.025 - 0.040
Rock	0.030 - 0.045
Unmaintained	0.050 - 0.140

Natural channels (minor streams, top width at flood stage < 30 m)	
Fairly regular section	0.030 - 0.070
Irregular section with pools	0.040 - 0.100

4.4.2 Rainfall-Runoff Models

For design of the storm water drainage systems, more advanced rainfall-runoff models shall be employed. Some examples of the applicable rainfall-runoff models are:

- Non-linear Reservoir (Stormwater Management Model SWMM)
- NRCS Dimensionless Unit Hydrograph Method
- NRCS Triangular Unit Hydrograph Method
- Colorado Urban Hydrograph Procedure (for catchments < 1,300 hectares)
- Snyder Unit Hydrograph Method
- Clark Unit Hydrograph Method
- Delmarve Unit Hydrograph
- Epsey Unit Hydrograph
- Santa Barbara Urban Hydrograph Method
- San Diego Modified Rational Hydrograph

Note: Please be aware that all stormwater projects must adhere to the Stormwater Management Model. All other methods are for informational purposes only.

4.4.2.1 Non-Linear Reservoir

The conceptual view of surface runoff used by SWMM method is illustrated in the following **Figure 0-6**. Each sub-catchment surface is treated as a nonlinear reservoir. Inflow comes from precipitation and any designated upstream sub-catchments. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, Q, occurs only when the depth of water in the "reservoir" exceeds the maximum depression storage, d_p, in which case the outflow is given by Manning's equation. Depth of water over the sub-catchment (d in feet) is continuously updated with time (t in seconds) by solving numerically a water balance equation over the sub-catchment.

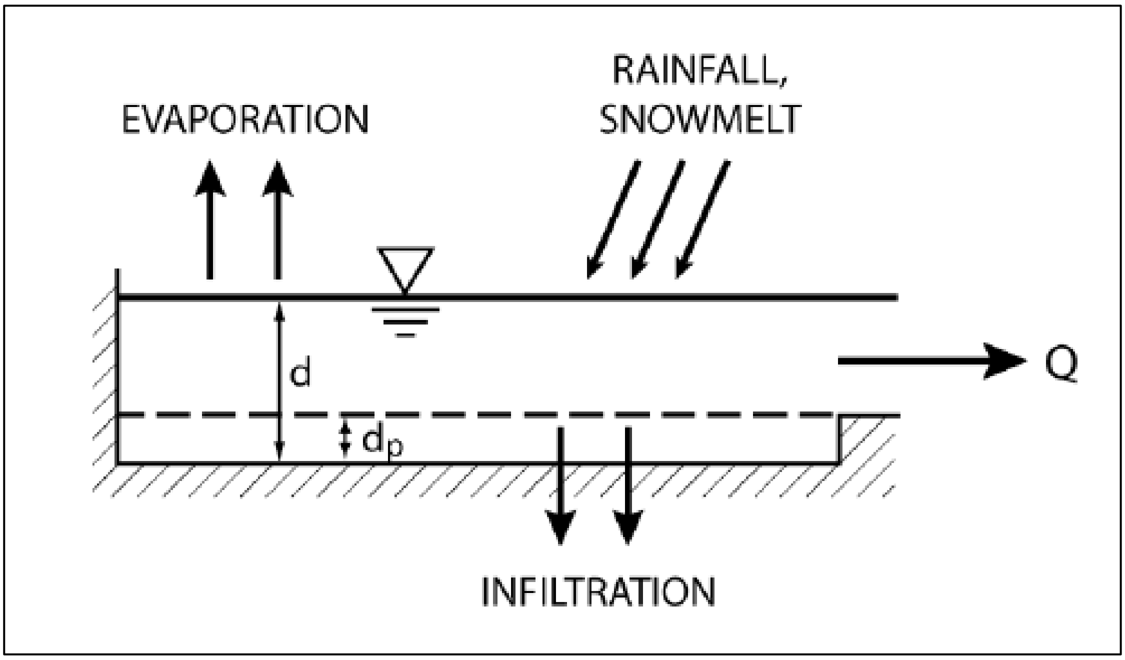


Figure 0-6—Non-Linear Reservoir (Innovyze, 2011)

4.4.2.2 NRCS Dimensionless Unit Hydrograph Method

NRCS Dimensionless Unit Hydrograph is shown in **Figure 0-7** below and the coordinates of this unit hydrograph are presented in **Table 0-10.**

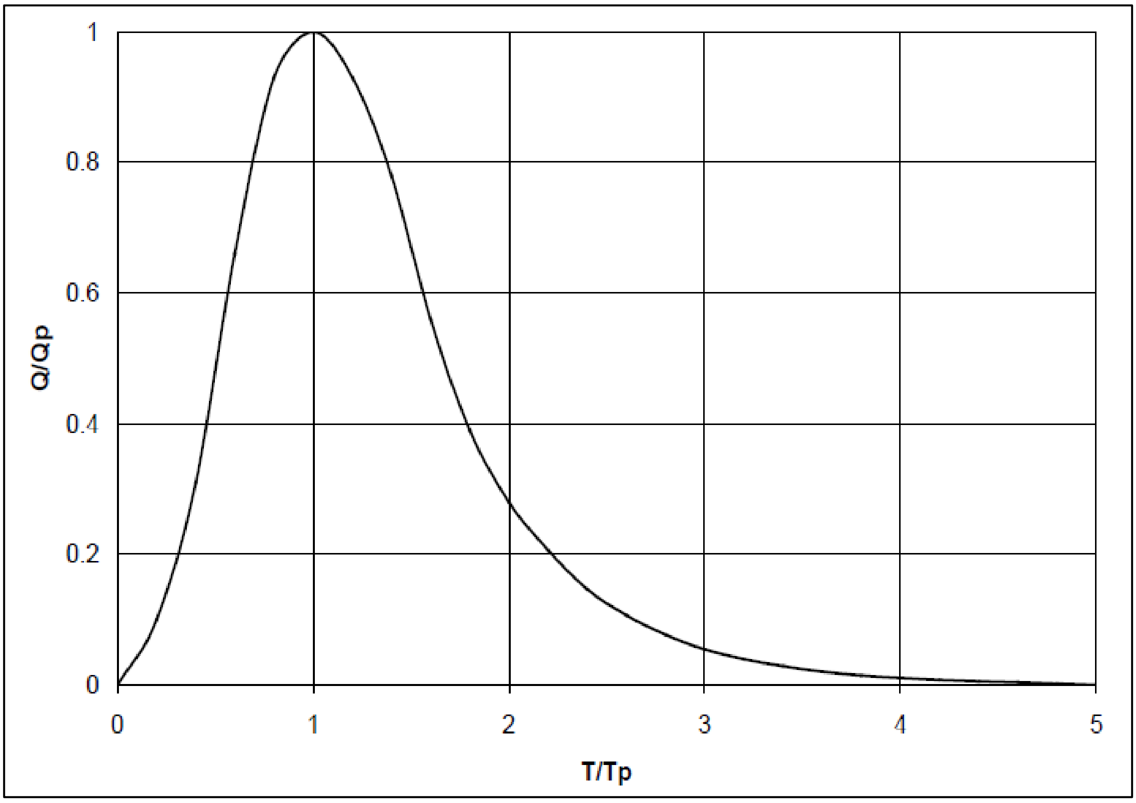


Figure 0-7— NRCS Dimensionless Unit Hydrograph (Innovyze, 2011)

Table 0-10— Coordinates of NRCS Dimensionless Unit Hydrograph (NRCS, 2007)

Time ratio (t/t_p)	Discharge ratio (Q/Q_p)	Time ratio (t/t_p)	Discharge ratio (Q/Q_p)
0.0	0.000	1.7	0.460
0.1	0.030	1.8	0.390
0.2	0.100	1.9	0.330
0.3	0.190	2.0	0.250
0.4	0.310	2.2	0.207
0.5	0.470	2.4	0.147
0.6	0.660	2.6	0.107
0.7	0.820	2.8	0.077
0.8	0.930	3.0	0.055
0.9	0.990	3.2	0.040
1.0	1.000	3.4	0.029

Time ratio (t/t_p)	Discharge ratio (Q/Q_p)	Time ratio (t/t_p)	Discharge ratio (Q/Q_p)
1.1	0.990	3.6	0.021
1.2	0.930	3.8	0.015
1.3	0.860	4.0	0.011
1.4	0.780	4.5	0.005
1.5	0.680	5.0	0.000
1.6	0.560		

To generate a tr-hour hydrograph for a catchment, time to peak (T_p) and the peak flow rate (Q_p) are determined using catchment characteristics as below:

$$T_p = \frac{t_r}{2} + t_l \quad \textbf{(Equation 6)}$$

$$t_l = 0.6\, t_c \qquad \textbf{(Equation 7)}$$

where:

t_r = duration of effective rainfall (hr)

t_l = lag time of the catchment (hr)

t_c = time of concentration (hr)

Lag time represents the time from the centre of mass of effective rainfall to the time-to-peak of the hydrograph. In other words, lag time is a delay in time, after a rain over a catchment, before the runoff reaches its peak. Lag time can be calculated using the following equation (NRCS, 2007):

$$t_l = \frac{L^{0.8}\,(\frac{1000}{CN} - 9)^{0.7}}{1900\, s^{0.5}} \qquad \textbf{(Equation 8)}$$

where:

t_l = lag time of the catchment (hr)

L = hydraulic length of the catchment (ft) which refers to travel distance of water from the most upstream location of the catchment to the point where the unit hydrograph is required

NRCS Curve Number which is a measure of runoff generating capacity of a watershed and depends on the soil, the antecedent moisture condition, the cover and the hydrologic conditions of the watershed. Recommended CN values are presented in

Table 0-12.

s = average slope of the catchment

And the peak flow rate of the unit hydrograph (for 1 inch of runoff) is calculated as:

$$\mathbf{Q_p} = \frac{\mathbf{484\ A}}{\mathbf{T_p}} \qquad \textbf{(Equation 9)}$$

where:

Q_p = peak flow rate of the unit hydrograph (for 1 inch of runoff) (cfs)

A = are of the catchment (mi^2)

T_p = time to peak of the unit hydrograph (hr)

Once T_p and Q_p are known, actual time and flow rate coordinates of the t_r-hour unit hydrograph are determined by multiplying the dimensionless time (T/T_p) and the dimensionless flow rate (Q/Q_p) from **Table 0-10** by T_p and Q_p, respectively. The result will be the unit hydrograph of the catchment. The actual synthetic hydrograph of the catchment can be generated by multiplying the unit hydrograph to the depth of runoff.

It shall be noted that **Equations 9** and **Equations 10** are empirical equations, so they shall be used in imperial units (as presented) and the final results be converted to the metric system.

NRCS Curve Number (CN)

As described before, Curve Number (CN) is one of the parameters of the NRCS Dimensionless Unit Hydrograph method which needs to be estimated.

Rainfall infiltration losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors known as Runoff Curve Numbers. These represent the runoff potential of an area. The higher the CN, the higher the runoff potential will be.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. NRCS divides soils into 4 hydrologic soil groups based on infiltration rates as shown in **Table 0-11**. The effects of urbanisation on the natural hydrologic soil groups shall be considered as well. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes shall be made in the soil group selected.

Table 0-11— Hydrologic Soil Groups (TxDOT, 2011) (NRCS, 2007)

Soil group	Description	Soil type	Infiltration rate (mm/hr)
A	Low runoff potential due to high infiltration rates even when saturated	Less than 10% clay and more than 90% sand or gravel Deep sand, deep loess, well-aggregated loamy sand, sandy loam, loam and silty loam	7.6 – 11.4
B	Moderately low runoff potential due to moderate infiltration rates when saturated	10%-20% clay and 50%-90% sand Shallow loess, sandy loam and loamy sand, well-aggregated silty loam, silt and sandy clay loam	3.8 – 7.6

Soil group	Description	Soil type	Infiltration rate (mm/hr)
C	Moderately high runoff potential due to slow infiltration rates	20%-40% clay and less than 50% sand Clay loam, shallow sandy loam, loam, silt loam, sandy clay loam, clay loam, silty clay loam, well-aggregated silty clay and sandy clay	1.3 – 3.8
D	High runoff potential due to very slow infiltration rates	Greater than 40% clay and less than 50% sand Soils that swell significantly when wet, heavy plastic clays, certain saline soils	1.3

Table 0-12 provides a list of suggested runoff curve numbers. These values assume medium Antecedent Moisture Conditions (AMC II). If necessary, CN shall be adjusted for wet or dry antecedent moisture conditions. A five-day period shall be used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period. Average AMCs (AMC II) are recommended for most hydrologic analysis, however in areas with high water table conditions, AMC III conditions may be used. When a drainage area has more than one land use, a composite CN may be used.

The below equations adjust values for expected dry soil conditions (AMC I) and wet soil conditions (AMC III):

$$\mathbf{CN\ (AMC\ I)} = \frac{\mathbf{4.2\ CN(AMC\ II)}}{\mathbf{10 - 0.058\ CN(AMC\ II)}} \qquad \textbf{(Equation 10)}$$

$$CN\ (AMC\ III) = \frac{23\ CN(AMC\ II)}{10+0.13\ CN(AMC\ II)}$$ **(Equation 11)**

Table 0-12— Runoff Curve Numbers (CN) for Urban Areas (NRCS, 2007)

Cover type	Average percent of impervious area*	Hydrologic soil group			
		A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.)					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Paved parking lots, roofs, driveways, etc. excluding Right-of-Way (RoW)		98	98	98	98
Streets and roads					
Paved, curbs and storm drains (excluding RoW)		98	98	98	98
Paved, open ditches (including RoW)		83	89	92	93
Gravel (including RoW)		76	85	89	91
Dirt (including Row)		72	82	87	89
Desert urban areas					
Natural desert landscaping (pervious areas only)		63	77	85	88

Cover type	Average percent of impervious area*	Hydrologic soil group			
		A	B	C	D
Artificial desert landscaping (impervious weed barrier, desert shrub with 2-5 cm sand or gravel mulch and basin borders)		96	96	96	96
Urban districts					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
500 m^2 or less	65	77	85	90	92
1000 m^2	38	61	75	83	87
1350 m^2	30	57	72	81	86
2000 m^2	25	54	70	80	85
4000 m^2	20	51	68	79	84
8000 m^2	12	46	65	77	82
Developing urban areas, newly graded (pervious areas only, no vegetation)		77	86	91	94

*- The average percent impervious area is used to develop the composite CNs.

4.4.2.3 NRCS Triangular Unit Hydrograph Method

NRCS Triangular Unit Hydrograph is an approximation to NRCS Dimensionless Unit Hydrograph described above. This hydrograph (**Figure 0-8**) is defined in terms of three points, Q_p, T_p and T_b.

The lag time, time to peak and peak flow rate are calculated using the same equations as for the Dimensionless Unit Hydrograph.

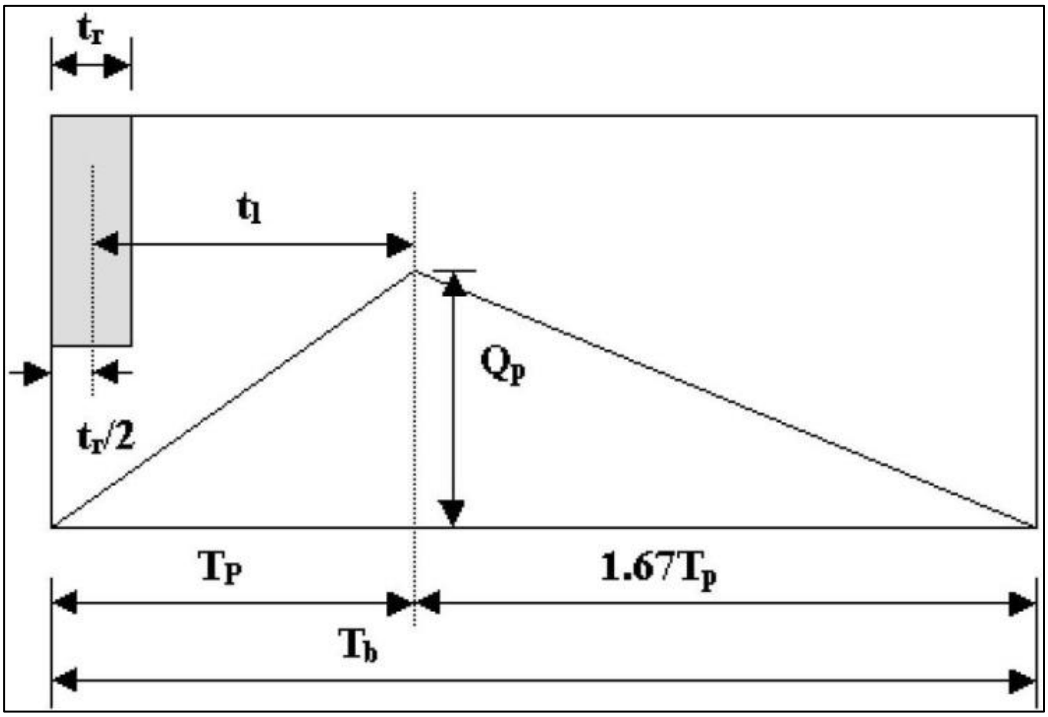


Figure 0-8— NRCS Dimensionless Unit Hydrograph

The above figure shows a dimensionless triangular unit hydrograph. To compute the peak discharge, the following equation is utilized:

$$Qp = \frac{0.0021\ \mathrm{QA}}{\mathrm{Tp}} \qquad \textbf{(Equation 12)}$$

$$Tp = 0.6\mathrm{Tc} + \frac{\mathrm{D}}{2} \qquad \textbf{(Equation 13)}$$

$$D = \frac{\mathrm{Tc}}{7.5} \qquad \textbf{(Equation 14)}$$

Where,

A is area (ha)

T_p is time to peak (hours)

T_c is time of concentration (hours)

D is duration (hours)

Q is total runoff (m3/sec)

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 shall be set at 0.6 Tc, the recession time to 1.67Tp and the default SCS peak rate factor shall be used, with no base flow. Above Figure 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 tc<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

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

4.5 Special Considerations

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 shall 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 shall be used. Where the remainder of the catchment could contribute significant addition flow, this should also be included in the model as separate catchment.

4.6 Flood Volume Calculations

To calculate flood volumes, as well as peak flows such as for 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.

STORM WATER SYSTEM DESIGN

Storm water drainage system in urban city like Dubai, has sole purpose of reducing the risk of surface water flooding through receiving and disposing of rainfall run-off. The storm water system should also capable to intercept subsurface runoff and help to maintain the maximum ground water level below than certain limit.

Propose storm water drainage system in new development areas shall discharge uncontaminated rainfall runoff to the existing storm water drainage system, to evaporation lagoons or direct to the sea. The new storm water system shall be designed by considering the hydraulic constraints imposed by existing storm water network owned by DM or developers. No surface flooding will be allowed unless agreed with DM, along with the necessary checks at the appropriate design storms and return periods.

DM 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. The permitted discharge shall be controlled by providing tank/storage with reasonable size or consider SuDs inside the development to accommodate the flows. Consultant shall discuss the proposal with DM and finalize.

5.1 Flow Attenuation

To lower the peak rate, flow attenuation shall be provided. The proposal of flow attenuation tank to produce more economic designs and to meet hydraulic constraints in existing networks and by reducing the overall costs.

The attenuation shall be provided on either.

- On-line tanks or oversized pipes with a flow control device at their downstream end to limit the pass-forward flow,
- Off-line storage tanks with a small outlet flow control device and high-level overflow back to the main storm water line. Ideally off-line tanks should have a separate tank to retain

the first flush. Therefore, online storage is the preferred method of attenuation as it reduces the maintenance requirements.

- Lagoon (detention pond) where levels and location permit.

Note:

- Maintenance friendly lagoons and off-line tanks 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. But such cases shall be discussed with DM before finalizing the design.
- The flow attenuation measures shall be provided as close as possible to the source of flow.
- The empty time of the storage will be checked with DM.

5.2 Conduit Capacity and Head loss

5.2.1 Colebrook-White Equation

The recommended hydraulic design formula for pipes is the 'Colebrook-White' formula, as this gives the greatest accuracy (ADM, 2016)

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

$$V = -2\,(2gDs)^{\frac{1}{2}} \log\left(\frac{k}{3.7D} + \frac{2.51v}{D\,(2gDs)^{\frac{1}{2}}}\right) \qquad \textbf{(Equation 15)}$$

V = velocity (m/s)

g = gravitational acceleration = 9.81 m/s2

D = pipe bore (m)

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

K = effective roughness (m)

ν = kinematic viscosity of fluid. Water = 0.727 x 10 -6 m2 s^{-1} (at 35 °C)

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

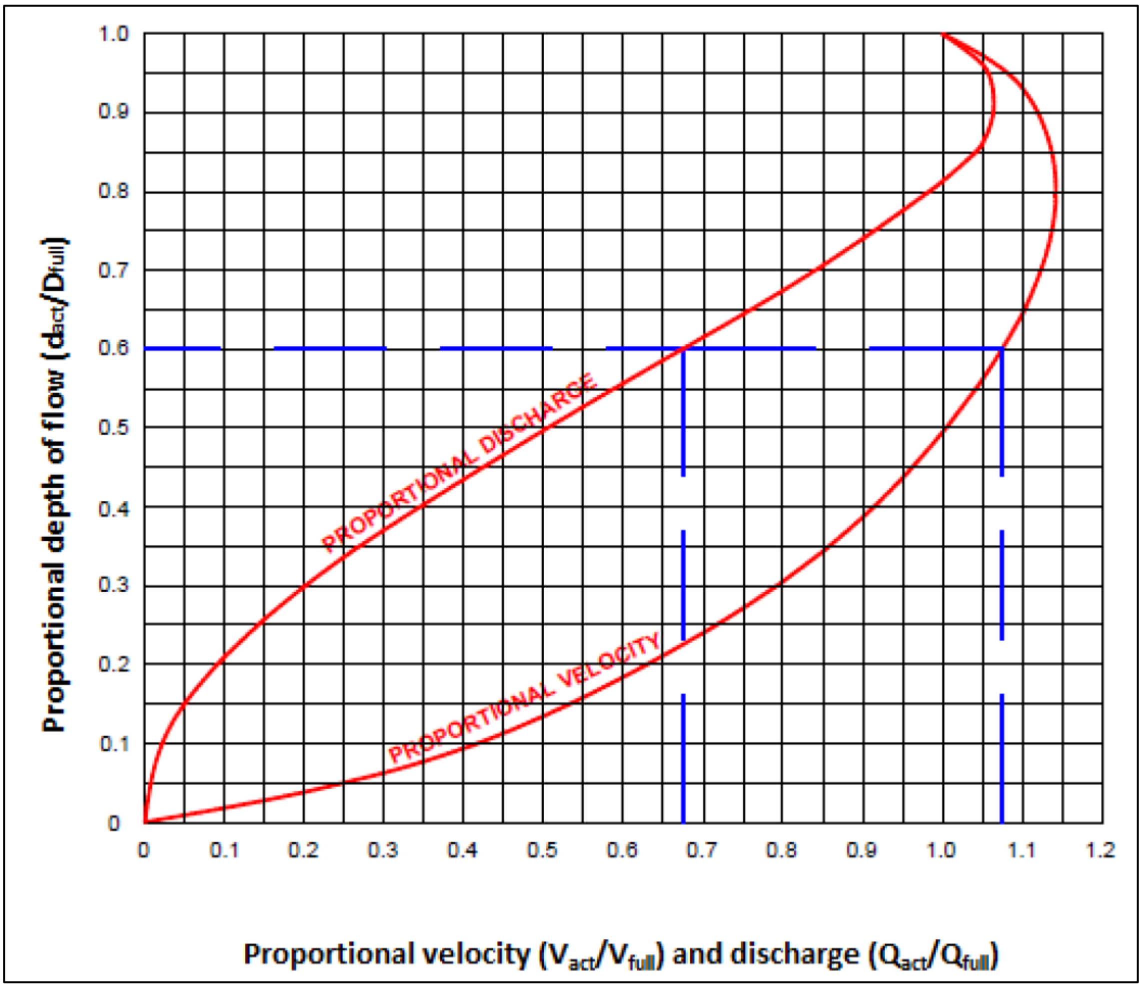


Figure 0-1**— Relative Velocity and Discharge in a Circular Pipe for any Depth of Flow (ADM, 2016)**

The roughness values to be used for storm water design are as follow:

Table 0-1— Colebrook-White Roughness Values

Pipe Material	Colebrook-White, K (mm)		
	Good	Normal	Poor
uPVC & HDPE	0.3	0.6	1.5
GRP Pipes	0.3	0.6	1.5
Line Concrete Pipe	0.3	0.6	1.5

For pressurized pipe Colebrook-White Roughness coefficient will be 0.15mm and 0.3mm for velocity (From 1.1 m/sec to 1.8 m/sec) and velocity (Less than 1.1 m/sec) respectively.

5.2.2 Darcy-Weisbach Equation

The Darcy-Weisbach formula was developed for use in the analysis of pressure pipe systems. However, the formula is sufficiently general so that it can be applied readily to open channel flow systems. In fact, the American Society of Civil Engineers (ASCE) Task Force on Friction Factors in Open Channels (1963) supported the use of the Darcy-Weisbach formula for free-surface flows.

Head loss due to friction can be calculated by Darcy-Weisbach equation as below:

$$\mathbf{h_f = f \frac{L}{D_h} \frac{V^2}{2g}}$$ **(Equation 16)**

where:

h_f = head loss due to friction (m)

f = Darcy-Weisbach friction factor

L = length of the conduit (m)

D_h = hydraulic diameter of the conduit (m), For a full circular pipe, this equals to pipe diameter and for a free surface flow this equals to 4xR in which R is the hydraulic radius

V = average velocity of the flow (m/s)

g = gravity acceleration (m/s²)

Hence, for free surface flow applications, Darcy-Weisbach formula can be written as below, where R is the hydraulic radius (m) and S is the channel slope:

$$\mathbf{V} = \sqrt{\frac{8g}{f}\, R\, S} \qquad \textbf{(Equation 17)}$$

Darcy-Weisbach friction factor for turbulent flows depends on two parameters, i.e. Reynolds Number (R_e) and Relative Roughness (ε/D) and can be determined by the Moody diagram (**Figure 0-2**) or calculated by the Colebrook-White formula.

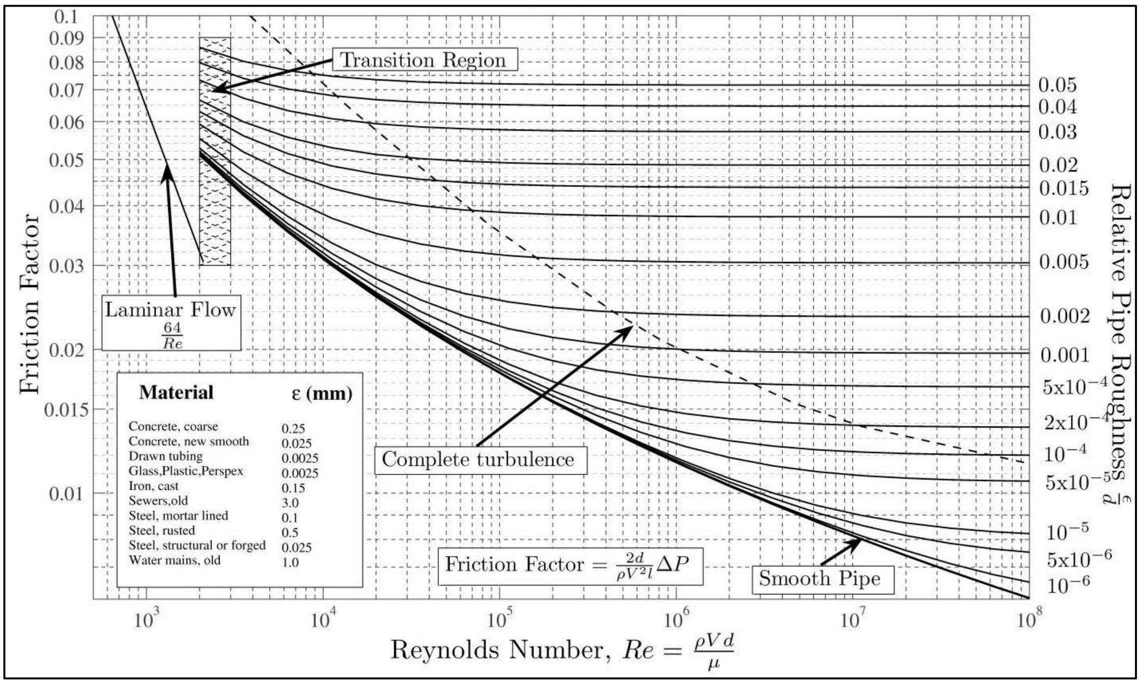


Figure 0-2— Moody Diagram for Estimation of Darcy-Weisbach Friction Factor

The Colebrook-White formula for full flow in pipes is as below:

$$\frac{\mathbf{1}}{\sqrt{\mathbf{f}}} = -2\,\log_{10}\left(\frac{\boldsymbol{\varepsilon}}{\mathbf{3.7\,D}} + \frac{\mathbf{2.51}}{\mathbf{R_e}\sqrt{\mathbf{f}}}\right) \qquad \textbf{(Equation 18)}$$

Or

$$\frac{\mathbf{1}}{\sqrt{\mathbf{f}}} = -2\,\log_{10}\left(\frac{\boldsymbol{\varepsilon}}{\mathbf{14.8\,R}} + \frac{\mathbf{2.51}}{\mathbf{R_e}\sqrt{\mathbf{f}}}\right) \qquad \textbf{(Equation 19)}$$

Where,

f = Darcy-Weisbach friction factor

ε = roughness height (m)

D = pipe diameter (m)

Re = Reynolds Number

R = hydraulic radius (m)

As it is clear, using Moody Diagram or Colebrook-White formula is an implicit procedure which requires an iterative solution. Hence, some approximations of Colebrook-White formula have been developed which explicitly calculate the Darcy-Weisbach friction factor. One of these formulas which calculates the Darcy-Weisbach friction factor in a full flowing circular pipe with acceptable accuracy is the Swamee-Jain equation, as presented below:

$$\mathbf{f} = \frac{0.25}{\left[\log\left(\frac{\varepsilon}{3.7\,D} + \frac{5.74}{R_e^{0.9}}\right)\right]^2} \quad \textbf{(Equation 20)}$$

Where,

f = Darcy-Weisbach friction factor

D = pipe diameter (m)

R_e = Reynolds Number

5.2.3 Hazen-Williams formula

The Hazen-Williams formula is an empirical equation which has been used as a practical equation for water flow in pressure conduits. The Hazen-Williams formula is written as below:

$$\mathbf{h_f} = \frac{10.7\,L\,Q^{1.852}}{C^{1.852}\,D^{4.87}} \qquad \textbf{(Equation 21)}$$

Where:

h_f = head loss due to friction (m)

L = length of the conduit (m)

Q = flow discharge (m^3/s)

C = Hazen-Williams roughness coefficient

D = pipe diameter (m)

Table 0-2 presents the Hazen-Williams roughness coefficient (C) for different materials.

Table 0-2— Hazen-Williams Roughness Coefficient (Hammer, 1988)

Material	Roughness Coefficient (C)
Glass Reinforce Pipe	130
Ductile iron	
Cement lined	130-150
New, unlined	130
5 years old, unlined	120
20 years old, unlined	100
Concrete	130
Copper	130-140
Plastic	140-150
Steel	
New, welded	120
New, riveted	110

5.2.4 Manning's formula

Manning's formula is widely used in open channel flow and is written as below:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \textbf{(Equation 22)}$$

where:

Q = Discharge (m^3/s)

n = Manning's roughness coefficient

A = Flow area (m^2)

R = hydraulic radius (m)

S = slope (m/m)

For design the storm water system, manning will be 0.013 regardless the pipe material as sensitivity, The manning values for different materials are as follow:

Table 0-3— Manning's roughness coefficients

Pipe Material	Manning Coefficient, n
uPVC	0.011
HDPE	0.012
GRP Pipes	0.012
Line Concrete Pipe	0.011

If the Manning and Darcy-Weisbach equations are combined, Manning's roughness coefficient can be calculated based on the Darcy-Weisbach roughness coefficient as below:

$$n = R^{\frac{1}{6}} \sqrt{\frac{f}{8\,g}} = 0.1129\, R^{\frac{1}{6}} \sqrt{f} \quad \textbf{(Equation 23)}$$

Where:

n = Manning's roughness coefficient

R = hydraulic radius (m)

f = Darcy-Weisbach roughness coefficient

5.3 Minor Losses

The previous section presented a number of methods for estimating the head loss caused by friction as flow moves along the length of a pipe. In addition to friction losses, energy losses occur due to changes in flow velocity through inlets, pipe entrances, outlets, transitions, bends, and other appurtenance. These losses, known as minor losses, shall be considered in the design of drainage systems and are computed by multiplying a coefficient K by the velocity head or change in velocity head. Typical values for K under various conditions can be found in relevant literature.

5.4 Flow Velocity

Design flow velocities shall be within the limits provided in **Table 0-4**. Minimum velocities are based on providing self-cleansing velocities and prevent sedimentation in the drainage pipes and channels. Maximum velocities are set to minimise the negative effect of abrasion on the pipes and manholes and erosion in open channels, ponds and other water bodies.

Table 0-4— Flow Velocity Limits in the Drainage Systems

Drainage system component	Minimum velocity (m/s)	Maximum velocity (m/s)
Pipes		
Gravity line	0.75	2.5
Pressure line	1.0	2.5
Open channels		
Un-lined	0.75	2.0

Drainage system component	Minimum velocity (m/s)	Maximum velocity (m/s)
Lined	0.75	2.5
Bio-retention swales	0.75	1.5
Filter drains	0.75	1.0
Detention/retention ponds	N/A	1.0

In rising mains, for cases where initial flows are significantly lower than future/ultimate flows, two or more rising mains may be considered. This could be the case with regional pumping facilities where some portion of the catchment will not be developed for several years.

It shall be noted that the velocity range shall be met on a daily basis, not necessarily each time the pumping station operates. Velocities shall be checked for each pumping scenario, especially in the case of dry weather season groundwater control.

Storm water flows in arid climates are normally accompanied by transport of large amount of suspended sediments. The flow properties in these situations are different from the properties of clear water. It has been found that an increase in suspended load tends to decrease conduit resistance (friction/roughness) and thus causes an increase in flow velocity (Vanoni, 1941). It is also shown that the decrease in roughness can be as much as 20% of its clear water value if the concentration of Suspended Sediment (SS) in the flow is as high as 4.0 g/l (Nouh, 2001). **Figure 0-3** shows the variation of roughness with increase in suspended sediment concentration. Hence, the decrease in roughness shall be considered if high concentration of suspended sediment is anticipated.

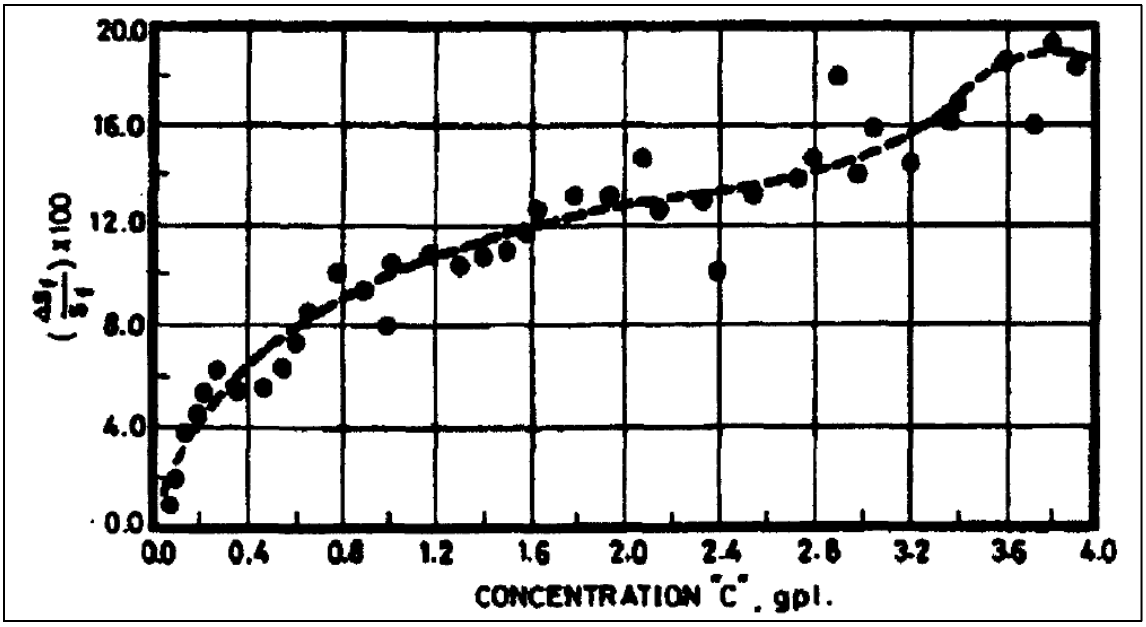


Figure 0-3— Decrease in Roughness with Increase in SS Concentration (Nouh, 2001)

5.5 Free-Surface Flow

Hydraulic conditions of free-surface flows in open channels and gravity pipes and mains can be classified as sub-critical and super critical based on the Froude Number, calculated as:

$$\mathbf{F}_r = \frac{v}{\sqrt{gD}} \qquad \textbf{(Equation 24)}$$

Where:

v = flow velocity (m/s)

g = gravity acceleration (m/s²)

D = hydraulic depth (m)

In hydraulic design of drainage systems, it is preferred to keep the flow conditions sub-critical, i.e. F_r<1.0. There is a theoretical threshold between sub-critical and super critical flows (F_r=1.0); therefore for practical reasons as well as safety factor, F_r shall be kept below 0.85 for sub-critical condition.

In super critical conditions (F_r>1.0), flow velocity and turbulence are higher than the equivalent sub-critical conditions and if the geometric and topographic conditions of the drainage system changes, e.g. a steep invert slope changes to a milder slope, at the location of this change, a hydraulic jump may occur which is a zone of high turbulence which can cause erosion and scouring. Hence, if occurrence of super critical condition is unavoidable, appropriate control measures such as erosion control and scour protection shall be provided at the outlet point of a super critical flow to another drainage system component or at the location where hydraulic conditions change from super critical to sub-critical.

5.6 Pressurised Flow

Pressurised flow conditions in pressure mains and rising mains shall be investigated in detail and Hydraulic Grade Line (HGL) and system curves shall be prepared to clearly demonstrate the performance of the system. The system shall be hydraulically designed to achieve the required pumped capacity within the design flow velocity as per **Table 0-4** and the size of the pipe shall be selected taking into consideration the Life Cycle Cost of the pumping system.

Upon steady state analysis study for the pumped system, the transient flow analysis shall be undertaken in which all the cases that produce transient flow conditions shall be addressed. The best design shall be based on avoiding create transient conditions or keep it to the minimum, failing to do that a surge suppression system shall be proposed to keep the system working at healthy and safe conditions. The choice of the modelling software package shall be approved by DM prior to commencement of the modelling task.

Hydraulic surges could have the potential to create a catastrophic failure at the pump station or within the pipeline. This aspect of surge analysis represents a substantial risk to the client, the designer and the operators. Design of surge mitigating measures shall be undertaken under the direct supervision of a Senior Hydraulic Specialist. No one solution applies to all pump stations.

During construction, a separate surge analysis will be required of the Contractor based on the actual supplied materials and installed equipment. Modifications to the surge mitigation measures and devices may be required at that time.

5.7 Tailwater

In order to carry out an HGL backwater analysis a storm water system, it is necessary to determine a starting/downstream Tailwater Level (TWL) for the calculations.

The designer shall consider in all cases to the adopted starting HGL and if necessary, liaise with DM to establish an agreement.

5.7.1 Tail water Levels for Tidal Outfalls

The starting water level used in the hydraulic analysis of storm water drainage systems discharging to tidal water bodies may be influenced by the following factors:

Tidal Variations

The UK Hydrographic Office publishes Admiralty Charts which predict tide levels throughout the year and define the average levels of the tidal planes at different locations globally. These charts can be used to identify tidal levels along the coastline of Dubai. Care must be taken when referencing the above tide tables to correctly translate the quoted levels to the survey datum used for the drainage design. It shall be noted that tide tables do not predict actual sea levels. Actual sea levels are the result of a combination of the different factors as described in this section. Therefore, Highest Astronomical Tide (HAT) only does not represent the likely highest possible sea.

Storm Surge

A storm surge (or meteorological tide) is an atmospherically driven ocean response caused by extreme surface winds and low surface pressure associated with severe weather conditions, usually cyclones. Strong offshore winds can generate significant ocean currents. When these currents approach a barrier such as a shoreline, sea levels increase (wind setup) as the water is forced up against the land. The low atmospheric pressures associated with cyclones can also raise sea levels well above predicted tide levels.

Storm induced wave action can produce both a wave setup (a rise in mean sea level as waves approach a shoreline) and wave run-up. Wave run-up is generally not considered in the selection of tail water level

When storm surge and wave setups are combined with the normal astronomical tide, the resulting Mean Water Level (MWL) reached is called the storm tide level.

Designers should note the following issues:

- Predicted storm surge elevations may vary significantly along the coastline.
- A storm surge is more likely to be associated with a long duration storm event such as a cyclone.
- The existence of a storm surge is highly probable during peak flooding events.
- A storm surge will likely be coincident with the peak outflow from storm events.

It is recommended that designers confer with DM in order to determine an appropriate tail water level for piped and open channel outfalls to tidal waterways.

Wave Setup

Wave setup is defined as the super-elevation of water levels due to the on-shore movement of water by wave action alone. Wave setup is the change in MWL due to wave action. It is not the actual wave height. It may occur during, or in the absence of, a storm event.

Wave setup is likely to occur during severe storms and shall be incorporated into the storm surge prediction for coastal waters. Wave setup can also occur on large water bodies such as the Dubai Creek. Consideration shall be given to the likely water level increase caused by wave setup when nominating the starting water level in the sea or the Creek. For example, a conservative estimate for Dubai Creek is a wave setup of up to 0.7 m for a 1 in 100 year storm. Guidelines for the determination of wave setup may be obtained from the "*Shore Protection Manual*" by the U.S. Army Corps of Engineers.

Climate Change

Designers should consider the impact of climate change on tail water levels. Predictions of the possible effect on sea level and other effects are given in the "*Intergovernmental Panel on Climate Change (IPCC) 4th Assessment Report IPCC-2007*".

The global risk guidance future scenarios for average sea level increases under low and high global warming scenarios shall be considered.

Designers should ensure they are familiar with the latest design/research information and should liaise with DM EPSS in this regard.

As mentioned, designers should confer with DM to establish an appropriate tail water level for the design of storm water outfalls to the sea or the Creek. Consideration shall be given to the joint probability of occurrence of the design storm, tide level and storm surge together with allowance for climate change.

Whilst it is not possible here to provide specific recommendations, some suggested levels are provided in **Table 0-5**. These suggestions should in no way replace the need to confer with DM and for the application of sound engineering judgement.

Table 0-5— Suggested Tail water Levels for Discharge to Tidal Waterways

Design condition	Design tailwater level*
Minor storm (5 to 10 year ARI)	MHWN to MHWS
Major storm (50 to 100 year ARI)	MHWS to HAT
Climate change	Additional 0.3 m

*- MHWN (Mean High Water Neap), MHWS (Mean High Water Spring), HAT (Highest Astronomical Tide). For more information on these terms, refer to (CIRIA, 1996).

5.7.2 Tail water levels for Non-Tidal Outfalls

The design of a drainage system which discharges to a non-tidal outfall, e.g. an open channel, a lake or a pond needs to take into account the expected tail water level in the receiving water body.

In cases where the tail water level is not affected by storm water runoff from an external catchment, e.g. in a detention basin or an open channel which receives water from only the subject drainage system, the tail water level shall be determined in accordance with the following:

Outlet to Lakes

Design tail water levels for outfalls discharging into lakes need to consider the potential seasonal variation in water level. As a design storm event is likely to occur following a period of a significant storm event, it is practical to assume that the lake will be at or approaching full capacity at the time the design storm occurs. The starting HGL for the design storm should therefore be set at the overflow level of the lake.

Note that under certain circumstances, the starting HGL may be lower than that discussed above. For example, where the ARI of design storm for the catchment is low (e.g. 2 years) and the lake is large, the lake may or may not be full. In such cases the starting HGL shall be determined in consultation with DM.

Outlets to Detention/Retention Basins

It is usual for a detention basin to be designed and checked for a number of ARIs. The starting HGL level for the design ARI of the pipe system shall be determined by analysing the detention basin for the same ARI as the pipeline being designed. If other pipe systems contribute and have catchment characteristics vastly different to those for the system being designed, then the designer must consider the behaviour of the system as a whole.

In subcritical outflow conditions, the position of the starting HGL will depend upon the relationship between the calculated tail water in the receiving waters, the critical depth (dc) of the particular flow under consideration in the outfall pipe and the obvert level (OL) of the pipe. The following general rules should apply (**Figure 0-4**):

a) If TWL > OL, then start HGL = TWL

b) If TWL ≤ OL and TWL ≥ dc, then start HGL = OL

c) If TWL < dc (i.e. free outfall), then start HGL = the normal flow depth (dn) in the outfall pipe for the given flow rate

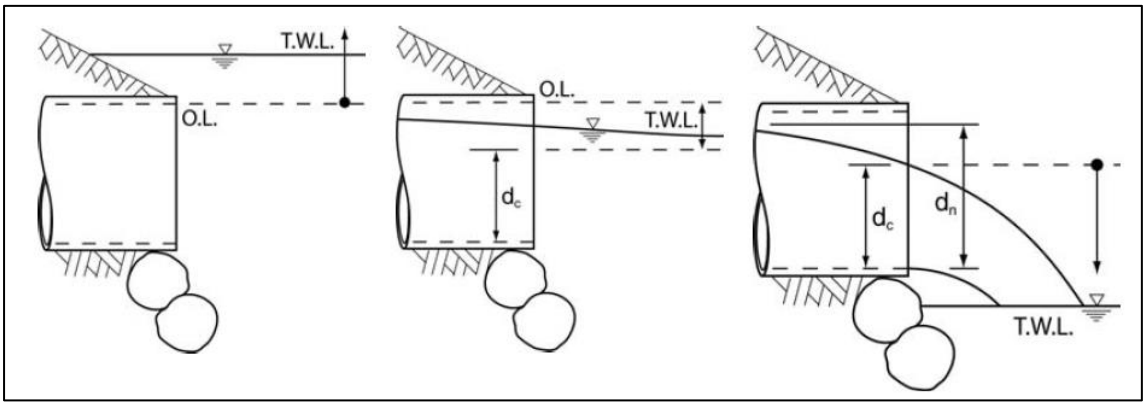


Figure 0-4— Hydraulic Conditions of Outfalls, from Left to Right, (a) Tail water above Obvert, (b) Tail water below Obvert, (c) Tail water below Invert

Note: The startling HGL conditions presented in (b) and (c) do not necessarily apply to the analysis of outflow from short pipes such as most culverts.

5.7.3 Tail water Levels in Existing Pipe Networks

The designer should determine the HGL of the existing system for the design ARI. Full account of structure losses shall be made in the existing system.

If this is considered impractical due to the complexity of the existing pipe network and lack of required information, then an appropriate estimation of the HGL in the existing network must be made.

5.7.4 Tail water Levels in Future Pipe Networks

Where design of a piped system is being undertaken in the upstream section of a catchment prior to the design of the downstream system, the designer should undertake sufficient preliminary planning of the downstream system to permit design of the upstream system. This planning should incorporate preliminary road layouts and levels along with preliminary drainage line locations and levels. To allow for possible inaccuracies associated with such a preliminary design, a factor of safety may need to be allowed. For example:

- allow a nominal height above the assessed HGL at the proposed connection to the downstream system

- adopt the HGL equal to the natural surface at the location of the next downstream structure in the proposed future pipe network

- adopt a starting HGL as approved by DM

5.8 Hydraulic Modelling of the Drainage System

All drainage system designs shall be modelled using internationally recognised numerical software such as InfoSWMM (on which the Dubai Drainage Master Plan model was developed). For other compatible software, DM approval is required.

5.8.1 Review of the Physical System

As a minimum, a broad understanding of the hydraulic behaviour of the physical system in question is essential to make an informed choice on the modelling approach and parameters. Whilst it is common that detailed hydraulic behaviour of the system is unknown, a good working knowledge of the study area and catchment is needed. Aspects such as study area shape, elevation and slope are important. The number and size of hydraulic structures and drainage dimensions shall be understood. Land-use is also an extremely important element of the physical system to consider.

5.8.2 Selection of the Numerical Model

Selection of the appropriate type of model is a critical decision. In this step, considering the physical system and its hydraulic characteristics, assumptions have to be made as to whether the flow can be considered as being one-dimensional (1D), two-dimensional (2D), or a combination of both, and whether the flow can be described as being steady (i.e. constant with time), or unsteady (time-varying). In all rural or urban storm water modelling, vertical accelerations in the flow field are considered to be negligible and a hydrostatic pressure distribution is assumed, with computations and results based around a depth-averaged velocity.

It is important to understand the distinction between models that are typically referred to as "full 2D" numerical models and those that are of a lesser standard or capability. A full 2D scheme for the purposes of urban and rural storm water modelling is accepted to be any model that numerically represents the complete depth-averaged or shallow water free-surface wave equations. Simplified numerical representations of the 2D free surface wave equations can be appropriate in many situations; however the modeller shall be aware of the limitations of such schemes. Further, the modeller should also be confident that any limiting assumptions will remain valid over the entire scope of the modelling project of interest, in terms of the range of potential geometries and flows that will be investigated.

It shall be noted that for the time being and until a combined 1D-2D model of the drainage system and overland flow across Dubai is available, 1D models of the drainage system are acceptable.

5.8.3 Development of the Site-Specific Model

The site-specific model is developed using the generic numerical hydraulic model (software package) through the selection of:

- A modelling domain
- Spacing/resolution and time step
- The input of site-specific data including topographic data, network geometry, structures, etc.
- The application of flow and/or water level boundary conditions

The site-specific model must then be calibrated and verified (if possible depending on the available information) to ensure it is capable of reproducing flow behaviour at the subject site. If for a particular reason, the model cannot be calibrated and verified, justifications shall be provided to DM and approval sought.

5.8.4 Boundary Conditions

Boundary conditions are required at the model boundaries.

The upstream boundary conditions are generally provided by a discharge hydrograph.

The downstream boundary conditions are generally specified in terms of water surface elevations (tail water). These may be specified as a constant, a times series, or computed internally using a rating curve. The joint probability of an inland flooding and that of the sea level may need to be considered. As in most instances the most severe flooding at coastal regions occurs when the flood from the inland faces high water level in the sea (high tide and/or storm surge). Therefore, for major outfalls, a joint probability analysis shall be undertaken to find out the probability of such flooding in coastal regions and identify accurate downstream boundary conditions.

5.8.5 Modelling Log and Naming Conventions

Establishment of a modelling log is essential. The log could be in Microsoft Excel or Word and should contain sufficient information to record model versions during development and calibration, file naming conventions and observations from simulations.

Model file naming conventions and locations are important in ensuring that simulations can be undertaken efficiently, with high traceability, and that old simulations can be reproduced as required. They also assist in minimising human errors. Successful model file naming conventions have the following characteristics:

Files are named using a logical and appropriate system that allows easy interpretation of file purpose and content

- A model version naming and numbering system (designed prior to modelling) shall be included in input data filenames
- A logical and appropriate system of folders is used that manages the files
- Documentation of the above in the Project Quality Control Document and/or Modelling Log

5.8.6 Historical Flood Data

Historical flood information is particularly important as it can often provide a significant improvement in the quality and reliability of the study outcomes. While data on historical floods may be difficult to obtain at times, efforts expended in finding and analysing these data are extremely valuable.

Typically, information on the historical flood behaviour is collected at the commencement of the study. It is recommended that the data collection process involve significant stakeholder consultation. In undertaking the data collection, there are many types of data that may be found.

Drainage pumping station working hours are usually available from DM which can help in understanding the behaviour of the system during storm events.

Anecdotal information is another source of information for model calibration which is usually qualitative in nature but can be very valuable in determining the system behaviour and subsequently verifying that the model behaves in a similar manner.

Photograph and video evidence can also be beneficial in this regard. However, it shall be noted that memories can sometimes fade or be skewed by other events that have occurred between. In addition, information providers may not be able to provide unbiased information. Detailed discussions with residents and stakeholders can provide the modeller/designer with a general feel for the reliability of all anecdotal evidence.

5.8.7 Calibration, Verification and Sensitivity Analysis

Calibration of a hydraulic model is a critical and important stage of the model development. Calibration demonstrates that the hydraulic model is capable of reproducing system behaviour within acceptable parameter bounds. In the absence of historical flooding information, attempt shall be made to cross-check the model against other modelling or desktop analyses, if possible.

Model verification shall also be undertaken with additional model simulations or analyses after the model calibration to “independently” proof or verify the model, if available information allows.

Before commencing calibration, the criteria for achieving an acceptable calibration shall be clearly defined in agreement with DM. For example, the criteria could simply be that the timing of the flood (pumping stations working hours) is to be “consistent” with the observations or similar criteria depending on the specific project conditions and requirements.

Calibration and sensitivity analysis are essential as they provide an indication of the uncertainty associated with the model results. Poor calibration results can highlight deficiencies in the schematisation of key features, or limitations of the historical data. Sensitivity analysis can provide direction during the calibration process, by indicating the model parameters or inputs on which the calibration results are most dependent, so the modeller can focus on reducing the uncertainty of those inputs. The primary consideration is that the calibration process should reflect the purpose for which the model is intended.

However, the sources of uncertainty in hydraulic modelling shall be understood and appreciated and the impacts of these uncertainties on the modellers’ ability to calibrate the model shall be considered. There can be significant uncertainties associated with the input data, recorded information, hydrological modelling, model schematisation and modelling software capability. During calibration, it is important that DM technical staff are engaged in constructive dialogs about these inaccuracies and their impacts. It is far more important to understand why a model may not be calibrating well at a particular location than to use unrealistic parameter values to ‘force’ the model to calibrate.

It is worth repeating that the goal of a calibration is to produce a model that is capable of adequately representing the physical system and, in doing so, producing reliable results. Community members and other stakeholders may have first-hand experience of observations during a storm event.

For the calibration and verification process, the following aspects shall be considered:

- Which historical events are likely to be used for calibration/verification?
- How much data exists for each event?
- What is the reliability and relevance of these data?

- Were the events recent enough for present members of the community to remember?
- Is the spread in magnitude of the historical events similar to the spread in magnitude of the proposed design events?
- Could the model be calibrated, and therefore considered reliable, for both small and large historical events, or through relatively wet and dry periods?
- What model parameters will be adjusted for calibration of the model?
- Are the available geometry, topography and control structure data consistent with the state of the catchment during the calibration events, or have there been major changes due to development, upgrades or some other cause?
- Based on the available data, how much confidence will there be in the calibration? Will it be sufficient to achieve the desired objectives?

Sensitivity testing of model parameters, uncertainties in input data and the model schematisation will be a part of the modelling task. It also plays a useful role for establishing the uncertainty of un-calibrated models.

For models that are well-calibrated to a range of storm events and later verified, considerable confidence can be had in the model ability to reproduce relatively accurate results. This in turn means that factors of safety such as the design freeboard applied to detention ponds can be kept to a minimum. However, for un-calibrated or poorly calibrated models, less confidence can be had in the model accuracy, and greater factors of safety (e.g. larger freeboards) shall be applied to reflect the greater uncertainty. To quantify these uncertainties, sensitivity testing shall be carried out where a model calibration is non-existent or poor.

Examples of sensitivity testing to help quantify a model uncertainty are:

- Adjust hydraulic roughness parameters up and down by a certain level, e.g. 20%
- Increase inflows by a certain level, e.g. 20%

- For downstream boundaries, not at a receiving water body such as the ocean, vary the stage discharge or water level upwards to check that the water levels in the area of interest are not greatly affected
- Apply blockages and greater losses to hydraulic structures and inlets
- Vary the time step and other computational parameters

5.8.8 Processing and Analysis of Results

The following aspects shall be considered in processing and analysis of modelling results:

- Required modelling scenarios and events
- Key assumptions against which the sensitivity of the results will need to be checked
- Required outputs
- Acceptable level of accuracy of the results
- Collating, analysis and presentation of the model outputs

5.8.9 Checking of Results

The first step to be applied during the calibration/verification phase is basic checking of the results for obvious errors and model numerical “health.” Every new option or model run would have some level of sanity checking to ensure that the results are consistent with what was expected.

A process shall be developed for checking that model results are sensible and consistent. As a minimum, the following checks shall be undertaken when interpreting results:

- Mass balance - errors greater than 1% to 2% should generally be investigated, and the cause of the errors identified and rectified where possible.

- Continuity - discharge hydrographs shall be obtained at several locations along the drainage lines, and at locations upstream and downstream of major intersections, to check that the continuity and attenuation of flows is reasonable.

- Stability - the results shall be checked for signs of instability, such as unrealistic jumps or discontinuities in flow behaviour, oscillations (particularly around structures or boundaries), excessive reductions in time step or iterations required to achieve convergence.

- Froude numbers - Froude numbers shall be checked to identify areas of trans-critical and super-critical flow, and the implications of this flow behaviour on the model results considered. In general, model results in areas of trans-critical flow shall be used with extreme caution. Flow at hydraulic control structures shall be roughly checked with suitable hand calculations, such as the weir and/or orifice equations.

- Structure head losses - head losses through structures such as gullies, culverts, manholes, etc. shall be checked against suitable hand calculations.

The model developed for the system under design shall be incorporated in the overall Dubai Stormwater Info Work ICM model in order to confirm compatibility with and connection to the existing system, and assess the impact that the new development/ drainage system will have on DM Stormwater system.

5.9 Soakaways, Open Channels & Ditches

DM pre-approval is required before proposing the soakaways and soakaway trenches, Open Channel/Ditches within Dubai.

DRAINAGE SYSTEM COMPONENTS

6.1 General

In this section, drainage components which contribute to attenuation and conveyance of storm water runoff will be described. Some other components which contribute to storm water runoff treatment will be described in **Section 0**.

The following general items shall be considered and applied in all future drainage projects:

- Drainage System Concept:
- General Planning: any future drainage project shall be set up with the following design stages:
  - Preliminary Investigations and Optioneering
  - Detailed Survey
  - Preliminary Design
  - Detailed Design
  - Construction Support and Supervision

6.2 Curbs and Gutters

During design storm events, the depth of open-channel flow in roadway gutters must be limited to prevent overflowing of curbs and consequent flooding of adjacent properties. The spread (top width) of the gutter flow should also be limited to keep water from extending too far into roadway travel lanes. Excessive spread can cause safety hazards such as vehicular hydroplaning and limited visibility due to splash and spray. At locations where the depth and/or spread of the flow into the travel lane begins to exceed the allowed value, storm water inlets or catch basins are required to intercept some or all of the water and convey it into the subsurface storm water network.

The relationship between the depth of flow in a gutter and its spread depends on the gutter type and the cross slope of the pavement and/or gutter. As illustrated in **Figure 0-1**, gutters fall into two basic categories, i.e. conventional curb and gutter sections and shallow swale sections. With conventional curb and gutter, the flow cross-section extends from the curb toward the roadway centreline. The basic types of curb and gutter illustrated in the figure are uniform, composite, and curved sections. For streets where curbs are not required, shallow swale sections along the side of the road or in the median can be used instead. Swale sections include V-shape gutters, V-shape median swales, and circular swales. In urbanized areas, composite gutters and V-shape gutters are the most common types (Durrans, 2007).

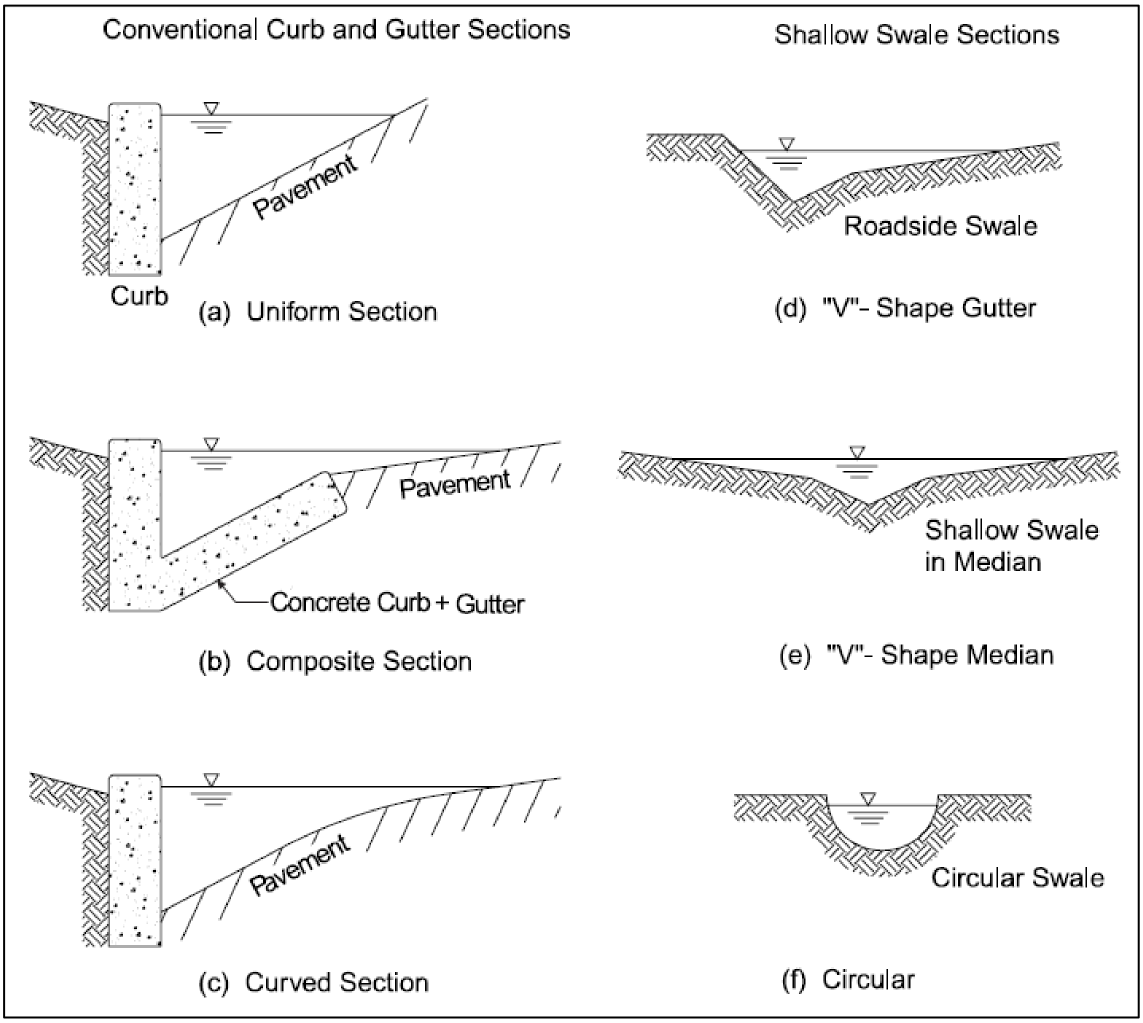


Figure 0-1— Typical Gutter Cross Sections (Durrans, 2007)

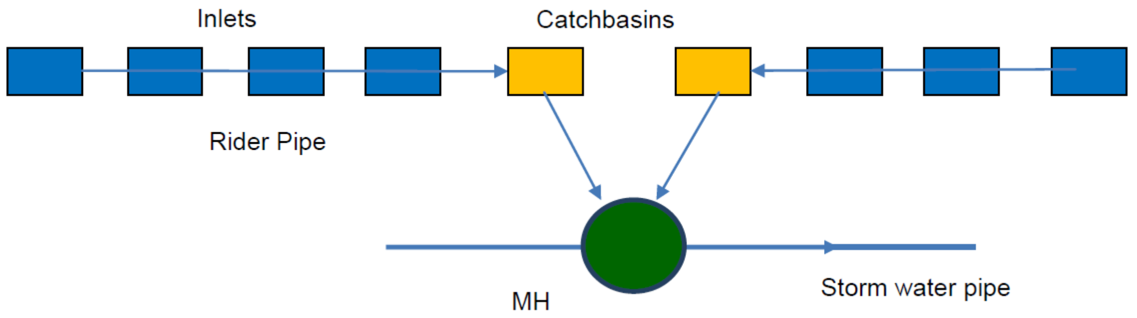
To maintain proper drainage, roadway longitudinal slopes (slopes along the length of the road) are usually specified to be no smaller than about 0.4%. It is difficult to maintain positive drainage with smaller slopes, resulting in localised puddling. Pavement cross-slopes (measured perpendicular to the road direction) necessary to induce flow off of the pavement surface and into the gutter range from 1.5% to as high as about 4%, but are typically specified at around 2% (Durrans, 2007).

The allowable spread of open-channel flow in a gutter section is generally a function of the roadway classification, the design traffic speed, and whether that portion of the gutter is in a sag location (that is, at the low point of a roadway vertical curve). Generally, the maximum spread of flow on the roadways shall be as follow:

- Ponding shall be minimized on the traffic lanes of high-speed, high-volume highways, where it is not expected to occur.
- Considering the hydroplaning effect while classified the road and highways.
- Minimize the impact and inconvenience to pedestrians and structures or buildings adjacent to road kerb which are located within the splash zone. Consultant has to discuss the splash zones with DM and obtain approval before proceeding in the design.
- Multi-lane kerbed roadways with slow speed and longitudinal grade less than 1 % in urban area, without parking/shoulders, the spread on the travel lane shall be acceptable, with certain spread depth and width over road. The spread and width shall vary based on the classification of roads and priority of the project and each time consultant shall obtain approval from DM.
- 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. Consultant shall provide spread as per DM requirements.

6.3 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 below. 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.



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 shall be spaced between 20-50m intervals. 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.

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 for details.

Table 0-1— Use of Inlets and Catch basins

Outlet pipe diameter up to (mm)	Maximum depth (D) to outlet pipe invert level (m)	Inlet Internal Dimensions (mm)	Catch Basins Internal Dimensions (mm)
300	D=<2.0	600 x 600	600 x 1600

Outlet pipe diameter up to (mm)	Maximum depth (D) to outlet pipe invert level (m)	Inlet Internal Dimensions (mm)	Catch Basins Internal Dimensions (mm)
			630 x 1600
500	2.0>D=<2.8	1000 x 1000	1000 x 1600

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

The inlet gullies are essential component of the storm water system. Therefore, inlets gullies shall be correctly considered in the hydraulic model. The storm water model shall not be considered completed without inclusion of such inlets to the road.

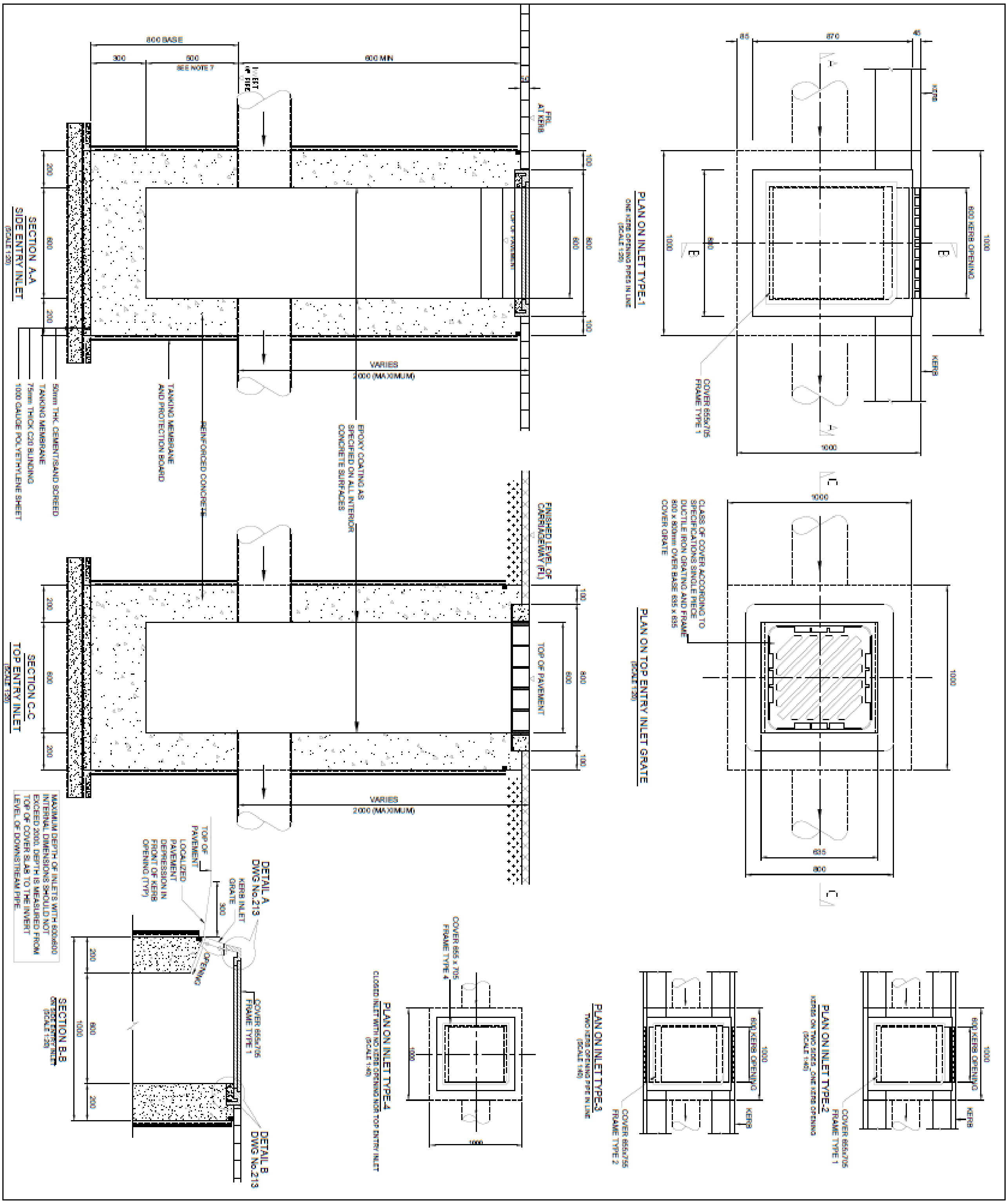


Figure 0-2— Inlet with Outlet pipe 300mm and D=<2.0 (ADM, 2016)

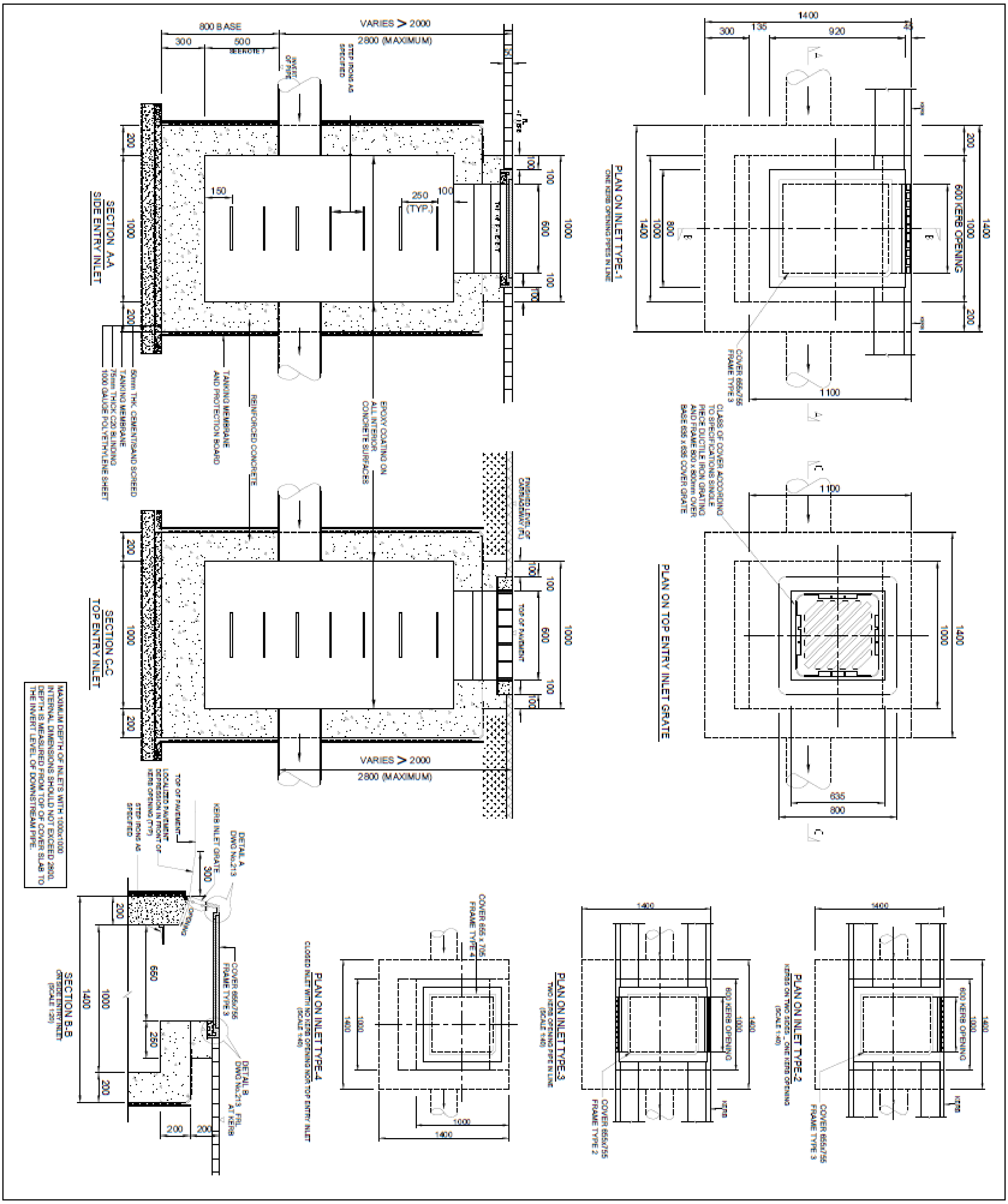


Figure 0-3— Inlet with Outlet pipe 500mm and 2.0>D=<2.8 (ADM, 2016)

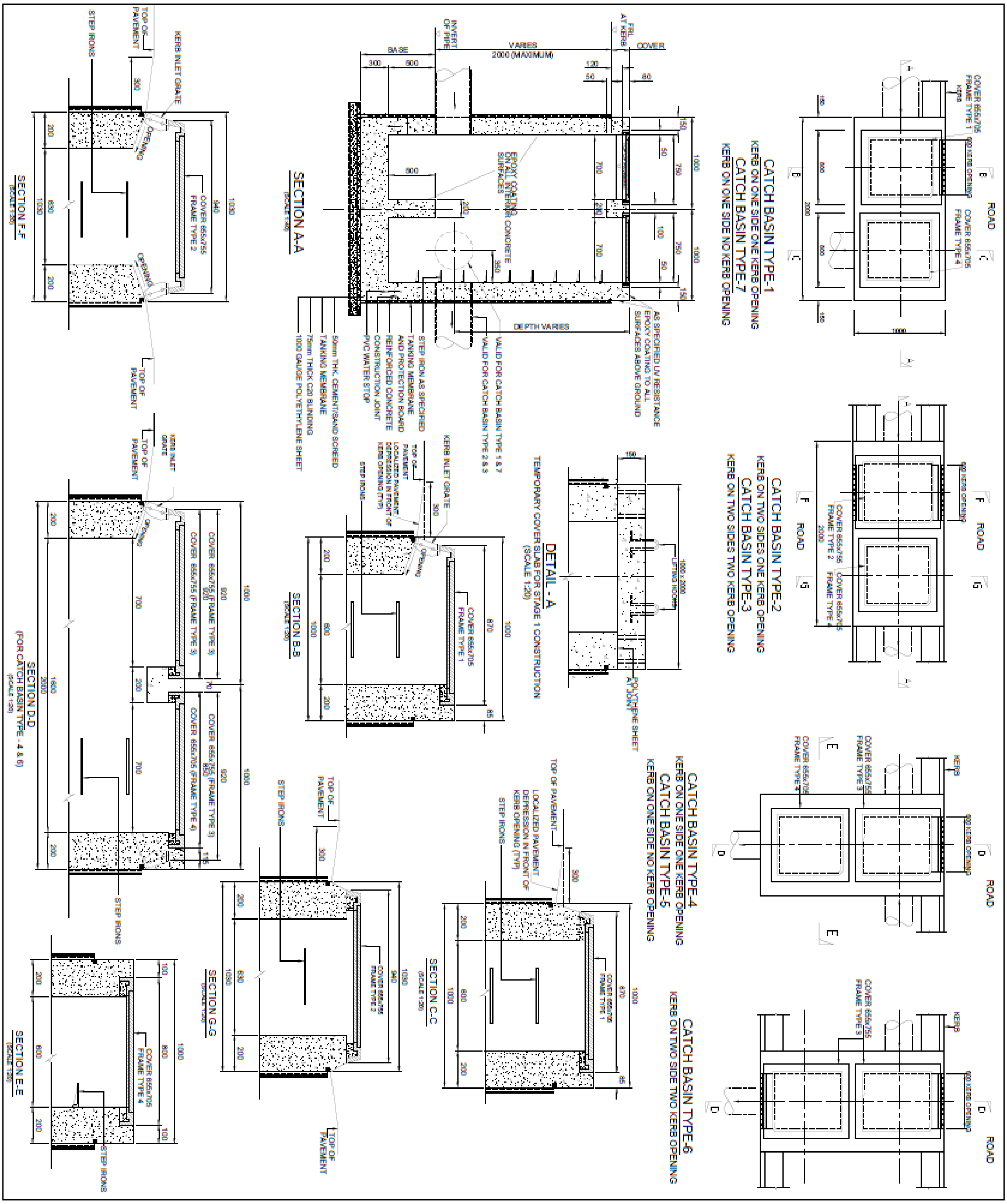


Figure 0-4— Catch Basin with Outlet pipe 300mm and D=<2.0 (ADM, 2016)

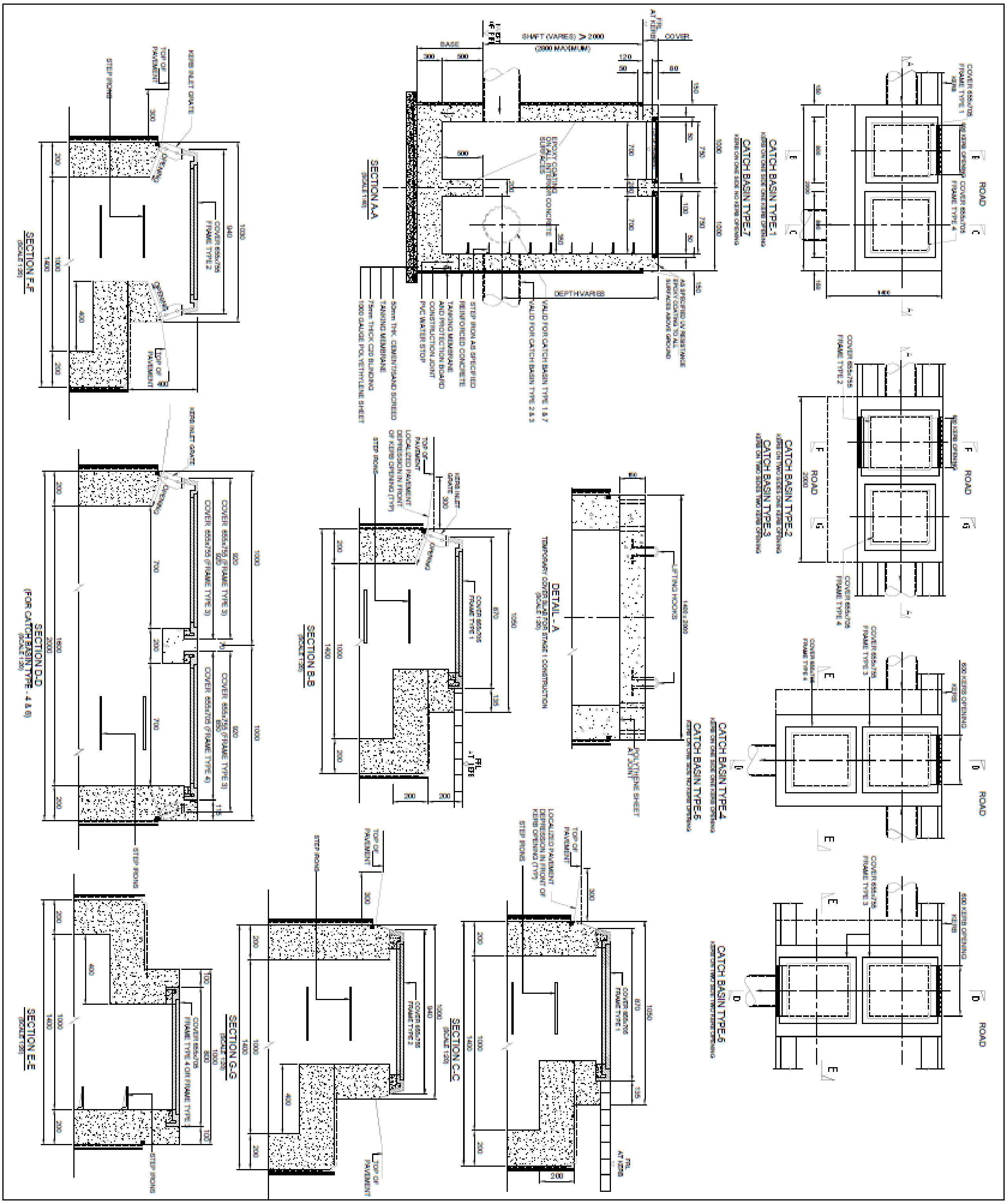


Figure 0-5— Catch Basin with Outlet pipe 500mm and 2.0>D=<2.8 (ADM, 2016)

6.3.1 Inlet Types and Their Applications

Common inlet types are:

- Grate inlets
- Curb-opening inlets
- Combination inlets

Perspective sketches of these types of gullies are presented in **Figure 0-6**.

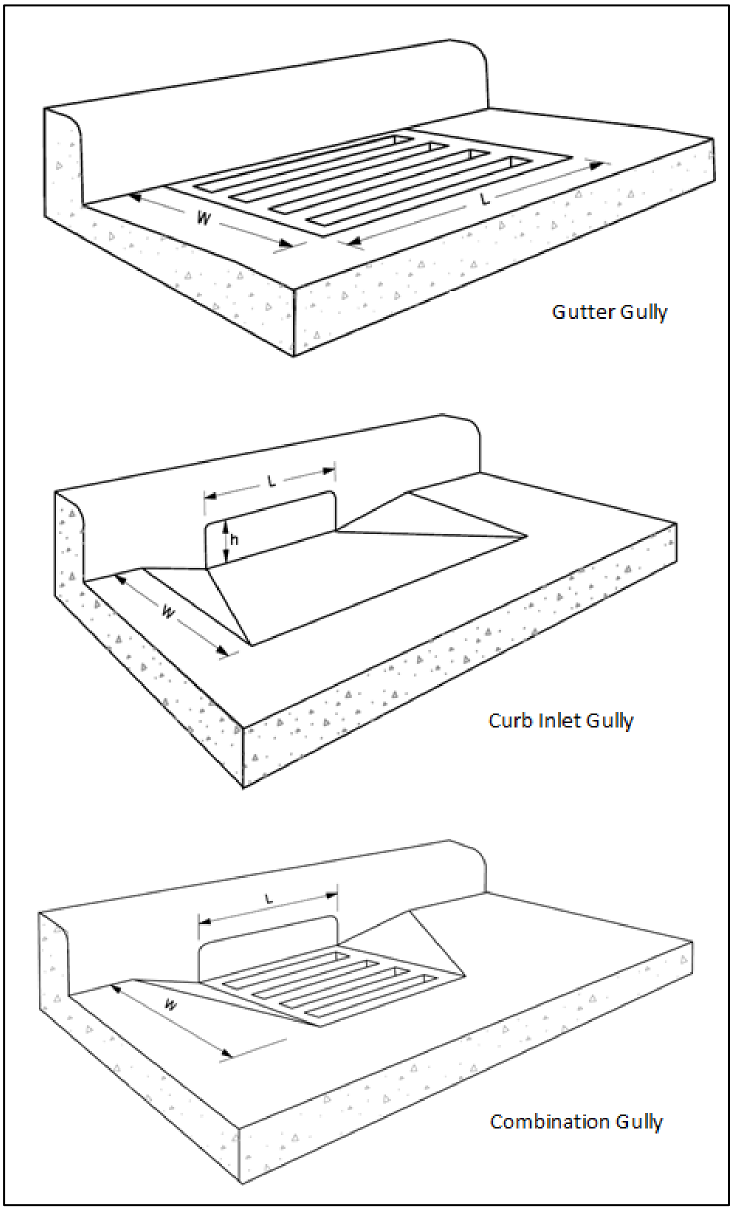


Figure 0-6— Perspective Sketches of Different Types of Gullies (ADM, 1998)

Gutter inlets consist of a metal grate placed over an opening in the gutter. Curb-opening inlets are openings in the curb face which are generally placed in a depressed gutter section. A combination of these two types of inlets may also be used, that is called a Combination Inlet.

Grate inlets perform satisfactorily over a wide range of gutter grades. Grate inlets generally lose capacity with increase in grade, but to a lesser degree than curb-opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing.

Their principal disadvantage is that they may be clogged by floating trash or debris. For safety reasons, preference shall be given to grate inlets where out-of-control vehicles might be involved. Additionally, where bicycle traffic occurs, grates shall be bicycle safe.

Curb-opening inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade steepens. Consequently, the use of curb-opening inlets is recommended in sags and on grades less than 3%.

The advantages of curb-opening inlets are that they are less susceptible to clogging and less hazardous to pedestrians, motorcycles and bicycles than the grated gutter gullies.

Combination inlets provide the advantages of both curb opening and grate inlets. This combination results in a high capacity inlet which offers the advantages of both grate and curb-opening inlets. When the curb opening precedes the grate in a "Sweeper" configuration, the curb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a curb opening on both sides of the grate.

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.

Design of gullies and their spacing shall be in accordance with FHWA – NHI – 10 – 009 Hydraulic Engineering Circular No. 22 (HEC 22) third edition and Sewer Systems Outside Buildings and HA102/00 Spacing of Road Gullies (HA, 2000).

6.3.2 Inlet Locations

The location of inlets is determined by geometric controls which require inlets at specific locations, the use and location of flanking inlets in sag vertical curves, and the criterion of spread on the pavement. In order to adequately design the location of the inlets for a given project, the following information is needed:

- Layout or plan sheet suitable for outlining drainage areas
- Road profiles
- Typical cross sections
- Grading cross sections
- Super-elevation diagrams
- Contour maps

Gully locations shall be generally in accordance with the following:

- Location of gullies shall be determined to minimise flooding of roadways and sidewalks and to limit the spread of flow to the values mentioned in **Section 6.2**. Additional gullies shall be placed at sag/low points and entrances to underpasses, hospitals and other strategic locations.
- Maximum length from gully to manhole shall be decided based on the design storm frequency and duration. The performance of gully shall be checked against next storm event to check the performance of the inlets. During the calculation, the efficiency shall be used as 50% - 60%.

There are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations shall be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations follow.

- At all low points in the gutter grade
- Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street intersections, i.e., at any location where water could flow onto the travel way
- Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks)

- Immediately downstream of bridges (to intercept bridge deck drainage)
- Immediately upgrade of cross slope reversals
- Immediately upgrade from pedestrian cross walks
- At the end of channels in cut sections
- On side streets immediately upgrade from intersections
- Behind curbs, shoulders or sidewalks to drain low area

In addition to the areas identified above, runoff from areas draining towards the highway pavement shall be intercepted by roadside channels or inlets before it reaches the roadway. This applies to drainage from cut slopes, side streets, and other areas alongside the pavement. Curbed pavement sections and pavement drainage inlets are inefficient means for handling extraneous drainage.

6.4 Drainage Pipes

6.4.1 Pipe Depths and Covers

A minimum cover of 1.2 m above the crown of the drainage pipes shall be maintained. This is to provide protection from external loads. Pipes with a cover of less than 1.2 m shall be protected with concrete.

Maximum depths to invert shall be determined on the basis of maintaining a cost-effective and safe design. The recommended maximum cover for drainage system components such as pipes and manholes is 10 m. If depths greater than this cannot be avoided for a particular reason, sufficient justification shall be provided and additional provisions be considered to protect the pipe, manholes, etc. from soil loads.

6.4.2 Pipe Materials and Sizes

Normally, all possible pipe materials with regards to the local environment of the site shall be considered. The durability of a drainage facility depends on the characteristics of soil and water.

These characteristics may vary from site to site. It is not cost-effective to declare a rule of thumb that the storm drainage system shall be of one material exclusive of all others.

Hence, the choice of material shall be based on careful consideration of durability, hydraulic operation, structural requirements, and availability.

To follow the SuDs principles, according to “BS EN 752:2008 Drain and Sewer Systems Outside Buildings”, such materials shall be used that minimise the depletion of finite resources, can be operated with the minimum practicable use of energy and can be constructed, operated, and at the end of their life, decommissioned with the minimum practicable impact on the environment.

Table 0-2 presents the preferred drainage pipe material, taking into account the environmental and economic aspects. Alternative pipe materials, e.g. Ductile Iron (DI) for pumping mains may be used subject to prior approval by DM.

Table 0-2— Preferred Drainage Pipe Material

Pipe Type	Diameter	Material
Gravity (including slotted pipes)	< 315 mm	uPVC, HDPE
	≥ 315 mm	GRP, RCP (for culverts)
Pumping mains	All	GRP, HDPE
Within chambers	All	GRP

The minimum permissible pipe size for storm water drainage gravity lines is 250 mm. Minimum size of land drainage pipes is 160 mm, however slotted carrier pipes, serving both as land drain and carrier drain must meet the 250 mm minimum requirement. The minimum permissible size for rising mains is 200 mm.

Below are some other design considerations for gravity pipes:

- The minimum pipe class shall be Class 2.

- Saltwater cover pipes or Fibre-Reinforced Concrete (FRC) pipes shall be used in tidal areas, i.e. where the invert is below the design TWL.
- Pipes shall generally be flush jointed, bandaged in an approved manner, in accordance with the pipe manufacture's specifications.
- In unstable ground where pipe movement is possible, the pipes shall be rubber ringed spigot and socket jointed or be laid in and backfilled with cement stabilised material.

6.4.3 Pipe Corridors

Pipe corridors shall be determined in accordance to the latest RTA RoWs and in consultation with all utility providers, e.g. DEWA, Etisalat, Du, etc.

6.4.4 Pipe Clearance

The minimum depth for pipes shall be 1.2 m to the crown of the pipe. This is required to provide pipe protection from external loads and to avoid interference with other utilities. If circumstances require installation of a pipe with depth less than 1.2 m above the crown, then concrete protection is required.

The required minimum covers from the finished ground level to the top of pipe shall be in accordance with **Table 0-3** below.

Table 0-3— Minimum Cover and Vertical Clearance for Pipes

Type of crossing	Minimum cover/Vertical clearance (m)
Without protection	1.2
With protection	0.5
Road crossing by non-destructive methods	2.5
Under exiting utilities (Vertical clearance)	

Water Pipeline	0.5 (for open cut)
Electricity, Telecommunication etc.	0.3
Oil and Gas	As per Dubai Supply Authority (DUSUP) requirements

A proper design check is required for the pipe at shallow depth beneath the major roads or highways.

Minimum horizontal clearance of 3 m is required. If utilities are in the same trench, the other utility shall be placed on a separate bench on un-disturbed soil.

These are minimum requirements. However, the exact required clearance shall be discussed in detail and confirmed with all utility providers.

6.4.5 Air Valves and Washouts

Air valves and washouts shall be included on all rising mains as necessary to improve performance and enhance access for maintenance. In general, such devices shall be considered at the following locations:

- Air valves: high points and as necessary based on surge analysis.
- Washouts: low points and as necessary for access and dewatering. Note: *Normal practice, the washout is required also upstream and downstream of any crossing of main road.*
- Access chambers: when the length of rising main is greater than 500 m between the air valves and/or washouts.

Air valves installed on rising mains shall be double orifice type (unless surge control considerations dictate otherwise) and shall be suitable for solid-bearing liquid. Air valve size shall be determined as per manufacturer's data sheets in reference to the pipe diameter. Air valves are mainly used to ventilate the system. However, they can be used for surge suppression as a secondary method especially for the long rising main and hydraulically complicated system.

The following considerations shall also be taken into account when designing and locating air valves:

- Air valves can fail to function correctly if there is a lack of seating pressure. This can occur when the valve location closely approaches the hydraulic gradient, their location shall therefore be checked against these criteria.
- All air valves shall be located so as to permit ease of access and maintenance.

Each device shall be located within a special chamber to ensure easy access. Connections to the pipes shall be included for flushing the lines and/or for dewatering.

Air valves and washouts shall be provided with a separate isolation gate valve, with bevel gearing, to enable removal of the valve without shutting off the main.

6.5 Drainage Manholes

Manholes shall be placed wherever necessary for clean-out and inspection purposes. It is good engineering practice to place manholes at changes in direction, junctions of pipe runs and intervals in long pipe runs where the size or direction may not have changed. It shall be aligned with the gully locations. All connections shall be made to manholes. No intermediate or in-line connections are permitted.

The recommended maximum spacing between manholes is presented in **Table 0-4**.

Table 0-4—Maximum Spacing between Manholes

Pipe diameter (mm)	Maximum spacing (m)
D ≤ OD 315 mm	100
From 350mm to 500mm	150

Pipe diameter (mm)	Maximum spacing (m)
From 600mm to 800mm	200
From 900mm to 1000mm	250
From 1100mm to 1300mm	300
From 1400mm to 1500mm	400
Pipe size > 1500 mm	600

Any alteration in the above specified spacing of manholes, consultant has to obtain pre-approval from DM.

Manhole shall be of sufficient size to permit access for maintenance activities. In addition, their design and material selection shall be such that to guarantee maximum performance for an extended service life.

Benching and channels in manholes shall be formed to permit safe access and to maximise hydraulic efficiency through the manhole. The correct manhole sizes shall be used in hydraulic model. For size of the manholes, consultant can refer to manhole drawings and specifications.

Table 0-5— Manhole Cover Levels (ADM, 2016)

Location	Cover Level
Paved areas	Final Paved level
Landscaped areas	Final Ground Level +0.1m
Open, unpaved areas	Final Ground Level +0.25m

6.6 Sustainable Urban Drainage System

6.6.1 Modern Plastic Soakaways

The following criteria shall be considered for sizing of Modern Plastic Soakaways:

Table 0-6— Design Criteria for Sizing of Modern Plastic Soakaways

Parameter	Design Criteria
Design with overflow arrangement ARI	10-year
Design without overflow arrangement ARI	25-year
Emptying time at low level	5 days
Emptying time at high level	2-3 weeks

It shall be noted that the emptying time for the higher level is approximate. Actual emptying time may vary depending on the infiltration rate or outlet sizing based on the lower level emptying time.

6.6.2 Swales/Depressed Landscapes

The following criteria shall be considered for sizing of Swales:

Table 0-7— Design Criteria for Sizing of Swales

Parameter	Design Criteria
Design with overflow arrangement ARI	10-year
Design without overflow arrangement ARI	25-year
Swales emptying time at low level	5 days
Swales emptying time at high level	2-3 weeks

The check dam shall be provided to divide the swales in different portions that will help to utilize the available volume of swales more effectively. Furthermore, overflow chamber or outfall arrangement shall be provided to connect the proposed swales with network that shall optimize the network pipe sizes and reduce the cost. Example is presented in **Figure 0-7.**

Figure 0-7— Swales with Check Dam and Overflow Arrangement (Blankenship, n.d.)

6.6.3 Detention and Retention Ponds

The following criteria shall be considered for sizing of detention/retention ponds (**Table 0-8**):

Table 0-8— Design Criteria for Sizing of Detention/Retention Ponds

Parameter	Design Criteria
Design ARI	50-year
Control ARI	100-year

Parameter	Design Criteria
Pond emptying time at low level	5 days
Pond emptying time at high level	2-3 weeks

It shall be noted that the emptying time for the upper level is approximate. Actual emptying time may vary depending on the outlet sizing based on the lower level emptying time. All ponds must meet this criteria for a given storm event, therefore upstream ponds must be cleared more quickly.

6.7 Pumping and Lifting Stations

6.7.1 General Design Considerations

The design philosophy for the Dubai Drainage Master Plan includes minimisation of the total number of the drainage pumping stations. Where pumping is required, the number of times a given flow is pumped should also be minimised. This philosophy has been adopted to reduce operation and maintenance associated with pumping stations.

The following sections provide design guidelines for drainage pumping stations.

6.7.2 Sizing of Pumping Stations

Drainage pumping stations must be designed to handle runoff flows based on the appropriate design storms.

All drainage pumping stations shall be designed to handle the estimated runoff from their respective catchments. In some cases, a pumping station may need additional capacity to ensure that clearing times for upstream detention ponds are met. This shall be investigated using the hydraulic model of the system.

6.7.3 Screens

Consideration to screening facility shall be given depending on the size of the pumping station and in the case of presence of debris and large solids.

6.7.4 Wet Well Configuration and Sizing

Wet wells are usually a below-grade structure (above grade is possible, but not typical) of a pumping station. It is the structure into which the liquid flows from, and where the pumps draw water. Wet wells serve the following purposes:

- Create a hydraulic break minimising the effects of the upstream system. The free water surface is allowed to rise and fall buffering the system from any fluctuations in flow and pressure.
- Provide storage volume to allow constant speed pumps to start and stop without exceeding the number of starts required for a certain size motor.
- Provide adequate submergence above the suction bell of pump to prevent formation of vortices and adequate Net Positive Suction Head (NPSH).
- Provide free-board to allow the water level to rise during upset or emergency operation without overflowing.

The wet well design shall meet the flow distribution based on the accepted criteria recommended by the American National Standards Institute - Hydraulic Institute (ANSI-HI) in the "*Pump Intake Design*" standard. These recommendations mitigate adverse hydraulic phenomenon that may occur in the pump station wet well. In summary, the geometry of the wet well, operation of the pumps, and the depth of water in the sump influence the approach flow hydrodynamics and can result in adverse hydraulic phenomena.

The wet well volume shall be designed with adequate storage to prevent frequent starting and stopping (cycling) of the pump. The maximum number of allowable starts is typically dependent on the characteristics of the electric motors and typically ranges between 6 for large motors and 15 for small motors. The design engineer is responsible for contacting the pump/motor manufacturer to obtain the minimum cycle time. Furthermore, the wet well shall be sized to allow for the pump starting sequence. The wet well shall be sized to provide adequate storage during this time period. The below described criteria for sizing the wet well shall be applicable for constant speed pump that operate in ON/OFF mode. In case of using Variable Speed Pump in which the pump speed will be adjusted to match the incoming flow rate, the wet well volume shall be large enough to keep the currents sufficiently low.

Initial sizing of wet wells for a single pump or a single-speed control step can be undertaken using the following equation:

$$\boldsymbol{V} = \frac{\boldsymbol{t\,q}}{\boldsymbol{4}} \qquad \textbf{(Equation 25)}$$

where:

V = required capacity (m^3)

t = minimum time in minutes of one pumping cycle (time between starts

q = pump capacity (m^3/min)

For multiple-speed pumps, the available storage volume in the wet well does not need to be as conservative. As flow rate is controlled by the speed of the pump, the pump does not need to start against a closed valve. The pumps can start, and increase speed to immediately contribute flow into the system.

One design criteria often overlooked is the storage volume required in the event of a power outage. With a constant flow rate entering the pump station wet well, a disruption in power supply will immediately be reflected with a rise in the water surface elevation. In this case, it is impossible to provide storage for an extended power outage. Therefore, the SCADA system shall be configured such that in the event of power failure in a downstream pump station, the upstream pump station shall be signalled to stop. In collection system applications, the flow can be allowed to back-up into the system, otherwise the wet well shall be designed with adequate storage volume or overflow potential during a power outage. The design engineer shall liaise with DEWA and DM regarding the design power outage duration.

If adequate volume cannot be provided to prevent short-cycling of the pump, multiple pumps or variable-speed pumps shall be considered to reduce the required volume.

When designing a wet well, the following items shall be considered:

- Provide an opening in the deck with adequate clearance to allow removal of any pump components or piping from the wet well.
- The wet well shall be provided with an air vent sized to release or admit outside air due to the rise and fall in water levels. Area of vent is typically equal to at least half of the inlet pipes area.
- Provide a grate (hatchway) for access to the wet well. Grate size to be at least 1.2 m by 1.2 m with appropriately sized safety net or equivalent safety system.

6.7.5 Pump Types

For small to medium size pump station, the preferable type of pumps are the Submersible close-coupled pumps driven by a submersible motor and are generally a vertical installed type. For large size pump station, a centrifugal (non-clog) pump with a horizontal or vertical shaft can be used. The unit is either frame-mounted or close coupled with the motor on the floor of a dry chamber (dry well). Pumps used at the Ponds or lakes that handling screened storm water, a submersible centrifugal pump installed inside a column pipe can be used.

6.7.6 Pump Selection Criteria

Typically, storm water drainage pumping stations are not considered to go through the multiple start-stop cycles experienced in sewage pumping stations. However, when the drainage pumping station is also used for dewatering, the multiple start-stop cycle may become an issue.

Pump selection shall be made in conjunction with the pipe size (system curve) determination in order to optimise conditions over the anticipated range of flows and should consider both storm water runoff and groundwater flows. Pumps shall be also selected so that they will able to handle wide range of inflows; the dry weather flow and wet weather flow efficiently. Pump selection should consider maximising pumping efficiency and meeting the clearing time requirements.

Actual pump selection can only be made once a system curve is developed. The following items shall be considered:

- Required range of head and flows
- Number of pumps (1 duty and 1 standby as a minimum requirement)
- Operation and control philosophy
- Efficiency
- Potential for upgrading capacity; to accept runoff from adjacent catchments on a long or short-term basis
- Flexibility in the system; in case design criteria such as design storm or clearing times become more restrictive in the future

Where possible, the selected pumps shall be in the mid-range of the available impeller sizes so that simple changes/rehabilitation can be made to improve pumping station capacity.

Multiple size pumps may be appropriate to meet groundwater, dry weather flow and wet weather pumping requirements.

6.7.7 Pumping Station Structures

Pumping station structures shall be designed to ensure a safe environment for operation and maintenance staff as well as maximising performance and minimising costs. The following items shall be considered:

- Depend on the type of the selected pump the structure of the pump station shall be either with only wet well or dry well and wet well.

- Wet wells should include provisions for appropriate ventilation prior to entry by trained personnel.
- Provisions shall be made to facilitate removing pumps, motors and other mechanical and electrical equipment.
- Suitable and safe means of access must be provided to dry and wet wells.
- Due consideration shall be given to the selection of materials because of the presence of aggressive groundwater flows, greases, oils and other constituents frequently present in the drainage system.
- Wet wells shall be configured to minimise turbulence, especially near the intake of the pumps.
- Wet well controls are typically of the encapsulated float-type; however more sophisticated control may be considered. In all cases, control sensors shall be located away from the turbulence of incoming flow and pump suction.

6.7.8 Corrosion and Erosion Considerations

Corrosion and erosion are a fluid characteristic with no effect on the hydraulics, but if not considered may be detrimental to the life of pumps, valves and piping. The effects of corrosion and erosion should always be considered when dealing with fluids other than potable water.

Corrosion is an undesirable degradation of material resulting from a chemical or physical reaction with the environment. Erosion is the deterioration of metals buffeted by the entrained solids in a corrosive medium. The corrosive or erosive potential of a service would dictate the materials of construction, hardness and ductility of material and special liners such as rubber or ceramic and cathodic protection are required.

Figure 0-8 and **Figure 0-9** show examples of corrosion and erosion on pump impellers.

Figure 0-8— Erosion due to Cavitation (Mahdi, n.d.)

Figure 0-9— Erosion on Pump Impeller (PES Solutions, 2015)

When designing a pump station with a fluid containing corrosive constituents, water known to be corrosive, or fluids other than water, a sample must be taken and tested. Results shall be reviewed by the Corrosion Engineer and the pump manufacturer for proper material selection of pump components.

6.7.9 Electrical and Instrumentation System Requirements

To enhance the operability of the pumping stations, the following provisions shall be included in the design:

- Supply and control circuits should allow for disconnection from outside of the wet well.
- Terminals and connectors shall be protected from corrosion through proper location and/or the use of water-tight seals. Separate strain relief is required.
- Motor control panels shall be properly sealed.
- Power cords shall be designed for flexibility and serviceability under conditions of extra hard usage. Field connections should also be facilitated.
- Ground fault interruption protection shall be used.

Regulations DEWA, Dubai Civil Defence and Etisalat/Du shall be considered in the design of electrical and instrumentations systems.

As mentioned in **Section 6.7.3**, a SCADA system is required to control the water levels in the wet well. The SCADA system should include flow metering, flow controls and level control as well.

Uninterruptible Power Supply (UPS) shall be considered to support the SCADA system and maintain its operation. UPSs require special provisions in location, ventilation, maintenance, and interconnection to building and other electrical power and equipment systems. The sizes and locations must be provided in the design.

The electrical system and equipment shall be designed to permit inspection and maintenance of individual items.

A single manufacturer shall be specified whenever possible, this is to overcome the issue of compatibility of diverse components of instrument and control system and other factors such as cost, required operator skill level, and owner preference.

6.7.10 Safety Aspects of Pumping Stations

A list of minimum safety requirements is provided below.

- Address the Confined Space and safety requirements as per international best practice, e.g. the UK Health and Safety Executive (HSE) guidelines.
- The pump station structure shall be designed incorporating the Dubai Civil Defence requirements for firefighting, lighting, access and exit.
- Safety Guards around rotating equipment shall be provided.
- Ventilation shall be provided.
- Requirement for protective clothing, gloves, boots and goggles for the operation and maintenance staff shall be specified in the Operation and Maintenance (O&M) manuals.
- Other safety equipment shall be specified and shown on the drawings where required. DM and ministry of Labour requirements shall be considered.

6.8 Outfalls

The design of outfalls is specialised and site-specific, so this section only provides some general aspects of outfall design.

Outfalls may discharge storm water runoff to the Creek, the sea or a detention/retention pond. Outfall locations near public beaches or hotels shall be avoided. Outfalls should also not be located in areas with environmental values, such as wildlife areas.

The invert level of the outfall shall be above the peak design water level of the receiving water body so as to provide free discharge conditions. Where periodic back-flooding cannot be avoided, a non-return valve shall be considered.

Outfalls shall be formed so as to avoid, or provide protection against, local erosion. It may be necessary to provide additional protection to the outfall opening to prevent damage, interference or entry. The visual impact of the outfall shall also be taken into account.

Tail water considerations of the outfalls shall be in accordance with the details described in **Section 5.7**.

SITE & ROAD GRADING

7.1 Background:

Site grading is an important part of the Land Development process and storm water system.

Most sets of Site Plan documents are multi sheet plan sets that will include among other plans, a Grading and Drainage plan. If the scale of the project is large enough, those two sub-disciplines may be broken into separate sheets with a standalone Grading Plan and a separate yet related Drainage Plan. This is typically divided at the discretion of the engineer.

In the case of separate grading and drainage design plans, the Drainage Plan will typically identify the information (rim/grate elevations, invert elevations (bottom of pipe)) associated with the inlets (catch basins), storm manholes, flared outlet structures, permanent erosion protection, and outlet control structures.

The Grading Plan defines the information that the proposed surface will exhibit post construction.

7.2 Existing Conditions:

When approaching the grading of a site, the engineer must first be provided with a document that defines the existing conditions, including the topography. This document is typically referred to as a Topographic Survey or “survey”. If the survey is received with adequate information for the purposes of re-grading a site by design, it will identify the existing surface features, spot elevations or “spot grades”, and contour lines. There may also be outfall elevation information surrounding the site so the drainage system can be connected if and as needed.

7.3 Contour Lines:

Contour lines identify all of the areas on the site that are at a certain elevation and how they connect to each other. The contour lines can be reflected at various intervals. The ideal contour interval is 1’. At a minimum, 2’ contours provided on the survey can be adequate, but the design should typically be in 1' intervals at a minimum. Contour lines as shown on surveys and proposed grading plans are frequently shown distinctly and referred to as "major" and "minor" contour lines, meaning that the 0.5m contours are shown differently than the 5' interval contours. Existing contour lines are often shown as "dashed" or "hidden" line types, while proposed contour lines are typically shown as solid line types. (Refer to **Figure 0-1**)

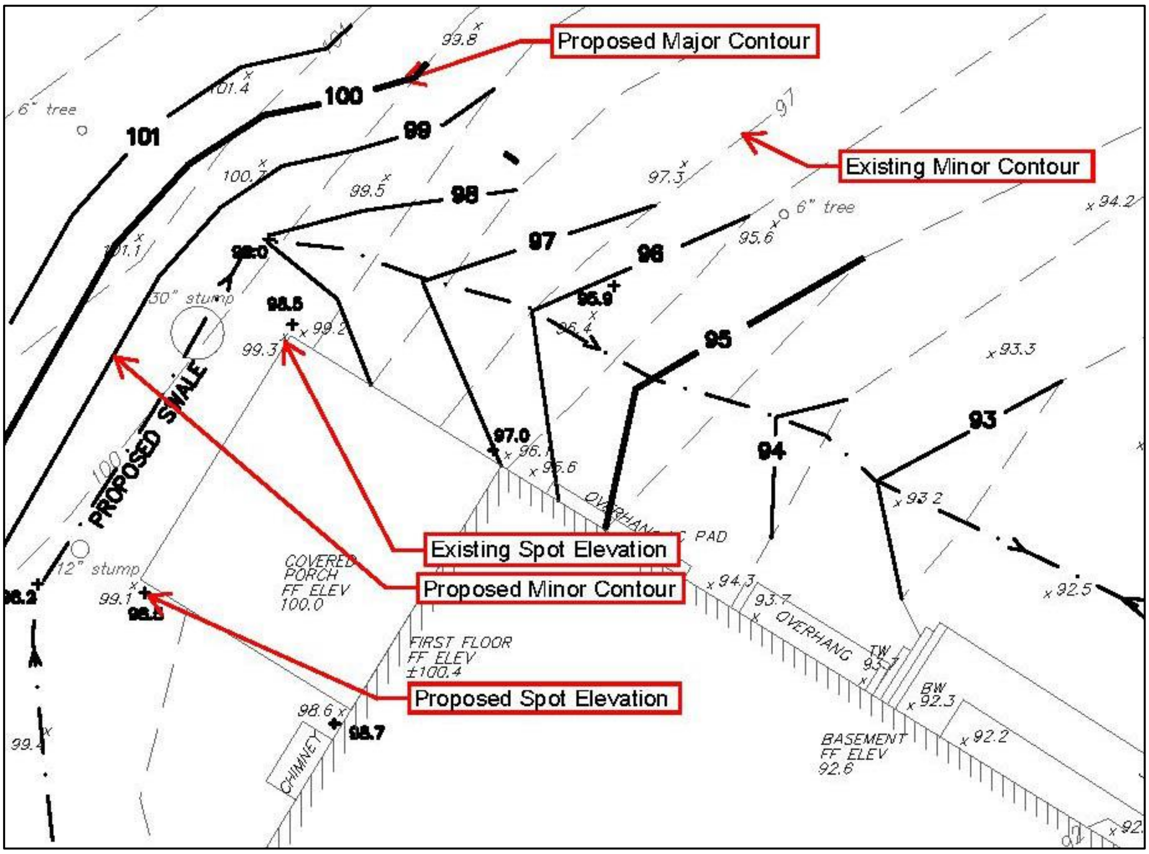


Figure 0-1 – Contours and Spot Grades (Tiner, 2014)

7.3.1 Spot Elevations / Spot Grades:

Spot grades identify the exact elevation of a point on the surface, or of a level flat area located upon the surface. Typically spot grades are taken to and shown as to the nearest tenth (0.1') or hundredth (0.01') of a foot (ft.) similar approach can be opted for the metric system. There are several ways for this information to be reflected. The precise location of the existing spot grades are typically identified with an "x" with the elevation identified in numerical form adjacent. The location that the engineer wants the proposed spot grade to be might be noted with a line from the noted numerical elevation, or perhaps with a "+" mark next to the elevation. (Refer to **Figure 0-1**)

7.3.2 Other Standard Annotations:

Spot grades at curbs and retaining walls are typically noted with two elevations, one associated with the top of the curb/wall, and one at the base. This will typically be shown with a CL: XXX.XX and BC: XXX.XX respectively.

7.3.3 Slope:

The mathematical definition of slope is expressed in below equation:

$$S = \frac{Change\ in\ Elevation\ (m)}{Change\ in\ Distance\ (m)}$$

Slope is often represented in terms of a percentage, in which case S x 100 = S%.

Steeper slopes are also frequently identified in terms of the ratio of horizontal to vertical. For example a 3:1 slope indicates that the every rise of 1' is separated by a distance of 3'. This is noted in Horizontal to Vertical, or H:V.

7.3.4 Plan Setup:

The most appropriate way to start site grading is to overlay the proposed site plan layout (also referred to as a "Horizontal Control Plan") on top of the survey. This is typically done with electronic conversion (i.e. in AutoCAD or Civil 3D) of the survey to the engineering team's graphical standards, and overlaying an X. ref of the conceptual proposed plans in top of it. Afterwards, the engineer is able to start the process of working through the site grading process.

7.3.5 Limit of Disturbance / Transition between Existing and New Grades

Although it is not typically graphically required to show the Limit of Disturbance (LOD) on grading plans, the LOD is often times the point on the proposed grading plan where the proposed contour lines will intersect with the existing contour lines. Sometimes the surface is disturbed and the grading does not change. In these cases, the LOD goes beyond the proposed work. Often times the engineer will trace the existing contour lines with new contour lines just to show a disturbance has taken place. Either graphical method is acceptable. The bottom line is, the points at which the engineer wants the existing grade to meet the proposed grade must have the two contour lines (existing and proposed) meeting/intersecting on the proposed grading plan.

7.3.6 The Inverse Slope/Contour Calculation Method

Once the engineer begins to become comfortable with slope calculations and how to graphically reflect contours, it becomes apparent, fairly quickly, that there is a simple way to calculate the distance between contour lines (assuming non variable slope). By taking the inverse of the slope (in non-variable sections of slope), the engineer is provided with the distance between the minor 1' contour lines. This works itself out as follows, say for example the engineer wants to show the contour lines of a parking field that is sloped at 2%, and they can quickly calculate 1 / 0.02 = 50. So they know to place the contour lines apart by 50'. In a similar fashion, if the engineer wants to show a 3:1 slope, they just show each contour separated by 3' since the 3' horizontal corresponds to the 1' vertical rise.

As will be noted below, there are maximum and minimum slopes that the engineer needs to be aware of. Therefore they can also quickly check slope by looking at the distance between the contour lines and taking the inverse. For example, if on a parking lot design, the engineer separated the 67 and 68 contour lines by 120', the reviewing engineer could measure this distance with a scale and quickly determine that the slope of the parking lot was designed at 0.83%. This is calculated as follows: 1/120 = 0.0833 x 100 = 0.83%

The inverse slope/contour calculation method is a useful tool for the grading engineer to understand.

7.4 Design Parameters and Other Limitations

7.4.1 Design Parameters

The engineer may have developed their own design standards based on their professional experience and practice, and/or they may refer to international best principals to determine the various parameters that they will utilize in design. After determining the specific design parameters required, the engineer is ready to start grading the site. The engineer must determine the existing elevations at the interfaces between the existing work and the proposed work, such as the proposed driveway entrance to the existing road.

7.4.2 Positive Drainage

The primary goal of any engineers grading design is to make sure that storm water flows off of the site in a safe efficient manner. As noted above, grading goes hand in hand with drainage, and the engineer goal must be to avoid standing water ("standing water" by definition refers to water that has no place to go, and is therefore only dissipated by evaporation). As a result the primary design parameter of all grading designs is to maintain positive drainage ("positive drainage" by definition means that the water always has an ability to flow away from where it is).

If the engineer finds that they cannot avoid a low spot that will cause standing water, they must put some sort of drainage structure into the design so that the water can flow away via the subsurface pipe network designed associated with the drainage design.

7.4.3 Rules of Thumb

There are several reasons for some of the basic rule of thumb parameters that engineer's generally follow. The following will describe some of those parameters and some of the reasons they exist:

7.4.3.1 Maximum Access Drive Slope: 8%

It is a good practice to keep the slope of the main access drive less steep than 8%. This is typically driven by the goal of not causing cargo being transported by truck traffic to shift excessively.

7.4.3.2 Maximum Parking Lot Slope: 5%

It is good practice to keep the slope within the parking field of any large commercial or retail parking lot flatter than 5%. There are at least two (2) significant reasons for this parameter:

1. A slope much steeper than 5% increases the frequency of “runaway” shopping cart left by shoppers. This can cause damage to vehicles as well as the shopping carts.

2. A slope steeper that 5% had traditionally began to create difficulty in keeping the vehicle door open when parked perpendicular to the slope with the door on the high side. Additionally it begins to be more difficult when parked perpendicular to a 5% slope for a passenger to close the vehicle door when the door is opened on the low side of the vehicle.

Many developers have their own criteria that require the parking fields in front of their stores to be even flatter than, as low as 3% in some cases.

7.4.3.3 Maximum Slope in Maintainable Grassed Landscaped Areas 3:1

It is good practice to keep the slope within grassed areas no steeper than 3:1 or 33%. The generally cited reason for this is that this is the maximum “maintainable” slope. The term maintainable refers to the maintenance of the lawn by the landscaper. Landscapers have difficulty remaining stable on riding mowers when cutting the grass while driving parallel with slopes steeper than 3:1. It is not to say that it can’t be done, but ideally the best practice is to accommodate maintenance. Additionally, mulch beds placed on 3:1 slopes do not typically “erode” from storms of relatively strong intensity.

7.4.3.4 Maximum Slope in Stabilized Landscaped Areas 2:1

If the engineer needs to achieve slightly steeper slopes in landscaped areas, but also want to minimize cost, the maximum slope best practice is 2:1. As noted above, this exceeds the maximum “maintainable” slope. As a result, the engineer must stabilize the slope with vegetation that anchors the top soil, yet requires minimal maintenance in terms of needing to be cut by the landscaper.

7.4.3.5 Slopes exceeding 2:1

Slopes exceeding 2:1 should be stabilized with materials that do not erode, such as rip rap (stone erosion control), gabion baskets, paved or concrete finished slopes, or other kinds of retaining walls. Refer to the section below on other grading features for a more in depth discussion on retaining walls.

7.4.3.6 Minimum Slope of Asphalt: 1.5%

When designing the grade of the parking field, if it is an asphalt parking lot, maintaining a 1.5% minimum slope in the parking lot is a good goal. It is reported that it is difficult for any paving contractor to maintain any flat slope consistently over a long distances due to the nature of the installation and compaction process. So while means and methods are not usually the concern of the engineer, it is likely that an asphalt field much flatter than 1.5% will have small puddles and ponds developing throughout the parking field due to undulations that occur. If 1.5% is maintained as a minimum, water is less likely to be caught in the undulations and draining to the designated inlets and/or catch basins is maintained.

7.4.3.7 Minimum Slope of Concrete: 0.75%

When designing the grade of a large concrete area, be it the sidewalk in front of the building, or a large “garden center” area, maintaining a minimum slope of 0.75% should be the intent. The work associated with installing concrete can be much more precise in terms of formwork and how finishing occurs. As a result, the engineer has a little more discretion in terms of allowing these areas to be flatter.

7.4.3.8 Minimum Slope of Concrete Curb: 0.75%

When designing the slope of the curb adjacent to inlets (i.e. curbs that stormwater flow is directed toward), the best practice is for the slope to be no flatter that 0.75%. The absolute minimum of 0.5% may be permitted to occur in extreme circumstances, but just like the reason cited for the absolute minimum slope of concrete slabs and asphalt fields, it is difficult over long distances for the contractor to install the curb and asphalt abutting up to it to this precise slope for the entire length. As a result, regardless of how much care the contractor takes, due to slight undulations and variations in the install, puddling and ponding up against the curb will likely occur. Refer to **Figure 0-2** for a picture of an installation of curb that did not meet the minimum guideline.

It is worthy to note that the curb along the high side of a parking field can be proposed to be installed level if it suits the design. Sometimes new land development / engineers misunderstand this parameter, and try to keep a curb slope of 0.75% on the high side of the lot, which has no practical purpose.

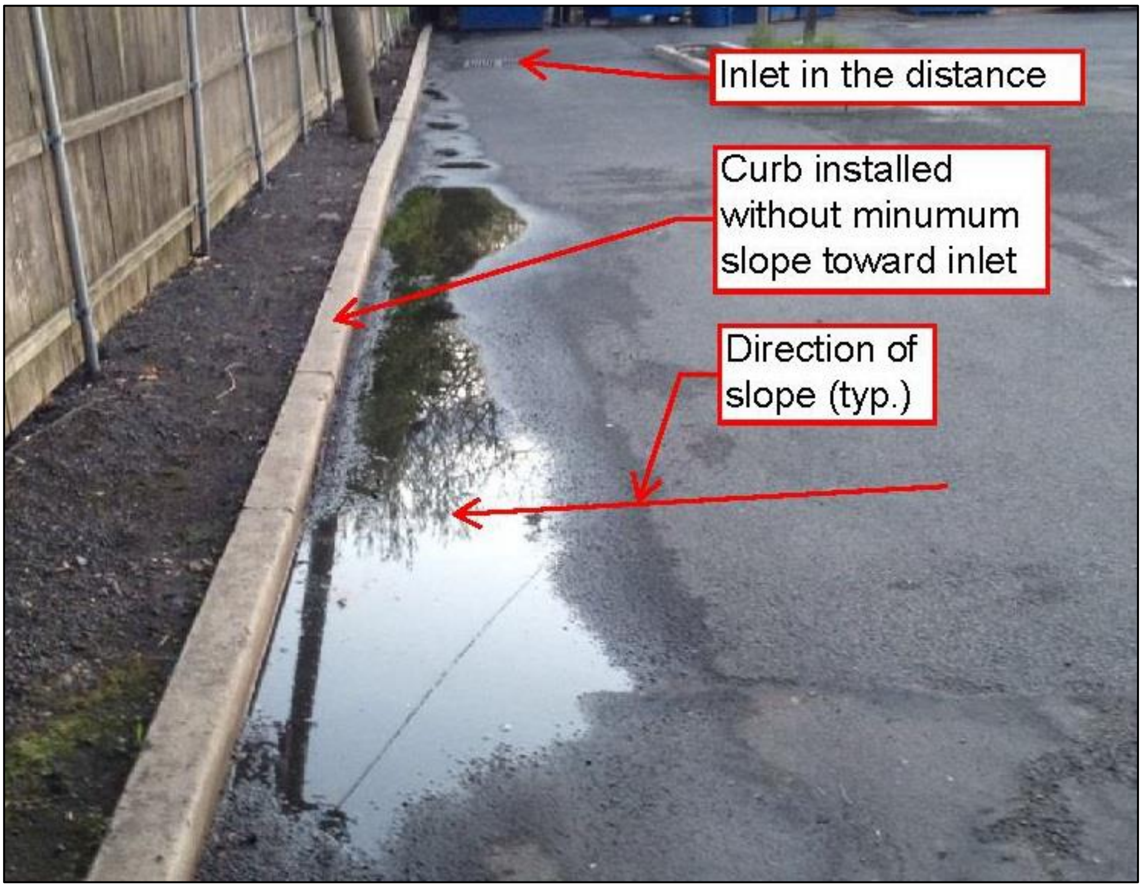


Figure 0-2— Picture of Curb Installed without Minimum Slope (Tiner, 2014)

7.4.3.9 Loading Dock grading: 2.0% for 60’

When designing the grade at a loading dock, the intent should be for the bed of the delivery vehicles to be relatively flat as they meet up with the dock. Since the majority of these vehicles is 60’ or less, it is a good practice to keep that first 60’ at 2%. If the slope is too steep coming into the dock, there have been reported cases of the top of the delivery vehicle colliding with the building above the loading dock door causing damage. Additionally, there should be minimal to no cross pitch in a loading dock so that the dock and the vehicle bed can be flush the whole width of the vehicle. This is why a significant number of loading docks are designed with trench drains at the end of the dock, and not a drain in the corner. Always consult with the owner for standards.

7.4.4 Cut-Fill Analysis

An important parameter that the engineer must consider is referred to as a "Cut-Fill Analysis" or earthwork calculation. It is an important topic to understand because of the cost implications. The proper grading plan will attempt to minimize:

1. The amount of earthwork required on site.

2. The amount of net import or export to or from the site.

These two topics are somewhat related, but the strategy to minimize each is attempting as much as possible to follow and maintain the existing topography of the site. Sometimes the engineer will have to compromise minimizing proposed earthwork in order to reduce the amount of import or export of soil required. This could require a large cut, the soil from which is utilized on another low lying area of the site resulting in a balanced site. Typically setting the finished floor elevation of the proposed buildings on site is done after considering the proposed elevations of a balanced site and then working from there.

On an exceptionally "steep" site, the engineer must look at the maximum slopes between the entrance to the site and the proposed building location, and then also work into the grading strategy a Cut-Fill Analysis to try to cost effectively balance all of the limitations.

7.4.5 Rock Ledge walls

If the geotechnical report indicates that the sub-surface consists of rock, and the geotechnical analysis supports it, the grading design can incorporate rock ledge walls into the design. Rock ledge walls can be proposed virtually vertically as well as at any positive slope the engineer may determine appropriate. In order for these to be installed, costly rock excavation and/or blasting may be required. As a result, the engineer should consider the cost implications of proposing such features rather than trying to solve grading challenges in other less costly ways.

Figure 0-3— Rock Ledge wall

7.5 Other Grading Features

There are several other features that the design engineer will have available to them to accomplish the grading goals on the site. These include, berms, swales, ridgelines, and retaining walls. These features are described in further detail below.

7.5.1 Berms

Berms are "mounds" of soil, or small hills that may be proposed in order to direct flow in a certain direction or around an area for some purpose. Often, berms are merely proposed as a screening feature, or part of a buffer, in order to accommodate the landscaping and create a partial screen. Berms can be a useful place to "lose" excess soil in the engineer's attempt to maintain a balanced site. Refer to **Figure 0-4** for a cross section of a constructed berm, and **Figure 0-5** for some pictures of berms.

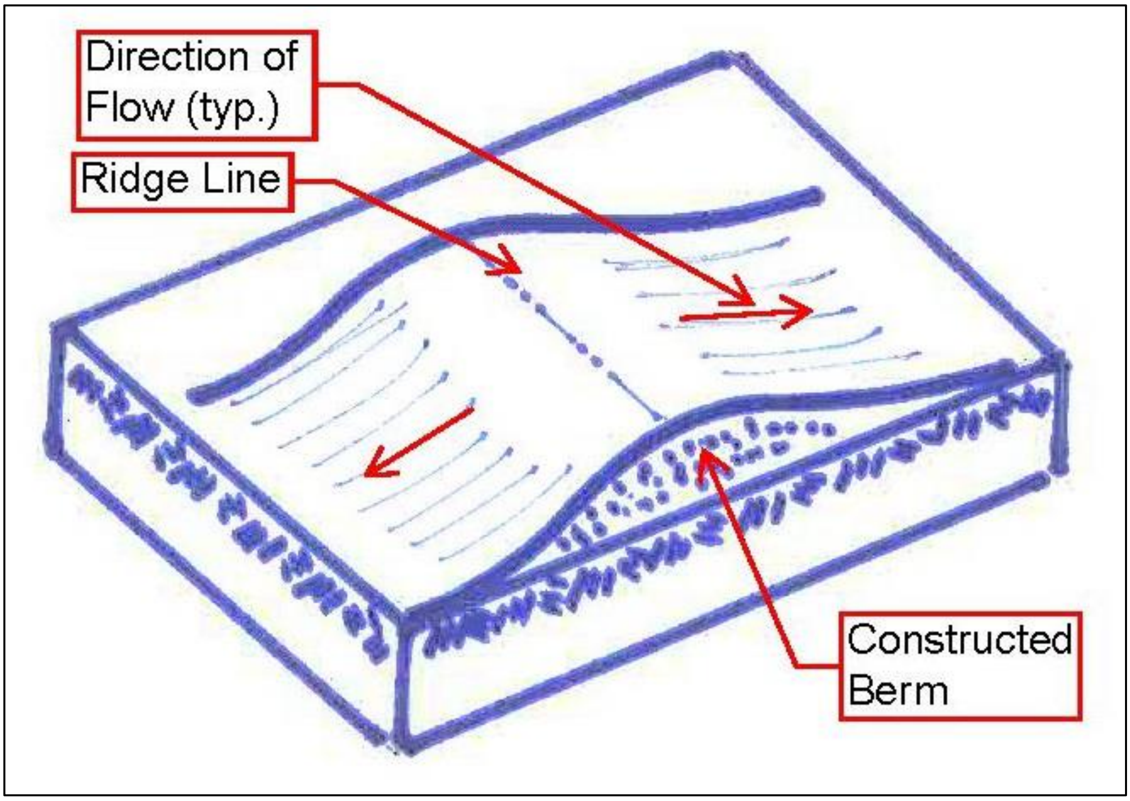


Figure 0-4— Cross Sectional Diagram of a Constructed Berm (Tiner, 2014)

Figure 0-5— Some Pictures of Berms (Dhas, 2020), (Construction Week Middle East, 2019)

7.5.2 Swales

Swales are valleys or linearly defined low lines that are used to convey flow in some direction. Sometimes a swale can be used just to divert a portion of flow, and sometime they can be used in order to save on another inlet and the associated branch pipe. If the required slope in the valley of a swale becomes too steep, additional stabilization measures may be appropriate. If the area of surface flow contributing to a swale is large enough, the engineer may need to define the swale as a channel, in which case, channel flow calculations and stabilization details are described in DM Design Guidelines that will be applicable to incorporate into the design. Refer to **Figure 0-6** for a cross section of a swale, and **Figure 0-7** for some pictures of swales.

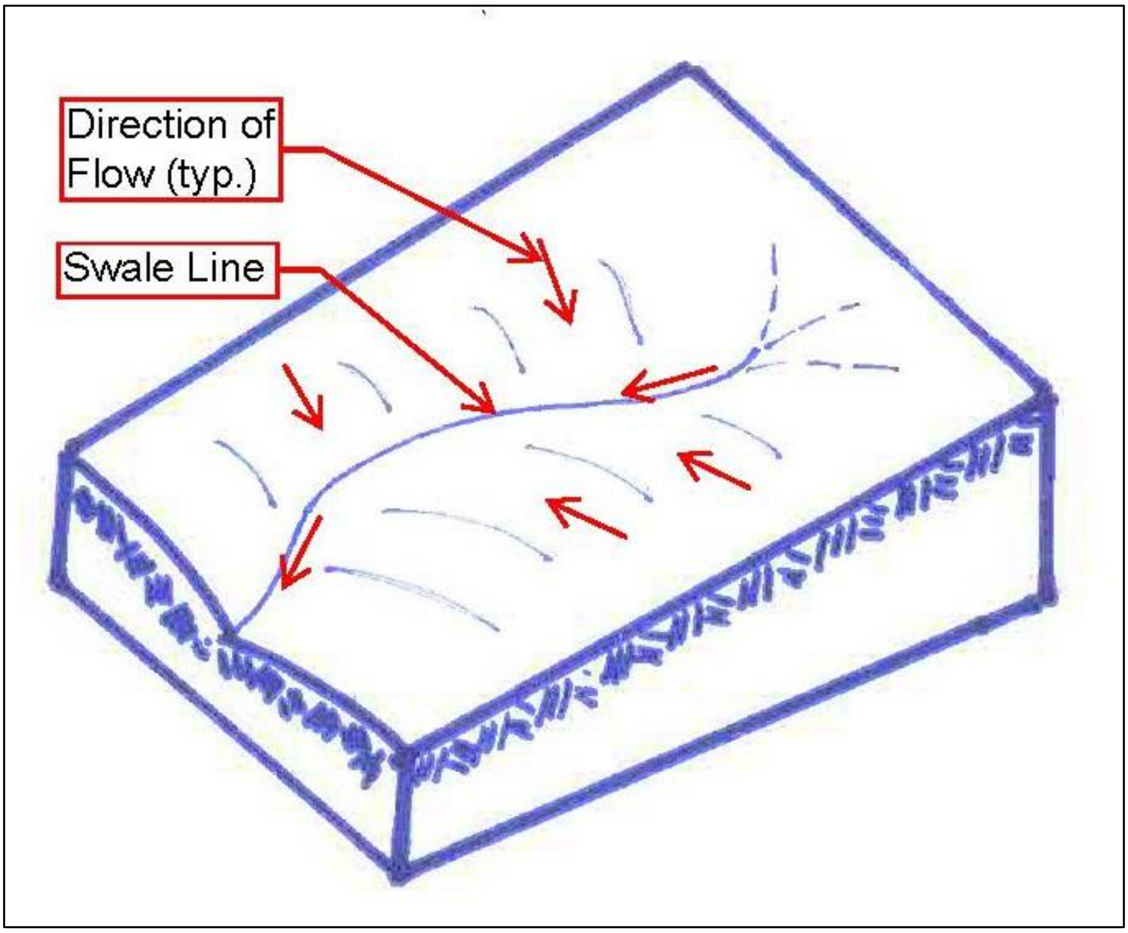


Figure 0-6— Cross Sectional Diagram of a Swale (Tiner, 2014)

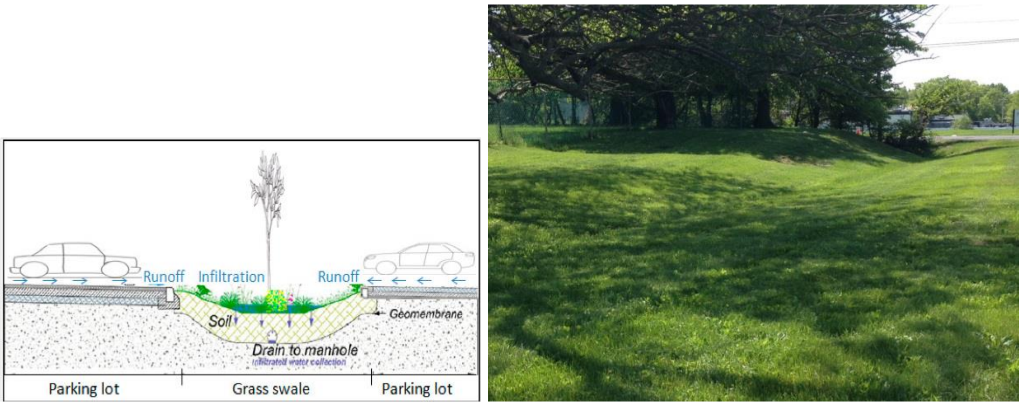


Figure 0-7— Some Pictures of Swales (Shafique, et al., 2018), (Tiner, 2014)

7.5.3 Ridge Lines

Ridge lines are linearly defined high points. An appropriate practice for an engineer to incorporate in grading design is to clearly delineate ridge lines by noting the location of the line as well as providing identification of the spot grades at either end of the ridge line. In large parking fields, the plan may become too "busy" or cluttered if every ridge line is proposed, so at their discretion the engineer may define the ridges with high point spot grades and the appropriate contour lines.

Large flat sites may have a significant number of ridge lines since the entire site needs to undulate, rising away from the inlets, and returning down to the next row of structures. If the site was to be designed to continue to rise, excessive earthwork would be required, as well as the need for costly import.

7.5.4 Retaining Walls

Retaining walls are an important tool for the engineer to propose when grading will require slopes exceeding 2:1 in order to accomplish the site grading goals. There are several types of retaining walls available (such as pre-cast, cast in place concrete, modular block, and timber). Additionally there are methods of reinforcing proposed "steep/vertical" soil embankments such as methods involving soil nails, etc. that serve the same purpose as retaining walls. Unfortunately each of these "retaining wall" methods, including basic modular block retaining walls, are expensive when compared to natural grading. As a result, the engineer must take care to minimize the amount of proposed retaining walls on a project. However, there is no doubt that as developers continue to develop challenging sites, the need to construct large retaining walls will continue.

The rule of thumb is that small retaining wall less than 4' in height can be detailed by the civil engineer with little or no geotechnical analysis. However if a retaining wall greater than 4' in height is required, the engineer is well served to consult with a structural engineer who will analyse the geotechnical report and issue signed and sealed retaining wall drawings that have structural calculations to support the wall design. Refer to **Figure 0-8** for some pictures of retaining walls.

Figure 0-8— Pictures of Retaining Wall

It is worth noting that many modular retaining wall system vendors will provide signed and sealed wall design calculations and drawings at no cost if their system is specified.

7.6 Problem Areas and Other Locations of Importance

7.6.1 Landscaped Islands and Peninsulas

Since the parking lot is typically a large field of asphalt, the small curbed in landscaped areas that contain, grass, trees, shrubs, and mulch are referred to as "Islands". Additionally, there are often landscaped areas at the end of the row of parking spaces that accommodate a tree, but are connected back to the main lawn outside of the parking field. These mid-course "end caps", if connected back to the main lawn, are referred to as "peninsulas".

Islands and peninsulas can be considered grading "problem areas" because they require a little extra attention to detail. If one is not careful, the intent of the surrounding grading, plus the detail that calls out for the height of the exposed curb face to be 6" can put an installing contractor in a grading dilemma. If the contractor does not pick up on the situation and send in a Request for Information, the installed condition may be areas that are too flat and/or areas that collect water that should not. (Refer to **Figure 0-9** for pictures of puddling adjacent to an island)

Figure 0-9— Picture of Puddling Adjacent to Island

The best practice is for the engineer to identify spot grades at the islands and peninsulas so that there is no confusion.

7.6.2 Longitudinal Islands with Sidewalks

Longitudinal Islands are islands that run the length on many parking spaces down the "spine" of the parking bay. Often times when a large parking field is proposed, the longitudinal island is a great location to achieve a grade change in the direction perpendicular to the circulation (i.e. parallel to the store fronts). During the iterative process of design, sometimes it is decided to add a sidewalk to these islands. When this occurs, the engineer should take care to reassess and confirm there was not a significant change in elevation across the island, and the assumed/required maximum 2% cross slope of the sidewalk may cause a discrepancy that cannot be easily remedied by the time it is recognized.

7.6.3 Flush Ramps

Flush Ramps can be a problem when grading because often times when the surrounding grades were initially developed by the engineer, the fact that a ramp would be proposed was not considered. As a result, sometimes detailed grading associated with the ramp is not provided. As another consequence, the installing contractor may not recognize that a potential puddle is going to occur.

One scenario where this occurs frequently is when the sidewalk is oriented up a modest grade, and the ramp is cut into the curb with relatively flat surrounding grades, but directed toward the curb that the ramp is cut into. Since the curb has dropped 6” down to the depressed/flush location, suddenly a low spot where water will gather is introduced.

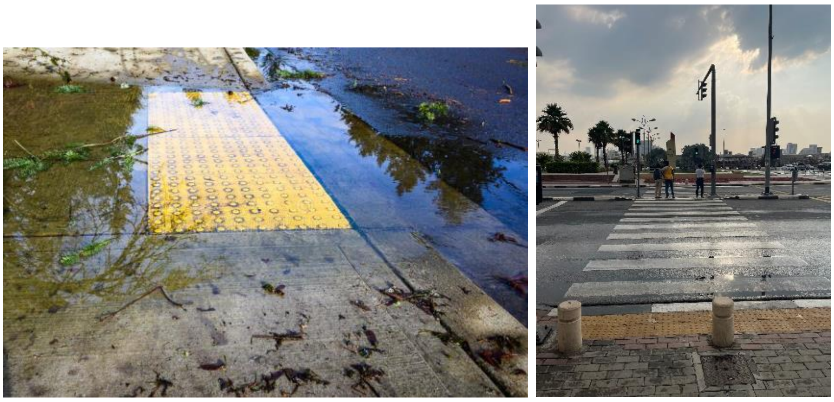


Figure 0-10— Pictures of Puddle Located on a Flush Ramp Condition (Nelson, 2022)

Unfortunately this is the worst place for a puddle to occur. Refer to **Figure 0-10** for some examples. The major purpose in many cases for installing the flush ramp is to accommodate those with disabilities especially.

It is not difficult to overcome the risk of this in design with a few additional proposed spot grades, but left undetailed a puddle may exist in the post constructed condition. The engineer should always analyse proposed flush ramp locations so that they can provide the extra spot grades and/or other detailing if warranted.

7.6.4 Property Line grading

Grade changes at the property line can be a problem, and is often overlooked. However, it is important that the engineer consider the existing grading characteristics following onto and off of the site at the property line.

Assume for example an existing condition where a large meadow is situated such that stow water will flow onto the site, perpendicularly across an undulating property line. In the proposed condition, a new parking lot or building only several feet off the property line is being proposed. If accommodations are not made to address the flow coming onto the potentially unrecognized low spot that is made, this could create standing water, which could lead to water in the building or groundwater into the basement. Perhaps the finished floor is a slab and is set high enough to remove that risk, but without a swale or other drainage structures placed in the area, the yard may develop standing water and become muddy for long seasonal periods. Muddy conditions make the property less desirable and usable, and standing water fosters mosquito breeding and disease.

7.6.5 Drainage Outfall Location

Typically the drainage of the site will flow toward the existing low end of the site. Establishing how low the surface grade can be at the lowest portions of the site requires taking several things into account, for example: Where will the outfall of the outlet control structure be tied into? What type of detention basin is being proposed, and are there adequate clearances in order to achieve the drainage goals? What is the seasonal high groundwater elevation in the area of the anticipated basin?

7.6.6 Setting the Finished Floor Elevation

Setting the most appropriate finished floor elevation is one of the most challenging aspects of site grading. Besides all of the various parameters that must be met associated with creating a vehicular accessible site and building, and a pedestrian accessible route from the parking lot to the building, maximizing the balance of the earthwork can be significantly impacted by the elevation that the building is set. The engineer should always make sure to set the finished floor elevation of the buildings on site based on good practices and by working within the guidelines established by the Authority Having Jurisdiction.

SUSTAINABLE DRAINAGE

8.1 Proposed Approach

Conventional urban drainage systems were designed to collect storm water as quickly as possible and dispose of it from the built environment. These systems are now struggling to cope with further urbanisation and are unlikely to cope with greater extremes in rainfall because of climate change. Hence, conventional urban drainage systems are considered unsustainable in the long term.

There is a growing acceptance that we need to have a more sustainable approach to managing storm water, in line with the concept of Sustainable Development which is provoking a profound rethinking in our approach to urban water management. Sustainable Development is defined as meeting the needs and aspirations of the present generation without compromising the ability of future generations to meet their own needs. An alternative definition by United Nations Environment Programme (UNEP) asserts that Sustainable Development is that which improves the quality of life while living within the carrying capacity of supporting ecosystems.

Sustainable services must be environmentally friendly, socially acceptable and financially viable. The sustainability concept calls for overall rethinking and this implies paying attention to particular situation in the local area (Nouh, 2001).

The growing trend towards a sustainable approach to managing storm water has resulted in the concept of Sustainable Drainage Systems (SuDs) which is a component of Sustainable Development and Integrated Water Resources Management (IWRM) and mimic natural drainage processes to reduce the effect on the quality and quantity of runoff from developments and provide amenity and biodiversity benefits. This is in line with the objectives of urban drainage management described in **Section 0.**

SuDs mimic natural drainage patterns by:

- Storing runoff and releasing it slowly (attenuation)
- Allowing water to soak into the ground (infiltration)

- Filtering out pollutants
- Allowing sediments to settle out by controlling the flow
- Creating attractive environment for people and wildlife

Sustainable drainage requires a new approach to surface water management, moving away from traditional piped drainage systems and promoting wider environmental objectives and meeting the requirements of new legislation. Sustainable drainage is different to traditional drainage because it:

- delivers a higher environmental performance expected by the society
- is often visible above ground, enabling easier inspection and management
- is often easier to manage
- Is often multi-purpose, e.g. providing drainage and public open space, car parking, etc.
- reduces the rate and volume of runoff from development with more natural approaches
- can remove pollutants from runoff
- is easier to adapt to climate change and developmental pressures

Therefore, any measures, solutions and designs of urban drainage schemes in Dubai shall be consistent with the recognized principles of SuDs.

8.2 SuDs Management Train

For SuDs to best mimic the natural drainage, a Management Train approach shall be adopted. The SuDs Management Train and hierarchy of techniques that shall be considered are as follows:

Prevention: The use of good site design and site housekeeping to reduce and manage runoff and pollution, e.g. sweeping to remove surface dust and detritus from car parks, rainwater reuse/harvesting, land-use planning and reduction of paved surfaces. Prevention policies should generally be included within the site management plan

Source control: Control of runoff at or very near its source, e.g. Modern Plastic Soakaways/ Other Infiltration Methods, Green Roofs, Pervious Pavements, Swales, Basins and filter strips.

Site control: Management of water in a local area or site, e.g. routing water from building roofs and car parks to a large Modern Plastic Soakaway/ Infiltration Or Detention Basin, Swales and Basins.

Regional control: Downstream management of runoff for a whole or several sites/catchments, typically in a Detention or Retention Pond.

Hence, wherever possible, storm water shall be managed in small, cost-effective landscape features located within small sub-catchments rather than being conveyed to and managed in large systems at the bottom of drainage areas.

The techniques that are higher in the hierarchy are preferred to those further down so that prevention and control of water at source should always be considered before site or regional controls.

However, where upstream control opportunities are restricted, a number of lower hierarchy options shall be used in series. Water shall be conveyed elsewhere only if it cannot be dealt with on site.

8.3 Infiltration Systems

According to the SuDs principles, where possible, local site drainage shall be preferred to centralise regional drainage. Hence, wherever possible, storm water shall be managed in small, cost-effective landscape features located within small sub-catchments rather than being conveyed to and managed in large systems at the downstream of the drainage areas. Infiltration systems are one of the methods for local site drainage as per the SuDs principles.

Experiences in the UAE show that the performance of soakaway infiltration systems mostly depends on groundwater levels. They normally work fine where there is no high groundwater level. But the main issue on fast developing areas is that groundwater levels increase once developments have been populated, mainly due to excessive irrigation. If SuDs are proposing in development or along the road. The hydraulic model must include the proposed SuDs option in order to replicate the actual hydraulic performance of the proposed storm water system. This is designer's responsibility to check the DM requirements before initiating the design.

Infiltration systems shall be designed based on site-specific investigations.

Given the soil permeability, depth of groundwater in and around Dubai and considering the performance of surface water infiltration options and limitation in space due to existing and future developments in Dubai, infiltration trenches seem to be the best option for storm water infiltration. These systems could be located in the RoWs of the roads or under the roads to avoid space limitations.

Storm water infiltration systems need to be able to perform as part of the overall integrated and sustainable drainage system for Dubai. These systems need to be installed at shallow enough depths not to interfere with the groundwater levels and have combined soakage and storage performances that allow for the efficient disposal of the storm flows. A key constraint to the successful operation of the system is the depth to groundwater level. This means that for the soakaway system to function as intended, in some areas it will need to be installed after the local groundwater control system has been commissioned.

8.4 Maintenance, Operation and Management Aspects of SuDs

Like all drainage systems, SuDs components shall be inspected and maintained. This ensures efficient operation and prevents failure. Usually SuDs components are on or near the surface and most can be managed using landscape maintenance techniques. For below-ground SuDs components such as permeable paving and soakaways, the manufacturer or designer should provide maintenance advice. This should include routine and long-term actions that can be incorporated into a maintenance plan.

The design process should consider the maintenance of the components including access, waste management and any corrective maintenance to repair defects or improve performance, etc.

Generally, maintenance is required from time to time to protect the integrity of drainage facilities.

Normal maintenance practice in arid climates includes (Nouh, 2001):

- Embankment and slope protection against sliding by placing granular materials and/or planting grass cover on the slopes
- Retardation of sheet erosion by using grass cover plantation and/or riprap placement on the surface
- Instalment of concrete sediment racks in the detention basin to avoid the transport of debris and boulders. The area of the rack shall be large enough to hold up quite a large mass of material without impeding the flow of water.
- Instalment of trash racks to hold rubbish, papers, leaves, etc.

Routine maintenance in such climates shall be carried out after each rainstorm checking the embankments and repair the damage, checking the concrete and metal components of the drainage system and make necessary restoration, clean both the concrete sediment racks and trash racks, and clean the settled sediments and the rubbish materials from the streets.

SUBSURFACE FLOW AND DRAINAGE DESIGN

9.1 Subsurface Flow

Groundwater movement is governed by variations in piezometric head and soil hydraulic conductivity (permeability). Typical permeability data appropriate to soil conditions are provided in **Table 0-1** 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.

Table 0-1— Typical Soil Permeability Values

Permeability (m/s)	Soil type	Drainage characteristics
10^{-0}	Clean gravel	Good drainage
10^{-1}		
10^{-2}		
10^{-3}	Clean sand, clean sand and gravel mixture	Acceptable drainage
10^{-4}		
10^{-5}		
10^{-6}	Very fine sand, organic and inorganic silt, clay, subkha, stratified clay deposits	
10^{-7}		
10^{-8}		
10^{-9}		
10^{-10}	Homogeneous clay below weathering zone	Practically impervious

9.2 Groundwater Levels

A critical parameter in designing a drainage system is the height of the water table midway between two consecutive rows of pipes (refer to **Figure 0-1**). 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.

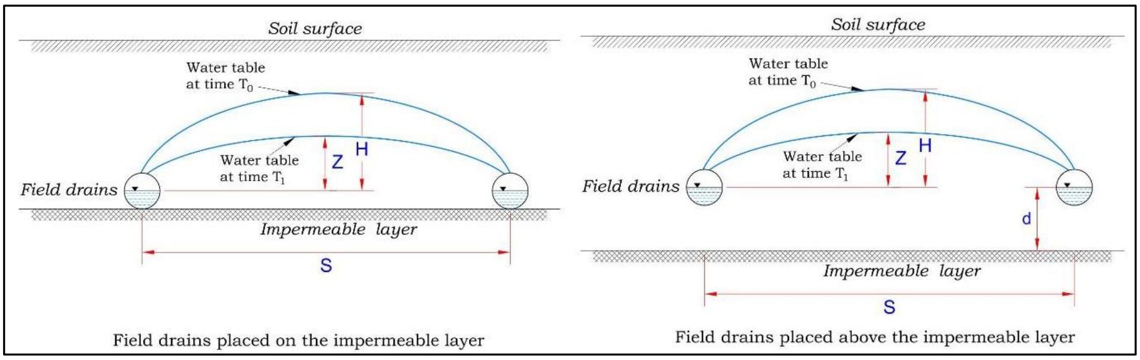


Figure 0-1— Placement of Field Drains with Respect to the Impermeable Layer (ADM, 2016)

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.

In urban landscaping requirements, the water table at the midway point must also be at least 15 cm below the plantation root zone. For streetscape landscaping this 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 1 m below the surface.

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.

9.3 Subsurface Drainage Planning

The main phases of planning a subsurface drainage project are shown in **Table 0-2**.

Table 0-2— Main Phases of Planning a Subsurface Drainage Project

Design Stages	Details
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.

9.3.1 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 0-2**.

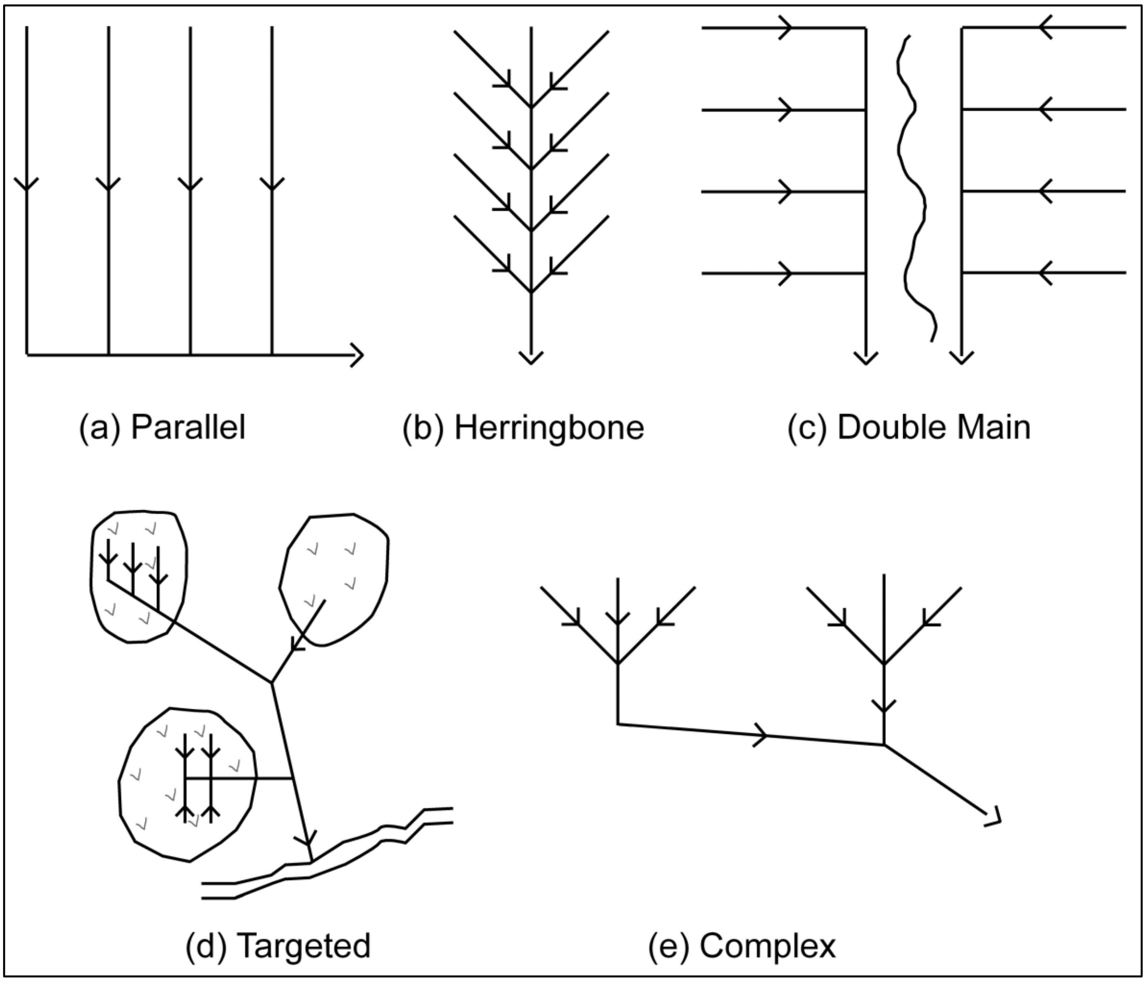


Figure 0-2— Example Drainage System Layout Alternatives (Koganti, et al., 2020)

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 0-3**.

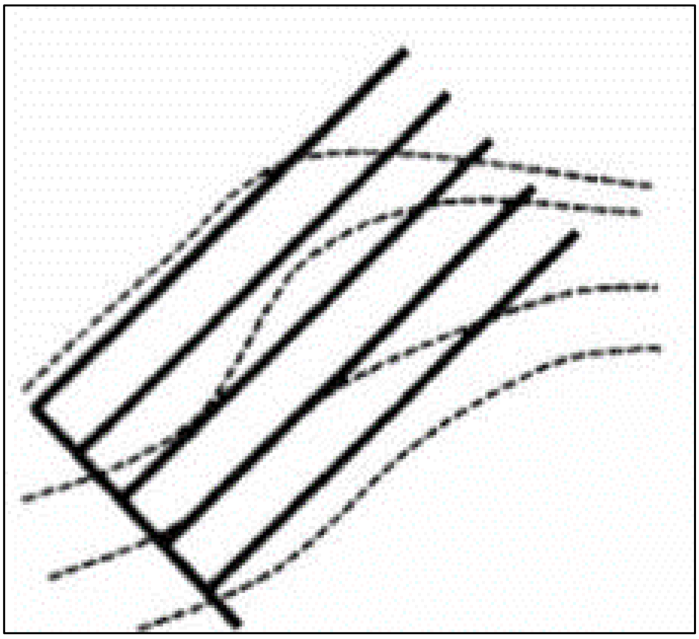


Figure 0-3— Alignment of Field Laterals with Contours (Wright & Sands, 2001)

9.3.2 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 spacing for a range of ground conditions. Average pipe spacing for soils of different textures are presented in below table:

Table 0-3—Average pipe spacing for soils of different textures

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

Soil Texture	Hydraulic Conductivity		Spacing
	Class	(m/day)	(m)
Irrigated soils	Variable	Variable	45 - 200

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.

9.3.3 Determining Discharge from Subsurface Drains

With reference to **Figure 0-4** below, the steady state discharge of spaced drains can be computed using the following formulas:

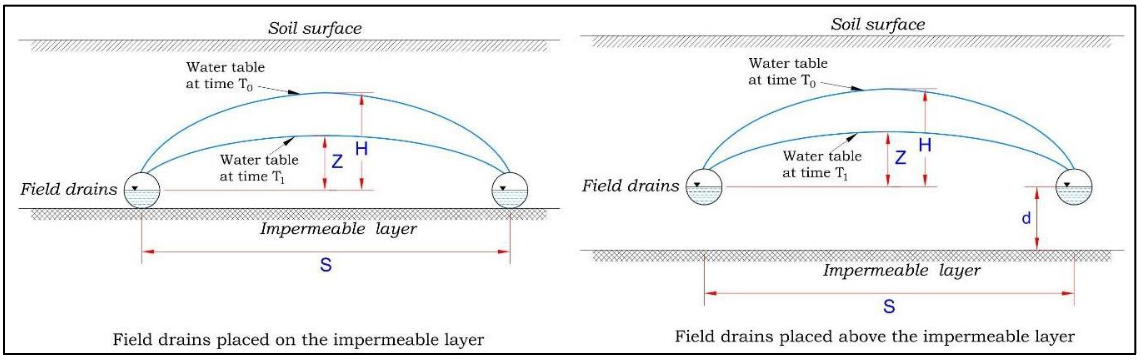


Figure 0-4— Placement of Field Drains with Respect to the Impermeable Layer (ADM, 2016)

1. Drains placed above the impermeable layer

$$Q = CA\frac{2\pi KH(d + H/2)}{S^2}$$

2. Drains placed on the impermeable layer

$$Q = CA\frac{4KH^2}{S^2}$$

3. 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

K = the hydraulic conductivity weighted over the affected soil profile in

d = the height above impermeable surface

H = the initial height of water above the centreline of the drains

S = spacing between consecutive drains

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)

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

4. Radius of Influence

$$R = 2000M\sqrt{K}$$

Where:

M = Drawdown (L)

K = Coefficient of Permeability (L/T)

The steady state discharge of single line drains can be computed using the following formulas:

5. Drains placed above the impermeable layer

$$Q = CA\frac{2\pi KH(d + H/2)}{4R^2}$$

6. Drains placed on the impermeable layer

$$Q = CA\frac{KH^2}{R^2}$$

9.3.4 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 similar region like 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 above.

Minimum size for perforated pipes is 110mm DN.

9.3.5 Pipe Materials

Approved pipe materials parameters are shown in **Table 0-4**.

Table 0-4— Approved Drainage Pipe parameters

Description	Parameters
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 center line of the pipe.

9.3.6 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.

9.3.7 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).

9.3.8 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.

9.3.9 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.

9.3.10 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.

9.4 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), by the drainage area

- Using the formulae in **Section 9.3.3**, “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 Manning equation:

9.5 Drainage Installation

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.

9.6 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.

WATER QUALITY AND WATER QUALITY MANAGEMENT

10.1 Introduction

Based on the discussions in sections above, water quality management is one of the important aspects of urban drainage and SuDs to prevent the transfer of pollutants to receiving water bodies.

Design criteria for storm water quality are intended to provide treatment of the first flush, i.e. the first 10 mm of runoff depth.

The design of the storm water quality management measures should follow "The SuDs Manual, CIRIA 2007"; however, the particular climatic and geographic conditions of Dubai shall be considered in the design of such systems.

10.2 Water quality control

In SuDs, the aim is to utilise the natural water quality treatment processes.

In the design of storm water treatment systems, the below items shall be considered:

- Priority given to the treatment of road runoff from areas where there is high concentration of vehicle braking and turning, i.e. roundabouts and intersections.
- Incorporation of Grassed swales (where appropriate) to reduce the total pollutant loadings of receiving waters.
- Incorporation of water treatment systems into roadway features, e.g. bio-retentions filters into traffic calming devices.
- Incorporation of litter collection systems into car parks and surrounding roadways of shopping centres, takeaway food centres, community areas, entertainment facilities and sporting fields.

- Incorporation of public education messages onto the face of storm water inlet lintels, e.g. PROTECT OUR ENVIRONMENT – FLOWS TO CREEK.

The range of water quality control measures are described below.

10.2.1 Pre-Treatment

As mentioned in **Section 0**, pre-treatment (removal of oil, gross pollutants and sediment loads) is vital to ensure the long-term effectiveness of the SuDs components. Pre-treatment measures are described below. Pre-treatment components shall be used in the drainage system as appropriate.

Pre-treatment components that shall be considered in the design of storm water drainage systems are described below.

Gross Pollutant Traps (GPTs):

GPTs are devices for the removal of solids conveyed by runoff that are typically greater than 5 millimetres. There is a variety of GPTs currently suitable for use in urban catchments including gully baskets, in-ground GPTs, trash racks and pipe nets. For more information on the design of GPTs, the designer can refer to the Water Sensitive Urban Design (WSUD) guidelines published by different Australian authorities.

Oil separators:

Oil separators shall be specified for treating runoff from areas where hydrocarbon products are handled, e.g. petrol stations, storage areas, bus and truck parking areas, airports, etc. or where small oil spills may occur. They shall be installed close to the potential pollution source to minimise risks.

Oil separators are available as pre-fabricated proprietary systems from different vendors, but can also be built in-situ. These components shall be designed in accordance with BS EN 858-1, Separator Systems for Light Liquids (BSI, 2002).

Sedimentation manholes/Catch basins:

Sedimentation manholes may be located above or below ground, should remove sediment from the storm water runoff, must be easily maintainable and must be safe to operate.

A sedimentation manhole is a manhole with an enlarged sediment sump which maintains a permanent water pool to promote settling of solids and to store settled sediments. It can also include a baffle to retain oils and floating debris. Due to a high potential for re-suspension of sediments, water quality treatment performance of sedimentation manholes is limited; however, they may be appropriate as a cost-effective and simple solution to protection of downstream SuDs components where there is a high proportion of sands and other coarse sediments in the catchment. Suitability of these components shall be considered in the design.

Vortex separators:

Vortex separators are structures with a gravity settling or separation unit to remove sediments and other gross pollutants from storm water. Water moves in a centrifugal manner and the centrifugal forces created by the circular motion make suspended particles to move to the centre of the device where they settle at the bottom. They can either be designed to accommodate the full flow or can be installed downstream of a bypass structure, so that high flows are routed around the device. Suitability of these components shall be considered in the design.

Proprietary filtration systems:

Filtration systems are offered by a variety of manufacturers and can be obtained as pre-fabricated standard units of custom-made to suit site conditions. Some manufacturers combine vortex separation and online filtration in one system. Suitability of these components shall be considered in the design.

10.2.2 Treatment

SuDs components contributing to storm water runoff treatment are described below. Treatment components shall be used in the drainage system as appropriate.

Filter strips:

Filter strips are vegetated strips of land which accept runoff as overland sheet flow from the upstream catchment. They treat runoff by vegetative filtering and promote settlement of particulate pollutants and infiltration. They are particularly applicable at the edges of car parks.

Infiltration trenches:

Infiltration trenches have been described in Section above.

Bio retention swales:

Bio retention swales are linear vegetated drainage components in which runoff can be stored, conveyed and treated. They should promote low flow velocities to allow much of the suspended particles to settle out. **Figure 0-1** shows a typical bio retention swale located along a road.

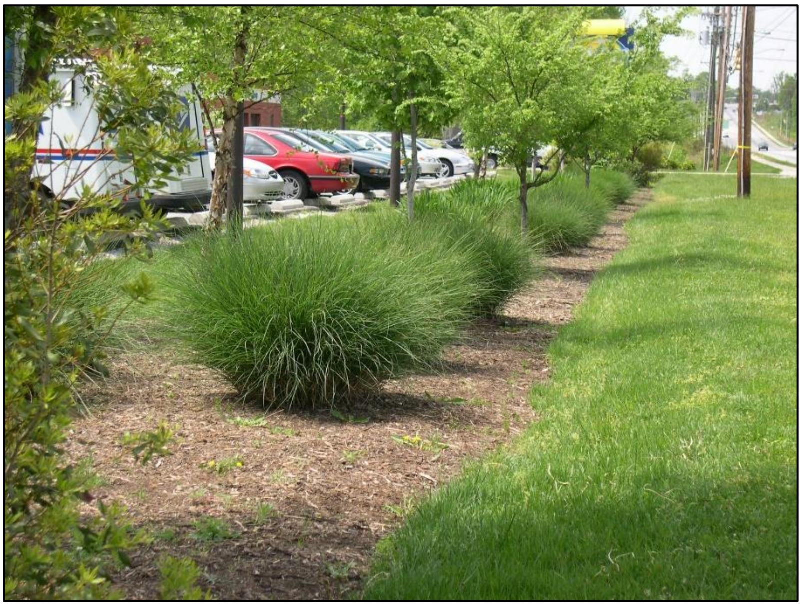


Figure 0-1— A Typical Bio retention Swale Located along a Road (Austin, n.d.)

Bio retention swales provide both storm water treatment and conveyance functions. A bio retention system may be installed in the base of a swale that is designed to convey storm water as part of a drainage system. The swale component provides pre-treatment of storm water to remove coarse to medium sediments while the bio retention system removes finer particulates and associate contaminants. Bio retention swales provide flow retardation for frequent storm events and are particularly efficient at removing nutrients.

Bio retention swales also act to disconnect impervious areas from downstream waterways and provide protection from frequent storm events by reducing flow velocities compared with piped systems. The bio retention component is typically located at the downstream end of the overlying swale 'cell', i.e. immediately upstream of the swale overflow pit as shown in **Figure 0-2** or along the full length of a swale as a continuous trench.

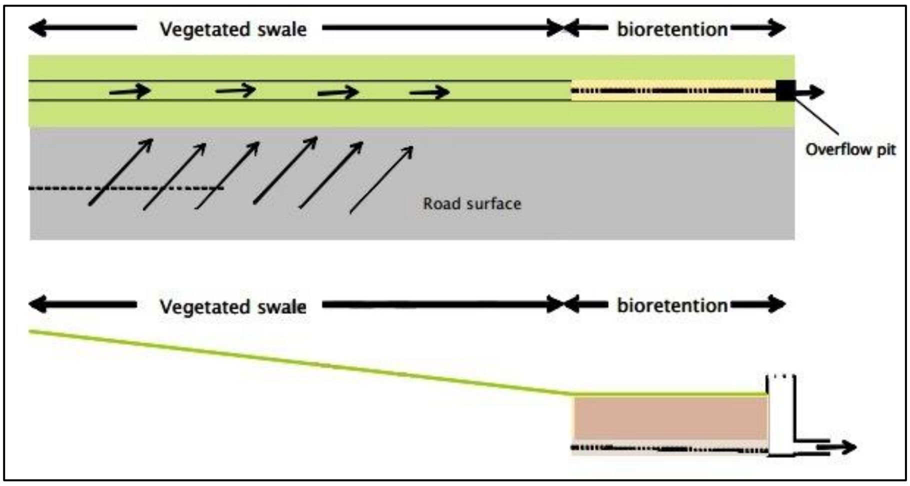


Figure 0-2— Conceptual Layout of Bio retention Swale (Gold Coast Planning Scheme Policy, 2007)

The choice of bio retention location within the overlying swale will depend on a number of factors, including area available for the bio retention filter media and the maximum batter slopes for the overlying swale. Typically, when used as a continuous trench along the full length of a swale, the desirable maximum longitudinal grade of the swale is 4%. For other applications, the desirable grade of the bio retention zone is either horizontal or as close as possible to encourage uniform distribution of storm water flows over the full surface area of bio retention filter media and allowing temporary storage of flows for treatment before bypass occurs.

The bio retention swale treatment process operates by filtering storm water runoff through surface vegetation associated with the swale and then percolating the runoff through a prescribed filter media, forming the bio retention component which provides treatment through fine filtration, extended detention treatment and some biological uptake.

Bio retention swales are not intended to be 'infiltration' systems in that the intent is typically not to have the percolating storm water runoff exfiltrate from the bio retention filter media to the surrounding in-situ soils. Rather, the typical design intent is to recover the percolated storm water runoff at the base of the filter media within perforated under-drains for subsequent discharge to receiving waterways or for storage for potential reuse. In some circumstances however, where the in-situ soils allow and there is a particular design intention to recharge local groundwater, it may be desirable to permit the percolated storm water runoff to infiltrate from the base of the filter media to the underlying in-situ soils.

A key hydraulic design consideration for bio retention swales is the delivery of storm water runoff from the swale onto the surface of a bio retention filter media. Flow must not scour the bio retention surface and needs to be uniformly distributed over the full surface area of the filter media. In steeper areas, check dams may be required along the swale to reduce flow velocities discharged onto the bio retention filter media.

It is important to ensure that velocities in the bio retention swale from the runoff events over the range of design return periods (2 to 50 years) are kept sufficiently low, preferably below 0.5 m/s and not more than 2.0 m/s to avoid scouring. This can be achieved by creating shallow temporary ponding, i.e. extended detention, over the surface of the bio retention filter media via the use of a check dam and raised field inlet pits. This may also increase the overall volume of storm water runoff that can be treated by the bio retention filter media.

Selection of an appropriate bio retention filter media is a key design step involving consideration of the below inter-related factors:

- Saturated hydraulic conductivity required to optimise the treatment performance of the bio retention component given site constraints on available filter media area
- Depth of extended detention provided above the filter media
- Suitability as a growing media to support vegetation growth, i.e. retaining sufficient soil moisture and organic content

Figure 0-3 shows the typical section of a bio retention swale.

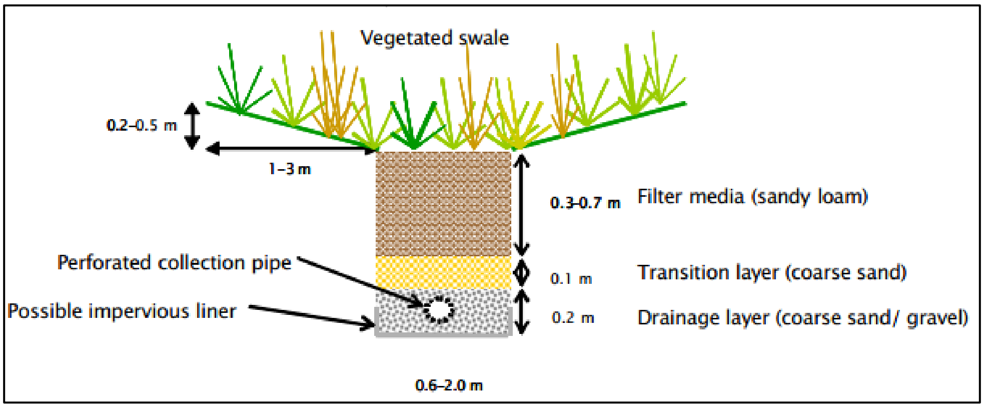


Figure 0-3— Typical Section of a Bio retention Swale (Gold Coast Planning Scheme Policy, 2007)

The filter media layer provides the majority of the pollutant treatment function, through fine filtration and also by supporting vegetation. The vegetation enhances filtration, keeps the filter media porous and provides some uptake of nutrients and other storm water pollutants. As a minimum, the filter media is required to have sufficient depth to support vegetation. Typical depths are usually between 300-1000 mm with a minimum 800 mm for tree planting.

The saturated hydraulic conductivity of the filter media is established by optimising the treatment performance of the bio retention system given site constraints of the particular site. Saturated hydraulic conductivity should remain between 50-200 mm/hr.

The particle size difference between the filter media and the underlying drainage layer shall be not more than one order of magnitude to avoid the filter media being washed through the voids of the drainage layer. Therefore, a transition layer of minimum 100 mm thick is recommended.

In order to size the perforated collection pipe, a 50% blockage of the perforation is recommended. Also, to size the overflow pit, a 50% blockage of the inlet grate shall be considered.

Bio retention Basins:

Bio retention basins are shallow landscaped depressions which are typically under-drained and rely on vegetation and infiltration to remove pollution and reduce runoff.

10.3 Water Quality Modelling

The performance of the water quality management measures shall be checked and justified by modelling the drainage water treatment train. This can be undertaken by Model for Urban Storm water Improvement Conceptualisation (MUSIC) developed by eWater initiative in Australia or similar tools such as the quality modelling module of InfoSWMM.

The MUSIC software serves as a planning and decision support system, and packages the current knowledge of the performance of a range of storm water treatment measures into a modelling tool. MUSIC is designed to operate at a range of temporal and spatial scales, suitable for modelling storm water quality treatment systems for individual plots up to regional scales. It provides the ability to simulate both quantity and quality of runoff from catchments and the effect of treatment facilities on these components. MUSIC is an aid to decision making. It enables designers and/or stakeholders to evaluate conceptual designs of storm water management systems to ensure they are appropriate for their catchments. By simulating the performance of storm water quality improvement measures, MUSIC determines if the proposed systems can meet specified water quality objectives.

It shall be noted that the MUSIC model shall be customised for the particular location and application with the relevant input data.

By using water quality modelling tools such as MUSIC or InfoSWMM, the performance of these systems shall be checked against the current environmental legislations and in particular the water quality limits of discharge to the environment and water bodies in the UAE and Dubai.

The main pollutants to be considered in storm water treatment are Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The design requirement for reduction of pollutants to be achieved through provision of storm water treatment systems is 75% for TSS, 45% for TP and 35% for TN.

The nominal size of a bio retention facility (swale or basin) in terms of surface area to achieve the target reduction factors is 0.5% to 3% of the catchment area.

DRAINAGE MANAGEMENT PLAN

A Drainage Management Plan for new developments shall be prepared by the designer/developer and submitted to DM for approval.

The main aim of the Drainage Master Plan is to confirm to DM that the proposed drainage system has been developed according to the Drainage Design Criteria and international best practice, and that it follows the concept of the Drainage Master Plan.

The Drainage Management Plan shall include but not be limited to:

- The overall concept of the proposed drainage system and how it follows the Drainage Master Plan and SuDs principles
- Hydrologic and hydraulic model of the drainage system, confirming that the proposed drainage system for the new development is compatible with the existing DM drainage system, the Drainage Master Plan and the Drainage Design Criteria, and the impacts of the proposed system on the DM drainage system. This model shall also confirm that the post-development peak outflows from the development site are not greater that the pre-development peak outflows, i.e. Drainage Impact Zero as required by the Drainage Master Plan.
- The designer shall be able to prove that the overland flows from the development, in events larger than the Design ARI (Level of Service), will not impact the external and arterial roads and adjacent developments. The recommended approach to analyse this impact is two-dimensional (2D) overland flow modelling, however the designer may chose other methods to prove that there will not be an impact.
- Water quality model, confirming that the outflows from the site meet the environmental regulations and discharge quality limits
- The lawful point of discharge shall be nominated and approvals sought from DM.

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