STORMWATER DRAINAGE MANUAL

Update in the fifth edition highlighted in blue

Planning, Design and Management

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DRAINAGE SERVICES DEPARTMENT

Government of the Hong Kong
Special Administrative Region

CONTENTS

1. INTRODUCTION

1.1 SCOPE

This Manual offers guidance on the planning, design, operation and maintenance of stormwater drainage works which are commonly constructed in Hong Kong. Such works include stormwater pipelines, box culverts, nullahs, river training works, polders and floodwater pumping facilities.

Some sections of the manual are also relevant for the management of natural watercourses. Drainage Services Department (DSD) has also promulgated Practice Note No. 1/2011 for "Design Checklists on Operation & Maintenance Requirements" which can be reached on DSD's internet home page : www.dsd.gov.hk. Readers are requested to go through the Practice Notes in the course of designing the drainage system to ensure that the final products satisfy the operation and maintenance requirements of the maintenance authority.

The major changes in the fifth edition of this Manual comparing with the previous edition include review on rainfall analysis and sea level analysis, consideration of climate change and promotion of blue-green concept in drainage infrastructure. There are also minor amendments included in this edition taking into account the updated information. All of the changes are reflected in Chapter 1 to Chapter 6 and Chapter 13 of this Manual.

1.2 ABBREVIATIONS

The following abbreviations are used throughout this Manual:

AFCD	Agriculture, Fisheries and Conservation Department
Arch SD	Architectural Services Department
BD	Buildings Department
BS	British Standard
BS EN	European Standards adopted as British Standards
BSI	British Standards Institution
CEDD	Civil Engineering and Development Department
DLO/YL	District Lands Office/Yuen Long
DO/YL	District Office/Yuen Long
DSD	Drainage Services Department
EC	Eurocodes (i.e. European Standards EN1990 to EN1999)
EPD	Environmental Protection Department
ETWB	Environment, Transport and Works Bureau
FEHD	Food and Environmental Hygiene Department
FSD	Fire Services Department
GCO	Geotechnical Control Office
GEO	Geotechnical Engineering Office (formerly known as Geotechnical Control Office)
GRP	Glass Reinforced Plastic

HDPE	High Density Polyethylene
HKO	Hong Kong Observatory
HyD	Highways Department
HKPF	Hong Kong Police Force
LCSD	Leisure and Cultural Services Department
LD	Labour Department
MDPE	Medium Density Polyethylene
NENT	North East New Territories
NWNT	North West New Territories
PWD	Public Works Department
SCS	Soil Conservation Service (United States)
TELADFLOCOSS	Territorial Land Drainage and Flood Control Strategy Study
TD	Transport Department
uPVC	Unplasticized Polyvinyl Chloride
UK NA	United Kingdom National Annexes to Eurocodes
USBR	United States Bureau of Reclamation
WBTC	Works Bureau (or Works Branch) Technical Circular
WSD	Water Supplies Department

1.3 DESIGN STANDARDS

1.3.1 Planning and Investigation of Drainage System

BS EN 752, or its latest versions, is to be adopted, except otherwise stated for planning and investigation of drainage system.

1.3.2 Drainage Structures

In Hong Kong, drainage structures are currently designed to BS, either directly as in the case for water retaining structures to BS 8007, or indirectly as in the case for structures subject to highway loading to BS 5400 customized by the local guiding document. In view of the progressive replacement of BS by EC (EN 1990 to EN 1999) and their UK NA through the promulgation of BS EN standards since March 2010, the Government has planned to migrate from BS to EC and UK NA in 2015. To cope with the migration, a transition period from 2013 to 2014 is set out during which the designer may opt for using BS or EC and UK NA in conjunction with local guidance/documents as appropriate for structural design of the drainage structures (e.g. box culverts, manholes, floodwater pumping stations, etc.). Starting from 2015, the use of EC and UK NA cum local guidance/documents as appropriate will become mandatory. The following design standards, or their latest versions, are to be adopted, except otherwise stated:

Design Elements/Loads	Design Standards
Imposed loads	Code of Practice for Dead and Imposed Loads, BD
Traffic loads	Structures Design Manual for Highways and Railways, HyD

Design Elements/Loads	Design Standards
Wind load	Code of Practice on Wind Effects in Hong Kong
Reinforced concrete structures	BS EN 1990 and BS EN 1992 (in general)
- Pumping station	- BS EN 1992 (superstructure) and BS EN 1992-3 (substructure)
- Box culvert	- BS EN 1992-3 and Structures Design Manual for Highways and Railways, HyD (if subject to highway loading)
- Tunnel lining	- BS EN 1992-3 (liquid retaining properties) and GEO Manuals, Guidelines and Publications (geotechnical)
- Manholes (other than standard manholes in DSD standard drawings)	- BS EN 1992-3
Foundation	
- Deep and shallow foundations	- Code of Practice for Foundations, BD (structural design) and GEO Manuals, Guidelines and Publications (geotechnical design)
- Reinforced concrete design for raft and pile cap	- BS EN 1992
Earth retaining structures	Guide to Retaining Wall Design, GEO

Recommended design parameters for concrete and steel reinforcement are given in Table 27.

2. STORMWATER DRAINAGE IN HONG KONG

2.1 THE HONG KONG SITUATION

Stormwater drainage and sewerage are part of the essential infrastructure of a modern city. In Hong Kong, separate systems are provided for the collection and disposal of stormwater and sewage.

In Hong Kong, life and property are from time to time under the threat of flooding due to heavy rainfall. The average annual rainfall of Hong Kong is about 2400 millimetres. Rainfall distribution is seldom uniform spatially and temporally and remarkable extremes in storm rainfall are also experienced. Such heavy rainfall, sometimes coupled with high sea levels associated with storm surges during the passage of tropical cyclones, can cause flooding.

Apart from natural causes, human activities can also influence the prevalence of flooding. Examples are changes in land use resulting in increase in runoff and depletion of flood storage; blockage of natural drainage systems by refuse, agricultural wastes or silt arising from both natural erosion and construction activities; indiscriminate land filling; and lack of comprehensive maintenance of natural watercourses due to land access problems.

In addition to the provision of a comprehensive system of stormwater pipelines, culverts and nullahs in the urbanized areas, it is necessary to undertake flood mitigation measures in the rural areas such as the construction of river training works to improve the flow capacity and the installation of polders and floodwater pumping systems for low-lying villages.

When planning new drainage projects, the conditions of the associated existing drainage system and the proposed measures for the improvement of the system recommended by the recent relevant studies should be checked for reference. For example, DSD had progressively commenced eight Drainage Master Plan Studies and three Drainage Studies since 1994 for completion in 2010 to review the performance and conditions of the existing drainage system covering almost the entire territory of Hong Kong. Figure 1 shows the boundaries of the study areas of these Drainage Master Plan Studies and Drainage Studies. Since 2008, the Review of these Drainage Master Plan Studies has progressively commenced and the review process is on-going. There are also other studies relating to the drainage system of Hong Kong, including TELADFLOCOSS Studies. These studies should be referred to when planning for new drainage projects.

Efforts have also been stepped up to implement a wide range of non-structural measures to relieve the problem of flooding. These include:

(a) Development control requiring Drainage Impact Assessments for new development proposals which are likely to have a significant impact on the existing drainage systems.

(b) Drainage legislation, Land Drainage Ordinance (enacted on 31.3.1994) empowering Government to carry out maintenance of main natural watercourses.

(c) Enhancement of the flood warning service and distribution of advisory/educational information pamphlets on flood prevention to people living in flood-prone areas.

(d) Operation of an Emergency and Storm Damage Organisation to deal with emergency cases of flooding.

3. GENERAL PLANNING AND INVESTIGATION

3.1 GENERAL

A stormwater drainage system should be designed to collect and convey run-off generated within a catchment area during and after rainfall events, for safe discharge into a receiving watercourse or the sea. The magnitude of peak flows that have to be accommodated will depend primarily on the intensity of rainfall and the size, topography, soil type, configuration and land use of the catchment. General information on the planning and investigation required for stormwater drainage systems is given in BS EN 752 (BSI, 2008). This Chapter gives further guidance on this subject.

3.2 SYSTEM PLANNING

3.2.1 Overview

System planning involves the assessment of the performance of the existing stormwater drainage system within a catchment and the design of a new or upgraded system to allow for the impact of new development within the catchment and/or to assess the necessity and feasibility of bringing the flood protection standards up to the levels recommended in this Manual. Catchments in Hong Kong vary considerably from rural areas with natural watercourses to old, highly congested, intensively developed urban areas and, particularly for large scale or strategic system planning, it is important to fully investigate and validate the adoption of the criteria, parameters and recommendations contained in the guidelines in this Manual. After a detailed analysis of all the characteristics of a catchment and the performance of the drainage systems, an experienced practitioner may propose alternatives to the guidelines to suit particular circumstances. However the adoption of any such alternatives shall be fully justified. Considerations on this issue may include proper analyses conducted on risk assessment; consequences of flooding including risk to life and limb; potential disruption to the community of major new works and the cost/benefit of new works. In the ultimate, however, the planning and implementation of drainage systems shall allow for the eventual provision of the flood protection standards recommended in this Manual.

3.2.2 Blue-green Concept

Blue-green drainage infrastructure (with "blue" representing the water bodies and "green" referring to plants) is a form of development aiming at improvement of the sustainability and resilience of Hong Kong's drainage system. It facilitates the infiltration of rainfall and the process of natural filtering to reduce the quantity and improve the quality of runoff. Therefore, blue-green drainage infrastructure could reduce surface runoff, water pollution, heat island effect, carbon footprint and energy consumption, and blend the natural water environment into the city. It also supplements the conventional drainage system by reducing the surface runoff and attenuating the peak flow, rendering the entire drainage system more resilient against unexpected extreme events. Examples of blue-green drainage infrastructure include swales and conveyance channels, filtrations (filter strips, filter trench and bioretention areas), infiltrations (soakaways, infiltration trenches, infiltration basins and rain gardens), retention and detention facilities (detention basins, retention ponds, geocellular storage systems) and wetlands. Various ecological features could be adopted in river improvement works to provide habitats for aquatic life, thus promoting conservation of biodiversity and sustainable use of biological resources. Blue-green drainage infrastructure also embraces sustainable drainage system such as green roofs, porous pavements and rainwater harvesting facilities. Given the scarce land resource in Hong Kong, the design of blue-green drainage infrastructure could probably integrate with other public facilities to co-use the same piece of land with a view to unleashing the drainage reserve space for public enjoyment. Blue-green concept is highly recommended for consideration in the early planning stage of drainage infrastructure and new development.

3.2.3 Progressive and Early Improvement

During the planning and design stages of a drainage improvement project, consideration should be given to provide some urgent works in the initial period of the construction stage in order to achieve progressive and early improvement in drainage capacity of the drainage system.

3.2.4 Detailed Considerations

Specific guidance on aspect of hydrological and hydraulic analyses for system planning and design are addressed in Chapters 4 to 8, however there are other factors that require special attention at the system planning stage and some examples are as follows.

(a) If the new drainage system is at the downstream end of an existing network, the designer shall take into account the possibility of future improvement of the upstream systems. The new system should be designed to accept the increased flow after improvement of the existing upstream network.

(b) The provision of adequate protection of low-lying areas within floodplains in the rural New Territories.

(c) The effect on sea levels of tropical cyclones.

(d) Substantial overland flow may occur due to performance failure or partial failure of stormwater drains during heavy rainfall events, e.g. blockage of a major culvert by a fallen tree or failure of a stormwater pumping station. This may have very serious consequences and cause serious flooding. A risk management approach should be adopted to cater for performance failure of stormwater drains. If the risk and/or consequences are high, mitigation measures such as provision of fail-safe design, design redundancy (e.g. provision of oversized drainage conduit or bypass drainage conduits in case of failure) and/or provision of safe overland flood paths may be necessary. Areas where serious consequences may occur are areas where many lives and properties will be threatened and/or serious disruptions to economic and social activities may occur. They include both the New Territories floodplains and urban areas. They also include old hinterland areas with ground level lower than that of the surrounding reclaimed area, long steep roads, depressed roads, road or railway tunnels, and pedestrian underpasses.

For overland flood paths along roads, the route of the flood water should be checked based on topographic data. The flood water should be channeled back to underground stormwater drains with spare capacity as early as possible through existing or additional stormwater inlets, gullies, etc. For this purpose, the road pavement drainage design should be reviewed with reference to the Guidance Notes on Road Pavement Drainage Design of Highways Department or its more up-to-date replacement.

3.2.5 Location of Public Drainage Systems

As far as possible, stormwater drainage systems should be located on Government Land and all nullahs, culverts and pipelines should be located either in road reserves or specially designated drainage reserves, which are non-building areas. Such reserves are essential in order to ensure that there is free and unrestricted access at all times for construction, repairs and maintenance.

Drainage reserves should be included where necessary on the various statutory and non-statutory town plans. The width of a reserve should be determined from the requirements for working space, vehicular access for construction plant, depth of the stormwater drain and clearance from adjacent existing structures and foundations. In general, a minimum width of 6 m plus the outside diameter of the pipeline or outside width of culvert is recommended.

For the implementation of public projects, the acquisition and allocation of land should follow the prevalent Government procedures. Attention should be drawn to the general principle that the land intake for each project should be kept to the minimum. If land is required from LCSD, necessary consultation and arrangement with LCSD should be initiated at the earliest possible stage.

The Hong Kong Planning Standards and Guidelines stipulate that no discharges from new stormwater outfalls or nullahs should be allowed to drain into a typhoon shelter, marina or boat park.

3.3 INFORMATION FOR SYSTEM PLANNING

3.3.1 Maps, Town Plans and Drainage Records

Government regularly publishes maps and town plans from which information on land use and topography of catchment areas can be extracted. For large-scale works, aerial photographs may provide an essential source of reference. Reference should also be made to Drainage Services Department's drainage records for information on the existing stormwater drainage systems.

3.3.2 Location of Utilities

(a) General

Utility companies and the appropriate Government authorities should be consulted regarding the effect of a project on their existing and proposed services and regarding any facilities required of the project. In particular, attention should be drawn to the fact that there are some underground tunnels and the associated structures constructed or being proposed by the Hong Kong Electric Company Limited, Mass Transit Railway Corporation, WSD, HyD, TD, DSD, etc. Hence these underground tunnels and structures have to be protected against damage by the construction or site investigation works of a new project.

The installation of services by utility companies on Government land is in general governed by block licences, permits, etc. Under the block licences, Government can order the private utility companies to carry out diversion works without any charge. The diversion/resiting of tram tracks of Hong Kong Tramways Limited and the associated posts and cables is an exception to this general rule.

(b) Existing Utility Services

The procedure for obtaining approval for the removal and/or diversion of existing services belonging to utility companies can be lengthy and may require the sanction of the Chief Executive-in-Council in exceptional circumstances. Engineers should therefore make the necessary arrangement and obtain agreement with the utility companies in concern at the earliest possible stage. Relevant Ordinances block licences and permits should be referred to if necessary.

(c) Utility Companies (The list may not be exhaustive)

(i) Public utility companies may include:

CLP Power Hong Kong Limited
Hong Kong Broadband Network Limited
Hong Kong Cable Television Limited
Hong Kong Tramways Limited
Hutchison Telecommunications (Hong Kong) Limited
i-Cable Communications Limited
Mass Transit Railway Corporation Limited
New T&T Hong Kong Limited
New World Telephone Limited
Pacific Century CyberWorks HKT
Rediffusion (HK) Limited
The Hong Kong & China Gas Company Limited
The Hong Kong Electric Company Limited

(ii) Government departments having utility installations may include:

Drainage Services Department
Highways Department
People's Liberation Army Hong Kong Garrison
through Security Bureau
Transport Department
Water Supplies Department

3.4 ENVIRONMENTAL CONSIDERATIONS

3.4.1 Aesthetics/Landscape

All the drainage works should be designed to blend in with the environment. Special attention should be paid to the aesthetic aspects of the structures and landscaping works. Landscape architect of the relevant office in CEDD, Arch SD or HyD may be consulted for advice on landscape treatment.

3.4.2 Natural Streams and Rivers

Natural streams and rivers are good habitats supporting a variety of wildlife and may have important ecological functions and carry high aesthetic and landscape values. Construction works should be restrained to minimize possible disturbance to the ecosystem. For projects that may affect natural streams or rivers, the project proponents should ensure that comments and advice received from AFCD and appropriate departments are incorporated into the planning, design and construction of the projects as far as practicable. If there is vegetation or landscaping features forming part of the mitigation requirements, the project proponent should also identify the maintenance party during the design stage. Designer should refer to DSD PN 1/2015 for more detailed guidelines on environmental considerations for river channel design.

3.4.3 Environmental Assessment

The necessity for and the extent of a Project Profile and an Environmental Impact Assessment (EIA) for stormwater drainage projects should be determined in accordance with the prevailing Government procedures. The Environmental Impact Assessment Ordinance (EIAO) was enacted on 4 February 1997 and came into operation on 1 April 1998. All projects and proposals that are covered under Schedule 2 or 3 of the EIA Ordinance shall follow the procedures as laid down in the Ordinance. In addition to the air, noise, dust and water aspects which are usually considered for most civil engineering works, issues such as dredging and disposal of contaminated mud and the impact of large-scale drainage works on the ecology of the surrounding areas should also require detailed assessment. Mitigating measures such as wetland compensation should be devised accordingly.

3.4.4 Environmental Nuisances

It should be well cognizant of the possibility that stormwater drainage systems in Hong Kong may, under certain circumstances, be contaminated by different pollution sources including sewage through expedient connections and hence giving rise to odour nuisance. Siltation and odour problems should therefore be considered at planning, design, construction and operation stages of stormwater drainage system, in particular where it is within the tidal zone or where significant pollution, such as discharge of livestock's waste into watercourses, channels, nullahs etc., is identified. Once expedient connection or pollution source is found in the stormwater drainage system, it should be reported to EPD and/or AFCD and relevant maintenance parties of the stormwater drainage system. Where a significant pollution source is identified, remediation at source is usually the most effective solution to curb the pollution impacts. However this may not be achieved by the relevant authorities within a short time, in which case identification and agreement of proper interim measures among all related parties to alleviate the impacts on operation and maintenance would be needed.

3.5 SITE INVESTIGATIONS

Reference should be made to GCO (1987) for guidance on good site investigation practice and GCO (1988) for guidance on description of rocks and soils in Hong Kong.

3.6 SAFETY ISSUES

Every project has its own particular and distinctive features (e.g. general arrangement/layout of the works, site location and constraints, accessibility of the works by the public, etc). It is necessary for the designer to identify all potential risks arisen from the proposed works and to design the works in such a way as to remove, reduce and/or control the identified hazards present during the course of construction, operation, maintenance, and finally decommissioning and demolition. In general, consideration should be given to the following aspects when carrying out risk assessment at the design stage:

(a) The anticipated method of construction – site constraints, technique involved, plant and materials to be used.

(b) The operation of works – warning signs, fencing, life buoys, grilles, means of emergency communication.

(c) The maintenance of the works – confined space, works on or near water, desilting, replacement of pumps, penstocks and flap valves, and diversion of flow for the pumping station.

(d) The decommissioning and demolition of the works – pre-stressed members, contaminated grounds.

Designer may refer to DSD (2010) or its latest version for information on the hazards of different types of works and the suitable control measures.

4. RAINFALL ANALYSIS

4.1 GENERAL

Rainfall analysis is based on historic or synthetic rainstorms. Historic rainstorms are usually applied in flood investigation or model calibration and synthetic rainstorms are commonly used in the planning and design of drainage systems.

4.2 HISTORIC RAINSTORMS

4.2.1 Applications

Historic rainstorms are used in actual storm event simulations, which are carried out in conjunction mostly with the calibration/verification of hydrological/hydraulic models, and with flood-forecast and post-event flood evaluations.

4.2.2 Point Rainfall

There are 193 operational rain gauge stations in Hong Kong, as summarized in Table 1. The locations of automatic reporting rain gauge (i.e. telemetered) and other conventional rain gauges which include ordinary and autographic types are indicated in Figure 2a and Figure 2b respectively. Some of the gauging stations may contain both ordinary and autographic (monthly) gauges at the same location.

The density of rain gauges in Hong Kong, approximately 5.7km^2 per station, is higher than the World Meteorological Organization’s minimum standards for urban areas of 10-20km^2 per station. Nevertheless, the variations of local rainfall are rather extreme both spatially and temporally, and additional rain gauges may still be needed for individual projects, either on long-term or short-term basis, for defining the areal rainfall.

4.2.3 Areal Rainfall

The areal rainfall of a sub-catchment or catchment should be derived from the records of a number of rain gauges based on an appropriate technique, such as the isohyetal method.

4.3 SYNTHETIC RAINSTORMS

4.3.1 Applications

For design purpose, synthetic rainstorms are recommended for adoption to simplify the planning, design and management of stormwater drainage systems. They are artificial design storms built upon statistics of the historic rainfall records. The commonly used statistical distribution models include, but are not limited to, Log-normal, Pearson Type 3, Log-Pearson Type 3, Generalized Extreme Value (GEV), Generalized Pareto, Generalized Logistic and Gumbel.

4.3.2 Variation of Rainfall

The mean annual rainfall from 1981 to 2010 in Hong Kong is about 2400mm. However, there are some variations in extreme rainfall across the Territory. For instance, Tai Mo Shan acquired with the highest mean annual rainfall of more than 3000mm. For some areas such as in North District, a relatively lower annual rainfall is recorded. It is revealed that orographic effect is the major reason for the large spatial variation of rainfall in Hong Kong. Similar pattern of variation has also been observed on different rainstorm durations. It is therefore recommended to adopt different synthetic rainstorms to reflect rainfall characteristics at various rainfall zones. The rainfall statistics at HKO Headquarters* are recommended for application in the whole Territory except Tai Mo Shan area, West Lantau area and North District area. Different design rainfall profiles are established for Tai Mo Shan area, West Lantau area and North District area. Delineation of rainfall zones is presented in Figure 3 and digital files of the rainfall zones can also be downloaded at DSD webpage.

4.3.3 Intensity-Duration-Frequency (IDF) Relationship

The rainfall statistics at HKO Headquarters* are recommended for general application (except Tai Mo Shan area, West Lantau area and North District area) because of its long-term and good quality records. The recommended IDF Relationship is based on the GEV distribution model, which is the best-fit model for different rainstorm durations on average and also adopted by HKO, in the frequency analysis of the annual maximum rainfall recorded at HKO Headquarters*. The relationships are presented in Table 2a and Figure 4a for various durations not exceeding 4 hours.

For Tai Mo Shan, West Lantau and North District areas, it is recommended to adopt the annual maximum rainfall for various durations recorded by the local rain gauges within the 3 areas in the statistical analysis. The distribution models which fit the respective durations the best are applied and regional frequency analysis of extreme rainfall has also been employed to develop the IDF Relationships. These relationships are presented in Tables 2b, 2c and 2d and Figures 4b, 4c and 4d for various durations not exceeding 4 hours.

The IDF data can also be expressed by the following algebraic equation for easy application:

$$i = \frac{a}{(t_d + b)^c}$$

where

i	=	extreme mean intensity in mm/hr,
t_d	=	duration in minutes ($t_d \leq 240$), and
a, b, c	=	storm constants given in Tables 3a, 3b, 3c and 3d.

* See Notes 2 & 3 of Table 2a

For durations exceeding 4 hours, the rainfall depth instead of the mean intensity is normally used. The Depth-Duration-Frequency (DDF) Relationships for duration exceeding 4 hours are given in Tables 4a, 4b, 4c and 4d. The IDF data can be generated by dividing rainfall depth with duration.

4.3.4 Storm Duration

The design rainstorm duration should make reference to the time of concentration or time to peak water level of the catchment under consideration as appropriate. The time of concentration is defined as the time for a drop of water to flow from the remotest point in the catchment to its outlet. For computational modeling analysis, a longer storm duration may be required if the recess arm of the hydrograph is required.

4.3.5 Design Rainstorm Profile

The time distribution of the design rainstorm should be taken as:

(a) For the Rational Method of runoff estimation, a uniformly distributed rainfall with an intensity determined by the IDF relationship should be used.

(b) For other methods of runoff estimation and for storm durations equal to or shorter than 4 hours, a symmetrically distributed rainfall is recommended with the following formulation based on RO (1991):

$$F(t) = \begin{cases} \dfrac{a[b+2(1-c)t]}{(2t+b)^{c+1}} & , \quad 0 \le t \le \dfrac{t_d}{2} \\ F(-t) & , \quad -\dfrac{t_d}{2} \le t \le 0 \end{cases}$$

where F(t) = rate of rainfall or instantaneous intensity in mm/hr at time t (in minutes)

t_d = rainstorm duration (in minutes) ($t_d \le 240$)

a, b, c = storm constants given in Tables 3a, 3b, 3c and 3d, which are the same as those given for the algebraic equation of the IDF relationship

The recommended rainstorm profiles for various return periods are given in Figures 5a, 5b, 5c and 5d and tabulation of the relationships are shown in Tables 5a, 5b, 5c and 5d. The connection between the tabulated data in Tables 5a, 5b, 5c and 5d and the curves in Figures 5a, 5b, 5c and 5d is elaborated in Figure 6.

For storm durations longer than 4 hours, the rainstorm profile can be derived from the IDF or DDF relationship for the portions outside the middle 4 hours.

4.3.6 Areal Reduction Factor

The design rainstorm profile relates to point rainfall only. The areal rainfall of a catchment can be obtained by multiplying the point rainfall with an areal reduction factor (ARF). DSD (1990) gave the following ARF based on a Depth-Area-Duration (DAD) analysis on local rainstorms:

Catchment Area *A (km²)*	ARF
≤ 25	1.00
> 25	$\dfrac{1.547}{(A+28)^{0.11}}$

4.3.7 Frequent Rainstorms

Sometimes, for the design of certain drainage components, rainfall with a frequency of more than once per year is used. The IDF data of such frequent rainstorms are given in Table 6**, according to Cheng & Kwok (1966).

** no recent research on frequent rainstorms has been carried out for updating

5. SEA LEVEL ANALYSIS

5.1 GENERAL

Sea level forms the downstream hydraulic boundary condition of stormwater drainage systems.

5.2 HISTORIC SEA LEVELS

5.2.1 Applications

Historic sea levels are used in actual event simulations for the calibration and verification of hydraulic models.

5.2.2 Data Availability

There are 15 operational tide gauges in Hong Kong managed by the Hong Kong Observatory, the Hydrographic Office of Marine Department, the Airport Authority and the Drainage Services Department. Brief particulars of the tide gauges are given in Table 7 and their locations are shown in Figure 7.

Tidal data are normally recorded in Chart Datum (CD) which is 0.146 m below Principal Datum (PD). The relationship between the two datums can be represented as

$$\text{mCD} = \text{mPD} + 0.146 \text{ m}$$

where mCD is metre above Chart Datum and mPD is metre above Principal Datum.

5.2.3 Astronomical Tides

Tides arise from the gravitational attractions of the moon and the sun on the sea water masses. Periodic hourly tidal fluctuations are mainly due to the moon's effect. Tides in Hong Kong are of the mixed dominantly semi-diurnal type with significant daily inequality. Daily tidal fluctuations throughout the month are due to the combined effect of the moon and the sun, with spring tides at new and full moons, and neap tides at the first and last quarters. Each year HKO publishes a tide table giving the astronomical tide predictions (based on a Harmonic Analysis) for most of the operational tide gauge stations.

5.2.4 Storm Surges

The predicted astronomical tides are based on normal meteorological conditions, and the observed tides may differ from those predicted when the conditions deviate from the normal. For instance, during tropical cyclones, such differences (i.e. storm surges) can be large.

A storm surge is induced by a low pressure weather system. The sea level rises through barometric suction. The associated wind field also piles up water through surface friction (wind set-up). Other factors affecting storm surges include the Coriolis Effect, coastline configuration and sea bed bathymetry.

During tropical cyclones, HKO predicts the height of storm surge at Hong Kong using the SLOSH (Sea, Lake and Overland Surges from Hurricanes) storm surge model developed by the National Oceanic and Atmospheric Administration (NOAA) of USA.

5.3 SYNTHETIC SEA LEVELS

5.3.1 Applications

Synthetic sea levels are recommended for adoption to simplify stormwater drainage planning, design and management.

5.3.2 Design Extreme Sea Levels

Table 8 shows the design extreme sea levels at North Point/Quarry Bay, Tai Po Kau, Tsim Bei Tsui and Tai O, based on the GEV distribution model, with the parameters estimated by the Method of L-moments. In comparison to the Gumbel Distribution with only 2 distribution parameters traditionally adopted in Hong Kong, the flexible 3-parameter GEV Distribution is more superior to describe the probability distribution of the extreme sea levels. The Mean Higher High Water (MHHW) levels for the 4 tidal stations are shown in Table 9. The data have been converted to mPD for easy application.

5.3.3 Design Sea Level Profile

For simplicity, the sea level should be assumed constant with time.

6. FLOOD PROTECTION STANDARDS

6.1 GENERAL

Flood protection standard is generally defined as the design standard for drainage system that is adequate to accommodate a T-year flood, whereas T is the design return period of the flood event. Appropriate flood protection standards should be chosen to suit the type, category and design life of the drainage systems. Definitions of stormwater drainage systems are discussed in Section 6.6. The consequential losses ranging from major casualties to minor inconvenience to daily life due to inadequate flood protection standards should be carefully considered in major improvement works. Suitable freeboard and reduction in flow capacity due to sedimentation should be allowed in flood level computations.

6.2 DESIGN RETURN PERIODS

Ideally, the choice of a design return period should be based on an economic evaluation in which the costs of providing the drainage works are compared with the benefits derived. However, comprehensive local flood damage data are normally not available to the degree of precision required for cost-benefit analysis. For this reason, a general policy decision based on such considerations as land use, hazard to public safety and community expectations is more appropriate. Table 10 gives the recommended design return periods based on flood levels.

Admittedly, for new drainage systems or drainage upgrading in some existing areas, particularly low lying ones or those in congested urban locations, the recommended standards may not be suitable or achievable. A pragmatic approach should be considered.

In general, the recommended standards should be followed in carrying out the design of temporary works and temporary flow diversion with a view to ensuring the proposed works will not cause an unacceptable increase in flood risk. Notwithstanding this, DSD in assessing acceptability of any increase in level of flood risk may take into account relevant factors such as whether the increase in level of flood risk can be practically and cost-effectively mitigated in a commensurate manner. The designer should refer to the Drainage Services Department Technical Circular No. 1/2017 or the prevailing technical circulars / guidelines in formulating the temporary works and temporary flow diversion.

6.3 PROBABILITY OF DESIGN FAILURE

It should be noted that a drainage system designed for a T-year return period event does not mean that its capacity will only be exceeded once in every T years. Suppose the drainage system has a design life of L years, the probability (P) of the system's capacity being exceeded at least once over its design life is given by:

$$P = 1 - (1 - \frac{1}{T})^L$$

For instance, for a drainage system designed for a 50-year design life and a 200-year return period, there is a 22% chance of flooding at least once during the design life. For the same design life, the chance is 64% for a system sized for a 50-year return period.

6.4 DEFINITION OF FLOOD LEVELS

The following approximate pragmatic rule for determining the T-year flood level in the fluvial-tidal zone of a drainage system is recommended. The T-year flood level is taken as the higher of those flood levels due to the following two cases:

Case I a T-year sea level in conjunction with a X-year rainfall,

Case II a X-year sea level in conjunction with a T-year rainfall.

In the above rule,

X = 10, when T = 50, 100 or 200
X = 2, when T = 2, 5 or 10

The design return periods for combined rain and tide events are tabulated in Table 11 for easy reference.

6.5 FREEBOARD

The freeboard is the vertical distance between the crest of a river embankment, or manhole cover level in the case of an urban drainage system, and the design flood level. Freeboard should be provided to cover super-elevations at bends, wave run-ups, etc. For normal condition, a 200mm allowance is generally considered adequate to cover super-elevations at bends and wave run-ups if both apply. For locations where excessive super-elevations at bends and wave run-ups are expected, these shall be assessed separately. Allowance should also be made for ground settlement and bank erosion if considered necessary. In addition to other allowances made, a margin of safety (300 mm minimum) is recommended to account for inaccuracies in flood level computations. Sediment thickness at the bed should be excluded from the freeboard calculation and provision for such thickness may be achieved through a lower design bed level. For the amount of sedimentation in stormwater drains, please refer to Section 9.3.

6.6 STORMWATER DRAINAGE SYSTEMS

Table 10 stipulates the flood protection standards for five categories of stormwater drainage systems according to the nature of catchment served or the hierarchy of the drains within the overall drainage system. These are:

(a) Intensively Used Agricultural Land
(b) Village Drainage including Internal Drainage System within a Polder Scheme
(c) Main Rural Catchment Drainage Channels

(d) Urban Drainage Trunk Systems
(e) Urban Drainage Branch Systems

While the meaning of Agricultural Land in Category (a) is self-evident, those of the others are outlined below.

6.6.1 Village Drainage and Main Rural Catchment Drainage Channels

'Village Drainage' refers to the local stormwater drainage system within a village. A stormwater drain conveying stormwater runoff from an upstream catchment but happens to pass through a village may need to be considered as either a 'Main Rural Catchment Drainage Channel' or 'Village Drainage', depending on the nature and size of the upstream catchment. In any case, the impact of a 50-year event should be assessed in the planning and design of village drainage system to check whether a higher standard than 10 years is justified.

6.6.2 Urban Drainage Branch and Urban Drainage Trunk Systems

The classification is basically a hierarchical grouping of the drainage network for assigning the flood protection standards according to the perceived importance of the individual drainage system. The higher standard is needed for a trunk drain because it conveys higher flows and comparatively serious damage or even loss of life could occur if it floods. In addition, any surcharge in a trunk drain will prevent the adjoining branch drains from draining the catchment and discharging the stormwater into the trunk drain effectively.

An 'Urban Drainage Branch System' is defined as a group or network of connecting drains collecting runoff from the urban area and conveying stormwater to a trunk drain, river or sea. For a simple definition, the largest pipe size or the equivalent diameter in case of a box culvert in a branch system will normally be less than 1.8 m.

An 'Urban Drainage Trunk System' collects stormwater from branch drains and/or river inlets, and conveys the flow to outfalls in river or sea. Pipes with size or box culverts with an equivalent diameter equal to or larger than 1.8 m are normally considered as trunk drains. It is however noted that small catchments do not necessarily have to have a trunk drain at all.

Notwithstanding the above delineation of Urban Drainage Trunk and Branch Systems, reference should always be made to the relevant Drainage Master Plan Study. In view of the ongoing evolvement of drainage development and requirements, advice from DSD should be sought in case of doubt.

For the design of gully system for road pavements, reference should be made to HyD (2010).

6.7 INTERFACE WITH RESERVOIRS/CATCHWATERS

When part of a drainage basin is a WSD catchment, the stormwater drainage should be designed for the greater of the following:

(a) maximum runoff assuming the absence of the WSD catchwaters, and

(b) runoff from the catchment excluding the part of the WSD catchment but include the estimated overflows from the catchwaters and reservoir spillways as provided by WSD.

6.8 DESIGN CONSIDERATIONS OF RAINFALL AND SEA LEVEL DUE TO CLIMATE CHANGE

In Hong Kong, rainfall is projected to increase and mean sea level is projected to rise under climate change. Excess rainfall will overload the stormwater drainage system. Sea level rise will increase the flood risk in low-lying and coastal areas. The Hong Kong Observatory utilized the results of simulations made by global climate models in the IPCC Fifth Assessment Report to conduct a projection of rainfall increase and sea level rise due to climate change in Hong Kong up to the late 21st century.

In general, new drainage provision or developments with potential drainage impact should consider the climate change effects up to mid 21st century. A longer projection year up to the end of 21st century should be adopted if the marginal benefit of the design outweighs the marginal cost. It should be well justified if the above criteria could not be achieved.

To consider the effect of climate change in the drainage design, the projection of rainfall increase percentage and sea level rise given in Table 28 should be added to the respective design rainfall intensities/synthetic rainstorm profiles and design extreme sea levels given in the corresponding figures and tables in this manual.

The projection of rainfall increase and sea level rise comes up from a number of variables and research technology available at the time of assessment and it will be regularly reviewed based upon the latest IPCC Assessment Report or other relevant studies. The designer should make reference to the latest findings of climate change assessment promulgated by relevant authorities and, appropriately follow the prevailing design standards and guidelines.

7. RUNOFF ESTIMATION

7.1 GENERAL

Methods to estimate runoff from single storm event can be based on statistical or deterministic approaches. The common deterministic methods are the rational method, the time-area method, the unit hydrograph method and the reservoir routing method. The two fundamental pre-requisites for any reliable runoff estimates are good and extended rainfall/ evapotranspiration data and adequate calibration/verification of the rainfall-runoff model parameters by sufficient number of gauging stations.

7.2 DATA AVAILABILITY

7.2.1 Rainfall

The rain gauge network in Hong Kong is described in Section 4.2.2. Design rainstorms are described in Section 4.3.

7.2.2 Evaporation/Evapotranspiration

Daily evaporation data are measured at the HKO King's Park Meteorological Station. Three lysimeters to measure potential evapotranspiration are also available at the station.

7.2.3 Streamflow

WSD operates a network of stream flow gauges for water resources planning purposes. The locations of these gauges are given in Figure 8. Flow-Duration curves are available in the Annual Report on Hong Kong Rainfall and Runoff – WSD (annual). Rating curves for the gauges can be obtained from WSD. Locations of DSD’s river stage gauges are also shown in Figure 8. These DSD gauges are primarily for flood monitoring.

7.3 NEED FOR CALIBRATION/VERIFICATION

7.3.1 Choice of Runoff Estimation Method

There is no single preferred method for runoff estimation. A chosen model for any given application should be calibrated and validated with rainfall-runoff data, whenever possible.

7.3.2 Flow Gauging Methods

The commonly used flow gauging methods are listed below:

(a) Velocity-Area Method
(b) Slope-Area Method
(c) Weirs and Flumes

(d) Dilution Method
(e) Ultrasonic Method
(f) Electromagnetic Method
(g) Float Gauging Method

Details of various methods of flow gauging are described in Herschy, R.W. (1985).

7.3.3 Practical Difficulties

There are practical difficulties in gauging the runoff in drainage systems within floodplains and in those subject to tidal influence. In the former, the flow cross-section may be too wide for flow gauging to be practical. In the latter case, the discharge in the drainage system may be affected by the sea level as well as rainfall. In both cases, the parameters in the rainfall-runoff model have to be carefully calibrated and validated in the hydraulic modeling process.

7.4 STATISTICAL METHODS

The statistical approach to runoff estimation can give good results if the streamflow records are long enough. The limitation is that it only gives the peak of the runoff and not the whole hydrograph. Also, runoff may be subject to changes by urbanization and drainage improvements. Such changes can better be estimated by Deterministic Methods.

The statistics on the streamflow records are expressed in the form of a frequency analysis of the flow data. This relates the magnitude of flows to their frequency of occurrence through the use of probability distributions. The flow data series can be treated in the following manner:

(a) *A Complete Duration Series.* This consists of all the data available.

(b) *A Partial Duration Series.* This consists of data which are selected so that each is greater than a predefined threshold value.

(c) *An Annual Maximum Series.* This consists of the maximum value recorded in each year.

A complete duration series is commonly used in low-flow analysis and estimation of frequent events. On the other hand, for extreme events, the frequency analysis is normally done on the partial duration series for shorter records and on the annual maximum series for longer records. Full details of frequency analysis are given in Chow, Maidment & May (1988).

7.5 DETERMINISTIC METHODS

7.5.1 Introduction

Deterministic methods are based on a cause-effect consideration of the rainfall-runoff processes. Such methods are used when:

(a) There are limited streamflow records for frequency analysis. Runoff data have to be generated from rainfall data which are usually more plentiful.

(b) There are changes to the rainfall-runoff responses due to land use changes, drainage improvements, etc. which have upset the homogeneity of the streamflow data. This has introduced complications to the statistical analysis of the data.

(c) A runoff hydrograph is required. However, it should be noted that the Rational Method is normally used to estimate the peak runoff but cannot provide a runoff hydrograph.

Commonly used deterministic methods are outlined below.

7.5.2 Rational Method

The Rational Method dates back to the mid-nineteenth century. Despite valid criticisms, it is a traditional method for stormwater drainage design because of its simplicity. Once the layout and preliminary sizing of a system has been determined by the Rational Method, the design can be refined by dynamic routing of the flow hydrographs through the system. Details on the application of the Rational Method are described below:

(a) *Basic Formulations.* The idea behind the Rational Method is that for a spatially and temporally uniform rainfall intensity i which continues indefinitely, the runoff at the outlet of a catchment will increase until the time of concentration t_c, when the whole catchment is contributing flows to the outlet. The peak runoff is given by the following expression:

$$Q_p = 0.278\, C\, i\, A$$

where Q_p = peak runoff in m^3/s
C = runoff coefficient (dimensionless)
i = rainfall intensity in mm/hr
A = catchment area in km^2

For a catchment consisting of m sub-catchments of areas A_j (km^2) each with different runoff coefficients C_j, the peak runoff at the drainage outlet is given by the following expression:

$$Q_p = 0.278 i \sum_{j=1}^{m} C_j A_j$$

Due to the assumptions of homogeneity of rainfall and equilibrium conditions at the time of peak flow, the Rational Method should not be used on areas larger than 1.5 km^2 without subdividing the overall catchment into smaller catchments and including the effect of routing through drainage channels. The same consideration shall also be applied when ground gradients vary greatly within the catchment.

(b) *Runoff Coefficient.* C is the least precisely known variable in the Rational Method. Proper selection of the runoff coefficient requires judgement and experience on the part of the designer. The value of C depends on the impermeability, slope and retention characteristics of the ground surface. It also depends on the characteristics and conditions of the soil, vegetation cover, the duration and intensity of rainfall, and the antecedent moisture conditions, etc. In Hong Kong, a value of C = l.0 is commonly used in developed urban areas.

In less developed areas, the following C values may be used but it should be checked that the pertinent catchment area will not be changed to a developed area in the foreseeable future. Particular care should be taken when choosing a C value for unpaved surface as the uncertainties and variability of surface characteristics associated with this type of ground are known to be large. It is important for designer to investigate and ascertain the ground conditions before adopting an appropriate runoff coefficient. Designers may consider it appropriate to adopt a more conservative approach in estimation of C values for smaller catchments where any consequent increase in cost may not be significant. However, for larger catchments, the designers should exercise due care in the selection of appropriate C values in order to ensure that the design would be fully cost-effective.

Surface Characteristics	*Runoff coefficient, C**
Asphalt	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Grassland (heavy soil**)	
Flat	0.13 - 0.25
Steep	0.25 - 0.35
Grassland (sandy soil)	
Flat	0.05 - 0.15
Steep	0.15 - 0.20

* For steep natural slopes or areas where a shallow soil surface is underlain by an impervious rock layer, a higher C value of 0.4 - 0.9 may be applicable.

** Heavy soil refers to fine grain soil composed largely of silt and clay

(c) *Rainfall intensity*. i is the average rainfall intensity selected on the basis of the design rainfall duration and return period. The design rainfall duration is taken as the time of concentration, t_c. The Intensity-Duration-Frequency Relationship is given in Section 4.3.2.

(d) *Time of concentration.* t_c is the time for a drop of water to flow from the remotest point in the catchment to its outlet. For an urban drainage system,

$$t_c = t_o + t_f \qquad\qquad t_f = \sum_{j=1}^{n} \frac{L_j}{V_j}$$

where t_o = inlet time (time taken for flow from the remotest point to reach the most upstream point of the urban drainage system)

t_f = flow time

L_j = length of j^{th} reach of drain

V_j = flow velocity in j^{th} reach of drain

The inlet time, or time of concentration of a natural catchment, is commonly estimated from empirical formulae based on field observations. In Hong Kong, the Brandsby William's Equation is commonly used:

$$t_o = \frac{0.14465 L}{H^{0.2} A^{0.1}}$$

where t_o = time of concentration of a natural catchment (min.)

A = catchment area (m^2)

H = average slope (m per 100 m), measured along the line of natural flow, from the summit of the catchment to the point under consideration

L = distance (on plan) measured on the line of natural flow between the summit and the point under consideration (m)

7.5.3 Time-Area Method

This method is modified from the Rational Method. It consists of the combination of a rainstorm profile with an incremental time-area diagram. Given a rainstorm profile in which the average rainfall intensities within successive time increments are i_1, i_2, i_3, the successive ordinates of the runoff hydrograph can be written as:

$$Q_1 = 0.278\ C\ i_1 A_1$$

$$Q_2 = 0.278\ (C\ i_1 A_2 + C\ i_2 A_1)$$

$$Q_3 = 0.278\ (C\ i_1 A_3, + C\ i_2 A_2 + C\ i_3 A_1) \qquad \ldots \text{ etc.}$$

where C = runoff coefficient
A_1, A_2, etc = successive increments of the time-area diagram

The above formulation is the basis of the Hydrograph Method in Watkins (1962) used in the United Kingdom for urban drainage design since it was published in the first edition of Road Note No. 35 in 1963. Flow routing in pipes was later incorporated in the second edition of Road Note No. 35 in 1974.

7.5.4 Unit-Hydrograph Method

The classical theory of unit hydrograph refers to the relationship between net rainfall and direct runoff. The catchment is treated as a black box with the net rainfall as input and the direct runoff as response. If the input is a uniform net rainfall with a duration t_{dur} and a unit depth, the response is the t_{dur} - unit hydrograph. Moreover, the system is considered linear and time-invariant. The direct runoff due to any net rainfall with different depths for successive increments of t_{dur} is obtained by linear superposition of the responses of the various net rainfall depths at each increment of t_{dur}. This process is called convolution. The direct runoff is added to the base-flow to give the total runoff. Application of the Unit Hydrograph Method requires:

(a) *Loss Model.* There are 3 classical methods of determining the net rainfall hyetograph from the rainfall hyetograph:

(i) initial loss + constant rate of losses (∅-index)

(ii) initial loss + continuous losses (for example, SCS losses)

(iii) constant proportional losses (runoff coefficient)

The loss model parameters can be derived from rainfall-runoff data. The above three methods are explained with worked examples in Chow, Maidment & May (1988).

(b) *Unit Hydrograph.* The unit hydrograph for a catchment can be derived from rainfall-runoff monitoring. For an ungauged catchment, the unit hydrograph may be derived synthetically from known unit hydrographs of gauged catchments of similar characteristics. In Hong Kong, the WSD mean dimensionless unit hydrograph was developed for upland catchments. Details are given in PWD (1968). Other examples of synthetic unit hydrographs are those according to Soil Conservation Service (1972).

7.5.5 Reservoir Routing Methods

The net rainfall-direct runoff routing can be looked at as a reservoir routing process with the inflow (I) due to the net rainfall falling on the catchment and the outflow (Q) as the direct runoff from the catchment. The flood storage volume (S) in the catchment is assumed to be a function of the outflow. Linearity of this function determines whether the reservoir is linear or non-linear. Moreover, the reservoir can either be single or a series of reservoirs in cascade. Examples of Reservoir Routing Methods are the Australian RORB model for rural catchment (Laurenson & Mein (1986)) and the hydrological component in the Wallingford Procedure (HRL (1983)) for urban catchment. As with unit hydrograph method, reservoir routing methods need to work in conjunction with appropriate loss models.

8. HYDRAULIC ANALYSIS

8.1 GENERAL

Hydraulic analysis for drainage planning or design makes use of the runoff results of the various subcatchments and the characteristics of the drainage system to determine flood levels throughout the system. In the tidal reaches of the system, flood levels are also affected by the downstream boundary condition at the drainage outfall as defined by a sea level analysis.

8.2 FLOW CLASSIFICATIONS

8.2.1 Laminar vs Turbulent Flow

Laminar flow is characterized by fluid moving in layers, with one layer gliding smoothly between the adjacent layers. In turbulent flow, there is a very erratic motion of fluid particles, with mixing of one layer with the adjacent layers. Nearly all practical surface water problems involve turbulent flow. The Reynolds Number (Re) is used to distinguish whether a flow is laminar or turbulent.

$$\mathrm{Re} = \frac{\bar{V}R}{\nu}$$

where	R	=	hydraulic radius (m)
		=	A/P
	ν	=	kinematic viscosity (m^2/s)
	A	=	cross-section area of flow (m^2)
	P	=	wetted perimeter (m)
	$\bar{V}$	=	cross-sectional mean velocity (m/s)

The transition from laminar to turbulent flow happens at

$$\mathrm{Re} = 500 \text{ to } 2{,}000$$

It is considered to be good practice to check the value of Re before applying the laws of turbulent flow.

8.2.2 Surcharge vs Free-surface Flow

In surcharged flow (or pipe flow), the whole conduit conveys flow and there is no free surface. The flow cross-section is the cross-section of the conduit and this does not vary with the flow. In free surface flow (or open channel flow) which predominates in stormwater drainage systems, there exists a free surface and the hydraulic cross-section varies with the flow.

8.2.3 Subcritical vs Supercritical Flow

In open channel flow, it is important to compare the mean flow velocity ($\overline{V}$) and the surface wave celerity (c). The Froude Number (Fr) is defined as:

$$Fr = \frac{\overline{V}}{c}$$

$$= \frac{\frac{Q}{A}}{\sqrt{\left(\frac{gA}{\alpha B}\right)}}$$

$$\therefore Fr^2 = \frac{\alpha B Q^2}{g A^3}$$

where α = Coriolis Coefficient (or Energy Coefficient)
B = width of free surface (m)
Q = discharge (m^3/s)
A = flow area (m^2)
g = acceleration due to gravity (m/s^2)

When Fr < 1, the flow is subcritical and a wave disturbance can travel both upstream and downstream.

When Fr = 1, the flow is critical. Critical flow has a minimum energy for a given discharge or a maximum discharge for a given energy.

When Fr > l, the flow is supercritical and a wave disturbance can only travel downstream.

Chow, V. T. (1959) quotes values of α.

8.2.4 Steady vs Unsteady Flow

In steady flows, flow conditions (viz. discharge and water level) vary with the position only. In unsteady flow, flow conditions vary with position as well as time. Steady flow can be either uniform or non-uniform.

8.2.5 Uniform vs Non-uniform Flow

If the flow is also independent of position, the flow is uniform. Otherwise, the flow is non-uniform.

8.2.6 Gradually Varied vs Rapidly Varied Non-uniform Flow

In non-uniform flow, if the flow conditions vary slowly with location and bed friction is the main contribution to energy losses, the flow is gradually varied. Otherwise, the flow is rapidly varied.

8.3 UNIFORM FLOW

8.3.1 Frictional Resistance Equations

Most of these equations apply to turbulent uniform flow in open channels. The common equations are given in Table 12 using a consistent set of notations. All the equations are converted to the Chézy form for easy comparison.

The notations are:

$\overline{V}$	=	cross-sectional mean velocity (m/s)
R	=	hydraulic radius (m)
S_f	=	friction gradient (dimensionless)
C	=	Chézy coefficient ($m^{½}/s$)
n	=	Manning coefficient ($s/m^{1/3}$)
f	=	Darcy-Weisbach friction factor (dimensionless)
k_s	=	surface roughness (m)
ν	=	kinematic viscosity (m^2/s)
g	=	acceleration due to gravity (m/s^2)
C_{HW}	=	Hazen-William coefficient (dimensionless)

Amongst the equations in Table 12, Manning and Colebrook-White are the most popular in local applications. Design values of n and k_s are given in Tables 13 and 14 respectively[1]. Manning equation is more convenient to work with in open channel flow calculations. Colebrook-White equation is presented in design charts in HR Wallingford (2006). In Table 14, the term "sewer" should include both sanitary sewers and stormwater drains with possible polluted flow.

8.3.2 Compound Roughness

Suppose the flow area is divided into N sub-sections of which the wetted perimeters $P_1, P_2, ..., P_N$ and areas $A_1, A_2, ..., A_N$ are known. If the corresponding Manning roughness coefficients are $n_1, n_2, ..., n_N$, the equivalent roughness coefficient is

$$n = \frac{\dfrac{A^{5/3}}{P^{2/3}}}{\sum \dfrac{A_i^{5/3}}{n_i P_i^{2/3}}}$$

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[1] H.R. Wallingford Ltd., Barr, D.I.H. and Thomas Telford Ltd. are acknowledged for their consent to the reproduction of the Table on Recommended Roughness Values in the publication "Table for the Hydraulic Design of Pipes, Sewers and Channels, 8th Edition (2006)" in Table 14 of this Manual.

If the surface roughnesses are k_1, k_2, ..., k_N, the equivalent surface roughness is

$$k_s = \frac{\sum P_i k_i}{P}$$

8.3.3 Partially Full Circular Sections

Charts for partially full circular sections are available for both Manning and Colebrook-White equations. See Chow, V.T. (1959) and HRL (1990).

8.4 GRADUALLY VARIED NON-UNIFORM FLOW

The determination of free-surface profiles in non-uniform flow is of fundamental importance in steady-state hydraulic analysis. An example is the backwater curve analysis in the tidal section of a drainage system. The computational methods are summarized below. For simplicity, the Manning equation is used below to describe the frictional gradient. All local losses have been ignored.

8.4.1 Basic Formulations

Using the definition sketch at Figure 9, the basic equation is:

(a) In differential form:

$$\frac{dy}{dx} = \frac{s_o - s_f}{1 - Fr^2}$$

$$= \frac{s_o - \dfrac{Q^2 n^2}{A^2 R^{4/3}}}{1 - \dfrac{\alpha Q^2 B}{g A^3}}$$

(b) Alternatively, in finite difference form:

$$s_o \Delta x + y_1 + \frac{\alpha_1 \overline{V}_1^2}{2g} = s_f \Delta x + y_2 + \frac{\alpha_2 \overline{V}_2^2}{2g}$$

or

$$s_o \Delta x + y_1 + \frac{\alpha_1 Q^2}{2g A_1^2} = s_f \Delta x + y_2 + \frac{\alpha_2 Q^2}{2g A_2^2}$$

8.4.2 Types of Flow Profiles

Depending on the bed slope and the flow depth, 13 types of flow profiles can be distinguished. The classification is described in Chow, V.T. (1959).

8.4.3 Solution Techniques

First, it is necessary to establish the normal flow depth (y_n) and critical flow depth (y_c) by solving:

$$s_o = \frac{Q^2 n^2}{A_n^2 R_n^{4/3}}$$

and

$$1 = \frac{\alpha Q^2 B_c}{g A_c^3}$$

Next, it is important to distinguish:

(a) For supercritical flows, the calculation starts at some known section at the upstream side and proceeds in a downstream direction.

(b) For subcritical flows, the calculation starts at some known section at the downstream side and proceeds in an upstream direction.

Finally, as for the actual calculations, these can be:

(a) a numerical integration of the differential equation above, or

(b) solution of the unknown flow section from the known one using the difference equation above and the Standard Step Method or the Direct Step Method.

8.5 RAPIDLY VARIED NON-UNIFROM FLOW

8.5.1 General

Rapidly varied non-uniform flows are commonly encountered in drainage applications. This complex subject is treated thoroughly in Chow, V.T. (l959). Examples of such flows happen at:

(a) Weirs or spillways
(b) Gates
(c) Sudden channel expansions or contractions
(d) Hydraulic jumps
(e) Bends
(f) Stepped channels
(g) Channel junctions
(h) Constrictions due to bridge piers, culverts, etc.

Energy dissipators should be provided at transitions from supercritical flows to subcritical flows.

For important applications in rapidly varied flows, physical modelling is recommended to verify or refine the design.

8.5.2 Rapidly Varied Supercritical Flows

Rapidly varied supercritical flows may involve large changes in momentum with large changes in flow depth, formation of waves and vortices, aeration, etc. and may result in splashing, overflow, the flow flying into the air, rapid erosion of the pipeline/channel, etc. These effects should be duly taken into account in the analysis, and measures should be taken to reduce such risks. Some of these measures are as follows. Further guidelines are given in DSD Practice Note No. 3/2003.

(a) Dissipate the energy by means of hydraulic jump, stepped channel, stilling basin, etc.

(b) Measures to convey the flow smoothly at bends and junctions :

  (i) Horizontal bends should have radius not less than three times the width of the channel (for velocity of flow up to 2 m/s). For flows of higher velocity and/or bends of large angle, flow separation may occur at the inner bend and significant increase in flow depth may occur at the outer bend. In addition, shockwaves, choking phenomenon and spiralling of flow (in pipes) may result. For channels, increase in wall height at the outer bend (Vischer & Hager) or cover-up of channel top may be required. Chokage and flow spiralling may also need to be addressed;

  (ii) Guide walls, bends and transitions should be provided at junctions between channels to allow the flow from different incoming channels to turn and merge smoothly;

  (iii) For vertical bends in which the downstream gradient is suddenly increased, transitions of ogee curve or circular curve (radius >3H, H being the specific energy head at the bend concerned) should be provided to prevent high velocity flow from shooting into the air;

  (iv) The side walls of the channel should be sufficiently high to prevent overshooting/spillage, especially at positions where change in gradient/ flow direction occur, where obstructions exist, or where there are other circumstances creating turbulence, aeration, splashing, etc.

(c) Drainage channels/conduits should be designed to resist possible erosion under the anticipated velocities, undermining by scour and uplift forces due to high velocity over the channel/conduit surface.

(d) If hydraulic jump occurs, the jump should be contained to where it is designed to occur, and in no circumstances be allowed to occur at an erodible section of the channel/conduit. The position of the jump may be stabilized by means of physical controls. The channel/conduit should preferably have rectangular section at the position of the jump to simplify analysis.

(e) Bridge piers or other obstructions (including trash grilles) should not be placed inside channels with high velocity supercritical flow. If the flow velocity is not too high, piers/obstructions may be allowable if they have streamlined sections and are properly designed so as not to create unwanted hydraulic jump, cause overshooting of flow or trap floating debris/vegetation.

(f) If the gradient is very steep (inclination to horizontal greater than, say, 65^o) and there is insufficient space to allow for splashing etc., surface channel/conduit may be replaced by a downpipe or covered up on top. In such case, the downpipe/covered channel should be designed to prevent blockage and to facilitate inspection/clearance if necessary.

(g) If soil, boulders and debris may be washed down due to erosion or landslide especially during rainstorms, erosion protection measures such as lining the embankments and slopes with concrete or shotcrete, use of gabions, erection of retaining walls, tree planting, hydroseeding and provision of check dams should be adopted where appropriate.

8.5.3 Stepped Channel

Stepped channels are commonly used to convey flow along slopes. They are effective in dissipating the energy and in reducing the velocity of the flow. Flow in step channels can be classified into 3 regimes :

(a) Nappe flow regime – The water drops freely at each step, sometimes with a hydraulic jump. The nappe flow regime occurs in low flows or flow at slopes of flatter gradient.

(b) Skimming flow regime – The water flows down in a coherent stream skimming over the steps cushioned by recirculating vortices at the steps and significant air entrainment. The skimming flow regime occurs in high flows or in step channels of steeper gradient.

(c) Transition flow regime – Change from the nappe flow to skimming flow regime due to increase in flow or increase in slope will pass through the transition flow regime. Significant spray is present and the flow pattern may vary significantly from step to step in transition flow regime.

GEO Technical Guidance Notes No. 27 (TGN 27) provides a formula for checking the type of flow regime applicable to the design and the situation concerned.

Stepped channels should be designed according to GEO TGN 27 in general. For stepped channels under the nappe flow regime or transition flow regime, reference should be made to Chanson (1994) and Chanson (2002). Stepped channels of width greater than 900 mm and under the skimming flow regime should be designed in accordance with Annex TGN 27 A2 of TGN 27 except that :

(a) The discharge per unit channel width q_w should not be greater than 10 m^2/s (Chanson & Toombes);

(b) The minimum L/h ratio and the minimum channel length L in order to establish uniform aerated flow should be found from the following formula based on Chanson (2002)

$$\left(\frac{L}{h}\right)_{min} = \cos\propto \left(\frac{1}{0.1193\cos\alpha\,(\sin\alpha)^{0.259}}\,\frac{d_c}{h}\right)^{1/0.935}$$

where L = length of stepped channel
h = channel step height
α = channel angle to the horizontal
d_c = critical flow depth for the given discharge per unit width
= $\sqrt[3]{q_w^2/g}$
q_w = discharge per unit channel width

In addition :

(a) The possibility of jet deflection at the crest of the channel, i.e., the flow shooting out as a free-falling jet bypassing the steps, should be checked. For the flow to remain on the steps, the following equation should be satisfied :

$$\frac{d_c}{h} < \frac{Fr_b^{\,2/3}\sqrt{1+\frac{1}{Fr_b^{\,2}}}}{1+2\,Fr_b^{\,2}\left(1+\frac{1}{Fr_b^{\,2}}\right)^{3/2}\left(1-\frac{\cos\alpha_b}{\sqrt{1+\frac{1}{Fr_b^{\,2}}}}\right)}$$

where Fr_b = Froude no. at top end of the first step
= $V_b/\sqrt{g\,d_b}$
d_c = critical flow depth for the given discharge per unit width
= $\sqrt[3]{q_w^2/g}$
q_w = discharge per unit channel width
h = channel step height
α = channel angle to the horizontal

If the above condition is not satisfied, the step height h of the first few steps should be reduced.

(b) If the residual head and the Froude no. of the flow at the bottom of the stepped channel are still high, additional energy dissipation device should be provided downstream of the stepped channel.

The residual head H_{res} at the bottom of a long stepped channel in which uniform aerated flow is reached can be found from the following formula:

$$H_{res} = d_c\left[\left(\frac{f_e}{8\ \sin\propto}\right)^{1/3}\cos\propto + \frac{1}{2}\left(\frac{f_e}{8\ \sin\alpha}\right)^{-2/3}\right]$$

where d_c = critical flow depth for the given discharge per unit width

= $\sqrt[3]{{q_w}^2/g}$

q_w = discharge per unit channel width

α = channel angle to the horizontal

f_e = Darcy's friction factor of aerated flow

$$= 0.5f\left\{1 + \tanh\left[0.628\frac{0.51 - C_e}{C_e(1 - C_e)}\right]\right\}$$

where tanh(x) is the hyperbolic tangent function, i.e. $\tanh(x) = (e^x - e^{-x}) / (e^x + e^{-x})$;

f is the Darcy's friction factor of non-aerated flow. It is recommended to use f = 1.0 as an order of magnitude of the friction factor.

(c) For stepped channels which are curved or with intermediate breaks / intermediate branch connections, the design guidelines above may not be applicable. In such case, physical model test may need to be carried out for hydraulic design.

The details of stepped channels should be in accordance with Figure 10.

8.5.4 Stilling Basin

The most common types of stilling basin make use of the hydraulic jump to dissipate energy and change the flow regime from supercritical to subcritical. Hydraulic jump will occur when the flow channel flattens out (the slope of the channel becomes hydraulically gentle) and the tailwater depth (downstream flow depth) is sufficiently large (if the tailwater depth is too small, the jump will be swept out to the downstream). A hydraulic jump can be induced near the point where the channel slope changes from steep to gentle through :

a) provision of chutes, baffles and sills at the bottom of the channel;
b) widening of the channel; or
c) introduction of slotted bucket.

Reference should be made to U.S. Bureau of Reclamation (1960) and Vischer & Hager (1998). Figure 11 show stilling basins of types I, II and III developed by the U.S. Bureau of Reclamation, together with the design procedures and guidelines.

The impact type stilling basin can be used in both open and closed conduits, and is effective for flow velocity which is not so large. Details are given in Figure 12 (U.S. Bureau of Reclamation (1960)).

8.6 FLOW ROUTING

8.6.1 Introduction

Flow routing is a procedure to determine the flow hydrograph at a point in a drainage system based on known inflow hydrographs at one or more points upstream. Two methods of flow routing can be distinguished: hydrologic routing and hydraulic routing. In hydrologic routing, the flow at a particular location is calculated as a function of time, based on the upstream inflows and attenuation due to storage. In hydraulic routing, the flow is calculated as a function of space and time throughout the system. It is based on the solution of the basic differential equations of unsteady flow.

8.6.2 Hydrologic Routing

The basic formulation of hydrologic routing is the solution of the outflow hydrograph Q(t) from the inflow hydrograph I(t) through the continuity (or storage) equation and storage function, S:

$$\frac{dS}{dt} = I(t) - Q(t)$$

Depending on the choice of the storage function, two hydrologic routing methods can be distinguished:

(a) Reservoir Routing

$$S = f\,(Q)$$

(b) Muskingum Method

$$S = K\,[XI + (1\text{-}X)\,Q]$$

where K = proportional constant
X = weighting factor, $0 \leq X \leq 0.5$

8.6.3 Hydraulic Routing

The basis of hydraulic routing is the solution of the basic differential equations of unsteady flow (the Saint Venant Equations). Using the notations in Figure 13, these equations can be written as follows:

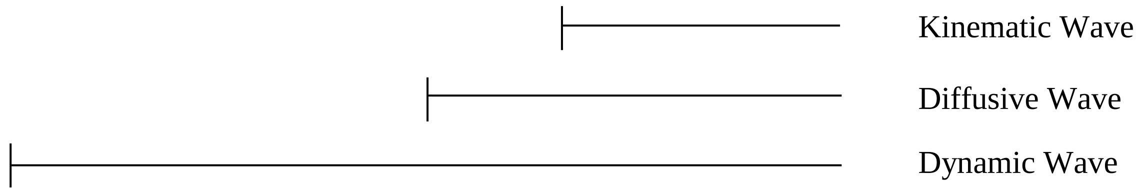
Continuity Equation:

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q$$

Momentum Equation:

$$\frac{1}{A}\frac{\partial Q}{\partial t}+\frac{1}{A}\frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right)+g\frac{\partial y}{\partial x}-g\left(s_o-s_f\right)=0$$

Local acceleration term	Convective acceleration term	Pressure force term	Gravity force term	Friction force term



where β is the momentum correction factor and S_f is the friction gradient.

Depending on the number of terms to be included in the Momentum Equation, the routing may be kinematic wave, diffusive wave and dynamic wave. Kinematic wave routing and dynamic wave routing are commonly used. These are briefly described below:

(a) *Kinematic Routing.* This is analogous to hydrologic routing. One of the kinematic routing methods is the Muskingum-Cunge Method. Cunge, J.A. (1969) gives hydraulics-based definition of the parameters K and X in the Muskingum Method. The limitations of kinematic routing are the failure to consider out-of-bank flows, loops, and downstream influence due to, for instance, tidal backup. These limitations can be overcome by Dynamic Routing.

(b) *Dynamic Routing.* The full Saint-Venant Equations are not amenable to analytical solutions. They are normally solved by numerical methods. The calculations for flow Q and water depth y are performed on a grid placed over the x-t plane. Common finite-difference schemes used are the Priessman 4-point Implicit Scheme and Abbot-Ionescu 6-point Implicit Scheme. Initial flow conditions and the design of the computation grid are important factors affecting the stability of the calculations.

8.7 LOCAL HEAD LOSSES IN PIPE FLOWS

In order to minimize the head losses in pipe flows, the selection of the pipe materials and the joint details are very important. The resistance in pipes will be influenced by the pipe material but will be primarily dependent on the slime and sediment that deposit on the pipe surface. Other factors such as discontinuities at the pipe joints, number of manholes, number of branch pipes at manholes and their directions of flow in relation to the main stream, etc will all affect the head losses.

Other sources of resistance which occur in pipes include inlets, outlets, bends, elbows, joints, valves, manholes and other fittings and obstructions can all be referred to as head loss and formulated as:

$$h_L = \frac{K\overline{V}^2}{2g}$$

in which K may refer to one type of head loss or the sum of several head losses.

The head loss coefficient K can be found elsewhere in the literatures of hydraulics. Table 15 contains some of the most commonly used head loss coefficients in Hong Kong as abstracted from the Preliminary Design Manual for the Strategic Sewage Disposal Scheme. Reference should also be made to Streeter, V.L. and Wylie, E.W. (1985), BSI (1997/2) and Chow, V.T. (1959).

8.8 COMPUTATIONAL HYDRAULIC MODELLING

Computational hydraulic modeling is becoming more popular nowadays and is a standard tool of hydraulic analysis. It is a subject for text books, manuals and courses of hydraulic softwares. Reference can be made to Cunge et al. (1980), Adri Verwey (2005) and prevailing Divisional Instruction issued by Land Drainage Division of DSD on this subject.

9. EROSION AND SEDIMENTATION

9.1 GENERAL

Erosion of natural and artificial sediments in a drainage basin, their transport along the drainage systems, and their subsequent deposition at the lower reaches of such systems are natural processes in the hydrological cycle. This is an evolving subject known as sediment transport. This section deals with its common applications in the drainage field including, amongst others, river bed and bank protection, velocity design in channels and pipes, scour around bridge piers and the quantification of sedimentation at the lower reaches of drainage systems.

There are different forms of river bank protection available, such as concrete lining, masonry facing and gabion wall. Chapter 13 of this Manual gives a more comprehensive list of different forms of channel linings. Designers shall check the allowable maximum velocity with the supplier or manufacturer when selecting the form of channel lining.

9.2 RIVER BED AND BANK PROTECTION BY ARMOUR STONE

The sizing of non-cohesive stones for river bed and bank protection against scouring induced by river flows is given by the following expression adapted from Zanen (198l):

$$D_m \geq \frac{\alpha}{\Delta} \cdot \frac{\overline{V}^2}{2g} \cdot \frac{1}{K_\beta} \cdot \frac{1}{K_\gamma}$$

where	D_m	=	mean grain size of armour stone (m)
	α	=	a dimensionless factor given below
	K_β	=	dimensionless adjustment factor for side-slope of bank
	K_γ	=	dimensionless adjustment factor for river sinuosity
	g	=	acceleration due to gravity (m/s^2)
	Δ	=	difference between the relative densities of stone and water
	$\overline{V}$	=	mean flow velocity (m/s)

(a) α *values.* These are given in the following table:

Researcher	α
Lane/Shield	0.3 to 0.5
Isbash	$\frac{0.7}{y^{(1/3)}}$ (where y is the flow depth in m)
USBR	0.2 for Minor turbulence
	0.5 for Normal turbulence
	1.4 for Major turbulence

(b) *Δ values.* Common Δ values are given in the following table:

Material	*Δ*
dense sand, gravel	1.65
concrete	1.2 to 1.4
asphalt concrete	1.3 to 1.4
granite	1.5 to 2.1

(c) *K_β values.* K_β adjusts for reduced shear stress on the bank and reduced stabilizing forces due to side slope. This factor is not applicable to the bed, for which a factor of 1 can be assumed.

$$K_\beta = \sqrt{1 - \frac{\sin^2 \beta}{\sin^2 \phi}} \frac{1}{0.8}$$

where β = side slope of river bank in degrees
φ = angle of repose in degrees

(d) *$K\gamma$ values.* Lane suggested the following table for $K\gamma$ to account for river sinuosity:

Degree of Sinuosity	*$K\gamma$*
straight canal	1.00
slightly sinuous river	0.90
moderately sinuous river	0.75
very sinuous river	0.60

The sizing of armouring stones for wave resistance in the estuarine reach of drainage channels can be carried out in accordance with guidelines in CED (1996).

9.3 VELOCITY DESIGN IN CHANNELS AND PIPES

Deposition of sediment in stormwater channels and pipes is inevitable and suitable allowance should be made in the design. For the permissible degradation between desilting cycles, the following guideline is proposed to take into account the effects to flow capacity due to materials deposited on the bed:

(a) 5% reduction in flow area if the gradient is greater than 1 in 25.
(b) 10% reduction in flow area in other cases.

Recent research on sediment movement in channels and pipes has shown that there is no unique design self-cleansing velocity since it depends on sediment type, grading, concentration, and transport rates as well as the size of the channel or pipe. Details can be found in DSD (1990).

Even if self-cleansing velocities could be derived, it would be difficult to achieve them in designs except in steep upland catchments. Large stormwater drainage systems, particularly those within new reclamations, generally have the potential for siltation due to flat gradients and also due to the phasing of their handing over upon completion. Sedimentation must therefore be expected in the middle and lower reaches of drainage systems. While some allowance for this could be made in sizing the channels and pipes, facilities must also be provided for regular desilting works to safeguard the drainage capacities.

9.4 SCOUR AROUND BRIDGE PIERS

Scour around bridge piers is a combination of three phenomena:

(a) Local scour near the bridge pier caused by the disturbance of the flow field around the pier.

(b) Long term degradation of the river bed due to increased flow velocity caused by the contraction of the river cross section at the bridge site.

(c) Short term degradation of the river bed around the bridge site during floods.

The first aspect is discussed in detail in a review article by Breusers, H.N.C., Nicollet, G., and Shen, H.W. (1977). The last two aspects are discussed comprehensively in Neill, C.R.(ed) (1973).

9.5 QUANTIFICATION OF SEDIMENTATION

There is currently insufficient data to enable a comprehensive estimate of the sedimentation rates in drainage systems. However, some crude quantification can be made, as a reference for the evaluation of maintenance commitments, based on the likely quantity of sediments arising in the drainage basins:

(a) *Natural Erosion.* Quantitative assessments of annual natural erosion are available in DSD (1990) for various land categories.

(b) *Artificial Sediments.* The estimated livestock populations and sediment loads from agricultural sources are available from EPD. Sediments from construction sites are extremely variable in quantity.

Actual sedimentation rates can be determined by repetitive hydrographic surveys on the cross-sections of the drainage systems. Measurements of sediment transport can also be carried out but this requires specialist techniques (Van Rijn, L.C., 1986).

10. DESIGN OF BURIED GRAVITY PIPELINES

10.1 GENERAL

This Chapter provides guidelines on the materials, level and structural design of buried gravity pipelines laid by cut and cover method.

In recent years, there have been technological improvements on the use of trenchless methods including pipe jacking, microtunnelling, directional drillings, auger boring and on-line replacement techniques for laying pipelines in congested urban areas. Reference should be made to the relevant literature and manufacturers' catalogue in designing pipelines laid by trenchless method.

10.2 MATERIALS

In general, concrete pipes have been used extensively for stormwater pipelines throughout the Territory and are normally available in sizes up to 2500 mm diameter in the local market. Where the stormwater flow is severely polluted, consideration may be given to the use of vitrified clay pipes to provide better protection against corrosion. Other pipeline materials are available and may be considered in relation to their advantages and disadvantages for particular situations. If such alternative materials are proposed, full account should be taken of their acceptability from the operation and maintenance point of view.

10.3 LEVELS

As the size of a stormwater drain increases downstream, it is preferable to maintain the soffits at the same levels at the manhole. This is to prevent the drain being surcharged by backwater effect when the downstream pipe is flowing full. Similarly when a lateral drain joins a main drain, the soffit of the lateral shall not be lower than that of the main drain. If the situation allows, it is preferable to have the lateral at a higher level to minimize possible surcharge of the lateral.

Designing for flush soffit requires adequate fall along the stormwater drain, and may not be achievable especially over reclaimed land. Under this circumstance, the inverts shall be kept at the same level to achieve a smooth flow when the stormwater drain is flowing partially full.

10.4 DEPTH OF PIPELINE

Designers should avoid deep underground pipeline. In general, the maximum depth of a pipeline should not be more than 6 m. Below such depth, maintenance and reconstruction of the pipeline will be very difficult. If the situation warrants such deep pipeline, one should always consider other alternatives including the use of intermediate pumping station.

Normally, the minimum cover from the surface of the carriageway to the top of the pipeline shall be 900 mm. For footway, the minimum cover shall be 450 mm.

10.5 STRUCTURAL DESIGN

10.5.1 Introduction

Pipes can be categorised into rigid, flexible and intermediate pipes as follows:

(a) Rigid pipes support loads in the ground by virtue of resistance of the pipe wall as a ring in bending.

(b) Flexible pipes rely on the horizontal thrust from the surrounding soil to enable them to resist vertical load without excessive deformation.

(c) Intermediate pipes are those pipes which exhibit behaviour between those in (a) and (b). They are also called semi-rigid pipes.

Concrete pipes and clay pipes are examples of rigid pipes while steel, ductile iron, uPVC, MDPE and HDPE pipes may be classified as flexible or intermediate pipes, depending on their wall thickness and stiffness of pipe material.

The load on rigid pipes concentrates at the top and bottom of the pipe, thus creating bending moments. Flexible pipes may change shape by deflection and transfer part of the vertical load into horizontal or radial thrusts which are resisted by passive pressure of the surrounding soil. The load on flexible pipes is mainly compressive force which is resisted by arch action rather than ring bending.

The loads on buried gravity pipelines are as follows:

(a) The first type comprises loading due to the fill in which the pipeline is buried, static and moving traffic loads superimposed on the surface of the fill, and water load in the pipeline.

(b) The second type of load includes those loads due to relative movements of pipes and soil caused by seasonal ground water variations, ground subsidence, temperature change and differential settlement along the pipeline.

Loads of the first type should be considered in the design of both the longitudinal section and cross section of the pipeline. Provided the longitudinal support is continuous and of uniform quality, and the pipes are properly laid and jointed, it is sufficient to design for the cross-section of the pipeline.

In general, loads of the second type are not readily calculable and it affects the longitudinal integrity of the pipeline. Differential settlement is of primary concern especially for pipelines to be laid in newly reclaimed areas. The effect of differential settlement can be catered for by using either flexible joints (which permit angular deflection and telescopic movement) or piled foundations (which are very expensive). If the pipeline is partly or wholly submerged, there is also a need to check against the effect of flotation on the empty pipeline when it is not in operation or prior to commissioning.

The design criteria for the structural design of rigid pipes are the maximum load at which failure occurs while those for flexible pipes are the maximum acceptable deformation and/or the buckling load. The design approach for rigid pipes is not applicable to flexible pipes. For the structural design of flexible pipes, it is necessary to refer to relevant literature such as manufacturers' catalogue and/or technical information on material properties and allowable deformations for different types of coatings, details of joints, etc.

10.5.2 Design Procedures for Rigid Pipes

The design procedures for rigid pipes are outlined as follows:

(a) Determine the total design load due to:

(i) the fill load, which is influenced by the conditions under which the pipe is installed, i.e. narrow trench or embankment conditions.

(ii) the superimposed load which can be uniformly distributed or concentrated traffic loads.

(iii) the water load in the pipe.

(b) Choose the type of bedding (whether granular, plain or reinforced concrete) on which the pipe will rest. Apply the appropriate bedding factor and determine the minimum ultimate strength of the pipe to take the total design load.

(c) Select a pipe of appropriate grade or strength.

Specific guidance on the design calculations is given in Sections 10.5.3 to 10.5.8.

10.5.3 Fill Loads

(a) *Narrow trench condition*. When a pipe is laid in a relatively narrow trench in undisturbed ground and the backfill is properly compacted, the backfill will settle relative to the undisturbed ground and the weight of fill is jointly supported by the pipe and the shearing friction forces acting upwards along the trench walls. The load on the pipe would be less than the weight of the backfill on it and is considered under 'narrow trench' condition by the theory and experimental work of Marston:

$$W_c = C_d \, w \, B_d^2$$

$$C_d = \frac{1}{2k\mu'} \left[1 - \exp\left(-2k\mu' \frac{H}{B_d}\right)\right]$$

$$k = \frac{\sqrt{(\mu^2+1)}-\mu}{\sqrt{(\mu^2+1)}+\mu}$$

where W_c = fill load on pipe in kN/m
w = unit weight of fill in kN/m^3
B_d = the width of trench in metre measured at the top level of the pipe (as shown on the relevant DSD Standard Drawing)
C_d = narrow trench coefficient
H = actual height of fill above the top of pipe in metres
k = Rankine's ratio of lateral earth pressure to vertical earth pressure
μ, μ' = coefficient of friction of backfill material and that between backfill and trench side respectively

For practical applications, take $\mu = \mu'$, and use Figure 14 to obtain values of C_d.

(b) *Embankment condition*. When the pipe is laid on a firm surface and then covered with fill, the fill directly above the pipe yields less than the fill on the sides. Shearing friction forces acting downwards are set up, resulting in the vertical load transmitted to the pipe being in excess of that due to the weight of the fill directly above the fill. The load on the pipe will then be determined as in the 'embankment' condition. The equation for the embankment condition as proposed by Marston is as below:

$$W_c = C_c\, w\, B_c^2$$

$$C_c = \frac{exp\left(\frac{2k\mu H_e}{B_c}\right) - 1}{2k\mu} + \left(\frac{H - H_e}{B_c}\right) exp\left(\frac{2k\mu H_e}{B_c}\right)$$

It is given by:

$$\left[\frac{exp\left(\frac{2k\mu H_e}{B_c}\right) - 1}{2k\mu}\right]\left[\frac{1}{2k\mu} + \frac{H - H_e}{B_c} + \frac{r_{sd}\, p}{3}\right] + \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$

$$+ \frac{r_{sd}\, p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) exp\,\frac{2k\mu H_e}{B_c} - \frac{H_e}{2k\mu B_c} - \frac{H\, H_e}{B_c^2} = r_{sd}\, p\, \frac{H}{B_c}$$

where W_c = fill load on pipe in kN/m
w = unit weight of fill in kN/m^3
B_c = external diameter of pipe in metres
C_c = load coefficient under embankment condition
H_e = height of plane of equal settlement above the top of pipe in metres
H = actual height of fill above the top of pipe in metres
r_{sd} = settlement ratio
p = ratio of projection of pipe's crown above firm surface to the external pipe diameter
k = Rankine's ratio of lateral earth pressure to horizontal earth pressure

μ = coefficient of internal friction of backfill material

For practical applications, values of C_c can be obtained from Figure 15.

Use r_{sd} = 1.0 for rock or unyielding foundations
= 0.5 – 0.8 for ordinary foundations
= 0.0 – 0.5 for yielding foundations

Narrow trench and embankment conditions are the lower and upper limiting conditions of loading for buried rigid pipes. Other intermediate loading conditions are not very often used in design.

One method for deciding whether the narrow trench condition or embankment condition of the Marston equations is to be used to determine the fill load on pipes was proposed by Schlick. Calculations are carried out for both conditions. The lower of the two calculation results is suggested to be adopted in design. Method of construction will be specified in accordance with the design trench conditions if necessary.

Under certain site conditions, when restricting the trench width is not practical because of the presence of underground utilities, consideration should be given to design the pipe for fill loads under the worse scenario of narrow trench and embankment conditions.

If the width of the trench, B_d, and external diameter of the pipe, B_c, are fixed, there is a unique value of cover depth at which the embankment or narrow trench calculations indicate the same load on the pipe. This value of cover depth is termed the 'transition depth' T_d, for this trench width and external diameter of pipe.

At depths less than the transition depth, the pipe is in the 'embankment' condition and the fill load will be dependent on the external diameter of the pipe. No restriction to trench width is required. In other cases, when the depth is greater than the transition depth, the fill load is dependent on the assumed trench width. The tabulated fill load on the pipe in Table 16 will be exceeded unless the trench width is restricted to the assumed value in order that the pipe is in the 'narrow trench' condition.

The fill load on a pipe and value of transition depth, assuming a saturated soil density of 2000 kg/m^3, are shown in Table 16. If the actual soil density Φ differs from 2000 kg/m^3, the fill load may be adjusted by a multiplying factor of Φ/2000. The values of kμ assumed in deriving this table are 0.13 for narrow trench condition and 0.19 for embankment condition. r_{sd} p for embankment condition is taken as 0.7 for pipes up to 300 mm nominal diameter and 0.5 for larger pipes.

10.5.4 Superimposed Loads

The equivalent external load per metre of pipe transmitted from superimposed traffic loads can be calculated by the Boussinesq Equation, by assuming the distribution of stress within a semi-infinite homogeneous, elastic mass:

$$p = \left(\frac{3L}{2\pi}\right)\left(\frac{H^3}{H_s^5}\right)\alpha$$

where L = concentrated load applied at surface of fill in kN
p = unit vertical pressure at a specific point within the fill in kN/m^2
H = Depth of such point below the surface in metres
H_s = slant distance of such point from the point of application of concentrated load at surface in metres
α = impact factor

The traffic load will be calculated as below:

$$W_p = \Sigma p\, B_c$$

where W_p = design traffic load in kN/m
Σp = unit vertical pressure due to the various concentrated loads in kN/m^2
B_c = external diameter of pipe in metres

Values of traffic loads for design are shown in Table 17 with the following assumptions:

Main road : pipelines laid under main traffic routes and under roads to be used for temporary diversion of heavy traffic, where provision is made for eight wheels loads, each of 90 kN acting simultaneously with an impact factor of 1.3 and arranged as in BSI (1978) Type HB Loading.

Light road : pipelines laid under roads except those referred in main roads, where provision is made for two wheel loads, each of 70 kN static weight, spaced 0.9 m apart, acting simultaneously with an impact factor of 1.5.

10.5.5 Water Load

The weight of water in a pipe running full generates an additional load, the equivalent external load on the pipe can be calculated from the following equation:

$$W_w = 9.81\left(\frac{3}{4}\right)\left(\frac{\pi D^2}{4}\right)$$

where W_w is the equivalent water load in kN/m,
D is the internal diameter of pipe in metres.

In general, the water load is not significant for small pipes of less than 600 mm diameter. The equivalent water load of pipes of 600 mm to 1800 mm diameter are as below:

Nominal Diameter (mm)	*Equivalent Water Load (kN/m)*
600	2.1
750	3.3
900	4.7
1050	6.4
1200	8.3
1350	10.6
1500	13.0
1650	15.8
1800	18.8

10.5.6 Bedding Factors

The strength of a precast concrete or vitrified clay pipe is given by the standard crushing test. When the pipe is installed under fill and supported on a bedding, the distribution of loads is different from that of the standard crushing test. The load required to produce failure of a pipe in the ground is higher than the load required to produce failure in the standard crushing test. The ratio of the maximum effective uniformly distributed load to the test load is known as the 'bedding factor', which varies with the types of bedding materials under the pipe and depends to a considerable extent on the efficiency of their construction and on the degree of compaction of the side fill.

The various methods of bedding used with precast concrete pipes are shown on the relevant DSD Standard Drawing. The values of the bedding factors below are average experimental values and are recommended for general purposes: -

(a)	granular bedding	1.9
(b)	120^{o} plain concrete bedding	2.6
(c)	120^{o} reinforced concrete bedding with minimum transverse steel area equal to 0.4% of the area of concrete bedding	3.4
(d)	concrete surround	4.5

On the basis of the experimental and numerical modelling work carried out, bedding factors used with vitrified clay pipes for class F, B and S bedding are shown in Figure 16.

10.5.7 Design Strength

For design, it is required that the total external load on the pipe will not exceed the ultimate strength of the pipe multiplied by an appropriate bedding factor and divided by a factor of safety.

The design formula is as follows:

$$W_e \leq \frac{W_t F_m}{F_s}$$

where W_e = total external load on pipe
W_t = ultimate strength of pipe
F_m = bedding factor
F_s = design safety factor of 1.25 for ultimate strength of pipe

Based on the assumed design parameters in paragraphs 10.5.3, 10.5.4, 10.5.5 and 10.5.6, values of the total external design loads in main roads and light roads are shown in Table 18.

Alternatively, Table 19 may be used for direct evaluation of the minimum crushing strength or grade of precast concrete or vitrified clay pipes using different bedding factors in main roads.

Worked Example: -

Given: Nominal pipe size 375 mm (outside diameter 500 mm)
Class B bedding (bedding factor = 1.9)
Cover depth range 1.8 m to 4.6 m
Pipe to be laid in main road

To determine strength of concrete pipe required.

Solution:

Bedding factor, F_m is 1.9

For nominal pipe size 375 mm, the effective trench width, B_d, is 1.05 m (see Hong Kong Government (1992) Table 5.9)

Transition depth, T_d, is 3.0 m

From Tables 16 and 17

Case A: check narrow trench condition when cover depth is 4.6 m

maximum design load = fill load + traffic load
= 57.6 +7.9 = 65.5 kN/m

Case B: check wide trench condition when cover depth is 3.0 m

maximum design load = fill load + traffic load
= 44.0 + 13.7 = 57.7 kN/m

Therefore, take the maximum design load,

W_e = 65.5 kN/m for whole section of pipe

Required ultimate strength of pipe,

$$W_t = W_e \text{ x } F_s \,/\, F_m = 65.5 \text{ x } 1.25 \,/\, 1.9 = 43.1 \text{ kN/m}$$

From BSI (1988/2) Part 100, Class H Pipe with ultimate strength of 45 kN/m is required.

10.5.8 Effect of Variation in Pipe Outside Diameters

The outside diameters in Table 18 are the general maxima for the majority of pipes. However, a few pipes with outside diameters exceeding the tabulated dimensions may be encountered. Provided the excess is not greater than 5%, the effect can be ignored. If the pipes employed have an outside diameter less than that being assumed, the load in Table 18 will then err on the safe side. It may be worthwhile making a more accurate computation of the design load as described in sections 10.5.3 and 10.5.4 with a view to achieving economy where the difference in outside diameter is considerable.

10.6 PIPE AT SLOPE CREST

Any leakage from pipeline which is close to the crest of a slope may affect the stability of the slope. Attention shall be paid to avoid routing of pipeline near slope crest. If pipeline is to be laid within the crest of a slope, appropriate leakage collection system shall be provided to prevent any adverse effects to the slope in case of pipe leakage. Reference shall be made to GCO (1984).

11. MANHOLES

11.1 GENERAL

This chapter provides guidelines on the design of manholes.

11.2 LOCATION

Manholes should be provided at:

(a) intersections of stormwater drains.

(b) junctions between different size of stormwater drains.

(c) where a stormwater drain changes direction/gradient.

(d) on long straight lengths at the following intervals:

Diameter of Pipe (mm)	*Maximum Intervals (m)*
$\leq$675	80*
>675 and $\leq$1050	100
>1050	120

* For pipe of size smaller than or equal to 675mm, the maximum interval should be reduced to 60m if:

- the flow carried by the pipe may likely cause frequent chokages;
- the manhole covers of the pipes are located on busy road such that opening 2 adjacent manhole covers at the same time may involve difficult traffic arrangement; or
- the pipe is located in a village with narrow roads which are inaccessible to standard water-jetting units.

Should there be any doubt on the maximum interval to be adopted for any special circumstances, the relevant O&M divisions should be consulted at the design stage.

In addition, manholes should, wherever possible, be positioned such that the disruption to the traffic will be minimal when their covers are lifted under normal maintenance operations.

11.3 ACCESS OPENINGS

Access openings are generally of two types, one for man access and the other for desilting purposes. A desilting opening should not be smaller than 750 mm by 900 mm, and it should be placed along the centre line of the stormwater drain to facilitate desilting. A man access opening should not be smaller than 675mm by 675mm. If cat ladders are installed in a manhole, the minimum clear opening should be 750mm by 900mm. A man access opening should be placed off the centre line of the stormwater drain for deep manholes and along the centre line of the stormwater drain for shallow manholes with depths less than 1.2 m.

11.4 ACCESS SHAFTS

Access shafts should be sufficiently large for a person to be able to go down in comfort and yet give him a sense of security. The minimum size of an access shaft is 750mm by 900mm. The access shaft should be orientated such that the step irons or cat ladders are provided on the side with the smaller dimension.

11.5 WORKING CHAMBERS

For manholes less than or equal to 1.2 m deep, work in them generally can be performed from ground level, i.e., the workmen standing on the ground can reach the invert of the stormwater drain without great difficulty. A working chamber is generally not required for this type of manhole.

For manholes deeper than 1.2 m, work in them generally cannot be easily carried out from ground level. Manholes of this type should be provided with working chambers and access shafts leading from ground level. The working chambers should enable a person to work inside.

11.6 INTERMEDIATE PLATFORMS

When the invert of a manhole is more than 4.25 m from the cover level, intermediate platforms should be provided at regular intervals. The headroom between platforms should not be less than 2 m nor greater than 4 m. The size of the platform should not be smaller than 800 mm by 1350 mm. The platform should be fitted with handrailing and safety chains at the edge to protect persons from falling down.

In order to facilitate rescue operation in case an accident occurred, designers are advised to provide an additional manhole opening where space permits.

11.7 INVERTS AND BENCHINGS

Inverts and benchings of the manholes should be neatly formed. The socket ends of pipes should be cut off and not projected into the manholes. The inverts should be curved to the radius of the inverts of the pipes and carried up in flat vertical faces, and should match the cross-sections, levels and gradients of the respective stormwater drain. The benching should be a plane surface sloping gently downward towards the stormwater drain. Suitable gradient of the benching is 1 in 12.

11.8 COVERS

Manhole covers should be sufficiently strong to take the live load of the heaviest vehicle likely to pass over, and should be durable especially under corrosive environment. Heavy duty manhole covers should be used when traffic or heavy loading is anticipated, otherwise medium duty covers can be used.

Manhole covers should not rock when initially placed in position, or develop a rock with wear. Split triangular manhole covers supported at the three corners are commonly used to reduce rocking. The two pieces of triangular cover should be bolted together to avoid a single piece of the cover being accidentally dropped into a manhole.

Foul sewer and stormwater drain manhole covers should be differentiated by the grid patterns which are shown on the DSD Standard Drawings.

11.9 STEP-IRONS AND CAT LADDERS

Step-irons should be securely fixed in position in manholes, and should be equally spaced and staggered about a vertical line at 300 mm centres. Cat ladders should be used in manholes deeper than 4.25 m or where manholes are frequently entered. It is safer and easier to go down a ladder when carrying tools or equipment.

Step-irons and ladders should start at not more than 600 mm below the cover level and continue to the platform or benching.

Step-irons and ladders, being constantly in a damp atmosphere and prone to corrosion, should be made of or protected with corrosion resistance materials, e.g. galvanized iron, glass-fibre reinforced plastic, plastic-coated steel, or stainless steel.

11.10 BACKDROP MANHOLES

Backdrop manholes are used to connect stormwater drains at significantly different levels, and should be used where the level difference is greater than 600 mm.

The backdrop can be provided by means of:

(a) a vertical drop in the form of a downpipe constructed inside/outside the wall of a manhole.

(b) a gradual drop in the form of cascade or ramp.

A cascade is preferred for drains larger than 450 mm diameter. Downpipes are suitable for drains less than 450 mm diameter. When downpipes are used, the following are recommended:

(a) proper anchoring of the backdrop at the bottom in the form of a 90^o pipe bend surrounded by concrete.

(b) a T-branch at the top fitted with a flap valve inside the manhole to avoid splashing.

12. DESIGN OF BOX CULVERTS

12.l GENERAL

Box culverts are required where precast pipes cannot be obtained in a sufficiently large size or where a box culvert configuration would better suit the available space between or adjacent to other structures or utilities. For hydraulic design of box culvert, reference can be made to FHWA (1985), THD (1962) and CIRIA (1997). The selection of the size and number of cells in a culvert depends not only on the hydraulic capacity but also on the requirement for maintenance and desilting. To facilitate the use of mechanical plant inside box culverts, the internal dimensions of each cell of a box culvert should not be less than 2.5 m × 2.5 m. The minimum width should be further increased if corner splays are used. For cells smaller than this size, agreement should be sought from maintenance authority. Smaller culverts may be used in special situations, such as steep gradients, where siltation or sedimentation will not be a problem.

12.2 DESIGN INVERT LEVEL AT DOWNSTREAM END

The design invert level of the box culvert at the downstream end should be kept at a high level as far as possible to allow for future extension of the culvert. The invert level of the box culvert should be designed to maintain free discharge of the flow at the outlet and to avoid backwater effect.

12.3 DESIGN LOADS

If the box culvert is subject to permanent vehicular or pedestrian live loads, Chapter 3 of HyD (4th Edition) or its latest version should be followed. Loadings due to construction and maintenance activities should also be considered when determining the design actions on box culverts. If the filling material above the culvert contributes to the major superimposed load and the culvert is not classified as a highways structure, design load shall be referred to BD (2011) or its latest version. Load combinations and the partial factors as specified in BSI (2002) or its latest version should be adopted in the design.

12.4 DURABILITY

The durability of a reinforced concrete box culvert depends mainly on the concrete grade, cover and crack width. Reference should be made to Section 4 of BSI (2004) or its latest version, unless modified by this Manual.

12.4.1 Exposure Condition

In general, as culverts at the downstream reaches of a drainage system are constantly in contact with sea water, exposure class “XS3” as defined in Table 4.1 of BSI (2004) or its latest version is recommended. At the upstream reaches where the culverts are not subject to sea water attack, “XC4” classification may be used.

12.4.2 Strength of Concrete

Given the required water retaining properties of box culverts, minimum concrete Grade ($f_{ck,cube}$) 40 is recommended. Since the design equations in the structural design to EC and UK NA are made use of the cylinder strength (f_{ck}), the designer may make reference to Table 3.1 of BSI (2004) or its latest version, or the following approximate equation for conversion of the cube strength to cylinder strength when carrying out the design.

$$f_{ck} = 0.8 \times f_{ck,cube}$$

where f_{ck} = characteristic compressive cylinder strength of concrete
$f_{ck,cube}$ = equivalent characteristic cube strength of concrete

Reference should also be made to the latest guideline on Concrete Specification for Reinforced Concrete Structures in Marine Environment by the Standing Committee on Concrete Technology.

12.4.3 Maximum Permissible Crack Width

The tightness class of box culverts shall be "Tightness Class 1" in accordance with BSI (2006) or its latest version.

The reinforced concrete structures of box culverts shall be designed so that design crack widths should be limited to w_{k1} as defined in BSI (2006) or its latest version. The limiting crack width w_{k1} varies according to the ratio (i.e. h_D/h) of hydrostatic pressure, h_D to the wall thickness of the containing structure, h. For h_D/h less than 5, w_{k1} shall be equal to 0.20 mm while for h_D/h greater than 35, w_{k1} shall be equal to 0.05 mm. Linear interpolation of w_{k1} is allowed for intermediate values of hydraulic gradient in accordance with BSI (2006) or its latest version. For box culverts which are subject to highway loading, reference should be made to Table 5.2 of HyD (4th Edition) or its latest version.

12.4.4 Concrete Cover to Reinforcement

Concrete cover to reinforcement should be provided in accordance with the following nominal values for the envisaged conditions of exposure. The value of nominal cover shall be the minimum concrete cover as recommended in BSI (2004) or its latest version plus the allowance for deviation taken as 10 mm. The value of nominal cover shall be indicated on the drawings and be used in design of reinforced concrete structure. For box culverts which are subject to highway loading, reference should be made to Table 5.2 of HyD (4th Edition) or its latest version.

Exposure Class	Nominal Cover (mm)	
	Concrete Grade $f_{ck,cube}$ = 40	Concrete Grade $f_{ck,cube}$ = 45 or above
XC4	35	35
XS3	-	60

12.5 MOVEMENT JOINTS

Movement joints at suitable spacing should be provided to control cracking in box culverts. In general, joints in the structure should pass through the whole box in one plane.

If movement joints are provided at spacing not exceeding 15 m, there is no need to consider cracking due to temperature difference from seasonal variations. For further details, Chapter 9 of HyD (4^{th} Edition) or its latest version should be referred to.

12.6 FOUNDATIONS

A layer of rock fill material is usually placed below the culvert. Where the subsoil comprises residual soil (completely decomposed volcanic or completely decomposed granite) or suitable filling materials, a 300 mm to 500 mm thick layer of rock fill material will generally suffice. For adverse foundation conditions, consideration should be given to removing the unsuitable sub-soil and replacing it with rock fill. The thickness of rock fill to be used in such cases depends very much on the subsoil conditions and needs to be assessed under the particular circumstances.

Generally, culverts do not need to be supported on piles. One of the exceptions is in newly reclaimed land or in other areas where substantial or unacceptable differential settlement is expected. For the design of piled foundation, reference should be made to BD (2004/2) or its latest version.

12.7 OPERATION AND MAINTENANCE REQUIREMENTS

12.7.1 Access

Wherever possible, an access ramp should be provided for mechanical plants to enter the box culvert. The minimum width and maximum gradient of the access ramp should be 3.5 m and 1 in 12 respectively. However, if a ramp is not feasible due to site constraints, then desilting openings should be provided.

12.7.2 Desilting Opening

Desilting opening is normally designed for box culvert to facilitate inspection and maintenance. In consideration of the methods commonly used for desilting, desilting opening should be provided at each cell at a maximum interval of 160m for large or multi-cell box culvert, or 120m for single cell box culvert of cross sectional area less than $5m^2$. Desilting openings of 900 mm x 750 mm and multi-part cover type of either 2m x 3m or 3m x 4m should be provided alternatively along the box culvert. For multi-cell box culvert, consideration can be given to align the desilting openings of 900 mm x 750 mm and multi-part cover type at alternate position across the box culvert to facilitate maintenance.

Desilting opening should be provided at locations suitable for desilting operations and just behind each seawall outfall if stop logs are installed as described in Section 12.7.7 below. Desilting opening on busy carriageway should be kept within one traffic lane and in any case should not be located at road junction where traffic would be seriously affected.

Desilting opening close to the signalized junction where braking of vehicles occurs frequently should be avoided. Where space is available, preference should be given to off-road desilting opening. The opening shaft should be orientated to match with the direction of the carriageway as far as possible in order to avoid unnecessary additional lane closures.

For multi-part desilting opening within carriageway, the cover should be a heavy-duty metal cover of the following loading grades (i) Heavy grade (D400) for carriageways with normal traffic loads (ii) Medium grade (C250) for light traffic load and light vehicles like carparks and on footpaths when occasional traffic is possible (iii) Extra heavy grade (E600) where exceptionally heavy traffic or frequent heavy wheel loads on the carriageways is expected such as docks and container terminals. More guidelines for desilting openings are given in DSD PN No. 1/2008.

12.7.3 Access Shafts

Access shafts of 900 mm × 750 mm should be provided for each cell of a box culvert at no more than 160 m intervals for large or multi-cell box culvert, or 120m for single cell box culvert of cross sectional area less than $5m^2$. The shaft can serve as an inspection manhole and for ventilation purpose during maintenance operation. A cat ladder should be provided in the culvert on one side of the shaft. Safety hoops/cages should be provided for ladders except that the installation of the safety hoops/cages inside the box culvert or flow area section would not be reasonably practicable such as catching the debris carried by the flood flow. Under such circumstances, other suitable safety precautions shall be considered when maintenance work is carried out. The location of access shaft should be suitably located in line with or at close distance to the desilting opening to facilitate desilting operation and inspection.

12.7.4 Internal Openings

For multi-cell culverts, internal openings with minimum size of 2 m × 2 m should be provided at the partition walls at 160m intervals for access between cells. The openings can serve as balancing holes and can also be used for flow diversion during maintenance of the box culvert. For ease of plant movement inside the multi-cell box culvert, the internal openings should be large enough and flush with the invert of the culvert to enable the desilting plant maneuvering across the cells. The internal openings should also be located in line with or at close distance to the desilting opening and access shaft locations to facilitate entry across different cells.

12.7.5 Freeboard

Under normal maintenance conditions, there should be a minimum freeboard of 500mm inside the box culvert for the consideration of the safety of maintenance personnel. As far as this Clause is concerned, the term "freeboard" refers to the vertical distance between the soffit level and the expected highest water level during normal maintenance condition of the box culvert. For box culverts with tidal influence, such highest water level should not be lower than the normal high tide level (2.5mPD). In case of site constraints or topography rendering the minimum freeboard cannot be practically achieved, the relevant maintenance parties should be consulted. However, the soffit of tidal influence box culvert should not be lower than the normal high tide level of 2.5mPD as far as possible.

12.7.6 Safety Provisions

Grilles should be provided at the entrance to box culverts from open channel to prevent people from being washed into the culvert. The grilles should be so designed that they will not prevent mechanical plant from entering the culvert to carry out maintenance operations. The height and spacing of the grilles should avoid the excessive collection of debris/vegetation which may cause stormwater overflowing from the system. The grilles should be placed at least 2 m upstream from the box culvert entrance to avoid sealing up the inlet when the grilles are substantially blocked by debris carried along by the flood flow.

12.7.7 Additional Provisions for Tidal Box Culvert

To facilitate maintenance operation, provisions for the installation of stop log or gate and winching equipment both in the structure of the box culvert and at the seawall should be considered. Maintenance access must be provided at each seawall outfall. Access to the culvert shall be by full width openings to allow the installation of stop logs or gates and to allow pumping over the stop logs or gates. For the maintenance of box culvert affected by tidal flows, an area for temporary storage of silt removed from the box culvert should be considered.

13. DESIGN OF NULLAHS, ENGINEERED CHANNELS AND RIVER TRAINING WORKS

13.1 GENERAL

Where land use permits, open channels should be the preferred option when compared with underground pipelines and culverts since the latter are more expensive to construct and maintain than open channels, in particular for those box culverts affected by the tide.

In general, open trapezoidal channels provide the most economical cross-section for the conveyance of stormwater, both in terms of construction and maintenance costs. Rectangular open channels are usually most costly to construct and have limited scope for improving the aesthetics when compared with trapezoidal channels. An analysis of land availability, acquisition and the channel appearance should be carried out before a rectangular section is adopted. Whichever section is adopted, the design should make due allowance for the appearance of the channel, the selection of suitable lining materials and the provision of landscaping.

In order to ensure effective protection against flooding of low-lying area behind a drainage channel, which is also adjacent an estuary, the top level of the proposed channel embankment or wall should tie in with that of the seawall at the estuary.

Blue-green concept is highly recommended to be considered in the early planning stage of nullah, channel and river projects. Designer should refer to DSD PN 1/2015 for more detailed guidelines on blue-green concept for river channel design.

13.2 CHANNEL LININGS

13.2.1 General

Side slope and bottom lining should normally be provided along the whole channel if the flow velocity exceeds 1 to 2 m/s (Chow, V.T., 1959). In the downstream reaches where the flow velocity is likely to be low, bottom lining is usually not required.

13.2.2 Types of Channel Linings

There are two types of channel linings as follows:

(a) *Rigid Linings*. Rigid linings are usually made of concrete, shotcrete, precast concrete slabs, stone masonry or grassed cellular concrete paving. They should only be used in locations where little settlement is anticipated. Stone masonry is preferred for aesthetics reasons in the past, while grassed cellular concrete paving is becoming more popular for ecological reasons. If grassed cellular concrete paving is to be used in locations subject to tidal influence, careful considerations should be given to ensure that there are suitable grass species to be established under such condition. Weep-holes should be provided in the lining for the free passage of groundwater.

(b) *Flexible Linings.* Flexible linings may consist of rip-rap, grass, gabions or random rubble. They are used in locations where large embankment settlement is anticipated. If the channel is subjected to tidal influence, the flexible linings should be designed to withstand wave action.

13.2.3 Design of Armour Layer

The armour layer of a flexible lining is susceptible to erosion by wave forces and flow-induced drag forces. Proper design of the armour layer is essential to protect the stability of the embankment/revetment. For the design of the armour layer for the tidal reach of drainage channels, reference to CED (1996) is recommended. For protection against scour due to river flows, the guidance in Section 9.2 of this manual should be followed.

13.3 CHANNEL SHAPE

The lining is generally the most expensive component of a lined channel. For economical reasons, the perimeter of channel cross-section should be minimised. Theoretically, a semi-circular shape provides the maximum hydraulic capacity for the minimum channel perimeter. A trapezoidal section which is a pragmatic approximation to the semi-circular shape is often adopted because it is easier and cheaper to construct, and can accommodate a wider range of flows than the simple rectangular channel. The side slope of the trapezoidal section normally ranges from 1 in 1.5 to 1 in 3.0, depending on the sub-surface condition and maintenance method.

13.4 COLLECTION OF LOCAL RUNOFF

In low-lying areas behind a drainage channel embankment/floodwall, the surface runoff_has to be collected by a system of U-channels and discharged into the drainage channel via pipework or box culvert through the embankment/floodwall. Usually, flap valves are installed at the drainage outlets to prevent back flows when water levels in the drainage channel are high. For large diameter drain pipes or large box culverts, multi-cell flap valves should be used so as to reduce the possibility of malfunctioning of the flap valves due to blockage. Desilting facilities shall be provided near flap valve in the drain pipe/box culvert if the section of drain pipe/box culvert is too long. The hydraulic impact of the flap valves on the upstream flow should be checked against potential flooding risk.

13.5 OPERATION AND MAINTENANCE REQUIREMENTS

13.5.1 Access Ramp

Concrete or similar hard paved access ramps should be provided along the drainage channel at intervals of about 600 m for the access of maintenance vehicles. For locations where there is a maintenance access at channel bottom and that the channel bottom is neither subject to tidal effect nor submerged most of the time, access ramps may be provided at more than 600m apart. The access ramp should have a width of 3.5 m and a slope ranging from 1 in 12 to 1 in 15. It should slope down in the same direction of flow. Along the edge of the access ramp, concrete upstands instead of railings are preferred for ease of maintenance.

For channel which base slab is always submerged due to tidal effect, intermediate platform should be provided in the ramp at level of 2.5 mPD with minimum size of 5 m x 20 m to facilitate mobilization of dredging plant and loading and unloading of dredged materials.

13.5.2 Dry Weather Flow Channel

To minimize siltation during low flow conditions in non-tidal channels, a dry weather flow channel should be provided in the invert of the main channel. It should be shallow and narrow and of trapezoidal or rectangular cross-section. A self-cleansing velocity of about 0.75 to 1 m/s is normally used to size the dry weather flow channel. For a narrow drainage channel, the dry weather flow channel may be located on one side of the invert to accommodate maintenance plant. It is also considered good practice to construct small channels branching off from the dry weather flow channel to intercept runoff from the lateral drainage inlets. However, such channels should be so designed that they will not obstruct the movement of maintenance plant along the bottom of the channel.

The design criteria in sizing the dry weather flow channel is to ensure the dry weather flows should be confined within the dry weather flow channel at 80% of the time.

If the dry weather flow is polluted, consideration should be given to divert the flow to the nearby sewerage system in order to reduce the pollutants in the storm water drainage system. In this connection, the capacity of the sewerage system to cope with the increased flow due to the intercepted dry weather flow should be checked.

13.5.3 Maintenance Road

Under normal circumstances, the maintenance road should be provided as far as possible within the drainage channel along the bottom or on a berm of the embankment at a sufficient height to be agreed with the maintenance engineer. Adequate headroom not less than 3.5 m at crossings should be provided. A verge with 1.6 m clear width paved with grassed cellular concrete paving should be provided along one or both sides of the channel bank to facilitate regular inspection of the channel.

Where an existing public access road is to be replaced or there are strong justifications under other special circumstances, maintenance roads could be provided along one or both sides of the channel bank and opened for public use. In these cases, comments on the road layout design, pavement design, proposed road markings and street furniture, etc. should be obtained from relevant maintenance authorities. Fire hydrants, irrigation water taps for planters and proper road lighting should also be installed according to the current standards for public road.

If the maintenance road is to be built on the embankment and not to be opened to the public, it should be paved with grassed cellular concrete paving as far as possible. It should be blocked off to avoid illegal parking. With the agreement of TD, a notice declaring it to be a restricted zone should be posted at all entrances of the maintenance road.

If the provision of maintenance road is fragmented due to unavailability of land or sections of the maintenance road within the embankment are always submerged, access points at strategic locations should be provided such that the whole length of the channel can still be accessed by maintenance vehicles.

A crossing slab over the dry weather flow channel should be provided at convenient locations for use by maintenance personnel and vehicles.

13.5.4 Safety Barriers and Staircases

In order to safeguard the safety of the maintenance personnel and the public, both sides of the channel should be provided with handrailings or parapets. Gates with locks should be provided at the entrances of access ramps to prevent vehicles from inadvertently entering the channel.

Staircases should be provided at the channel sides at intervals of 400 m. The staircases should not protrude from the surface of the channel sides to obstruct the flow. No opening shall be provided in the parapet or handrailing as entrance to these staircases. Warning signs should be erected at the parapet or handrailings near these staircases and other prominent locations to remind the public not to enter into the channel.

13.5.5 Grit Traps/Sand Traps

For drainage channels at the upstream reaches of a drainage basin, grit traps/sand traps should be provided to intercept and collect the silt and grit conveyed along small watercourses in times of storms. The grit trap/sand trap is normally in the form of a sump or a chamber which should be accessible by grab-mounted lorries with for easy desilting. Some guidance on design details of the grit trap/sand trap can be found in GCO (1984).

13.5.6 Tidal Channels

For tidal channels where maintenance dredging is envisaged, prior consultation with Port Works Division (CEDD) is required to determine the minimum water depth for the marine plant. Consideration should also be given to the effect of the design invert level of a tidal channel on its rate of sedimentation which is the prime factor affecting recurrent cost of the channel.

13.5.7 Staff Gauge

Staff gauges should be installed at the channel sides for the checking of water level in the channel. Details and installation locations of staff gauges should be agreed with the respective operation and maintenance division of DSD.

13.5.8 Chainage Marker and Survey Marker

Chainage markers and survey markers are to be installed at 100 m and 200 m intervals respectively on the coping of both sides of the channel. Exact details and locations of these markers should be agreed with the respective operation and maintenance division of DSD.

13.5.9 Marine Access and Marine Traffic

For large drainage channel near the sea where the quantity of desilting is anticipated to be enormous, marine access should be considered to facilitate future desilting operation. Marine Department should be consulted regarding the requirements for marine traffic management if marine access is necessary.

13.5.10 Maintenance and Management Responsibilities among Departments

The maintenance and management responsibilities of various departments concerned including DSD should be clearly defined in early planning/design stage especially in abandoned meanders, fish ponds, wetlands adjacent to the drainage channels, maintenance roads and landscaping works. An example of the schedule of responsibilities for a completed main drainage channels project is shown in Table 20.

13.5.11 Operation and Maintenance Manual

For major drainage channel, an operation and maintenance manual should be provided by the design office upon the handing over of the project. It should include as-built channel profiles, system hydraulics, spare parts provided, division of maintenance responsibility among departments, trigger levels for maintenance dredging, suggested monitoring schedule during operational phase, environmental issues relating to maintenance dredging, geotechnical monitoring schedule of channel embankment, safety requirements in relation to the operation and maintenance of the works and other maintenance items.

13.6 BRIDGE AND UTILITY CROSSSINGS

The soffit of all bridges crossing open channels should be designed such that no part of the bridge soffit will be submerged in water under the design rainstorm. Due consideration should be given to ensure that sufficient headroom is provided for the passage of maintenance plant and equipment.

As far as possible, bridge supports should not be positioned within channels. But if this is unavoidable, the supports should be designed and streamlined so that the obstruction to the water flow is minimal and debris and boulders are not easily trapped.

Utilities crossing a drainage channel which will reduce the design flow capacity and prevent the passage of desilting plant should be avoided. Such utilities should be accommodated as part of the highway or utility bridge crossings at suitable locations so that no part of the services will be below the bridge soffit. Alternatively, the utility crossings should be placed beneath the open channel with a minimum cover of 1 m. Such under-crossings must be designed and constructed to a standard to minimize the chance of digging up the channel for repair or replacement in the future.

13.7 GEOTECHNICAL CONSIDERATIONS

13.7.1 Embankment Design

The embankment design should be checked for the stability of each of the following failure modes for both the short-term and long-term conditions and the finalized design should be endorsed by GEO:

(a) adequacy of bearing capacity of the foundation soil.

(b) global stability of the embankment and its foundations.

(c) internal stability of the embankment's riverside and leeward side slopes.

For the construction of a channel embankment over a layer of soft marine deposits, excess pore water pressure would be built up instantaneously during the filling of the embankment. Thus, undrained shear strength of the marine deposit should be adopted for the short-term condition. For the long-term condition, the consolidation process of soil underneath the embankment would be completed or nearly completed. As such, the long-term stability should be checked for the drained condition. Moreover, the settlement of the embankment and its rate of consolidation should also be checked and allowed for in the design.

13.7.2 Factors of Safety

The factor of safety for an embankment design is defined as the ratio of average available shear strength of the soil along the critical failure surface of the embankment to that required to maintain equilibrium.

The adopted factor of safety against the failure of an embankment would be related to risks causing loss of life or property. It is recommended to follow the guidelines as stipulated in GCO (1984).

However, in case of a temporary condition such as during the construction stage of the embankment, a balance should be made between the potential economic loss in the event of a failure and the increased costs of construction required to achieve a higher factor of safety.

13.7.3 Loading Cases

The design loading cases should taken into considerations the following:

(a) loading due to future maintenance vehicles on the embankment.

(b) loading due to overfill of the embankment.

(c) the effects of steady seepage through the embankment when the river is at high flows (critical for leeward side slope stability).

(d) the effects of rapid drawdown during flood recession (critical for riverside slope stability).

13.7.4 Methods of Analysis

Various methods commonly adopted for slope stability analysis can be applied for analyzing the stability of an embankment. The method that should be applied depends on the potential failure mode of the embankment. For those potential failures with a circular slip, the method by Bishop, A.W. (1955) is simple to apply; for those with a non-circular slip, the methods by Janbu, N. (1972) or by Morgenstern, N.R. & Price, V.E. (1965) are commonly adopted. A detailed comparison of various methods of slope stability analysis is given in Table 5.5 of GCO (1984).

13.7.5 Seepage

Where appropriate, subsoil drains with proper filters should be provided at the toe of the leeward slope of the embankment to keep the phreatic surface within the embankment.

The water quality of floodwater is not suitable for fish farming. Thus, the problem of seepage through the channel embankment should be addressed especially at locations where fish ponds are reinstated behind the embankment.

13.7.6 Sensitivity Analysis

A sensitivity analysis may be warranted to account for the variability of the ground conditions and the uncertainty associated with the design values of soil strength. Some guidance on the sensitivity analysis can be found in GCO (1984).

13.7.7 Methods for Stability Improvement

Various methods have been developed for improving the stability of an embankment on soft foundation soil as shown in Table 21.

13.7.8 Geotechnical Instrumentation

Geotechnical instrumentation, such as settlement marker and inclinometer, is a very useful tool in confirming the assumptions made in the design of an embankment. It can also review the performance of the method adopted in improving the stability of the embankment. In designing the geotechnical instrumentation details, it is advisable to liaise closely with GEO.

13.7.9 Sign Boards for Slopes

All embankment slopes formed in association with the construction of the channel should be registered with GEO according to the relevant technical circular. Standard DSD sign boards for slopes should be erected alongside the slope edges according to the relevant technical circular before the slopes are handed over to the maintenance department.

13.8 OTHER CONSIDERATIONS

13.8.1 Reprovision of Irrigation Water

If existing stream courses are to be intercepted by the drainage channel, AFCD should be consulted during planning/design stage to confirm whether flows in the stream courses need to be maintained for irrigation. Inflatable dams and pumping facilities may be considered for such purpose.

13.8.2 Use of Inflatable Dam as Tidal Control Structure

For tidal channel where the downstream receiving water body is polluted or for some other reasons that the channel at upstream has to be kept dry, an inflatable dam together with a low flow pumping station installed near the downstream end of the channel may be adopted to prevent tidal water from flowing into the channel. The inflatable dam will be automatically inflated/deflated and the operation of the low flow pumping station will be suspended according to the pre-set conditions.

Under normal weather conditions, the dam will be inflated to prevent tidal water from flowing into the channel and the dry weather flow retained at upstream of the dam will be pumped into downstream receiving water body.

During severe rainstorm, the inflatable dam will be deflated and the operation of the pumping station will be suspended.

Inflatable dam shall be equipped with an alarm system to inform the operator/controller/public in case of any unexpected deflation of the dam or loss of pressure inside the dam. This is to ensure that necessary measures can be taken immediately to evacuate any people who may be present either upstream or downstream of the dam and who may be threatened by the sudden release of flood water.

Based on experience in Hong Kong, inflatable dam is costly in construction, operation and maintenance. It is subject to ageing and damage of various kinds which may require total replacement of the whole dam. There is only very limited suppliers in Hong Kong thus the long term availabilities of reliable suppliers and maintenance services are in doubt.

If a project requires the installation of large scale tidal control structures, the pros and cons of different tidal control structures should be assessed in detail before a particular type is adopted. Life cycle cost of the control measures shall be studied and the potential impacts of various tidal structures on the local river-estuarine ecology should also be addressed. Such assessments are also needed in case any of the existing installations become dilapidated and need replacement.

In searching for potential alternatives, other types of tidal control structures such as stop-log, flap valve, self-regulating tidegate, penstocks, radial gate, Obermeyer Spillway Gate could be considered. It is necessary to evaluate the use of different tidal control structures based on the following factors:

(a) Specific needs of the drainage project;
(b) Hydraulic performance of the tidal control structures;
(c) Hydraulic impact of the structures, especially during heavy rainstorm;
(d) Effectiveness of the structures to stop tidal water flow;
(e) Constructability and maintainability;
(f) Reliability and fail safe provision;
(g) Environmental considerations; and
(h) Life cycle cost.

13.9 DECKING OF EXISTING NULLAHS

Decking of existing nullahs is not preferred in view of the adverse hydraulic impact, problems associated with tracking water pollution and utility crossings as well as maintenance difficulties of a decked nullah. An open nullah has the advantage of an effective flood relief path which can capture overland flows from both sides of the nullah and help to mitigate the heat island effect.

Water pollution problem in a nullah should be tackled at source by a proper wastewater collection system, instead of decking. If water pollution cannot be stopped, other options such as dry weather flow interception systems for collecting and diverting the polluted flow to the sewerage system should be considered.

When decking of a nullah is proposed, the proponent shall submit a Drainage Impact Assessment (DIA) to DSD for agreement pursuant to EWTB TC (W) No. 2/2006 and DSD Advice Note No. 1 to identify the beneficial use and ownership of the decking, and to ensure that the decking proposal must not cause an unacceptable increase in the risk of flooding in areas upstream of, adjacent to or downstream of the decked section.

Where utility crossings and obstructions are found in the nullah, their effects on the hydraulic performance shall be checked to ascertain the acceptability of the drainage impacts due to the proposed decking and these utility crossings. Opportunity shall also be taken to eliminate the utility intrusions under the nullah decking project.

14. POLDER AND FLOODWATER PUMPING SCHEMES

14.1 GENERAL

Polder and floodwater pumping schemes have been in use in Hong Kong since the early 1980s. They have been adopted to protect villages in low-lying catchments in NWNT and NENT. A polder refers to a piece of lowland enclosed within an embankment, in which the water level is independent of that outside the embankment. It can be formed by the construction of flood protection embankment or similar structures in association with roads and other developments. External floodwater are prevented from entering the polder and surface runoff collected inside the poldered area will be pumped to nearby existing watercourses outside the poldered area.

Basically, a polder and floodwater pumping scheme would comprise the following components:

(a) flood protection structure (e.g. flood protection embankment or flood protection wall)
(b) internal drains, flow control devices and associated hydraulic structures
(c) floodwater storage facilities (e.g. floodwater storage pond)
(d) floodwater pumping facilities (e.g. floodwater pumping station)

The flood protection structure polders and separates the low-lying villages from the surrounding land and prevents external floodwater from entering the poldered area. Under normal rainfall conditions, surface runoff within the polder would drain by gravity via the internal village drains to the floodwater storage pond for storage and subsequent disposal. When the water level has risen to a pre-determined level, the pumps will be operated automatically to discharge the stored stormwater to nearby watercourses outside of the polder. However, it is always favourable to provide some flow control devices to facilitate gravity drainage of stormwater when rainfall is small and the water level in the nearby watercourse outside the polder is not high. In this case, the flow control devices enable the runoff to by-pass the storage and pumping facilities, and to directly discharge into the downstream watercourse.

Typical layout and arrangements for a polder and floodwater pumping scheme are shown in Figures 17 & 18. Although floodwater pumping schemes in Hong Kong have mostly been implemented for villages under rural settings, it has scope for implementation in the urban areas.

14.2 PLANNING AND DESIGN CONSIDERATIONS

In the planning and design of a polder and floodwater pumping scheme, the following should be taken into consideration:

14.2.1 Land Requirement

A polder and floodwater pumping scheme requires substantial uptake of land to accommodate the flood protection embankment, floodwater storage facilities and floodwater pumping facilities. In order to reduce the area of land resumption, it is necessary to minimize the size of these major components. For example, deep floodwater storage tank could be adopted to replace the shallow floodwater storage pond, while underground pump chamber can be built with its cover reserved for other structures and facilities.

14.2.2 Surface Water Management

When planning for floodwater storage and pumping facilities, it is necessary to study carefully the hydrology of the catchment. The discharge from the surface runoff, the ground water level and the quantity of dry weather flow will affect the volume of storage required, the detention time as well as the pump start/stop frequency. With the polder in place, the catchment will become a separate water-body which is susceptible to contamination. Suitable measures should be provided to facilitate effective surface water management for the polder on a case by case basis.

14.2.3 Choice of Pump Type

The most commonly used pump for floodwater pumping stations is the Archimedian screw pump. However, centrifugal pumps have also been used in some of the floodwater pumping stations in Hong Kong. Both types have their own merits and demerits. The Archimedian screw pump has been proven to be robust and efficient. It is most suitable for situation in which large pumping rate at low head is required. However, a screw pumping station is generally massive, noisy and visually intrusive. On the contrary, centrifugal pump is prone to damage due to clogging. It is most suitable for pumping water at high pumping head. Due to the high pressure involved, design of a centrifugal floodwater pumping station and its associated rising mains should be carried out carefully to avoid hydraulic problems caused by hydraulic surge, cavitation and creation of vortex in pump sump. Model tests may be required if such major problems are anticipated. To reduce electricity consumption and start-and-stop frequency of the pump motor, variable frequency electric motor could be considered in particular for pumping station with large flow variation. Consultation with the relevant maintenance parties should be sought in case of doubt.

14.2.4 Environmental Considerations

Environmental considerations should be carefully integrated into the design of various components for a polder and floodwater pumping scheme. The pump motors and the standby generator are the major noise sources of a floodwater pumping scheme. Acoustic linings and other noise reduction facilities can be installed to reduce the noise level. Smoke emitted from diesel generator should be properly catered for to reduce air pollution. In view of the rising ecological concern, more environmental friendly design, such as wet pond, unlined pond and channel bottom, grassed slopes, etc. should be adopted, especially when wetland compensation is required.

14.2.5 Drainage Impact to Surrounding Area

The drainage impact of the proposed floodwater pumping scheme has to be assessed thoroughly with the necessary remedial actions considered. In the detailed design stage, it is also necessary to consider the effect of implementation programmes of other nearby projects.

The construction of a polder and floodwater pumping scheme will inevitably alter the hydrology of the drainage basin. Firstly, flood storage for the basin may be reduced and flood depth in the flood plain outside the polder may be increased. Under such circumstance, it is necessary to refer to the relevant Drainage Master Plan for the overall flood risk situation of the catchment. Secondly, concentrated discharge from the floodwater pumping station to an existing streamcourse with inadequate capacity would aggravate the flooding in the surrounding area. The designer should initiate necessary steps to upgrade the downstream streamcourse to accommodate the pumped discharge.

The size of the floodwater storage pond as well as the pump cut-in/cut-out settings can also affect the flooding situation in the discharging streamcourse. It is worthwhile to manipulate the pump setting so as to sensibly alter the outflow hydrograph of the polder to avoid clashing with the peak flow of the streamcourse. With careful design, the peak water level in the streamcourse could be reduced.

14.2.6 Harbourfront Enhancement

For new polder and floodwater pumping facilities to be provided on the Victoria harbourfront, the principles and guidelines sets out in General Circular No. 3/2010 issued by the Government Secretariat of the HKSAR on Harbourfront Enhancement shall be observed. In general, the occupation of harbourfront land by public facilities that are environmentally unpleasant or incompatible with the harbourfront are not supported. Where there are no better alternatives after taking into account cost and other relevant factors, the project proponent should keep the footprint to a minimum as far as possible, and implement necessary mitigation measures to reduce the impact on the harbourfront. In addition, harbourfront access should be reserved where practicable for public use and the project proponent should landscape the harbourfront access to compensate for its occupation of the harbourfront land.

14.3 FLOOD PROTECTION EMBANKMENT/WALL

Flood protection embankment is constructed to exclude external floodwater from intruding the polder. In case reduction of land uptake is required or due to other special reasons, flood protection wall has to be built instead.

The embankment design should follow the guidelines as stipulated in GCO(1984), Section 13.7 of this manual and the relevant technical circulars. For construction of embankment in soft ground, particular attention should be paid to the rate of filling to avoid catastrophic failure due to non-dissipation of pore water pressure. Moreover, seepage through the embankment should be checked to ensure that it is small enough to be ignored in the sizing of the pumps and the storage ponds. There are requirements on submission of geotechnical design to GEO for checking and the procedures laid down in the relevant technical circulars should be followed.

A vehicular access with minimum width of 3.5 metres should be provided on top of the flood protection embankment for maintenance purpose. Considerations should also be given to open the vehicular access for public use.

Railing/Fencing shall be provided at the top edges of the flood protection embankment if there is a risk of falling from height.

14.4 INTERNAL VILLAGE DRAINAGE SYSTEM

The objective of the internal drainage system is to collect and to convey surface runoff inside the polder to the floodwater storage pond for storage and subsequent disposal. The design standard for internal drainage system should be equivalent to that of the village drainage as shown on Table 10.

Flow control devices, such as penstocks and flap valves, have to be installed to facilitate flow diversion during routine maintenance and emergency repair of the floodwater pumping station and floodwater storage pond. Such flow control devices should preferably be connected to the floodwater pumping station by telemetry for central monitoring and control. Whenever possible, the design of the internal drainage system and the flow control devices should enable the by-passing of dry weather flow away from the floodwater storage pond and the pumping station. When necessary, water level sensors can be used to enable automatic operation of these flow control devices.

14.5 FLOODWATER STORAGE POND

The open-air floodwater storage pond is less expensive to build and easy to maintain. When there is land constraint, deep floodwater storage tank can be considered. However, the construction and maintenance cost of a storage tank is generally much higher.

The floodwater storage pond should best be located at the lowest point of the polder. During the initial planning stage, the villagers' view on the type, size, location and any other proposed facilities should be sought.

14.5.1 Type of Floodwater Storage Pond

Floodwater storage ponds are mainly classified into two types: wet and dry ponds.

A wet floodwater storage pond is a pond purposely kept wet by allowing some floodwater to remain in the pond, and it is often continuously recharged by dry weather flow and groundwater seepage. It is usually adopted for ecological reason, or upon the request of residents on "fung shui" reason. In the design of wet pond, care must be taken to ensure that water inside the pond will not stay stagnant and become septic.

A dry floodwater storage pond is normally kept dry. A dry pond is preferable to a wet pond from the maintenance point of view. However, proper signage should be provided to warn the public against the possibility of flash flood.

In general, the floodwater storage pond (wet or dry) shall be fenced off against entry by the public as far as practicable. It can also be utilised for other purposes such as basket ball field and playground. If for whatever reason it must be opened for public use, special precautionary measures (other than signage) shall be in place to ensure that all persons staying within the storage pond will be evacuated at times of heavy rainfall.

14.5.2 Sizing of Floodwater Storage Pond

The sizing of storage pond for a given pumping capacity is given by the following equation:

$$Q_{in} - Q_{out} = \frac{dS}{dt}$$

where Q_{in} = inflow into the storage pond at time t
Q_{out} = pumping rate at time t, and
S = storage (volume) in pond at time t

Figure 19 illustrates the case for a pumping scheme with two duty pumps. S_{max} represents the storage volume required.

The pumping capacity and the storage volume have to be balanced to arrive at a most cost-effective combination. If a larger storage is chosen, a smaller pumping capacity can be provided and hence there will be savings in future operation and maintenance costs. Of course, this will incur a higher land resumption cost. On the other hand, a smaller storage can be chosen if larger pumps are provided. The minimum cost can be obtained with the optimisation of the storage facilities and the pumping capacity.

14.5.3 Operation and Maintenance Requirements

The operation and maintenance requirements of the floodwater storage facilities are as follows:

(a) Dry Type Floodwater Storage Pond

Peripheral surface channels should be provided around the bottom of the floodwater storage pond to convey runoff to the gravity outlet under normal situations. Vehicular access to the pond should be provided for maintenance and desilting purposes. In addition, warning signs are required to warn the public of the fact that the area may be subject to flash flooding in case of heavy rain and of the slippery condition of the pond area. Pictorial illustrations to help illiterate people are suggested.

If the dry pond is to be used also for other purposes, e.g. a playground, a satisfactory arrangement with the relevant authorities should be made concerning the future management and maintenance of the pond and the facilities before this concept can be adopted.

(b) Wet Type Floodwater Storage Pond

The public must be kept away from the pond area by warning signs, safety fences etc., which also serve to prevent children from venturing in the pond. On the other hand, vehicular access to the pond shall be provided to enable mechanised maintenance at regular interval.

(c) Floodwater Storage Tank

A floodwater storage tank is usually an enclosed underground chamber and differs distinctly from a pond. Adequate ventilation and other necessary provisions for enclosed area should be considered. Openings, accesses and heavy duty lifting appliances should be provided to enable future maintenance. Cat-ladder shall be provided at the side of the access manhole. In addition, the floodwater storage tank shall be regularly drained to avoid any water/contaminants detained inside from becoming septic.

14.6 FLOODWATER PUMPING STATION

14.6.1 General Requirements

Floodwater pumping stations should be designed to operate automatically with appropriate type of pumps which are controlled by water level sensors installed in the pump sump. Accumulated stormwater from the flood storage facilities is pumped to the nearest main drainage channel outside the polder.

A good appearance is one of the functional requirements of pumping stations. This is particularly important for pumping stations located in urban areas and exposed to public views. For those in village areas, local residents should be consulted as far as possible. To enhance the appearance of pumping stations, external finishes are usually provided for superstructure above ground. The construction cost for finishes together with the future maintenance cost should be taken into account in selecting the type of finishes to be used. To save cost, fair concrete finishes may be considered for internal walls of unmanned pumping station. However, the floor of pumping station should be painted with anti-skidding coats. Architectural Services Department should be consulted for their advice on the aesthetic design of pumping stations.

14.6.2 Design Capacity

The design capacity for all duty pumps should be adequate to handle rainstorm runoffs collected inside the polder with a return period of 10 years. Stand-by pumps must be available and should be able to automatically take over the failed duty pumps. Both duty pumps and stand-by pumps should be interchangeable. The standby pumps should be so designed such that they can also be activated in case of exceptionally severe rainstorms.

In the determination of the pumping requirements, the following guidelines should be followed:

(a) The desirable design freeboard for floodwater storage facilities should be 300 mm for a storm with a 10-year return period.

(b) Combinations of stand-by pumps and duty pumps can be formulated for an individual village flood protection scheme so that should any of the duty pumps in the station be inoperable due to routine maintenance or mechanical failure, the design capacity of the pumping station can still be maintained. As a simple rule, the total capacity of the stand-by pumps should be at least 30% of the total capacity of the duty pumps.

(c) The total maximum pumping capacity (both duty pumps plus stand-by pumps) of a pumping station should be able to accommodate a storm with a 50-year return period.

The flood depths inside the polder during a 50-year and a 200-year rainstorm under the proper functioning of duty pumps and the floodwater storage facilities should be checked. If the flood depth is considered intolerable, the capacity of the storage facilities should be increased to suit.

14.6.3 Operation and Maintenance Requirements

In designing a floodwater pumping station, vehicular access and parking area should be provided for maintenance vehicles. Unhindered direct access from a public road to the pumping station is required for the acceptance of FSD for fire fighting purpose. Gateway of adequate size should be provided for vehicles to deliver and remove the bulky equipment.

To facilitate maintenance of the pumps, penstocks should be installed within the pump sump for isolation of pumps from the floodwater storage pond/tank. Lifting appliances should also be provided for lifting of pumps and other heavy equipment. Water level sensors controlling the operation of pumps should be installed at suitable locations which will not be subjected to local fluctuation of water level and the interference of floating debris. Water level sensors should also be adjustable such that the pump cut-in/cut-out levels could be varied to suit different operating conditions. For future maintenance of screw pumps, a loading platform is normally required.

Power supply should be adequate for running the control system and all the pumps. In addition, an emergency power generator must be provided within the station compound to provide back-up electricity automatically during power failure. The generator must be designed to supply sufficient power to operate the control system and the pumps. A fuel oil storage tank is required to be installed in compliance with FSD’s requirements. The minimum capacity of the fuel storage should allow for 36 hours of operation of the generator when running all duty and standby pumps. It would be preferable that a dual-feed power supply could be obtained from the electricity company.

In addition to the above, the following points should also be considered for the operation and maintenance of a pumping station: -

(a) Pumping station should be isolated from flood storage pond by fence or boundary wall to prevent trespassing.

(b) Noise abatement measures should be provided to minimise disturbance to nearby residents and the operation and maintenance (O&M) personnel.

(c) The configuration, structures, pump sump and designed level of lift of the pumping station should be such that in the event of a dire emergency, a portable pump can be put in place to pump as much floodwater as possible from the storage pond to the pumping station outlet.

(d) Details of the pump house including parapet walls on the roof, louvers, etc. should be agreed with the relevant maintenance parties.

14.6.4 Structural Design Requirements

The structure of floodwater pumping station should be able to sustain all combinations of loading during its design life. Both the serviceability and ultimate limit states should be checked.

The floodwater pumping station can be divided into two parts namely superstructure and substructure. The superstructure mainly consists of the motor room and control room which may be designed as slab, beam and column or as plane frames in accordance with the BSI (2002) and BSI (2004) or their latest versions. The substructure mainly consists of the discharge chamber and pump sump which should be designed as liquid retaining and containment structure in conjunction with BSI (2006) or its latest version.

For pumping station with superstructure, dead load, imposed load, wind load and impact load, if any, should be considered in the design. Attention should be paid to any machinery load or crane load under operation. For the substructure, earth pressure and loads from underground hydrostatic pressure should be considered. Durability and serviceability requirements of the structure are provided in Sections 14.6.4.1 to 14.6.4.4 below. The stability of the structure against flotation due to groundwater under most adverse situation should also be checked in accordance with BD (2004/2) or its latest version.

The locations of expansion joints, if required have to be carefully placed to avoid possible movements not anticipated.

The design memorandum and calculations for a pumping station should comprise the following:

(a) the design standards and parameter adopted;

(b) the design assumptions;

(c) the design data;

(d) the design approach and the critical loading cases;

(e) detailed structural analysis with cross reference;

(f) summary of output; and

(g) sketch showing the reinforcement arrangement for critical section.

14.6.4.1 Exposure Conditions

Since the discharge chamber and pump sump of floodwater pumping stations will generally be exposed to stormwater without sea water attack, the substructure of floodwater pumping stations should be designed for exposure class "XC4" as defined in Table 4.1 of BSI (2004) or its latest version. Exposure class "XC3" shall be used for the design of superstructure.

14.6.4.2 Strength of Concrete

Given the required water retaining properties of discharge chamber or pump sump, minimum concrete Grade ($f_{ck,cube}$) 40 is recommended under "XC3" and "XC4" exposure classes. Since the design equations in the structural design to EC and UK NA are made use of the cylinder strength (f_{ck}), the designer may make reference to Table 3.1 of BSI (2004) or its latest version, or the following approximate equation for conversion of the cube strength to cylinder strength when carrying out the design.

$$f_{ck} = 0.8 \times f_{ck,cube}$$

where f_{ck} = characteristic compressive cylinder strength of concrete
$f_{ck,cube}$ = equivalent characteristic cube strength of concrete

14.6.4.3 Maximum Permissible Crack Width

The tightness class of superstructure and substructure of floodwater pumping stations shall be "Tightness Class 0" and "Tightness Class 1" respectively in accordance with BSI (2006) or its latest version.

For structures under "Tightness Class 1", the designed maximum permissible crack width shall be designed so that design crack widths should be limited to w_{k1} as defined in BSI (2006) or its latest version. The limiting crack width w_{k1} varies according to the ratio (i.e. h_D/h) of hydrostatic pressure, h_D to the wall thickness of the containing structure, h. For h_D/h less than 5, w_{k1} shall be equal to 0.20 mm while for h_D/h greater than 35, w_{k1} shall be equal to 0.05 mm. Linear interpolation of w_{k1} is allowed for intermediate values of h_D/h in accordance with BSI (2006) or its latest version.

14.6.4.4 Concrete Cover to Reinforcement

Concrete cover to reinforcement should be provided in accordance with the following nominal values for the envisaged conditions of exposure. The value of nominal cover shall be the minimum concrete cover as recommended in BSI (2004) or its latest version plus the allowance for deviation taken as 10 mm. The value of nominal cover shall be indicated on the drawings and be used in design of reinforced concrete structure.

Exposure Class	Nominal Cover (mm)
	Concrete Grade $f_{ck,cube}$ = 40
XC3 or XC4	35

14.7 TRASH SCREENS

Trash screens at the inlets to the pump sump and those at the outlets from the floodwater storage pond are required to prevent large flooding objects from damaging and clogging the pumps. The screens are normally of manual hand-raked type. The bar spacing and the bar size of the trash screens should be properly designed to avoid possible blockage of the pumps. It is essential that proper access to each screen be provided and a working platform installed over every screen above flood levels to facilitate routine and emergency raking. If it is found to be cost-effective, mechanical raking devices should be provided to save manpower in raking.

The trash screen is considered to be one of the crucial components of a floodwater pumping scheme, and should be monitored by video surveillance.

14.8 MONITORING AND CONTROL SYSTEMS

By utilising the latest information technology, a monitoring and control system should be provided for monitoring, inspecting and control of floodwater pumping schemes. The system should comprise:

(a) a telemetry system for monitoring and control of plant operation status of the pumping station as well as other hydraulic structures, such as the inlet chambers. The monitoring and control signals should include water levels in the floodwater pond, status of power supply, pumps, penstocks, screens, generator set, fuel oil storage tank, fire alarms, telemetry fault, etc.

(b) a video surveillance system for the visual monitoring of crucial electrical and mechanical (E&M) and civil components (such as the pumping station outlets to existing watercourses) to ensure proper functioning of the whole scheme at any time.

A Master Control System (MCS) should be implemented to minimise the staffing requirement on manning the floodwater pumping stations. Under this system, a control centre will function as the master station with other pumping stations connected to it as outstations. The surveillance signals of outstations will be transmitted to the master station (a control centre) where remote monitoring and control of the system can be carried out. The control centre should preferably be able to serve several floodwater pumping stations and be situated at a convenient location with all-weather unhindered vehicular access. The control centre should be manned 24 hours when necessary throughout the wet season. In case operational fault in outstation is reported, the control centre should be able to provide emergency support to the outstations.

14.9 MISCELLANEOUS ISSUES

14.9.1 System Commissioning

To ensure proper functioning of the floodwater pumping station, a system commissioning test should be conducted successfully in at least three consecutive days. Prior arrangement for the provision and storage of water and the re-circulation facilities have to be made to enable smooth running of the pumps.

14.9.2 Operation and Maintenance Issues

The major operation and maintenance parties for floodwater pumping schemes are the Sewage Treatment Division, the respective Operation and Maintenance Division and the Building & Civil Maintenance Team of DSD. Their comments should be sought as early as the design stage to agree on the construction details.

Upon commissioning of a floodwater pumping scheme, system manual of the scheme, operation and maintenance manual of the plant equipment, a full set of civil, electrical and mechanical drawings, and all required test reports and warranty documents should be provided to the operation and maintenance parties to facilitate the future operation and maintenance activities.

In general, flood protection embankment slopes and associated retaining walls have to be registered in GEO according to the relevant technical circular. Prior arrangements have to be made with GEO and the maintenance agency as to the registration of the slopes, the maintenance responsibilities and the display of the registration numbers on site. Standard DSD sign boards for slope should be used for the display of the registration numbers.

14.9.3 Division of Maintenance Responsibility

The maintenance and management responsibilities of various departments concerned should be clearly defined in the early planning/design stage. An example of the schedule of responsibilities for a completed polder and floodwater pumping scheme is shown in Table 24.

14.9.4 Future Extension

Although the area of catchment inside a polder is usually constant, change in land use, especially from fish pond and agricultural land to paved area, will affect the response of the catchment and thus the amount of surface runoff to be pumped. In designing the general layout of the floodwater pumping scheme, the pumping station and the floodwater storage facilities should be carefully sited to enable future extension, such as the additional of some more pumps and the deepening of the storage pond.

15. OPERATION AND MAINTENANCE OF STORMWATER DRAINAGE SYSTEMS

15.1 GENERAL

The proper maintenance and operation of stormwater drainage systems is essential if the works are to achieve their designed objectives. This Chapter describes some recommended practices and gives guidance on this aspect to assist personnel who are involved in the day-to-day operation and maintenance of the stormwater drainage system.

15.1.1 Maintenance Objectives

The objectives for proper maintenance and operation include:

(a) to offer a quality of service that is acceptable, having regard to costs and the effects on the environment, and to remedy recognised deficiencies.

(b) to monitor the capacity of the system and to restore the flow capacity by removal of excessive accumulation of silt and grease, etc.

(c) to monitor and maintain the structural integrity of the system.

(d) to prevent excessive infiltration and inflow.

(e) to desilt for environmental reasons so as to mitigate nuisance to the public.

(f) to provide feedback when necessary on the need for improvement and upgrading works.

(g) to achieve the above service objectives making the best possible use of manpower and resources at the least cost and least disruption to the public.

15.2 HANDING OVER OF COMPLETED WORKS

15.2.1 Procedures for Handing Over

To ensure that the works can be readily handed over to the maintenance authorities on completion, the standard of design and maintenance requirements laid down in this Manual must be fully complied with. Additionally, close consultation and liaison should be maintained between the design office and the maintenance authorities at each stage of the project. During the planning and design stages of a project, a design memorandum should be prepared so that the design parameters, handing over requirements or partial handing over arrangement of large project can be agreed by the maintenance authority. For non-standard drainage items, detailed consultation is required such that the operation and maintenance requirements can be incorporated into the design. If unforeseen problems are encountered during construction and changes have to be made, the maintenance authority must be consulted as soon as possible so that the changes can be accepted. On completion, any changes made should be incorporated in the design memorandum before handing over of the completed works to the maintenance authorities.

Prior to handing over of the works, joint inspection must be carried out and any outstanding works agreed. On substantial completion of the works, a handing over inspection should be carried out to ensure that all outstanding works have been completed before the issue of the completion certificate. Within 3 months of issuing the completion certificate, the final operation and maintenance (O&M) manual for electrical and mechanical (E&M) works, as-built drawings and calculations should be submitted. Prior to the end of the Maintenance Period, a joint inspection should again be carried out to check if further works are required and that all outstanding or remedial works have been completed.

Reference should be made to the Project Administration Handbook and the relevant technical memoranda for details of handing over and taking over procedures.

15.2.2 Handing Over in Dry Conditions

All pipes, channels and culverts, etc. to be handed over should be inspected in dry conditions wherever possible. In the case where the pipes, culverts or channels have to be commissioned prior to handing over (e.g. due to the requirement to maintain the existing flow or staged completion) and a temporary diversion of flow is not feasible, an additional inspection should be arranged prior to the commissioning. In certain circumstances, closed circuit television (CCTV) survey of the pipes and internal faces of the manholes showing each connection pipe before commissioning can be adopted as an alternative to the joint inspection but prior agreement with the respective operation and maintenance division of DSD should be sought.

15.2.3 Documents to be submitted

After the satisfactory handing over inspection, the following documents should be submitted as soon as possible, but it should not be later than 3 months under any circumstance:

(a) as-built drawings, in hard-copy and electronic format if applicable.

(b) hydraulic and structural design calculations, in electronic format if available.

(c) construction records including major acceptance tests and material quality records, product specifications and warranties.

(d) O&M manual and system manual, where appropriate.

(e) maintenance manual for slope embankment, where appropriate.

In the event that as-built drawings are not available at the time of the handing over inspection, marked up prints of the working drawings showing the final amendments and the extent of works to be handed over should be provided. Records of material quality and acceptance tests should also be available for scrutiny.

15.3 INSPECTION AND GENERAL MAINTENANCE OPERATIONS

15.3.1 Inspection Programme

Inspection of all existing drainage installations should be carried out regularly to ensure that the systems operate properly. Special attention should be paid to any signs of deterioration in the systems both hydraulically and structurally, since any structural defect, blockage, leakage or siltation detected at its early stage of formation would allow preventive remedial works to be carried out at lower cost. The frequency of inspection should be determined principally based on the nature and importance of the installations, the likely consequence in the event of malfunctioning of the system, the frequency of drainage complaints received in the vicinity and the resources available. Priority should be given to the system installations where the result of any failure would be serious or the remedial works particularly expensive.

Table 25 shows the recommended frequency of inspections of some typical drains. However, drains in some locations may require more frequent inspections to meet particular needs. Each case should be considered individually according to its specific circumstances.

15.3.2 Closed Circuit Television Surveys

Apart from general visual inspections, closed circuit television (CCTV) surveys can also be used to investigate the condition, in particular the structural integrity of the drains in close details.

It is essential that CCTV surveys are conducted during low flow conditions. If the flow quantity is large, the drain upstream should be temporarily blocked and the flow diverted. An adequate lighting system should also be adopted so as to produce a clear picture of the drain. Pipes which are silted and the surfaces coated with grease should be cleansed prior to the survey.

The defect coding, structural assessment and scoring system of culvert or pipeline on which CCTV survey has been carried out shall be done in accordance with “Manual of Sewer Condition Classification” and “Sewerage Rehabilitation Manual” published by Water Research Centre" to determine the priority of remedial works and the future inspection programme.

15.3.3 Inspection of Special Drains

(a) Drains within Red Routes, Pink Routes and Expressway

‘Red’ and ‘Pink’ Routes are classified by HyD as the major road network in Hong Kong. The Red Routes and Pink Routes are sections of the major road network where the capacity and nature of the alternative routes is limited and the potential impact is very high if these routes are either partially or totally closed. Details of the Red and Pink Routes are shown in the relevant Highways Department Technical Circular. Due to the importance of these routes, it would be highly undesirable to carry out unplanned works within these areas.

In order to minimize the occurrence of emergency or crisis drainage repair and clearance works, regular inspections of the drainage system within the Red and Pink routes should be carried out so that preventive maintenance can be well planned and performed outside the peak traffic hours.

For works to be carried out within expressways, the requirements as stipulated in the Road Traffic Ordinance, and in particular the safety aspects, should be observed. It should be noted that all expressways are either Red Routes or Pink Routes and allowable working time may be restricted. Drainage inspection programmes should be carried out in conjunction with Highways Department's cyclic lane closure programmes. Detailed procedures and criteria for working in expressways are given in the relevant Highways Department Technical Circular.

(b) Drains behind Slopes

Persistent leakage of water from sewers and stormwater drains (including gravity pipes, channels, tunnels and rising mains) not only causes nuisance, but can also be a serious risk to the stability of slopes and retaining walls. Such leakage can deliver a significant amount of water into the ground and its potential effect on the stability of the slope or retaining wall should not be disregarded. Preventive measures in the form of regular inspection and maintenance should be carried out with reference to "Code of Practice on Monitoring and Maintenance of Water-carrying Services Affecting Slopes" (ETWB) and the latest Geoguide requirements. Where defective drains are found, repairs should be carried out immediately.

As a minimum, sewers and drains located within a distance of H from the crest of a man-made slope/retaining wall, where H is the maximum vertical height of the slope/retaining wall, should be inspected at the frequency in accordance with "Code of Practice on Monitoring and Maintenance of Water-carrying Services Affecting Slopes" (ETWB) depending on the type of slopes. The distance from the crest of the slope should be further extended where the sewers and drains are known to be leakage prone. Particular attention should be paid to pressurized rising mains as their leakage or bursting may lead to severe damage. More frequent leakage detection may be desirable for those rising mains behind slopes and retaining walls in the high Risk-to-Life (i.e. Consequence-to-life) Category as classified by GCO (1984). The frequency shall be decided based on the prevailing conditions of the slopes/retaining walls and the rising mains. Reference can be made to WB (1996). Records of inspections should be sent to the maintenance agent of the slope likely to be affected by the sewers or drains.

15.3.4 Desilting Programme

Desilting of pipes and culverts is required so as to maintain their capacity and in some cases to alleviate the odour problem. The frequency of desilting varies from pipeline to pipeline. This depends on the pipe size, gradient, flow condition, etc. and is subject to verification by inspection results. Past experience indicates that regular desilting at complaint black spots can reduce the number of complaints. However, in many cases desilting may not be the most effective method and consideration should be given to long-term solutions including improvement or modification of the existing drainage system.

Where open channels, nullahs and rivers are subject to tidal effect, regular monitoring or survey should be carried out to ascertain the degree of siltation so as to determine the frequency of desilting.

In open channels and inlets to box-culverts, desilting operations are sometimes necessary for aesthetic or environmental reasons such as odour problem. The most common situation is where squatter areas or agricultural activities are present upstream of the engineered channel. Under such circumstances, desilting operations are often required at a much more frequent interval and there are areas where desilting is carried out more than once every month.

15.3.5 Methods for Desilting/Cleansing

Manual rodding and scooping is the simplest method used in pipe cleansing. A rattan rod with its head mounted with a hook or spike is driven manually into the pipe to pierce the blockage. Solids produced will be collected at the downstream manhole and removed by scoops. This method requires the least equipment and the set-up time is minimal. It is very effective in clearing local blockages caused by refuse or debris. However, it does not clean the pipe thoroughly and the blockage may reoccur very shortly. Its application is restricted when the manhole is deep, length of the pipe is long or the pipe size is large.

Water jetting is a common method for pipe cleansing. A hose is led into the pipe, usually from the downstream, and water is jetted out under high pressure up to 20 MPa pushing the hose forward while at the same time washing away the substances accumulated inside the pipe. This method is particularly effective in clearing blockages caused by oil and grease. It is also very effective in clearing the grease coated onto the interior surface of the pipes so as to explore the pipe surface condition. However, the effectiveness of water jetting decreases with the increase in pipe diameter and is seldom used for pipes greater than 900 mm diameter. For pipes of length exceeding 100 m, the use of water jetting is also not effective due to the excessive headloss in the hose.

Apart from normal cleansing, there are proprietary products available in the market for mounting onto the head of the water jetting hose for breaking through hard material. Some of the products have been used in Hong Kong and are found to be useful for breaking out cement mortar in a semi-solid state deposited inside the pipe.

Winching is the most frequently used method for the thorough cleansing of pipes. A ‘ball’ or bucket is towed along a section of drainage pipe between two manholes by a pair of winches. This action is repeated several times and the silt and debris inside the pipe can be scraped out. This method can be used for various sizes of pipes and is very effective in removing silt and medium sized particles inside the pipe. Some specially made ‘ball’ can also be used for breaking out hard material.

For large size box-culvert subject to tidal effect, desilting under submerged conditions is labour intensive and very difficult. It is desirable to desilt the box-culvert in dry condition. This can be achieved by using stop-logs or other device together with pumping.

For large open channels, nullahs and rivers with invert level below the tidal range, specially made floating pontoons with excavators or cranes mounting on them have been used in conjunction with grabs, air lift/suction or dredging for desilting.

Clearance of livestock's waste from dry weather flow interceptor should be carried out according to ETWB TC(W) No. 14/2004 with close liaison with concerned government departments.

15.4 STORMWATER DRAIN REHABILITATION

15.4.1 Pipe Replacement

When pipes are found to be damaged, repair work should be carried out as soon as possible. Replacement of damaged pipes by open excavation is a commonly used method.

To replace defective pipes by open excavation method, attention should be drawn to the following:

(a) maintenance of the existing flow

(b) road traffic conditions

(c) presence of underground utilities

(d) nuisance and inconvenience to the public

(e) excavation dewatering

(f) working area and shoring requirements

Close liaison with the utility undertakings and traffic authorities is required before the replacement work is carried out so that suitable construction methods can be determined.

The drawback of the open excavation method is that it may occupy substantial road space for a long period of time. In the urban area with heavy traffic, the economic loss due to traffic disruption as a result of open excavation is becoming hard to justify. At the same time, people's aspiration is rising and they are becoming less tolerant of traffic disruption. As a result, when drainage repair or improvement is required, trenchless methods for pipe rehabilitation should be considered as alternatives to open excavation.

15.4.2 Trenchless Methods for Repairing Pipes

For most trenchless methods, the scope to increase the flow capacity is rather limited. In general, the unit cost of trenchless renovation is higher than conventional open cut technique where the pipeline to be replaced is shallow and there is no obstruction due to underground utilities or other physical structures. However, when the need for increased flow capacity is not a deciding factor, trenchless renovation method can be employed with benefits of keeping social costs and economic losses to a minimum as well as avoiding physical obstruction problems that would otherwise arise if conventional method is used. However, if the defective drains in the urbanized area have become under-capacity, opportunity should be taken during the remedial works to replace them by larger pipes so that the overall capacity of the network can be increased to cope with any anticipated developments.

Some typical trenchless methods which can be used for the rehabilitation of defective drains are described in the following paragraphs. It should be noted that the list is not exhaustive and other methods may also be applicable.

Localised Repairs and Sealing

(a) *Joint Grouting*. This method is applicable to drains which are leaking through the joints but the drains are still structurally sound. Chemical grout is injected into the leaking joint filling up the void surrounding it to stop further leakage. For small drains, the chemical grout is internally applied by an inflatable packer guided by a CCTV camera and the same packer is used to test for air tightness of the grouted joint. For large pipes, it may be more convenient to send people into the drains to carry out the grouting directly.

(b) *Mechanical Sealing*. This method involves the installation of a metal band or clip faced with an elastomeric material at the damaged section of pipe, which forms a seal with the inner surface of the pipe. It has the advantage of not relying on in-situ chemical reaction, and can also be installed quickly.

Mechanical sealing systems are available for spot repair of pipe of either man-entry or non-man-entry. For non-man-entry pipe, the repair modules are installed by means of an inflatable packer which expands the clip and presses the rubber against the pipe wall. The packer can then be deflated and withdrawn.

Internal Lining

(c) *Internal Lining using Epoxy Impregnated Liner*. This method uses a factory fabricated lining tube conforming with the internal dimension of the drain to be rehabilitated. The liner consists of one or more layers of polyester felt in contact with impervious polyurethane membrane, the thickness of which are chosen to suit individual requirements. The polyester lining is firstly impregnated with specially formulated resin in the factory. After delivery to site, the liner is inserted into the defective drain and properly expanded so that its external surface is in contact with the interior of the defective drain. A high temperature environment is introduced inside the liner to enable the resin impregnated polyester felt to cure, harden and form a continuous solid pipe inside the original pipe. Any branch connection to the relined drain can then be reopened with a remotely controlled hole cutting machine.

The method is generally applicable for small to large size pipes, and even for oval and egg shaped drains. It can negotiate through smooth bends but wrinkles may develop at sharp bends. It adds extra structural strength to the original pipe and by proper design of resin, it offers good chemical and corrosive protection from all sort of environment. It provides a smooth surface to the pipe, and may even improve the flow capacity.

Special equipment is required to ensure proper and even impregnation and to remove the air inside the polyester felt. The proper use of inhibitors and the control of the surrounding temperature are important to prevent premature curing before insertion. The set up for insertion and heating is also demanding. The method is expensive due to the high mobilization cost, especially if only a short length is to be lined.

(d) *Internal Lining using Pre-deformed Polyethylene Liner.* This method involves placing a factory made polyethylene liner inside the defective drain. The liner is first deformed, then wound on a reel and delivered to site. It is pulled through the existing drain by winches. Water at high temperature and pressure is then used to cure and to restore the circular shape so that the liner fits snugly inside the old pipe.

The method is applicable for drains ranging in size from 100 to 450 mm diameter. It can only negotiate through large radius bends, and is not as flexible as the liner in paragraph (c) above. It can be used to seal up joints and cracks, and to improve the flow characteristics and chemical resistance. However, the interior of the old pipe should be adequately smooth and no serious obstructions should exist in order that the liner can be pulled through.

(e) *Internal Lining using a Smaller Pipe*. This method involves pulling/pushing a thin walled pipe such as steel pipe, GRP pipe, HDPE pipe and so on, through a defective drain. The external size of the new pipe is smaller than the internal size of the old pipe. The pipes are generally jointed by welding as the pipes are pushed. The annular space between the new and the old pipes is grouted by cement/concrete.

The main disadvantages of this method are the large reduction in size and the need for a large working space at the inlet. Branch connections are quite difficult to restore except for large drains that are manually accessible.

<u>On-line Replacement</u>

(f) *Pipe Bursting.* This method, sometimes called the pipe eating system, employs powerful hydraulic expanders or bursters which progressively destroy and expand the old pipe as it advances itself through the pipe. The replacement pipe, generally of larger size, is pulled in behind the bursters. The method has been used overseas but its potential applicability in Hong Kong is rather limited as it may easily cause damage to other utilities in the close proximity. However, computer aided design checks are now available to assess whether this technique is safe. This method is normally employed only for small diameter pipes as huge bursting force is required for the burster. Specialist should be consulted when considering employing this method.

15.5 POLDER AND FLOODWATER PUMPING SCHEMES

15.5.1 Operation

Polder and floodwater pumping schemes (PFPS) are designed for unmanned automatic operation under the control of water level electrodes in the floodwater storage ponds. The monitoring is by way of video surveillance and a telemetry system to transmit fault alarm signals to the nearest manned DSD installation.

As PFPS's are designed for the protection of lives and properties of villagers, it is considered that a close surveillance is required. The operation and detailed regular inspection and maintenance activities should be well recorded. Any defect or irregularity identified should be reported immediately so that prompt action can be taken to maintain the service conditions and the PFPSs can operate effectively when the need arises.

For proper upkeep and running of PFPSs, it is necessary that regular inspections, testing and trial operations must be put into effect. The routine operation functions should include the regular inspection of the floodwater storage pond, drainage channels, operation of penstocks/flap-gates, test running of machinery, pumps, telemetry devices, etc. and the schedule of inspections in the following sub-section should be followed.

15.5.2 Schedule of Inspection

Table 26 shows a Schedule of Inspection for Polder and Floodwater Pumping Schemes.

The frequency of inspection should be adjusted according to the prevailing circumstances of the individual station or the weather conditions. It is recommended that an inspection should be made after each significant storm event.

15.5.3 Documentation

The staff responsible for the operation and maintenance functions as described in the above sub-section should record down the inspection results in a brief tabular report.

Dated photo reports with simple descriptions, taken at prescribed locations and angles, will generally be sufficient for the majority of the uses, especially as regards the routine inspections. Video recording may be used as a feasible alternative if more extensive viewing is required. It is suggested that all representative inspection reports should be kept in file for future review purposes.

15.5.4 Operation during Rainstorms, Tropical Cyclones or Similar Situations

During rainstorms and other similar situations, continuous monitoring of the situation will become necessary even though the PFPSs remain in the automatic mode of operation. Regional patrol teams should be set up and they should be called in to inspect and attend to the electrical-mechanical installations under such circumstances.

15.6 CONNECTIONS TO EXISTING DRAINAGE SYSTEM

15.6.1 Existing Capacity

When a connection to the existing drainage system is required, the capacity of the existing system should be checked to see whether it has adequate spare capacity to accommodate the additional flow from the proposed connection and whether enlargement or duplication work is required.

15.6.2 Terminal Manholes

For every drainage connection from a private development, government building, park or housing estate, etc., a terminal manhole in accordance with relevant DSD Standard Drawings should be provided and positioned within the allocated land as near to the site boundary as possible.

15.6.3 Provision of Manholes

Manholes should be provided for connections made to existing pipes. Direct connection by Y-junction to existing pipes should only be allowed in exceptional cases and should be well justified. For existing large diameter pipes where construction of new manholes will be difficult, connections should be made to existing manholes.

15.6.4 Provision of Water Seal Trap

Exclusive road drains of short length and roadside gullies directly connected to large diameter drains or box culverts in tidal zone identified to have pollution issue should be provided with water seal trap in order to alleviate potential odour problem.

15.7 DRAINAGE RECORDS

The existing drainage records should be continually updated to include all the newly constructed stormwater drains and installations.

For all new works handed over to DSD for maintenance, as-built drawings as specified in para. 15.2.3, in hard copy and electronic format, containing the geographical and topographical data should be passed to the drawing office for retention and incorporation into the existing drainage record drawings. All manhole positions with details on cover levels, invert levels, diameters and directions of all the connecting pipes should be given in the drawings. For other special installations and special manholes, detailed drawings are required. The hydraulic and structural calculations, in hard copy and electronic format containing the hydraulic models, should be provided to supplement the drawings.

For all repair works, drainage connections and minor improvement works carried out during maintenance operations, a survey should be conducted on completion of the works to record all changes in levels, positions and sizes. The results of survey should be passed to the drawing office of DSD for updating the drainage records.

15.8 SAFETY PROCEDURES

15.8.1 Safety Requirements for Working in Confined Space

Working in a confined space such as an underground drain, box culvert, tanks, etc., is potentially dangerous. Great care must be taken at all times, particularly when working under adverse weather conditions. The legislative requirements of the Factories and Industrial Undertaking (Confined Spaces) Regulation have to be followed. Reference should be made to LD (2000), DSD Practice Note No. 1/2007 and DSD Safety Manual (2010) or their latest versions, for the legislative requirements and good safety practice for working in confined space. The essential elements of which include: -

(a) Appoint a "competent person" to carry out a risk assessment and make recommendations on safety and health measures before undertaking work in confined space.
(b) Allow only "certified workers" to work in the confined space.
(c) Operate a "permit-to-work" system.
(d) Conduct atmospheric testing of the confined space before entry.
(e) Provide adequate ventilation.
(f) Isolate the confined space.
(g) Ensure a "standby person" is stationed outside the confined space to monitor the weather condition and maintain communication with the workers inside.
(h) Ensure the use of approved breathing apparatus (if recommended in the risk assessment report) and other necessary personal protective equipment by workers inside the confined space.
(i) Formulate and implement appropriate emergency procedures to deal with serious or imminent danger to workers inside the confined space.
(j) Provide necessary instructions, training and advice to all workers to be working within a confined space or assisting with such works from immediately outside the confined space.

Based on experience, it is considered necessary to add a second line of defence to enable an early warning signal to be given out so as to increase the possibility of escape or being rescued when the prescribed safety measures fail. The following enhanced safety measures are introduced for DSD confined space work, unless the risk assessment demonstrates that such measures produce no added benefit to safety at work.

(i) Continuous Gas Monitoring: The person entering a confined space shall bring along a gas detector, which can give out warning signals of the sudden presence of dangerous gases or oxygen deficiency, to continuously monitor the atmosphere so as to enable immediate evacuation, and

(ii) Personal Alarm: A personal alarm of dead-man type, which is able to give out signals soon after a person loses his mobility, shall be worn by all persons entering a confined space to facilitate early rescue.

15.8.2 Working under Adverse Weather Conditions and during Flooding

Officers should always take note of the prevailing warning messages issued by the Hong Kong Observatory, in particular the following:

(a) Thunderstorm Warning

(b) Rainstorm Warning Signals
(c) Tropical Cyclone Warning Signals
(d) Landslip Warning
(e) Strong Monsoon Signal
(f) Special Announcement on Flooding in the Northern New Territories
(g) Very Hot Weather Warning
(h) Cold Weather Warning
(i) Frost Warning
(j) Fire Danger Warning
(k) Rain Alert

Some safety guidelines for working under adverse weather conditions are given in DSD Safety Manual (2010) or its latest version.

16. TRENCHLESS CONSTRUCTION

16.1 INTRODUCTION

Trenchless (no-dig) construction methods have been used in various DSD projects and some of the successful applications were presented in the technical papers identified in the reference list. Trenchless construction method in this chapter refers to means for laying of pipeline (or construction of culvert) of less than 3 metres in diameter without opening up the ground surface above, which might be a more cost-effective alternative if it is at all allowed or permissible. The difficulties for opening up the ground surface in Hong Kong especially in the vicinity to the heavily inhabited area are various such as unbearable disruption to traffic or business activities, physical obstructions (above or below ground), prolonged construction period, construction problems and adverse factors on environmental & other technical grounds.

Trenchless construction methods can be classified based on whether man-entry would be permitted for the normal excavation operation along the alignment of pipeline (or culvert). In this regard, they are broadly classified into two main types, namely ‘man-entry’ type and ‘non-man-entry’ type for the context of this Manual. The main reason for such classification is safety orientated as the safety concern and requirements would be more stringent for former type.

For the avoidance of doubt, if some smaller diameter inner pipelines are to be laid within a larger sleeve pipe or lining pre-formed beforehand, only the operation for constructing the sleeve pipe or lining is to be governed by this classification. The safety measures or requirements for subsequent laying of the inner pipelines especially when such operation might render health or safety hazards to workers such as difficulties to swift evacuation should be dealt with by other provisions under the construction contract.

Similar to other confined space work under DSD’s jurisdiction, contractor’s workers who need to participate in the trenchless construction would need to comply with the competence enhancement training requirements stipulated in DSD Technical Circular No. 3/2012, or its latest version, as appropriate.

16.2 NON-MAN-ENTRY TYPE

The usual conditions associated with ordering this type of trenchless method includes pipeline at considerable depth, long distance between adjacent access shafts and susceptible ground conditions. After stipulating this type of trenchless method in the construction contract, strictly no man-entry for normal excavation operation of pipeline (or culvert) should be allowed for unless the Engineer is satisfied that the risks perceived at the planning and design stages have all been cleared or for performing rescue operation for the malfunctioning tunnel boring machine or other essential equipment trapped below ground.

Under this type of trenchless method, those types of tunnel boring machine (TBM) equipped with a remote-control for normal excavation operation can usually be adopted. However, some models of TBM which require frequent manual removal of foreseeable obstruction such as hard strata or rock are unlikely to meet the said ‘non-man-entry’ requirements in this connection. In addition, other means known as hand-dug tunnels, headings and hand shield methods that require constant manual input for normal excavation operation obviously cannot meet such requirements.

Some basic information in respect of these methods that can meet the ‘non-man-entry’ requirements are highlighted below which are by no means exhaustive. Project engineers and designers are to satisfy themselves that any particular method accepted can really perform the required functions.

16.2.1 Slurry Pressure Balance Method

It is a TBM with a bulkhead located behind the face to form a pressure chamber. Bentonite slurry or other medium is introduced into the chamber under appropriate pressure to equalise ground pressure and to be mixed with material excavated by rotary cutterwheel. The bentonite slurry forms a temporary filter cake on the tunnel face which the slurry exerts pressure to support the ground. The continuously forming filter cake will be cut away by the rotary cutterwheel as the TBM advances. The excavated spoil will be removed from the pressure chamber by the slurry circulation system.

16.2.2 Earth Pressure Balance (EPB) Method

The EPB method consists of a cutting chamber located behind the cutterhead. This chamber is used to mix the soil with water foam/soil conditioner. It is maintained under pressure by the screw mucking system. The ground at the cutting face is supported by the resultant pressure balancing the increase in pressure due to advancement of the TBM and the reduction in pressure due to discharge of the excavated spoil.

The underlying principle of the EPB method is that the excavated soil itself is used to provide continuous support to the tunnel face by balancing earth pressure against the forward pressure of the machine. The thrust forces generated from rear section of TBM is transferred to the earth in the cutterhead chamber so as to prevent uncontrolled intrusion of excavated materials into the chamber. When the shield advances, the excavated soil is mixed with the injected special foam/soil conditioner material which changes the viscosity/plasticity of the spoil and transforms it into a flowing material. With careful control of the advance thrust force of the TBM and the rate of discharge of the spoil, adequate pressure could be maintained in the pressure chamber for supporting the tunnel face during the excavation process.

16.3 MAN-ENTRY TYPE

As indicated by the category name, man-entry would be permitted under this type of trenchless construction. Thus, adequate underground working space should have been ascertained by the designer at the pre-contract stage. Thorough study on the record drawings of the existing structures and utilities for the entire alignment along with any necessary ground investigation (including geophysical survey) should be conducted prior to allowing this type of trenchless method in the construction contract. According to the current safety standards, underground access to workers should be of 1.2m diameter minimum. Besides, sufficient area at the access shafts should also be obtained as back-up area to cater for emergency.

Some basic information in respect of these methods that can meet the ‘man-entry’ requirements are highlighted below which are by no means exhaustive. Project engineers and designers are to satisfy themselves that any particular method accepted can really perform the required function.

16.3.1 Heading Method

Heading method is a simple form of constructing hand-dug tunnel using structural frame as ground support during manual excavation. Minimum excavation dimensions of at least 1.5m x 1.5m are typically required for installing usually short-length pipelines crossing underneath road junctions, underground utilities and services and entrance to car parks in shallow depth. The completed works have demonstrated its effectiveness due to fast mobilization of plant, simple set-up on site, and flexibility in construction method to suit the limited space available.

Safety and ground condition risks must be carefully considered for this form of construction. Designers shall exercise reasonable skills and care to check that no other form of construction is appropriate and the ground conditions that are expected to be encountered during excavation are unlikely to impose unacceptable safety risks to the workers before deciding to adopt heading construction.

16.3.2 Hand-dug Tunnel Method

Hand-dug tunnel (i.e. open mode) is the technique of installing pipes by forming a tunnel with manual excavation inside the handshield from the entry pits to the end pits. The excavated materials are transported to the ground level through a trolley system and lifted up by lifting gantry installed at entry pit. Segments of tunnel frames are constructed one after the other until reaching the end pits. It is effective when the alignment of a pipeline (or culvert) has to pass through mechanical obstructions like walls and artificial hard materials which can be removed by manual means. However, emphasis has to be placed on pre-treatment to the ground prior to excavation to avoid instability of tunnelling soil face. Excavation at tunnel face could be accelerated by using pneumatic tools or by mini-backhoe if the size of tunnels allows.

16.4 MAJOR CONSIDERATIONS

16.4.1 Planning & Design Stage

(a) Existing Underground Conditions

The roads in the urban areas of Hong Kong are generally congested with underground utilities and services of different types and sizes at different depths. When planning a pipeline using trenchless construction, the level and alignment shall be designed to avoid diversion of existing utilities and services as it involves open excavation. Sufficient information shall be obtained and trial pits or other means shall be carried out to verify the depth and locations of all underground utilities in the vicinity of the proposed alignment & level of trenchless pipeline.

(b) Alignment & Level

Construction of a pipeline (or culvert) normally follows the alignment of the road. Pipe jacking is well applicable for straight driving, though slightly curved driving is possible with the latest technology; therefore, pipe jacking is not feasible in areas that require sharp bends unless additional intermediate jacking/receiving shafts are constructed.

Trenchless construction is often applicable for deep pipes in order to avoid underground utilities, felling of valuable trees and unacceptable ground movement. Hence, it is not suitable for very shallow pipe installation. Construction of deep pipelines may encounter bedrock or boulder which presents difficulty in pipe jacking. Hand-dug tunnel construction would be an alternative in such scenario while microtunnelling could also drill through rock and artificial hard material. Comprehensive planning and implementation of site investigation would help to reduce the risk of unexpected ground conditions. It may not be practical to carry out thorough drilling along the whole alignment of the pipeline. In most circumstances, directional coring and geophysical methods should be considered along the whole alignment of the proposed trenchless pipeline.

For pipeline (or culvert) construction in long length, it may be worthwhile to consider adopting different forms of construction for different sections of the pipeline between intermediate temporary shafts to suit changing conditions along the pipe alignment.

(c) Locations of Jacking/Launching and Receiving Shafts

Jacking shaft and receiving shaft between a pipeline (or culvert) should be selected to avoid conflicting with traffic and major utilities or minimize their diversions. The choice of the locations of the launching shaft and the receiving shaft depends on a number of factors such as the positions of permanent manholes, the hydraulic design of water flow, the maximum length of pipelines for the ease of future maintenance and the required working space. Substantial working space is required for launching shaft, receiving shaft and slurry treatment plant (for slurry operated TBM method). Thus, the area occupied by the shafts and slurry plant shall be considered in space limited works fronts during design stage. Due consideration should be given to temporary traffic arrangement schemes and application of Excavation Permits in stages.

The underground condition generally governs the geometry of the shaft. Rectangular shaft is usually constructed because it can be modified without much difficulty to accommodate existing utilities and services. However, at locations where these features are absent, circular shaft is used due to the smaller member size of temporary works required.

The shape and size of a shaft need to be tailor-made to suit utility constraint, resulting in the possible use of a combination of sheetpiles and pipe-piles to overcome the problem. For deep shaft, grouting is always required along the perimeter to ensure watertightness and hydraulic failure at the base of the shaft, before excavation is to commence.

(d) Site Investigation and Design

Thorough site investigation shall be carried out at the design stage. The site investigation result is essential to the design and procurement of machine for trenchless construction. Designers shall take into account the tunnel face stability, face support pressure, dewatering effects, ground loss and ground movement estimation, etc in the design. With regard to ground control slurry TBM method, guidelines for design calculations and work procedures shall be made reference to GEO Report No. 249.

16.4.2 Construction Stage

(a) Alignment & Level Control

The pipe jacking works in Hong Kong using TBMs generally shall follow the "Specification for Tunnelling (BTS and ICE, 2010)" as guideline for controlling tunnel alignment, in that a tolerance of 50mm is specified for line. However, deviation in more than the tolerance may occur at locations with unfavourable ground conditions. The tunnel alignment is corrected by suitable extension or retraction of the steering cylinders installed in the TBM. The use of TBM with 4 nos. steering cylinders offers better control in alignment than that with 3 nos. It requires a long period of time to correct any out-of-tolerance in alignment in order to avoid causing damage to the jacking pipes. However, allowance has to be made to account for the irregular profile of the tunnel, due to different ground conditions encountered during excavation, which could affect installation of the permanent pipeline therein to the required alignment.

The level control shall also follow the "Specification for Tunnelling (BTS and ICE, 2010)", in that a tolerance of 35mm is specified for level. In case of the pipeline exceeded the tolerated 0.5 degree angular deflection at pipe joint, there is a need to carry out a detailed inspection to ensure that there is no dislocation thereat. To better control the hydraulic performance of a pipeline (or culvert), tighter tolerance on invert level in the order of few millimeters may be adopted. For excessive opening in pipe joint, remedial measures have to be carried out. This can be achieved by locally trimming the concrete at the pipe end for better bonding before applying non-shrinkage epoxy, with a strength equivalent to the pipe, to fill up the problematic location for prevention of ingress of water.

(b) Safety Concerns

(i) Working under compressed air

The adoption of compressed air hand-dug tunnelling method would face a problem associated with air loss in porous ground, giving rise to the necessity of carrying out ground treatment to safeguard the tunnel and the personnel working inside. The switch-on of compressors and generators round–the-clock to maintain the pressure in tunnel also causes noise problem. Although high cost and relative low production rate make this method only applicable to short drives, the removal of artificial obstructions can be warranted. However, following the rapid development of TBM technology which allows pipe jacking drives in curved alignment and detects obstructions ahead of TBM advancement, personnel working under compressed air shall be avoided as far as practicable due to the risk involved and the reasons stated above. In case working under compressed air is found essential, detailed justifications and risk assessment should be prepared before construction.

Compressed-air tunnel will be adopted for high groundwater table condition. Depending on depth, an air pressure of 1 to 2 bars is required to balance the water head in the excavation face and be maintained inside the tunnel round-the-clock to avoid flooding which may in turn affect tunnel stability. To ensure constant supply of air, a standby compressor is provided for emergency situations. Pressurization and depressurization process is required in the air-lock installed on top of an air deck erected in the jacking shaft, for personnel entering and leaving the tunnel respectively. A medical lock needs to be provided at the shaft location when the applied compressed-air pressure exceeds 1 bar.

(ii) Working in confined space

Workplaces for trenchless construction including shaft and tunnel are always enclosed nature and there are reasonably foreseeable risks such as sudden ingress of water and collapse of tunnel. Procedures for working in confined space shall be strictly followed including assessment on the tunnel face stability and ingress of groundwater. Reference shall be made to DSD Practice Note No. 3/2012 “Safety Supervision of Work in Confined Space”, or its latest version, DSD Safety Manual on “Use of Headings” and relevant regulations.

When working in confined space under compressed air condition, there have been some cases that air was found leaking through the porous ground during tunnel excavation, resulting in inflow of groundwater. This entails horizontal and vertical grouting from inside the shield to stabilize the ground before further excavation could be proceeded with.

Flooding of the heading and the access shaft can occur as a consequence of sudden inrush of water from exposed faces due to bursting of nearby watermains, heavy rainfall, etc. In the risk assessment, suitable measures to ensure the heading works watertightness to prevent flooding should be considered.

(c) Ground Movement Monitoring

Tunnelling and pipe jacking would induce settlement in surrounding ground. The magnitude of settlement is greatly affected by ground conditions, type of tunneling method, control of inflow of groundwater, depth of tunnel and jacking speed. The presence of underground utilities and services above the jacked pipeline would lead to under-measurement of surface ground settlement due to their rigidity. It is necessary to estimate the settlement influence zone and to assess its effect on nearby roads, structures and utility installations such that they can be safeguarded during the operation and remedial measures taken, if necessary. Maximum ground settlement occurs at the centre line of the pipeline and diminishes to zero at a distance from its two sides. Most settlements occur during and immediately after completion of tunneling and pipe jacking works. Further settlement would continue, and its stoppage depends on the ground and groundwater conditions above the jacked pipeline, for a few weeks to a few months.

In many cases, ground settlement is associated with change in groundwater level due to dewatering or groundwater inflow in the tunnel. The Contractor shall closely monitor the standpipe and piezometer readings with reference to the baseline record. If there is significant drawdown of groundwater level, assessment to the effect of settlement and subsequent remedial measures shall be carried out.

For monitoring settlement, sub-surface settlement markers, in the form of a steel rod, by coring through rigid pavement, are generally adopted, with their installation at suitable intervals along the alignment of the pipeline and with sufficient number offset at both sides, prior to commencement of a pipe jacking drive. In flexible pavement, nail markers are used. This is supplemented by visual inspection that if settlement occurs, cracks would develop in pavement. For structures sitting on shallow foundation, their condition has to be assessed before commencement of tunneling and pipe jacking such that suitable monitoring devices such as tilt markers and settlement markers can be installed to monitor the ground behavior during the course of works. If the measured ground settlement exceeds the predicted value, the tunneling and pipe jacking works have to stop and an investigation on the cause, and the damage, if any, carried out, with remedial measures such as ground treatment implemented, as necessary, prior to resumption of works.

16.4.3 Environmental Issue

Slurry pressure balance method using TBM requires a large amount of bentonite based slurry during the course of driving. Proper consideration should be made to recycle and dispose of the bentonite slurry after use to minimize the impact to the environment.

The bentonite based slurry of slurry shield TBM is mixed at the slurry tank and pumped to the work face through the cutterhead of the shield under a recycling system. The spoil excavated by the slurry shield machine is pumped to the slurry tank for separation and disposal. Proper monitoring system should be set up to avoid overflow of slurry from the recycle tank in case the outflow slurry pipe is clogged with spoil. The slurry tank should also be designed with sufficient free board to avoid the bentonite based slurry spilling out on public roads and drains.

16.4.4 Cost Consideration

Pipeline (or culvert) laid using trenchless construction method are usually of higher construction cost than those laid using open trench construction method. Thus, consideration shall be made on the cost-effectiveness of trenchless construction method and the benefits that may be brought to the public before adopting trenchless construction method instead of open trench construction method. However, for case of deep sewer, great difficulty in utility diversion or temporary traffic arrangement, trenchless construction method may prevail in terms of cost and constructability.

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LIST OF TABLES

Table 1 - Operational Rain Gauge Stations in Hong Kong

Type	Number of Stations	Location Map
HKO Conventional Rain Gauge Stations	20	Figure 2b
HKO Rain Gauge Stations - Telemetered	50	Figure 2a
GEO Rain Gauge Stations - Telemetered	90	Figure 2a
DSD Rain Gauge Stations - Telemetered	33	Figure 2a

Table 2a – Intensity-Duration-Frequency (IDF) Relationship of HKO Headquarters
for durations not exceeding 240 minutes

Duration (min)	**Parameters**			**Extreme Intensity x (mm/h) for various Return Periods T(year)**								
	ξ **(mm/h)**	**α**	**κ**	2	5	10	20	50	100	200	500	1000
240**	26.00	9.30	-0.009	29.4	40.0	47.1	54.0	62.9	69.7	76.4	85.4	92.2
120++	43.79	14.56	0.081	49.1	64.4	73.7	82.2	92.5	99.7	107	115	121
60++	64.42	19.34	0.092	71.4	91.5	104	115	128	137	145	156	163
30++	84.48	20.28	0.141	91.7	112	124	134	145	153	160	168	174
15++	106.47	21.34	0.157	114	135	147	157	169	176	183	191	197
10	*122.53	*24.90	*0.198	131	155	168	179	190	198	204	212	216
5	*145.27	*28.54	*0.235	155	181	195	206	218	226	232	239	243
2	*175.33	*34.18	*0.285	187	217	232	244	256	263	269	275	279
1	*198.07	*39.17	*0.322	212	245	261	273	285	292	298	303	307
0.50	*220.81	*44.90	*0.360	236	273	290	303	315	322	327	332	335
0.25+++	244.85	52.05	0.404	263	303	322	335	347	354	359	363	366

Notes:

1. For interpolation/extrapolation, $x = \xi + \left(\frac{\alpha}{\kappa}\right)\left\{1 - \left[-log\left(\frac{T-1}{T}\right)\right]^{\kappa}\right\}$
2. ++ based on continuous rainfall recorded at HKO Headquarters (1947 – 2014)
3. +++ based on Jardi rate-of-rainfall records at King's Park (1952 – 2014)
4. * interpolated data
5. ** based on hourly rainfall records at HKO Headquarters (1884 – 1939; 1947 – 2014)

Table 2b – Intensity-Duration-Frequency (IDF) Relationship of Tai Mo Shan Area
for durations not exceeding 240 minutes

Duration (min)	**Extreme Intensity x (mm/h) for various Return Periods T(year)**						
	2	5	10	20	50	100	200
240	37.0	48.7	55.8	62.4	70.3	76.0	85.2
120	56.6	75.0	86.7	97.7	112	122	132
60	80.5	102	116	127	142	152	162
30	112	136	151	165	184	199	215
15	142	171	188	203	221	233	246
10	154	183	201	220	245	265	286
5	178	209	232	257	294	324	356

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N14 (31 years), N36 (15 years), N37 (15 years) & N40 (15 years) up to 2014
2. rainfall IDF relationships are derived from regional frequency analysis of extreme rainfall of local rain gauges

Table 2c – Intensity-Duration-Frequency (IDF) Relationship of West Lantau Area
for durations not exceeding 240 minutes

Duration (min)	**Extreme Intensity x (mm/h) for various Return Periods T(year)**						
	2	5	10	20	50	100	200
240	34.6	47.3	56.4	65.6	78.4	88.7	99.4
120	54.3	72.0	83.6	94.7	109	120	130
60	78.6	100	113	124	138	148	157
30	108	132	146	158	171	181	190
15	141	170	187	201	219	231	242
10	155	186	204	221	242	258	272
5	171	206	229	251	279	299	319

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N17 (24 years), N19 (15 years) & N21 (15 years) up to 2014
2. rainfall IDF relationships are derived by regional frequency analysis of extreme rainfall of local rain gauges

Table 2d – Intensity-Duration-Frequency (IDF) Relationship of North District Area
for durations not exceeding 240 minutes

Duration (min)	**Extreme Intensity x (mm/h) for various Return Periods T(year)**						
	2	5	10	20	50	100	200
240	28.5	37.7	43.4	48.6	54.9	59.4	63.6
120	42.2	54.7	62.5	69.6	78.4	84.7	90.8
60	61.0	75.7	84.3	92.0	101	108	114
30	84.0	100	110	118	128	135	142
15	106	127	139	150	163	173	182
10	119	141	155	168	184	196	208
5	138	161	177	193	216	234	254

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N05 (31 years), N34 (15 years), N46 (15 years), N33 (15 years), N35 (15 years), N36 (15 years), N45 (15 years) and HKO rain gauges EPC (22 years), SSH (11 years), TKL (29 years), R24 (31 years), R29 (30 years), R30 (25 years), SEK (18 years) up to 2014
2. rainfall IDF relationships are derived from regional frequency analysis of extreme rainfall of local rain gauges

Table 3a – Storm Constants for Different Return Periods of HKO Headquarters

Return Period T (years)	2	5	10	20	50	100	200	500	1000
a	499.8	480.2	471.9	463.6	451.3	440.8	429.5	414.0	402.1
b	4.26	3.36	3.02	2.76	2.46	2.26	2.05	1.77	1.55
c	0.494	0.429	0.397	0.369	0.337	0.316	0.295	0.269	0.251

Table 3b – Storm Constants for Different Return Periods of Tai Mo Shan Area

Return Period T (years)	2	5	10	20	50	100	200
a	1743.9	2183.2	2251.3	2159.2	1740.1	1307.3	1005.0
b	22.12	27.12	27.46	25.79	19.78	12.85	7.01
c	0.694	0.682	0.661	0.633	0.570	0.501	0.434

Table 3c – Storm Constants for Different Return Periods of West Lantau Area

Return Period T (years)	2	5	10	20	50	100	200
a	2047.9	1994.1	1735.2	1445.6	1107.2	909.1	761.8
b	24.27	24.23	21.82	18.36	13.01	8.98	5.40
c	0.733	0.673	0.619	0.561	0.484	0.428	0.377

Table 3d – Storm Constants for Different Return Periods of North District Area

Return Period T (years)	2	5	10	20	50	100	200
a	1004.5	1112.2	1157.7	1178.6	1167.6	1131.2	1074.8
b	17.24	18.86	19.04	18.49	16.76	14.82	12.47
c	0.644	0.614	0.597	0.582	0.561	0.543	0.523

Table 4a - Depth-Duration-Frequency (DDF) Relationship of HKO Headquarters
for durations of more than 4 hours

Duration	**Parameters**			**Extreme Depth x (mm) for various Return Periods T(year)**								
	ξ (mm)	**α**	**κ**	2	5	10	20	50	100	200	500	1000
31 days	623.21	170.34	0.049	685	870	986	1094	1229	1325	1418	1536	1622
15 days	443.93	147.00	0.078	497	652	747	834	939	1012	1082	1168	1230
7 days	324.59	115.18	0.0069	367	496	582	663	768	846	927	1049	1144
5 days	292.94	109.01	-0.035	333	461	548	634	748	837	927	1049	1144
4 days	276.35	103.84	-0.043	315	437	522	605	717	804	893	1015	1110
3 days	257.58	99.99	-0.0034	294	408	483	556	650	721	792	885	956
2 days	223.82	89.68	-0.004	257	359	427	492	577	640	704	788	852
24 hours	190.12	72.90	-0.035	217	302	361	418	495	554	614	696	760
18 hours	170.85	63.45	-0.049	194	270	322	374	444	498	555	632	693
12 hours	148.81	53.69	-0.071	169	234	280	326	390	441	494	568	627
8 hours	130.32	44.33	-0.088	147	201	241	281	337	381	429	496	551
6 hours	119.29	41.20	-0.061	135	184	219	253	301	338	377	430	473

Notes:

1. For interpolation/extrapolation, $x = \xi + \left(\frac{\alpha}{\kappa}\right)\left\{1 - \left[-log\left(\frac{T-1}{T}\right)\right]^{\kappa}\right\}$
2. Based on hourly records measured at HKO Headquarters (1884 – 1939 ; 1947 – 2014)
3. The Intensity-Duration-Frequency (IDF) data can be generated from the above by dividing depth with duration

Table 4b - Depth-Duration-Frequency (DDF) Relationship of Tai Mo Shan Area
for durations of more than 4 hours

Duration	**Extreme Depth x (mm) for various Return Periods T(year)**						
	2	5	10	20	50	100	200
31 days	876	1177	1425	1711	2174	2608	3134
15 days	624	869	1035	1193	1395	1543	1690
7 days	460	647	772	891	1042	1154	1263
5 days	427	605	724	837	980	1085	1189
4 days	399	568	683	792	932	1035	1155
3 days	371	520	629	741	898	1023	1155
2 days	326	459	557	658	799	913	1033
24 hours	267	366	435	503	593	664	736
18 hours	250	342	401	454	520	568	614
12 hours	216	289	334	375	425	461	511
8 hours	187	249	288	323	379	440	511
6 hours	165	222	264	310	379	440	511

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N14 (31 years), N36 (15 years), N37 (15 years) & N40 (15 years) up to 2014
2. rainfall IDF relationships are derived from regional frequency analysis of extreme rainfall of local rain gauges

Table 4c - Depth-Duration-Frequency (DDF) Relationship of West Lantau Area
for durations of more than 4 hours

Duration	**Extreme Depth x (mm) for various Return Periods T(year)**						
	2	5	10	20	50	100	200
31 days	711	969	1176	1408	1766	2084	2451
15 days	510	700	835	970	1152	1295	1444
7 days	380	527	630	732	870	977	1088
5 days	358	500	599	698	830	933	1039
4 days	334	468	563	658	788	890	995
3 days	304	434	525	613	726	810	894
2 days	270	392	480	565	676	759	841
24 hours	230	330	402	475	575	654	737
18 hours	213	296	359	426	525	610	705
12 hours	194	268	324	383	470	542	621
8 hours	171	238	289	343	422	488	561
6 hours	157	219	265	312	380	436	496

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N17 (24 years), N19 (15 years) & N21 (15 years) up to 2014
2. rainfall IDF relationships are derived from regional frequency analysis of extreme rainfall of local rain gauges

Table 4d - Depth-Duration-Frequency (DDF) Relationship of North District Area
for durations of more than 4 hours

Duration	Extreme Depth x (mm) for various Return Periods T(year)						
	2	5	10	20	50	100	200
31 days	632	853	1034	1243	1575	1882	2251
15 days	450	635	760	874	1011	1105	1191
7 days	341	485	582	670	775	848	914
5 days	327	464	543	606	673	714	748
4 days	306	442	516	573	631	665	698
3 days	289	402	468	527	599	649	698
2 days	249	342	404	462	537	593	649
24 hours	201	269	314	358	416	460	505
18 hours	188	245	283	322	376	420	468
12 hours	165	218	250	279	315	341	365
8 hours	147	191	217	241	270	290	310
6 hours	131	176	203	226	253	272	292

Notes:

1. based on continuous rainfall recorded at GEO rain gauges N05 (31 years), N34 (15 years), N46 (15 years), N33 (15 years), N35 (15 years), N36 (15 years), N45 (15 years) and HKO rain gauges EPC (22 years), SSH (11 years), TKL (29 years), R24 (31 years), R29 (30 years), R30 (25 years), SEK (18 years) up to 2014
2. rainfall IDF relationships are derived from regional frequency analysis of extreme rainfall of local rain gauges

Table 5a – Design Rainstorm Profile Intensity-Duration-Frequency Relationships of HKO Headquarters

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)				
	2	10	50	200	1000
-0.5 – 0.5	220	272	297	309	318
0.5 – 1.5	172	211	234	245	253
1.5 – 2.5	135	169	191	204	215
2.5 – 3.5	113	145	168	181	194
3.5 – 4.5	98	130	152	167	180
4.5 – 5.5	88	119	141	156	170
5.5 – 6.5	80	110	132	147	163
6.5 – 7.5	74	103	125	141	156
7.5 – 8.5	69	98	120	135	151
8.5 – 9.5	65	93	115	130	147
9.5 – 10.5	61	89	111	126	143
10.5 – 11.5	58	86	107	123	139
11.5 – 12.5	56	83	104	120	136
12.5 – 13.5	53	80	101	117	134
13.5 – 14.5	51	77	99	114	131
14.5 – 15.5	49	75	96	112	129
15.5 – 16.5	48	73	94	110	127
16.5 – 17.5	46	71	92	108	125
17.5 – 18.5	45	70	90	106	123
18.5 – 19.5	44	68	89	104	121
19.5 – 20.5	43	67	87	103	120
20.5 – 21.5	41	66	86	101	118
21.5 – 22.5	40	64	84	100	117
22.5 – 23.5	40	63	83	98	115
23.5 – 24.5	39	62	82	97	114
24.5 – 25.5	38	61	81	96	113
25.5 – 26.5	37	60	80	95	112
26.5 – 27.5	36	59	79	94	111
27.5 – 28.5	36	58	78	93	110
28.5 – 29.5	35	57	77	92	109
29.5 – 30.5	34	57	76	91	108
30.5 – 31.5	34	56	75	90	107
31.5 – 32.5	33	55	74	89	106
32.5 – 33.5	33	55	73	88	105
33.5 – 34.5	32	54	73	88	105
34.5 – 35.5	32	53	72	87	104
35.5 – 36.5	31	53	71	86	103
36.5 – 37.5	31	52	71	85	102
37.5 – 38.5	30	51	70	85	102
38.5 – 39.5	30	51	69	84	101
39.5 – 40.5	30	50	69	83	100
40.5 – 41.5	29	50	68	83	100
41.5 – 42.5	29	49	68	82	99
42.5 – 43.5	29	49	67	82	99
43.5 – 44.5	28	49	66	81	98
44.5 – 45.5	28	48	66	81	97
45.5 – 46.5	28	48	65	80	97
46.5 – 47.5	27	47	65	79	96
47.5 – 48.5	27	47	65	79	96
48.5 – 49.5	27	46	64	78	95
49.5 – 50.5	26	46	64	78	95
50.5 – 51.5	26	46	63	78	94
51.5 – 52.5	26	45	63	77	94
52.5 – 53.5	26	45	62	77	94
53.5 – 54.5	25	45	62	76	93
54.5 – 55.5	25	44	62	76	93
55.5 – 56.5	25	44	61	75	92
56.5 – 57.5	25	44	61	75	92
57.5 – 58.5	25	43	60	75	91
58.5 – 59.5	24	43	60	74	91
59.5 – 60.5	24	43	60	74	91

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)				
	2	10	50	200	1000
60.5 – 61.5	24	43	59	74	90
61.5 – 62.5	24	42	59	73	90
62.5 – 63.5	24	42	59	73	90
63.5 – 64.5	23	42	59	73	89
64.5 – 65.5	23	41	58	72	89
65.5 – 66.5	23	41	58	72	89
66.5 – 67.5	23	41	58	72	88
67.5 – 68.5	23	41	57	71	88
68.5 – 69.5	22	40	57	71	88
69.5 – 70.5	22	40	57	71	87
70.5 – 71.5	22	40	56	70	87
71.5 – 72.5	22	40	56	70	87
72.5 – 73.5	22	40	56	70	86
73.5 – 74.5	22	39	56	69	86
74.5 – 75.5	22	39	55	69	86
75.5 – 76.5	21	39	55	69	85
76.5 – 77.5	21	39	55	69	85
77.5 – 78.5	21	39	55	68	85
78.5 – 79.5	21	38	54	68	85
79.5 – 80.5	21	38	54	68	84
80.5 – 81.5	21	38	54	68	84
81.5 – 82.5	21	38	54	67	84
82.5 – 83.5	20	38	54	67	84
83.5 – 84.5	20	37	53	67	83
84.5 – 85.5	20	37	53	67	83
85.5 – 86.5	20	37	53	66	83
86.5 – 87.5	20	37	53	66	83
87.5 – 88.5	20	37	53	66	82
88.5 – 89.5	20	37	52	66	82
89.5 – 90.5	20	36	52	66	82
90.5 – 91.5	20	36	52	65	82
91.5 – 92.5	19	36	52	65	81
92.5 – 93.5	19	36	52	65	81
93.5 – 94.5	19	36	51	65	81
94.5 – 95.5	19	36	51	64	81
95.5 – 96.5	19	35	51	64	81
96.5 – 97.5	19	35	51	64	80
97.5 – 98.5	19	35	51	64	80
98.5 – 99.5	19	35	50	64	80
99.5 – 100.5	19	35	50	64	80
100.5 – 101.5	19	35	50	63	80
101.5 – 102.5	18	35	50	63	79
102.5 – 103.5	18	34	50	63	79
103.5 – 104.5	18	34	50	63	79
104.5 – 105.5	18	34	49	63	79
105.5 – 106.5	18	34	49	62	79
106.5 – 107.5	18	34	49	62	78
107.5 – 108.5	18	34	49	62	78
108.5 – 109.5	18	34	49	62	78
109.5 – 110.5	18	34	49	62	78
110.5 – 111.5	18	33	49	62	78
111.5 – 112.5	18	33	48	61	77
112.5 – 113.5	18	33	48	61	77
113.5 – 114.5	17	33	48	61	77
114.5 – 115.5	17	33	48	61	77
115.5 – 116.5	17	33	48	61	77
116.5 – 117.5	17	33	48	61	77
117.5 – 118.5	17	33	48	60	76
118.5 – 119.5	17	33	47	60	76
119.5 – 120.5	17	32	47	60	76
120.5 – 121.5	17	32	47	60	76

Note: * Rate of Rainfall is the average value over consecutive time intervals

Table 5b – Design Rainstorm Profile Intensity-Duration-Frequency Relationships of Tai Mo Shan Area

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)			
	2	10	50	200
-0.5 – 0.5	197	246	309	407
0.5 – 1.5	181	230	285	351
1.5 – 2.5	162	211	259	300
2.5 – 3.5	147	195	237	264
3.5 – 4.5	134	181	219	239
4.5 – 5.5	123	169	203	219
5.5 – 6.5	114	159	190	203
6.5 – 7.5	106	149	179	191
7.5 – 8.5	99	141	169	180
8.5 – 9.5	93	133	160	171
9.5 – 10.5	87	127	152	163
10.5 – 11.5	82	121	145	156
11.5 – 12.5	78	115	139	150
12.5 – 13.5	74	110	133	145
13.5 – 14.5	71	106	128	140
14.5 – 15.5	67	101	123	136
15.5 – 16.5	64	97	119	132
16.5 – 17.5	62	94	115	128
17.5 – 18.5	59	91	111	125
18.5 – 19.5	57	87	108	122
19.5 – 20.5	55	85	105	119
20.5 – 21.5	53	82	102	117
21.5 – 22.5	51	79	99	114
22.5 – 23.5	50	77	96	112
23.5 – 24.5	48	75	94	110
24.5 – 25.5	46	73	92	108
25.5 – 26.5	45	71	89	106
26.5 – 27.5	44	69	87	104
27.5 – 28.5	43	67	85	102
28.5 – 29.5	41	66	84	101
29.5 – 30.5	40	64	82	99
30.5 – 31.5	39	63	80	98
31.5 – 32.5	38	61	79	96
32.5 – 33.5	37	60	77	95
33.5 – 34.5	37	59	76	94
34.5 – 35.5	36	57	74	92
35.5 – 36.5	35	56	73	91
36.5 – 37.5	34	55	72	90
37.5 – 38.5	33	54	71	89
38.5 – 39.5	33	53	70	88
39.5 – 40.5	32	52	69	87
40.5 – 41.5	31	51	67	86
41.5 – 42.5	31	50	66	85
42.5 – 43.5	30	49	66	84
43.5 – 44.5	30	48	65	83
44.5 – 45.5	29	48	64	82
45.5 – 46.5	29	47	63	82
46.5 – 47.5	28	46	62	81
47.5 – 48.5	28	45	61	80
48.5 – 49.5	27	45	60	79
49.5 – 50.5	27	44	60	79
50.5 – 51.5	26	43	59	78
51.5 – 52.5	26	43	58	77
52.5 – 53.5	26	42	57	77
53.5 – 54.5	25	42	57	76
54.5 – 55.5	25	41	56	75
55.5 – 56.5	24	40	56	75
56.5 – 57.5	24	40	55	74
57.5 – 58.5	24	39	54	74
58.5 – 59.5	23	39	54	73
59.5 – 60.5	23	38	53	72

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)			
	2	10	50	200
60.5 – 61.5	23	38	53	72
61.5 – 62.5	23	37	52	71
62.5 – 63.5	22	37	52	71
63.5 – 64.5	22	37	51	70
64.5 – 65.5	22	36	51	70
65.5 – 66.5	21	36	50	69
66.5 – 67.5	21	35	50	69
67.5 – 68.5	21	35	49	68
68.5 – 69.5	21	35	49	68
69.5 – 70.5	20	34	48	68
70.5 – 71.5	20	34	48	67
71.5 – 72.5	20	33	47	67
72.5 – 73.5	20	33	47	66
73.5 – 74.5	20	33	47	66
74.5 – 75.5	19	32	46	66
75.5 – 76.5	19	32	46	65
76.5 – 77.5	19	32	46	65
77.5 – 78.5	19	31	45	64
78.5 – 79.5	19	31	45	64
79.5 – 80.5	18	31	44	64
80.5 – 81.5	18	31	44	63
81.5 – 82.5	18	30	44	63
82.5 – 83.5	18	30	43	63
83.5 – 84.5	18	30	43	62
84.5 – 85.5	18	29	43	62
85.5 – 86.5	17	29	43	62
86.5 – 87.5	17	29	42	61
87.5 – 88.5	17	29	42	61
88.5 – 89.5	17	28	42	61
89.5 – 90.5	17	28	41	60
90.5 – 91.5	17	28	41	60
91.5 – 92.5	16	28	41	60
92.5 – 93.5	16	28	41	60
93.5 – 94.5	16	27	40	59
94.5 – 95.5	16	27	40	59
95.5 – 96.5	16	27	40	59
96.5 – 97.5	16	27	39	58
97.5 – 98.5	16	26	39	58
98.5 – 99.5	16	26	39	58
99.5 – 100.5	15	26	39	58
100.5 – 101.5	15	26	38	57
101.5 – 102.5	15	26	38	57
102.5 – 103.5	15	26	38	57
103.5 – 104.5	15	25	38	57
104.5 – 105.5	15	25	38	56
105.5 – 106.5	15	25	37	56
106.5 – 107.5	15	25	37	56
107.5 – 108.5	14	25	37	56
108.5 – 109.5	14	24	37	56
109.5 – 110.5	14	24	37	55
110.5 – 111.5	14	24	36	55
111.5 – 112.5	14	24	36	55
112.5 – 113.5	14	24	36	55
113.5 – 114.5	14	24	36	54
114.5 – 115.5	14	24	36	54
115.5 – 116.5	14	23	35	54
116.5 – 117.5	14	23	35	54
117.5 – 118.5	14	23	35	54
118.5 – 119.5	13	23	35	53
119.5 – 120.5	13	23	35	53
120.5 – 121.5	13	23	34	53

Note: * Rate of Rainfall is the average value over consecutive time intervals

Table 5c – Design Rainstorm Profile Intensity-Duration-Frequency Relationships of West Lantau Area

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)			
	2	10	50	200
-0.5 – 0.5	192	250	309	378
0.5 – 1.5	176	231	280	323
1.5 – 2.5	159	210	249	275
2.5 – 3.5	144	192	226	244
3.5 – 4.5	131	177	207	222
4.5 – 5.5	121	164	192	205
5.5 – 6.5	112	153	179	192
6.5 – 7.5	104	144	168	181
7.5 – 8.5	97	135	159	172
8.5 – 9.5	91	128	151	165
9.5 – 10.5	85	121	144	158
10.5 – 11.5	80	115	138	153
11.5 – 12.5	76	110	132	147
12.5 – 13.5	72	105	127	143
13.5 – 14.5	68	101	123	139
14.5 – 15.5	65	97	119	135
15.5 – 16.5	62	93	115	132
16.5 – 17.5	60	90	112	129
17.5 – 18.5	57	87	108	126
18.5 – 19.5	55	84	106	123
19.5 – 20.5	53	81	103	121
20.5 – 21.5	51	79	100	118
21.5 – 22.5	49	76	98	116
22.5 – 23.5	47	74	96	114
23.5 – 24.5	46	72	94	112
24.5 – 25.5	44	70	92	111
25.5 – 26.5	43	68	90	109
26.5 – 27.5	41	67	88	107
27.5 – 28.5	40	65	87	106
28.5 – 29.5	39	63	85	104
29.5 – 30.5	38	62	84	103
30.5 – 31.5	37	61	82	102
31.5 – 32.5	36	59	81	100
32.5 – 33.5	35	58	80	99
33.5 – 34.5	34	57	78	98
34.5 – 35.5	33	56	77	97
35.5 – 36.5	33	55	76	96
36.5 – 37.5	32	54	75	95
37.5 – 38.5	31	53	74	94
38.5 – 39.5	30	52	73	93
39.5 – 40.5	30	51	72	92
40.5 – 41.5	29	50	71	91
41.5 – 42.5	28	49	70	90
42.5 – 43.5	28	48	69	90
43.5 – 44.5	27	48	69	89
44.5 – 45.5	27	47	68	88
45.5 – 46.5	26	46	67	87
46.5 – 47.5	26	46	66	87
47.5 – 48.5	25	45	66	86
48.5 – 49.5	25	44	65	85
49.5 – 50.5	24	44	64	85
50.5 – 51.5	24	43	64	84
51.5 – 52.5	24	42	63	83
52.5 – 53.5	23	42	62	83
53.5 – 54.5	23	41	62	82
54.5 – 55.5	23	41	61	81
55.5 – 56.5	22	40	61	81
56.5 – 57.5	22	40	60	80
57.5 – 58.5	22	39	60	80
58.5 – 59.5	21	39	59	79
59.5 – 60.5	21	38	58	79
60.5 – 61.5	21	38	58	78
61.5 – 62.5	20	38	58	78
62.5 – 63.5	20	37	57	77
63.5 – 64.5	20	37	57	77
64.5 – 65.5	19	36	56	76
65.5 – 66.5	19	36	56	76
66.5 – 67.5	19	36	55	76
67.5 – 68.5	19	35	55	75
68.5 – 69.5	18	35	54	75
69.5 – 70.5	18	35	54	74
70.5 – 71.5	18	34	54	74
71.5 – 72.5	18	34	53	73
72.5 – 73.5	18	34	53	73
73.5 – 74.5	17	33	53	73
74.5 – 75.5	17	33	52	72
75.5 – 76.5	17	33	52	72
76.5 – 77.5	17	32	51	72
77.5 – 78.5	17	32	51	71
78.5 – 79.5	16	32	51	71
79.5 – 80.5	16	32	50	71
80.5 – 81.5	16	31	50	70
81.5 – 82.5	16	31	50	70
82.5 – 83.5	16	31	50	70
83.5 – 84.5	16	31	49	69
84.5 – 85.5	15	30	49	69
85.5 – 86.5	15	30	49	69
86.5 – 87.5	15	30	48	68
87.5 – 88.5	15	30	48	68
88.5 – 89.5	15	29	48	68
89.5 – 90.5	15	29	48	67
90.5 – 91.5	15	29	47	67
91.5 – 92.5	14	29	47	67
92.5 – 93.5	14	28	47	67
93.5 – 94.5	14	28	47	66
94.5 – 95.5	14	28	46	66
95.5 – 96.5	14	28	46	66
96.5 – 97.5	14	28	46	66
97.5 – 98.5	14	27	46	65
98.5 – 99.5	14	27	45	65
99.5 – 100.5	13	27	45	65
100.5 – 101.5	13	27	45	65
101.5 – 102.5	13	27	45	64
102.5 – 103.5	13	27	44	64
103.5 – 104.5	13	26	44	64
104.5 – 105.5	13	26	44	64
105.5 – 106.5	13	26	44	63
106.5 – 107.5	13	26	44	63
107.5 – 108.5	13	26	43	63
108.5 – 109.5	12	26	43	63
109.5 – 110.5	12	25	43	62
110.5 – 111.5	12	25	43	62
111.5 – 112.5	12	25	43	62
112.5 – 113.5	12	25	42	62
113.5 – 114.5	12	25	42	62
114.5 – 115.5	12	25	42	61
115.5 – 116.5	12	24	42	61
116.5 – 117.5	12	24	42	61
117.5 – 118.5	12	24	41	61
118.5 – 119.5	12	24	41	61
119.5 – 120.5	11	24	41	60
120.5 – 121.5	11	24	41	60

Note: * Rate of Rainfall is the average value over consecutive time intervals

Table 5d – Design Rainstorm Profile Intensity-Duration-Frequency Relationships of North District Area

Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)				Duration Interval (min)	*Rate of Rainfall (mm/hr) for Return Periods T (years)			
	2	10	50	200		2	10	50	200
-0.5 – 0.5	155	193	232	276	60.5 – 61.5	18	29	37	44
0.5 – 1.5	140	177	212	247	61.5 – 62.5	18	29	37	43
1.5 – 2.5	123	160	190	217	62.5 – 63.5	18	29	36	43
2.5 – 3.5	111	145	172	194	63.5 – 64.5	18	28	36	42
3.5 – 4.5	100	133	158	176	64.5 – 65.5	17	28	36	42
4.5 – 5.5	91	123	146	162	65.5 – 66.5	17	28	35	42
5.5 – 6.5	84	115	136	150	66.5 – 67.5	17	27	35	41
6.5 – 7.5	78	107	127	140	67.5 – 68.5	17	27	35	41
7.5 – 8.5	73	101	120	132	68.5 – 69.5	17	27	34	41
8.5 – 9.5	68	95	113	124	69.5 – 70.5	16	27	34	40
9.5 – 10.5	64	90	107	118	70.5 – 71.5	16	26	34	40
10.5 – 11.5	60	86	102	112	71.5 – 72.5	16	26	34	40
11.5 – 12.5	57	82	98	107	72.5 – 73.5	16	26	33	39
12.5 – 13.5	54	78	94	103	73.5 – 74.5	16	26	33	39
13.5 – 14.5	52	75	90	99	74.5 – 75.5	16	25	33	39
14.5 – 15.5	50	72	86	95	75.5 – 76.5	16	25	33	39
15.5 – 16.5	48	69	83	92	76.5 – 77.5	15	25	32	38
16.5 – 17.5	46	67	81	89	77.5 – 78.5	15	25	32	38
17.5 – 18.5	44	64	78	86	78.5 – 79.5	15	25	32	38
18.5 – 19.5	42	62	75	84	79.5 – 80.5	15	24	32	37
19.5 – 20.5	41	60	73	81	80.5 – 81.5	15	24	31	37
20.5 – 21.5	39	59	71	79	81.5 – 82.5	15	24	31	37
21.5 – 22.5	38	57	69	77	82.5 – 83.5	15	24	31	37
22.5 – 23.5	37	55	67	75	83.5 – 84.5	14	24	31	36
23.5 – 24.5	36	54	66	74	84.5 – 85.5	14	23	30	36
24.5 – 25.5	35	52	64	72	85.5 – 86.5	14	23	30	36
25.5 – 26.5	34	51	63	70	86.5 – 87.5	14	23	30	36
26.5 – 27.5	33	50	61	69	87.5 – 88.5	14	23	30	36
27.5 – 28.5	32	49	60	67	88.5 – 89.5	14	23	30	35
28.5 – 29.5	31	48	59	66	89.5 – 90.5	14	23	29	35
29.5 – 30.5	31	47	57	65	90.5 – 91.5	14	22	29	35
30.5 – 31.5	30	46	56	64	91.5 – 92.5	14	22	29	35
31.5 – 32.5	29	45	55	63	92.5 – 93.5	13	22	29	34
32.5 – 33.5	29	44	54	61	93.5 – 94.5	13	22	29	34
33.5 – 34.5	28	43	53	60	94.5 – 95.5	13	22	28	34
34.5 – 35.5	27	42	52	59	95.5 – 96.5	13	22	28	34
35.5 – 36.5	27	41	51	59	96.5 – 97.5	13	22	28	34
36.5 – 37.5	26	41	51	58	97.5 – 98.5	13	21	28	33
37.5 – 38.5	26	40	50	57	98.5 – 99.5	13	21	28	33
38.5 – 39.5	25	39	49	56	99.5 – 100.5	13	21	28	33
39.5 – 40.5	25	39	48	55	100.5 – 101.5	13	21	27	33
40.5 – 41.5	24	38	47	54	101.5 – 102.5	13	21	27	33
41.5 – 42.5	24	37	47	54	102.5 – 103.5	13	21	27	33
42.5 – 43.5	24	37	46	53	103.5 – 104.5	12	21	27	32
43.5 – 44.5	23	36	45	52	104.5 – 105.5	12	20	27	32
44.5 – 45.5	23	36	45	52	105.5 – 106.5	12	20	27	32
45.5 – 46.5	22	35	44	51	106.5 – 107.5	12	20	26	32
46.5 – 47.5	22	35	44	50	107.5 – 108.5	12	20	26	32
47.5 – 48.5	22	34	43	50	108.5 – 109.5	12	20	26	32
48.5 – 49.5	21	34	43	49	109.5 – 110.5	12	20	26	31
49.5 – 50.5	21	33	42	49	110.5 – 111.5	12	20	26	31
50.5 – 51.5	21	33	41	48	111.5 – 112.5	12	20	26	31
51.5 – 52.5	20	32	41	48	112.5 – 113.5	12	19	26	31
52.5 – 53.5	20	32	41	47	113.5 – 114.5	12	19	25	31
53.5 – 54.5	20	32	40	47	114.5 – 115.5	12	19	25	31
54.5 – 55.5	20	31	40	46	115.5 – 116.5	12	19	25	31
55.5 – 56.5	19	31	39	46	116.5 – 117.5	11	19	25	30
56.5 – 57.5	19	31	39	45	117.5 – 118.5	11	19	25	30
57.5 – 58.5	19	30	38	45	118.5 – 119.5	11	19	25	30
58.5 – 59.5	19	30	38	44	119.5 – 120.5	11	19	25	30
59.5 – 60.5	18	29	38	44	120.5 – 121.5	11	19	25	30

Note: * Rate of Rainfall is the average value over consecutive time intervals

Table 6 – Intensity-Duration-Frequency (IDF) Relationship
for Frequent Rainstorms

Duration (min)	**Extreme Mean Intensity (mm/hr) for various Frequencies (per year)**			
	10	5	2	1
480++	5.2	8.1	12.1	15.0
420++	6.0	9.1	13.2	16.3
360++	7.1	10.5	15.0	18.4
300++	8.6	12.4	17.4	21.2
240++	10.7	14.9	20.6	24.8
180++	13.1	18.2	25.1	30.2
120++	17.9	24.1	32.3	38.5
60++	27.5	36.1	47.6	56.2
30++	39.2	50.6	65.6	77.0
15++	53.1	67.3	86.1	100.3
10*	61.0	77.0	98.0	114.0
5*	77.5	96.0	120.5	139.0
2*	97.0	119.0	148.0	170.0
1*	114.0	138.1	169.9	194.0
0.25+++	147.9	173.7	207.7	233.5

Notes :
1. ++ based on autographic rainfall charts at RO (now HKO) Headquarters (1947 - 1965)
2. +++ based on Jardi rate-of-rainfall records at King's Park (1952 - 1965)
3. * based on interpolated data

Table 7 - Tide Gauges in Hong Kong

Name of Station	Data archived since year	Tide Gauge managed by	Remarks
North Point	1954		Discontinued in 1986
Chi Ma Wan	1961		Discontinued in 1997
Tai Po Kau	1963	Hong Kong Observatory	
Ko Lau Wan (old)	1974		Discontinued in 1995
Tsim Bei Tsui	1974	Hong Kong Observatory	
Waglan Island	1976	Hong Kong Observatory	
Lok On Pai	1981		Discontinued in 1999
Tamar	1984		Discontinued in 1991
Quarry Bay	1985	Hong Kong Observatory	
Tai O (old)	1985		Discontinued in 1997
Tai Miu Wan	1994	Hong Kong Observatory	
Shek Pik	1997	Hong Kong Observatory	
Chek Lap Kok	1999	Airport Authority	
Ko Lau Wan	2000	Hydrographic Office of Marine Department	
Kwai Chung	2001	Hydrographic Office of Marine Department	
Ma Wan	2004	Hydrographic Office of Marine Department	
Cheung Chau	2005	Hydrographic Office of Marine Department	
Sha Tau Kok*	2004	Drainage Services Department	
Sai Kung*	2004	Drainage Services Department	
Mui Wu*	2004	Drainage Services Department	
Tai O*	2005	Drainage Services Department	

Remarks: The location plan for tide gauges is shown on Figure 7.

*The tidal level readings were measured by ultra sonic sensors installed above water.

Table 8 – Design Extreme Sea Levels (in mPD)

Return Period (Years)	**North Point/ Quarry Bay (1954-2017)**	**Tai Po Kau (1962-2017)**	**Tsim Bei Tsui (1974-2017)**	**Tai O (1985-2017)**
2	2.73	2.91	3.07	2.87
5	2.94	3.20	3.31	3.16
10	3.09	3.45	3.51	3.36
20	3.24	3.73	3.74	3.57
50	3.45	4.19	4.09	3.84
100	3.63	4.60	4.40	4.06
200	3.81	5.10	4.77	4.28

Table 9 – Mean Higher High Water (MHHW) Levels (in mPD)

North Point/ Quarry Bay (1962-2017)	**Tai Po Kau (1981-2017)**	**Tsim Bei Tsui (1983-2017)**	**Tai O (1985-2017)**
2.01	2.02	2.32	2.13

Table 10 – Recommended Design Return Periods based on Flood Levels

Intensively Used Agricultural Land	2-5 years
Village Drainage including Internal Drainage System under a Polder Scheme	10 years [1,3]
Main Rural Catchment Drainage Channels	50 years [2,3]
Urban Drainage Trunk Systems	200 years [4]
Urban Drainage Branch Systems	50 years [4]

Notes:

1. The impact of a 50-year event should be assessed in each village to check whether a higher standard than 10 years can be justified.
2. Embanked channels must be capable of passing a 200-year flood within banks.
3. For definitions of Village Drainage and Main Rural Catchment Drainage Channels, refer to Section 6.6.1.
4. For definitions of Urban Drainage Branch and Urban Drainage Trunk Systems, refer to Section 6.6.2.

Table 11 – Determination of Flood Level in the Fluvial-Tidal Zone

Flood Level Return Period	**Case I**	**Case II**
200 years	200-year rain + 10-year sea level	10-year rain + 200-year sea level
100 years	100-year rain + 10-year sea level	10-year rain + 100-year sea level
50 years	50-year rain + 10-year sea level	10-year rain + 50-year sea level
10 years	10-year rain + 2-year sea level	2-year rain + 10-year sea level
5 years	5-year rain + 2-year sea level	2-year rain + 5-year sea level
2 years	2-year rain + 2-year sea level	-

Table 12 - Frictional Resistance Equations

Equations	Formulation	Limit of Applications
Chézy	$\bar{V} = C\sqrt{RS_f}$	rough turbulent
Manning	$\bar{V} = \frac{R^{1/6}}{n}\sqrt{RS_f}$	rough turbulent
Darcy-Weisbach	$\bar{V} = \sqrt{\frac{8g}{f}}\sqrt{RS_f}$	laminar/turbulent
Hagen-Poiseuille	$\bar{V} = \frac{gS_f R^2}{2\nu}$	laminar
Colebrook-White	$\bar{V} = -\sqrt{32gRS_f}\,\log\left[\frac{k_s}{14.8R} + \frac{1.255\nu}{R\sqrt{32gRS_f}}\right]$	transition between rough and smooth turbulent flow
Hazen-Williams	$\bar{V} = 0.85C_{HW}R^{0.63}S_f^{0.54}$	pipe flow $\bar{V} < 3m/s,$ diameter > 0.05m

Table 13 - Values of n to be used with the Manning equation

Source: Brater, E.F. & King, H.W. (1976)

Surface	**Best**	**Good**	**Fair**	**Bad**
Uncoated cast-iron pipe	0.012	0.013	0.014	0.015
Coated cast-iron pipe	0.011	0.012*	0.013*	
Commercial wrought-iron pipe, black	0.012	0.013	0.014	0.015
Commercial wrought-iron pipe, galvanized	0.013	0.014	0.015	0.017
Smooth brass and glass pipe	0.009	0.010	0.011	0.013
Smooth lockbar and welded “OD” pipe	0.010	0.011*	0.013*	
Riveted and spiral steel pipe	0.013	0.015*	0.017*	
Vitrified sewer pipe	0.010	0.013*	0.015	0.017
Common clay drainage tile	0.011	0.012*	0.014*	0.017
Glazed brickwork	0.011	0.012	0.013*	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015*	0.017
Neat cement surfaces	0.010	0.011	0.012	0.013
Cement mortar surfaces	0.011	0.012	0.013*	0.015
Concrete pipe	0.012	0.013	0.015*	0.016
Wood stave pipe	0.010	0.011	0.012	0.013
Plank flumes - Planed	0.010	0.012*	0.013	0.014
- Unplaned	0.011	0.013*	0.014	0.015
- With battens	0.012	0.015*	0.016	
Concrete-lined channels	0.012	0.014*	0.016*	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry-rubble surface	0.025	0.030	0.033	0.035
Dressed-ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
Semicircular metal flumes, corrugated	0.0225	0.025	0.0275	0.030
Canals and ditches				
1. Earth, straight and uniform	0.017	0.020	0.0225*	0.025
2. Rock cuts, smooth and uniform	0.025	0.030	0.033*	0.035
3. Rock cuts, jagged and irregular	0.035	0.040	0.045	
4. Winding sluggish canals	0.0225	0.025*	0.0275	0.030
5. Dredged-earth channels	0.025	0.0275*	0.030	0.033
6. Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035*	0.040
7. Earth bottom, rubble sides	0.028	0.030*	0.033*	0.035
Natural-stream channels				
1. Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
2. Same as (1) but some weeds and stones	0.030	0.033	0.035	0.040
3. Winding some pools and shoals, clean	0.033	0.035	0.040	0.045
4. Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055

Table 13 (Cont'd)

Surface	**Best**	**Good**	**Fair**	**Bad**
5. Same as (3) some weeds and stones	0.035	0.040	0.045	0.050
6. Same as (4) stony sections	0.045	0.050	0.055	0.060
7. Sluggish river reach, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8. Very weedy reaches	0.075	0.100	0.125	0.150

Notes: *Values commonly used for design.

Table 14 - Recommended Roughness Values k_s

Source : HR Wallingford et. al. (2006)

Material Classification (see note 1)	Suitable values of k_s (mm) Good	Normal	Poor
Smooth Materials (Pipes)			
Drawn non-ferrous pipes of aluminium, brass, copper, lead, etc. and non-metallic pipes of Alkathene, glass, perspex, uPVC, etc.	-	0.003	-
Asbestos Cement	0.015	0.03	0.06
Metal			
Spun bitumen or concrete lined	-	0.03	-
Cast iron, epoxy lining, coupling joints	-	0.015	0.03
Ductile iron, polyethylene lining, push-fit joints	-	0.06	0.15
Ductile iron, polyurethane lining, push-fit joints	0.015	0.03	0.06
Wrought iron	0.03	0.06	0.15
Rusty wrought iron	0.15	0.6	3.0
Uncoated steel	0.015	0.03	0.06
Rusty steel	-	0.15	0.3
Steel, epoxy lining, push-fit joints	0.03	0.06	0.15
Galvanised iron, coated cast iron generally	0.06	0.15	0.3
Uncoated cast iron	0.15	0.3	0.6
Tate relined pipes	0.15	0.3	0.6
152mm x 51mm corrugated plate, unpaved circular pipe, running full	50	60	-
76mm x 25mm corrugated plate, unpaved circular pipe, running full	25	30	-
Old tuberculated water mains as follows :			
Slight degree of attack	0.6	1.5	3.0
Moderate degree of attack	1.5	3.0	6.0
Appreciable degree of attack	6.0	15	30
Severe degree of attack	15	30	60
(Good: up to 20 years use; Normal: 40 to 50 years use; Poor : 80 to 100 years use)			
Wood			
Wood stave pipes, planed plank conduits	0.3	0.6	1.5
Concrete			
Prestressed	0.03	0.06	0.15
Precast concrete pipes with ‘O’ ring joints	0.06	0.15	0.6
Spun precast concrete pipes with ‘O’ ring joints	0.06	0.15	0.3
Monolithic construction against steel forms	0.3	0.6	1.5
Monolithic construction against rough forms	0.6	1.5	-

Table 14 (Cont'd)

Material Classification (see note 1)	**Suitable values of k_s (mm)**		
	Good	Normal	Poor
Clayware			
Glazed or unglazed pipe:			
With sleeve joints	0.03	0.06	0.15
With spigot and socket joints and ‘O’ ring seals dia < 150mm	-	0.03	-
With spigot and socket joints and ‘O’ ring seals dia > 150mm	-	0.06	-
Pitch Fibre (lower value refers to full bore flow)	0.003	0.03	-
Glass Reinforced Plastic (GRP)	0.03	0.06	-
uPVC			
Twin-walled, with coupling joint	0.003	0.006	
Standard, with chemically cemented joints	-	0.03	-
Standard, with spigot and socket joints, ‘O’ ring seals at 6 m to 9 m intervals	-	0.06	-
New Relining of Sewers			
Factory manufactured GRP	0.03	-	-
Brickworks			
Glazed	0.6	1.5	3.0
Well pointed	1.5	3.0	6.0
Old, in need of pointing	-	15	30
Slimed Sewers (see note 3 & Section 8.3.1)			
Sewers slimed to about half depth; velocity, when flowing half full, approximately 0.75 m/s:			
Concrete, spun or vertically cast	-	3.0	6.0
Asbestos cement	-	3.0	6.0
Clayware	-	1.5	3.0
uPVC	-	0.6	1.5
Sewers slimed to about half depth; velocity, when flowing half full, approximately 1.2 m/s :			
Concrete, spun or vertically cast	-	1.5	3.0
Asbestos cement	-	0.6	1.5
Clayware	-	0.3	0.6
uPVC	-	0.15	0.3

Table 14 (Cont'd)

Material Classification (see note 1)	**Suitable values of k_s (mm)**		
	Good	Normal	Poor
Sewer Rising Mains (see note 4 & Section 8.3.1)			
All materials, operating as follows:			
Mean velocity 0.5 m/s	0.3	3.0	30
Mean velocity 0.75 m/s	0.15	1.5	15
Mean velocity 1 m/s	0.06	0.6	6.0
Mean velocity 1.5 m/s	0.03	0.3	1.5
Mean velocity 2 m/s	0.015	0.15	1.5
Concrete Channels			
Trowel finish	0.5	1.5	3.3
Float finish	1.5	3.3	5.0
Finished with gravel on bottom	3.3	7.0	18
Unfinished	2.0	7.0	18
Shotcrete, or Gunite, good section	5.0	14	43
Shotcrete, or Gunite, wavy section	10	33	70
Unlined Rock Tunnels			
Granite and other homogeneous rocks	60	150	300
Diagonally bedded slates	-	300	600
Earth Channels			
Straight uniform artificial channels	15	60	150
Straight natural channels, free from shoals, boulders and weeds	150	300	600

Notes :

1. The classifications 'Good', 'Normal' and 'Poor' refer to good, normal and poor examples of their respective categories unless otherwise stated. Classifications 'Good' and 'Normal' are for new and clean pipelines. The range of roughness takes account not only of the quality of the jointing but also the variation in surface roughness to be found in pipes that are normally of the same material.

2. Figures in shaded bold print are the values particularly recommended for general design purposes.

3. The hydraulic roughness of slimed sewers vary considerably during any year. The 'Normal' value is that roughness which is exceeded for approximately half of the time. The 'Poor' value is that which is exceeded, generally on a continuous basis, for one month of the year. The value of k_s should be interpolated for velocities between 0.75 m/s and 1.2 m/s. In Hong Kong, sewers for permanent use should be classified as slimed sewers.

4. The hydraulic roughness of sewer rising mains varies principally with the amount of slime that builds up inside the pipe and is normally not significantly affected by factors such as the jointing or the construction. Primarily, the increasing roughness values are intended to cover for the loss of flow area. The 'Normal' value represents the mean value of the measured hydraulic roughness while the 'Good' and 'Poor' values represent the values which are two standard deviations on each side of the 'Normal' value.

Table 15 - Head Losses Coefficient K

Source: DSD (1992)

	Entry Losses		**K**
	Sharp-edged entrance		0.50
	Re-entrant entrance		0.80
	Slightly rounded entrance		0.25
	Bellmouthed entrance		0.05
	Footvalve and strainer		2.50
	Intermediate Losses		**K**
(i)	Elbows	22.5^0	0.20
	(R/D = 1 approx.)	45^0	0.40
		90^0	1.00
(ii)	Close Radius Bends	22.5^0	0.15
	(R/D = 1 approx.)	45^0	0.30
		90^0	0.50**
(iii)	Long Radius Bends	22.5^0	0.10
	(R/D = 2 to 7)	45^0	0.20
		90^0	0.40
(iv)	Sweeps	22.5^0	0.05
	(R/D = 8 to 50)	45^0	0.10
		90^0	0.20
(v)	Mitre Elbows		
	2 piece	22.5^0	0.15
	2 piece	30^0	0.20
	2 or 3 piece	45^0	0.30
	2 piece	60^0	0.65
	3 piece	60^0	0.25
	2 piece	90^0	1.25
	3 piece	90^0	0.50
	4 piece	90^0	0.30
(vi)	Tees		
	Flow in line		0.35
	Line to branch or branch to line:		
	Sharp-edged		1.20
	Radiused		0.80
(vii)	Angle Branches		
	Flow in line		0.35

	Intermediate Losses (cont'd)	**K**
	Line to branch or branch to line :	
	30^0 angle	0.40
	45^0 angle	0.60
	90^0 angle	0.80
(viii)	Sudden Enlargements*	
	Inlet dia : Outlet dia.	
	4:5	0.15
	3:4	0.20
	2:3	0.35
	1:2	0.60
	1:3	0.80
	1:5 and over	1.00
(ix)	Sudden Contractions*	
	Inlet dia : Outlet dia.	
	5:4	0.15
	4:3	0.20
	3:2	0.30
	2:1	0.35
	3:1	0.45
	5:1 and over	0.50
(x)	B.S. Tapers*	
	Flow to small end	negligible
	Flow to large end	
	Inlet dia : Outer dia.	
	4:5	0.02
	3:4	0.04
	1:2	0.12
(xi)	Valves	
	Gate Valve - fully open	0.12
	1/4 closed	1.00
	1/2 closed	6.00
	3/4 closed	24.00
	Globe valve	10.00
	Right angle valve	5.00
	Reflux valve	1.00
	Butterfly valve	0.30
	Exit Losses	**K**
	Sudden Enlargement	1.00
	Bellmouthed Outlet	1.00**

Notes: * Figure for enlargements, contractions and B.S. Tapers apply to smaller diameter.
** Value modified.

$$K = \frac{\textit{Head Loss}}{V^2/2g} = \frac{\textit{Head Loss}}{\textit{Velocity Head}}$$

Table 16 – Narrow and Wide Trench Fill Load

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. **B_c** (mm)	Type of Load	Assumed Trench Width **B_d** (m)	**T_d** (m)	**Fill Load** in kN/m of pipe length for cover depth in metres						
					0.9	1.2	1.5	1.8	2.4	3.0	4.6
150	190	Narrow	0.60	3.7	-	-	13.4	15.1	18.1	20.4	24.0
		Wide	-	-	5.4	7.1	9.0	10.8	14.4	18.1	27.1
225	280	Narrow	0.70	2.4	-	-	15.6	17.8	21.4	24.2	29.2
		Wide	-	-	7.9	10.5	13.1	15.7	21.1	26.4	39.7
300	380	Narrow	0.75	1.5	-	-	17.8	20.3	24.6	28.3	34.6
		Narrow	0.85	2.4	-	-	19.8	23.2	28.4	32.6	40.4
		Wide	-	-	10.6	14.3	17.9	21.6	28.7	36.0	54.1
375	500	Narrow	1.00	2.4	-	-	24.3	27.8	35.3	40.2	52.6
		Narrow	1.05	3.0	-	-	26.5	30.8	38.4	44.9	57.6
		Wide	-	-	12.8	17.5	21.9	26.4	35.2	44.0	66.4
450	580	Narrow	1.15	2.4	-	-	28.9	33.5	41.9	49.3	63.6
		Wide	-	-	14.3	20.6	25.8	30.9	41.4	51.9	78.2
600	790	Narrow	1.35	1.8	-	-	35.6	41.5	52.5	62.3	82.0
		Wide	-	-	17.8	25.7	34.4	41.5	55.7	69.7	105
750	950	Narrow	1.50	1.5	-	-	40.1	47.0	59.5	71.0	94.6
		Narrow	1.60	1.8	-	-	41.2	50.2	62.8	74.3	102
		Wide	-	-	20.6	29.3	39.2	50.0	67.0	84.1	127
900	1120	Narrow	1.90	2.1	-	-	51.3	60.4	77.5	93.0	127
		Wide	-	-	23.5	33.1	43.8	55.5	78.5	98.6	149
1050	1300	Narrow	2.05	2.1	-	-	55.9	65.8	84.6	102	140
		Wide	-	-	26.7	37.4	48.9	61.5	90.6	114	172
1200	1490	Narrow	2.30	2.1	-	-	62.7	74.0	95.3	115	159
		Wide	-	-	30.0	41.7	54.4	68.0	98.8	130	197
1350	1650	Narrow	2.45	2.1	-	-	67.2	79.2	102	124	173
		Wide	-	-	33.0	45.6	59.3	73.6	106	143	219
1500	1830	Narrow	2.60	2.1	-	-	71.8	84.8	110	133	187
		Wide	-	-	36.2	49.7	64.4	80.0	114	153	242
1650	2010	Narrow	2.80	2.4	-	-	78.5	93.0	121	147	206
		Wide	-	-	39.4	54.0	69.5	86.0	122	162	264
1800	2240	Narrow	3.05	2.4	-	-	85.4	101	131	160	226
		Wide	-	-	43.5	59.4	76.3	94.1	132	175	295

Table 17 – Traffic Loads

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. **B_c** (mm)	Type of Load	**Traffic Load** in kN/m of pipe length for cover depth in metres						
			0.9	1.2	1.5	1.8	2.4	3.0	4.6
150	190	Main road Light road	16.8 13.6	12.8 9.2	10.4 6.4	8.9 4.8	6.7 2.9	5.2 2.0	3.2 1.0
225	280	Main road Light road	24.5 19.7	18.6 13.2	15.3 9.5	13.0 7.0	9.9 4.2	7.7 2.9	4.4 2.3
300	380	Main road Light road	33.3 26.8	25.4 18.1	20.7 12.8	17.6 9.6	13.6 5.8	10.5 3.9	6.1 1.7
375	500	Main road Light road	42.9 34.4	32.7 23.2	26.7 16.6	23.0 12.4	17.5 7.6	13.7 5.0	7.9 2.3
450	580	Main road Light road	50.0 40.1	38.2 27.0	31.5 19.4	27.1 14.6	20.7 8.9	16.2 6.0	9.3 2.8
600	790	Main road Light road	66.6 53.0	51.5 36.2	42.3 26.0	36.5 19.3	27.7 11.8	21.7 7.9	12.5 3.6
750	950	Main road Light road	80.2 63.5	62.0 43.2	51.0 31.2	43.9 23.3	33.4 14.1	26.1 9.5	15.0 4.4
900	1120	Main road Light road	93.2 73.3	72.6 50.2	60.0 36.2	51.3 27.1	39.1 16.8	30.5 11.1	18 5
1050	1300	Main road Light road	106 82.7	84.3 57.6	69.5 41.5	59.8 31.1	45.2 19.2	35 13	20 6
1200	1490	Main road Light road	120 92	96.6 65.0	80.4 47.2	68.2 35.6	51.8 22.0	40 15	23 7
1350	1650	Main road Light road	131 99.5	107 71.0	89.4 51.7	76.4 39.4	58 24	45 16	26 8
1500	1830	Main road Light road	143 107	118 77.0	99.1 56.6	85.0 43.1	64 27	49 18	28 8
1650	2010	Main road Light road	154 114	129 82.7	109 61.0	93.8 46.8	70 29	54 20	31 9
1800	2240	Main road Light road	172 122	144 89.4	122 66.6	104 51.0	79 32	61 22	35 10

Table 18 - Design Loads for Rigid Buried Pipelines

(a) Design Load for Main Roads :

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. **B_c** (mm)	Assumed Trench Width **B_d** (m)	**Total Design Load** in kN/m of pipe length for cover depth in metres						
			0.9	1.2	1.5	1.8	2.4	3.0	4.6
150	190	0.60	22	19.5	19.5	19.5	21	23	27
225	280	0.70	32	29	28	28	31	32	34
300	380	0.75	44	40	39	38	38	39	41
375	500	1.05	55	50	48	50	53	58	66
450	580	1.15	64	58	57	58	63	68	73
600	790	1.35	86	79	79	80	83	86	96
750	950	1.50	105	95	93	95	96	100	115
900	1120	1.90	120	110	110	110	120	130	150
1050	1300	2.05	140	130	125	130	135	145	170
1200	1490	2.30	160	145	145	145	155	165	190

(b) Design Load for Light Roads :

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. **B_c** (mm)	Assumed Trench Width **B_d** (m)	**Total Design Load** in kN/m of pipe length for cover depth in metres						
			0.9	1.2	1.5	1.8	2.4	3.0	4.6
150	190	0.60	19	16	15.5	15.5	17.5	20	25
225	280	0.70	28	24	23	23	25	27	31
300	380	0.75	38	32	31	30	31	32	36
375	500	1.05	47	41	38	39	42	50	60
450	580	1.15	54	48	45	45	51	55	67
600	790	1.35	73	64	63	63	67	73	87
750	950	1.50	87	76	74	74	77	85	100
900	1120	1.90	100	88	85	87	99	110	135
1050	1300	2.05	115	100	98	99	110	120	155
1200	1490	2.30	130	115	110	110	125	140	175

Table 19 - Minimum Strength or Class of Pipes in Main Roads

(a) Class of Precast Concrete Pipes

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. $\mathbf{B_c}$ (mm)	Assumed Trench Width $\mathbf{B_d}$ (mm)	Bedding Factor $\mathbf{F_m}$	**Class of Precast Concrete Pipes** to BS 5911:Part 100:1988 for cover depth in metres						
				0.9	1.2	1.5	1.8	2.4	3.0	4.6
150	190	600	1.9	L	L	L	L	L	L	L
			2.6	L	L	L	L	L	L	L
			3.4	L	L	L	L	L	L	L
225	280	670	1.9	L	L	L	L	L	L	L
			2.6	L	L	L	L	L	L	L
			3.4	L	L	L	L	L	L	L
300	380	750	1.9	M	M	M	L	L	M	M
			2.6	L	L	L	L	L	L	L
			3.4	L	L	L	L	L	L	L
375	500	1050	1.9	M	M	M	M	M	M	H
			2.6	M	L	L	L	M	M	M
			3.4	L	L	L	L	L	L	L
450	580	1150	1.9	M	M	M	M	M	H	H
			2.6	M	M	M	M	M	M	M
			3.4	L	L	L	L	L	L	M
600	790	1350	1.9	M	M	M	M	M	M	H
			2.6	M	M	M	M	M	M	M
			3.4	M	M	M	M	M	M	M
750	950	1500	1.9	H	M	M	M	M	M	H
			2.6	M	L	L	L	L	M	M
			3.4	L	L	L	L	L	L	L
900	1120	1900	1.9	M	M	M	M	M	H	H
			2.6	L	L	L	L	L	M	M
			3.4	L	L	L	L	L	L	L
1050	1300	2050	1.9	M	M	M	M	M	H	H
			2.6	M	L	L	L	M	M	M
			3.4	L	L	L	L	L	L	L
1200	1490	2300	1.9	M	M	M	M	M	M	H
			2.6	M	L	L	L	M	M	M
			3.4	L	L	L	L	L	L	L

Table 19 (Cont'd)

(b) Class of Vitrified Clay Pipes

Nominal Pipe Dia. **DN** (mm)	Assumed Outside Dia. B_c (mm)	Assumed Trench Width B_d (mm)	Bedding Factor F_m	**Class of Vitrified Clay Pipes** to BS 65:1988 for cover depth in metres						
				0.9	1.2	1.5	1.8	2.4	3.0	4.6
100	130	600	1.9	F	F	F	F	F	F	F
			2.5	F	F	F	F	F	F	F
150	190	600	1.9	F	F	F	F	F	F	F
			2.5	F	F	F	F	F	F	F
200 &	245 &	700	1.9	F	F	F	F	F	F	F
225	280		2.5	F	F	F	F	F	F	F
300	370	800	1.9	F	F	F	F	F	F	B
			2.5	F	F	F	F	F	F	F
375 &	460 &	1100	1.9	F	F	F	F	F	F	F
400	500		2.5	F	F	F	F	F	F	F
450	550	1200	1.9	F	F	F	F	F	F	B
			2.5	F	F	F	F	F	F	F
500	615	1300	1.9	F	F	F	F	F	F	B
			2.5	F	F	F	F	F	F	F
600	730	1400	1.9	B	F	F	F	F	B	B
			2.5	F	F	F	F	F	F	F

Note :

1. *L, M, H denote class of concrete pipes in Table 7 of BS 5911:Part100:1988.*
2. *F and B denote Standard and Extra Strength respectively of vitrified clay pipes in Table 3 of BS65:1988.*

Table 20 – Schedule of Maintenance Responsibilities for a Completed Main Drainage Channel Project

Below is an example for reference only. Project engineer should liaise with the relevant authorities to identify the maintenance and management parties for their own works in the design stage.

Completed Works	Maintenance Department	Management Department
Main drainage channel and associated drainage works	DSD(1)	DSD(2)
Public roads (including footpath, road drain, road side verges and slopes) to HyD's standard, and associated street furniture including lamp posts and cable draw pits	HyD	TD & HKPF
Border road (including road side verges and slopes)	HyD	TD & HKPF
Security-associated furniture along border road, including border fence, lamp posts, cable draw pits, a main switch room, pillar boxes and security grills	Arch.SD	HKPF
Run-ins	HAD	HAD
Road cleaning	LCSD	--
Box culverts and stormwater drains	DSD	DSD
Borrow area	CEDD	CEDD
Roadside landscape softworks	LCSD	LCSD
Off-roadside landscape softworks in plantation area and landscape softworks inside verges and slope of border road	AFCD	AFCD
Mangrove plantation	CEDD	CEDD(3)
Grasscrete softworks	LCSD	LCSD
Grasscrete hardworks	DSD	DSD

Table 20(Cont'd)

+3.4mPD embankment platforms and surplus strips of land	Lands D	Lands D
Fishponds and associated landings, chain link fence and gates	Lands D	Lands D
Concrete parapets	DSD	DSD
Water points and water metre boxes	Arch.SD	--
Fire hydrants	WSD	WSD
Marking plates of slope registration numbers	DSD	DSD

Notes: (1) DSD is responsible for planning and carrying out desilting/ dredging work, and structural maintenance of the channel banks. FEHD is responsible for the removal of refuses, animal carcasses, dead fishes and vegetation.

(2) Future management of waters within the main drainage channel will be further discussed among Marine Department, Marine Police, LCSD and DSD. DSD's management role is currently limited to flood control aspects.

(3) CEDD will temporarily take care of mangrove plantation. Future operation and maintenance of this wetland will be addressed as part of Wetland Compensation Study by AFCD.

Table 21 - Methods of Stability Improvement

Method	Application	Limitation
Replacement of foundation soil	To replace completely or partially the poor quality foundation soil with suitable filling material so as to provide a better foundation for constructing the embankment	Huge quantity of unsuitable material has to be disposed of and a large quantity of imported fill is required
Modification of embankment geometry	To provide lateral berms on both sides of the embankment to act as counterweights on the least favourable potential failure surfaces	Additional land resumption/ clearance is required
Staged construction	To leave sufficient delay between each increase in height of the embankment to allow the pore pressure to dissipate so that the foundation soil can develop the necessary shear strength to support the weight of the next layer of embankment	Longer construction period is required
Provision of geosynthetic reinforcement	To introduce an reinforcing element at the base of the embankment to provide an additional restoring force to act against the potential slippage of the embankment	Not effective in improving the embankment against bearing capacity failure
Installation of vertical drain	To accelerate the consolidation process of the foundation soil so that it can develop the necessary shear strength to support the weight of the embankment in a shorter period	Not applicable for areas with thin layer of soft foundation soil
Installation of Geocell Mattress	To provide a rigid platform at the base of the embankment to increase the bearing capacity of the foundation soil and reduce the differential settlement	Not economical for areas with thin layer of soft foundation soil

Table 22 - Components of a Polder and Floodwater Pumping Scheme

Facility	Components	Functions
Protective Bund	Earth bund and associated accesses, such as berms along the bund, to facilitate inspection and maintenance	To prevent external floodwater from entering village
	Concrete grass lining	Landscaping and protection against scouring
	Hydroseeding	Landscaping and erosion protection to slopes
	Roads and footpaths	To facilitate operation and maintenance and to allow villagers to escape during flooding and complete failure of the scheme
Inlets to pump sump	Trash screens and associated accesses/working platforms to facilitate inspection and maintenance	To protect clogging the pumps by any debris brought there by the runoff
Gravity outlets from village floodwater storage pond	Flexible flap valves	To prevent reverse flow from the drainage channel to the floodwater storage pond
	Penstocks	To isolate the floodwater storage pond from the outside main drainage channel
	Concrete pipes linked with pond	To drain the cumulated stormwater from the floodwater storage pond to the drainage channel, by gravity
	Trash screens and working platforms	
	Accesses to these components to facilitate inspection and maintenance	

Table 22 (Cont'd)

Facility	Components	Functions
Floodwater storage pond	Pond area	To provide floodwater storage
	Peripheral channels	To convey runoff to outlet under normal rainstorm situations
	Maintenance access and working platform	To carry out maintenance works to trash screens, silt traps, flap valves, penstock, remove water hyacinth and so on
	Concrete grass lining	Landscaping and protection against scouring
	Hydroseeding	Landscaping and erosion protection to slopes
	Peripheral fence	To prevent unauthorized entry, illegal dumping and illegal occupation
	Raised peripheral amenity planter	To provide environmental improvements
	Warning sign boards	To warn members of the public to keep out of the floodwater storage pond
Surface channels in village	Channels, concrete pipes and catchpits	To convey stormwater runoff to floodwater storage pond under gravity

Table 22 (Cont'd)

Facility	Components	Functions
Floodwater pumping station	Reinforced concrete pumping station structure, pumps and electric motors	To pump the accumulated runoff from the floodwater storage pond to the watercourse outside the protective bund
	Control and sensor equipment	To provide automatic operation of mechanical and electrical plant in pumping station
	Penstocks	To isolate pumping station from floodwater storage pond
	Ventilation plant	To ventilate pumping station
	Transformer room of power company	To provide power for operation of pumping station during emergencies
	Emergency power generator	To provide power back-up in case electricity supply from transformer room fails
	Underground fuel oil storage tank	To provide fuel backup for operation of emergency power generator
	Fire fighting equipment	To comply with Fire Services Department’s regulations
	Flow measurement device	To measure and record the flow rate of the pumped runoff
	Gravity outlet	To convey the floodwater to the nearest main drainage channel outside the Polder under gravity conditions
	Telemetry control/alarm system	To transmit plant fault and flood alarm signals to a central control centre which will be manned on 24 hours basis
	Security fence	To prevent unauthorized entry
	Amenity areas within station compound	Landscaping
	Video Surveillance system	To monitor the operation of pumping stations

Table 23 – Some of the Existing Polder and Floodwater Pumping Schemes in the NWNT and NENT

Location	**Village Catchment Area**	**Pond Area**	**Pump Capacity**
1. Chau Tau Tsuen	55,700 m^2	1,500 m^2	2 duty + 1 standby (@ 1,800 l/s)
2. Kiu Tau Wai	15,500 m^2	4,750 m^2	1 duty + 1 standby (@ 100 l/s)
3. Lo Uk Tsuen	21,500 m^2	2,150 m^2	1 duty + 1 standby (@ 280 l/s)
4. San Tin Villages	510,000 m^2	17,800 m^2	3 duty + 2 standby (@ 2,500 l/s)
5. Sha Po Tsuen	59,000 m^2	2,420 m^2	1 duty + 1 standby (@ 1,100 l/s)
6. Sheung Shui Tsuen	260,200 m^2	37,000 m^2	2 duty + 1 standby (@ 2,500 l/s)
7. Sik Kong Tsuen	42,000 m^2	2,140 m^2	2 duty + 1 standby (@ 425 l/s)
8. Sik Kong Wai	55,000 m^2	2,800 m^2	2 duty + 1 standby (@ 550 l/s)

Table 24 – Schedule of Maintenance Responsibilities for
a Completed Polder and Floodwater Pumping Scheme

Below is an example for reference only. Project engineers should liaise with the relevant authorities to identify the maintenance and management parties for their own works in the design stage.

Completed Works	Maintenance Department	Management Department
Flood Protection embankment slopes and associated retaining walls, including marking plates of slope registration numbers	O&M Division /DSD	O&M Division /DSD
Internal drains, flow control devices and associated hydraulic structures	O&M Division /DSD	O&M Division /DSD
Public roads (including footpath, road drain, road side verges and slopes) to HyD's standard, and associated street furniture including lamp posts and cable draw pits	HyD	TD & HKPF
Road cleaning	FEHD	--
Roadside landscape softworks	LCSD	LCSD
Off-roadside landscape softworks in plantation area and landscape softworks inside verges and slope of border road	AFCD	AFCD
Floodwater storage pond	O&M Division /DSD	O&M Division /DSD
Pumps, pump controlling devices, monitoring and control systems and other E&M devices	ST Division /DSD	ST Division /DSD
Pumping station structure	Building & Civil Maintenance Team /DSD	ST Division /DSD

Table 25 - Inspection Frequency for Typical Stormwater Drains

Component	Frequency
Trunk Stormwater drains, major box-culverts and nullahs	Annually before wet season and every time after a heavy rainfall
Complaint black-spots	1 month to 6 months, depending on location
Stormwater drains and foul sewers within red routes, pink routes and expressways	Annually
Silt trap, surface water intake	Annually before wet season and every time after severe rainstorm
Sewer and drains within a distance of 4H from crest of slope or retaining wall (H = vertical height)	At least once every 5 years
Other stormwater drains	1-5 years

Table 26 - Schedule of Inspection for Polder and Floodwater Pumping Schemes

Facilities/Objectives	**Duties**	**Frequency**
a) gravity inlets to floodwater storage pond b) gravity inlets to and outlet from pumping station	Check and take necessary actions for siltation, trapped debris at trash screen and pipe blockage	Once every two weeks Monthly
Remove obstacles in floodwater storage ponds and drainage paths	Check and take necessary actions for illegal dumping, unauthorized occupation and weed growth in pond area Check and take necessary actions for siltation at pond area and peripheral channels	Monthly Monthly
Pumps	Check for satisfactory operation of pumps, both automatic and manual operations Check electric motors, pump bearing, belt drive tension, gearbox, lubricant, control and sensor equipment, flow measurement devices Test run the system	Once every two weeks Once every two weeks Once every two weeks
Power supply and Generator	Check emergency power generator, battery condition, fuel level (arrange re-filling of fuel if required), lubricant, engine cooling water Test run the system	Once every two weeks Once every two weeks
Telemetry system	Check function of telemetry devices, telemetry alarm system Test run the system	Once every two weeks Once every two weeks
Penstocks, Flap gates and sluice gates at gravity outlets from floodwater storage pond	Check for satisfactory operation of flap gates Check for satisfactory functioning of penstocks, both automatic and manual operations, check lubricant Check siltation at silt trap and pipe blockage	Monthly Once every two weeks Once every two weeks
Repair polder embankments	Check for cracks, erosion, settlement, and unauthorized excavation of the embankment	Half-yearly
Structural repair to drains	Check structural integrity of peripheral channels, access ramps and stairs accesses	Yearly
Desilting and repair to flood storage ponds	Check structural integrity of pond area	Yearly
Other structural repairs	Check reinforced concrete structure, flume channel, chamber Check fencing and gates	Yearly Once every two weeks
Flood Pumping Station	Attend to machinery in operation during thunderstorm, flood warning, tropical cyclone or similar situations	As required

Table 27 – Recommended Design Parameters for Concrete and Steel Reinforcement

Parameter	Recommended Value
Concrete	
Compressive strength	Design equations based on cylinder strength (f_{ck}), with its equivalent cube strength ($f_{ck,cube}$) given in Table 3.1 of BSI (2004) or its latest version
Exposure condition	Box Culvert - XC4 (Stormwater box culvert in upstream) - XS3 (Concrete adjacent to the sea) Floodwater Pumping Station -XC3 (superstructure) -XC4 (substructure) Manhole (other than standard manholes in DSD standard drawings) - XC4 (away from coast) - XS1 (nearest to coast) Tunnel XC4
Concrete grade	$f_{ck,cube}$ = 40 MPa (XC3, XC4 and XS1) $f_{ck,cube}$ = 50 MPa (XS3)
Concrete cover	Box culvert XC4 : 35 mm XS3 : 60 mm Floodwater pumping station XC3 : 35 mm XC4 : 35 mm Manhole (other than standard manholes in DSD standard drawings) XC4 : 35 mm XS1 : 50 mm Tunnel XC4 : 35 mm
Design crack width	Section 7.3 of BSI (2006) or its latest version Tightness Class 1 : manhole, box culvert and pumping station Tightness Class 0 : superstructure of pumping station Tunnel : 0.25 mm (maximum permissible crack width)

Parameter	Recommended Value
Stress-strain curve	Figure 3.8 of BD (2013) or its latest version
Modulus of elasticity	Table 3.2 of BD (2013) or its latest version
Coefficient of thermal expansion	Section 3.1.9 of BD (2013) or its latest version
Drying shrinkage	Section 3.1.8 of BD (2013) or its latest version
Creep	Section 3.1.7 of BD (2013) or its latest version
Steel Reinforcement	
Yield strength	500 MPa
Modulus of elasticity	200 GPa

Table 28 – Rainfall Increase and Sea Level Rise due to Climate Change

	Rainfall Increase	Sea Level Rise (m)
Mid 21st Century (2041 – 2060)	10.4%	0.23
End of 21st Century (2081 – 2100)	13.8%	0.49

LIST OF FIGURES

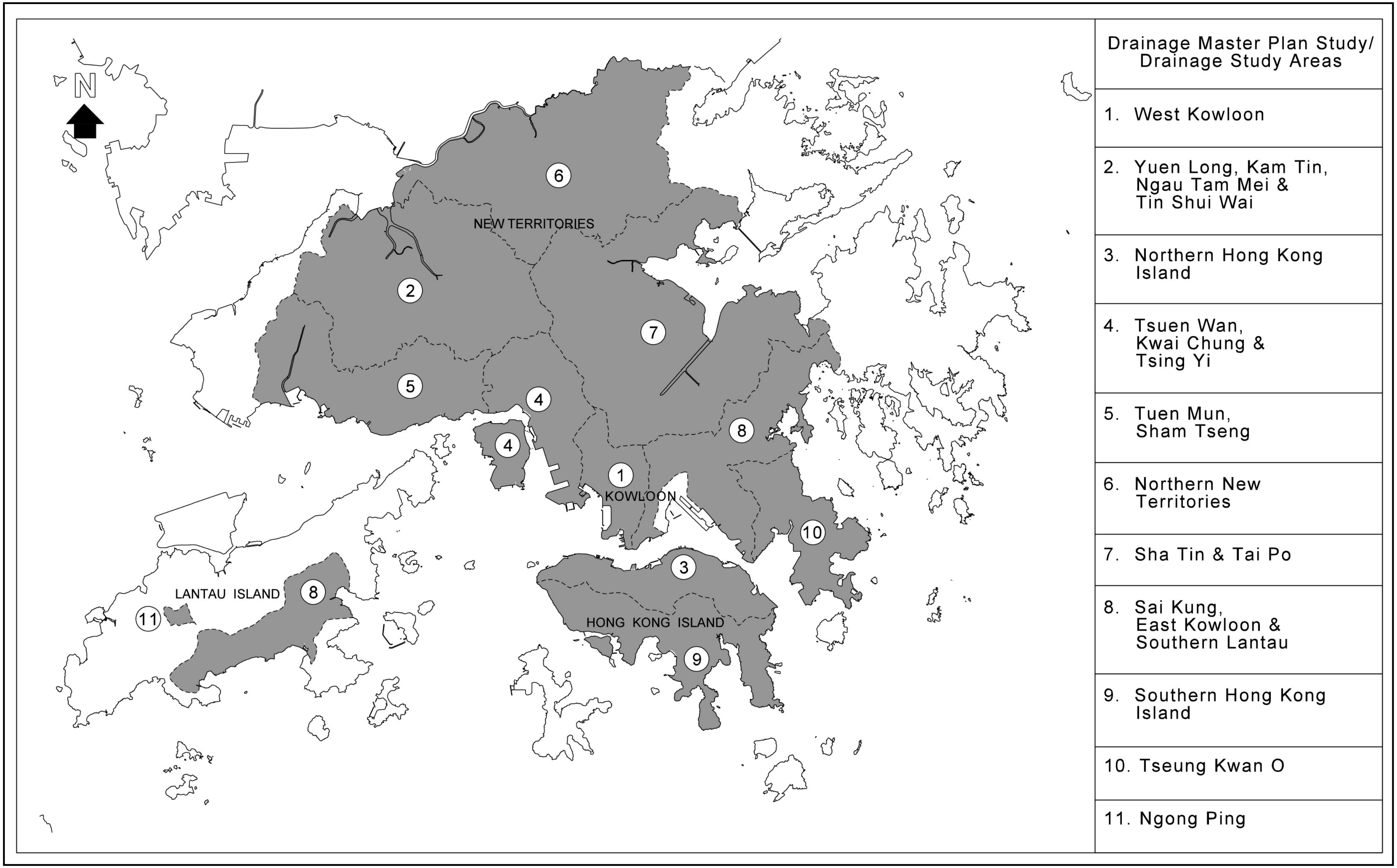


Figure 1. Drainage Master Plan Study / Drainage Study Areas in Hong Kong

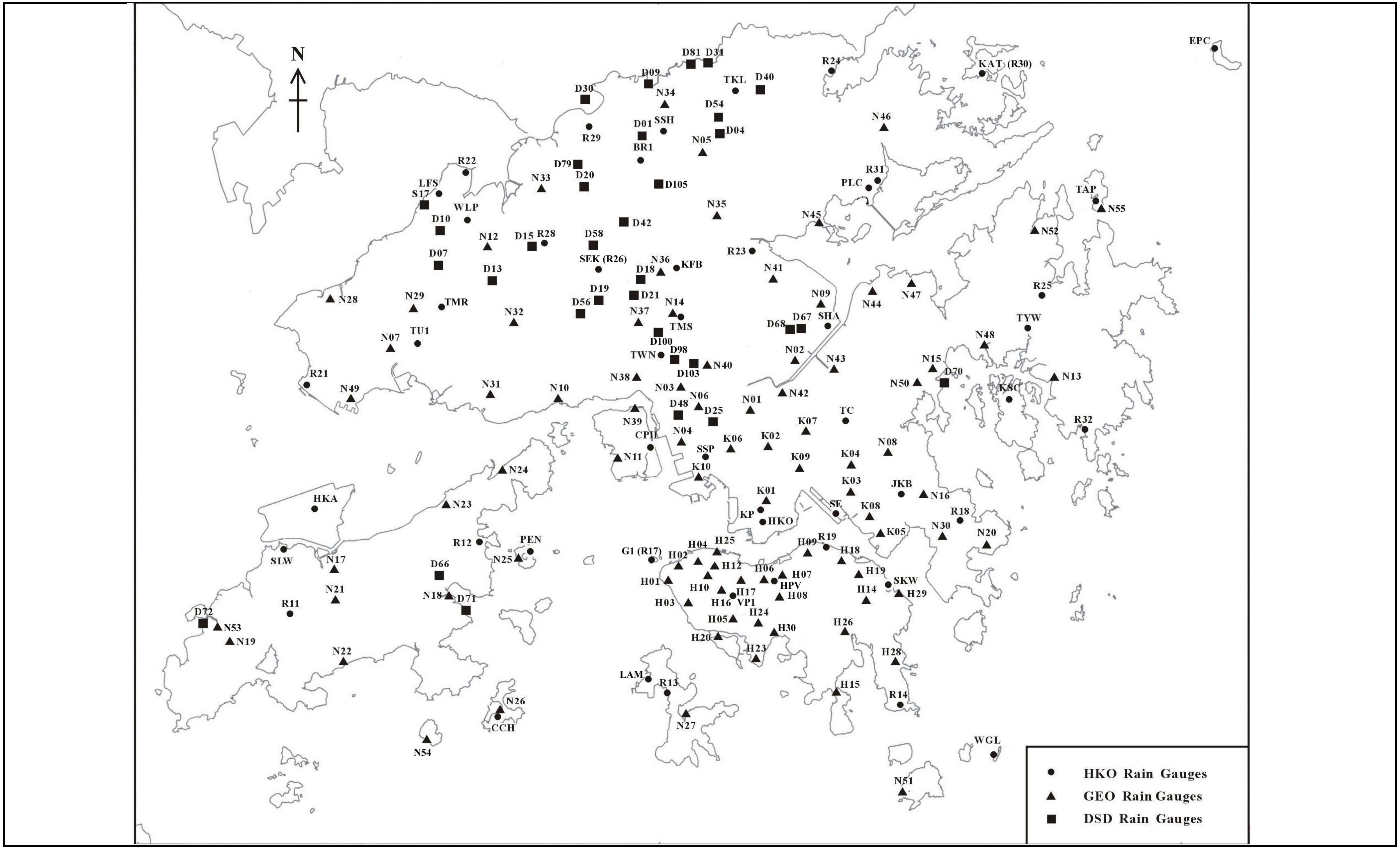


Figure 2a. Locations of Automatic Reporting (Telemetered) Rain Gauges

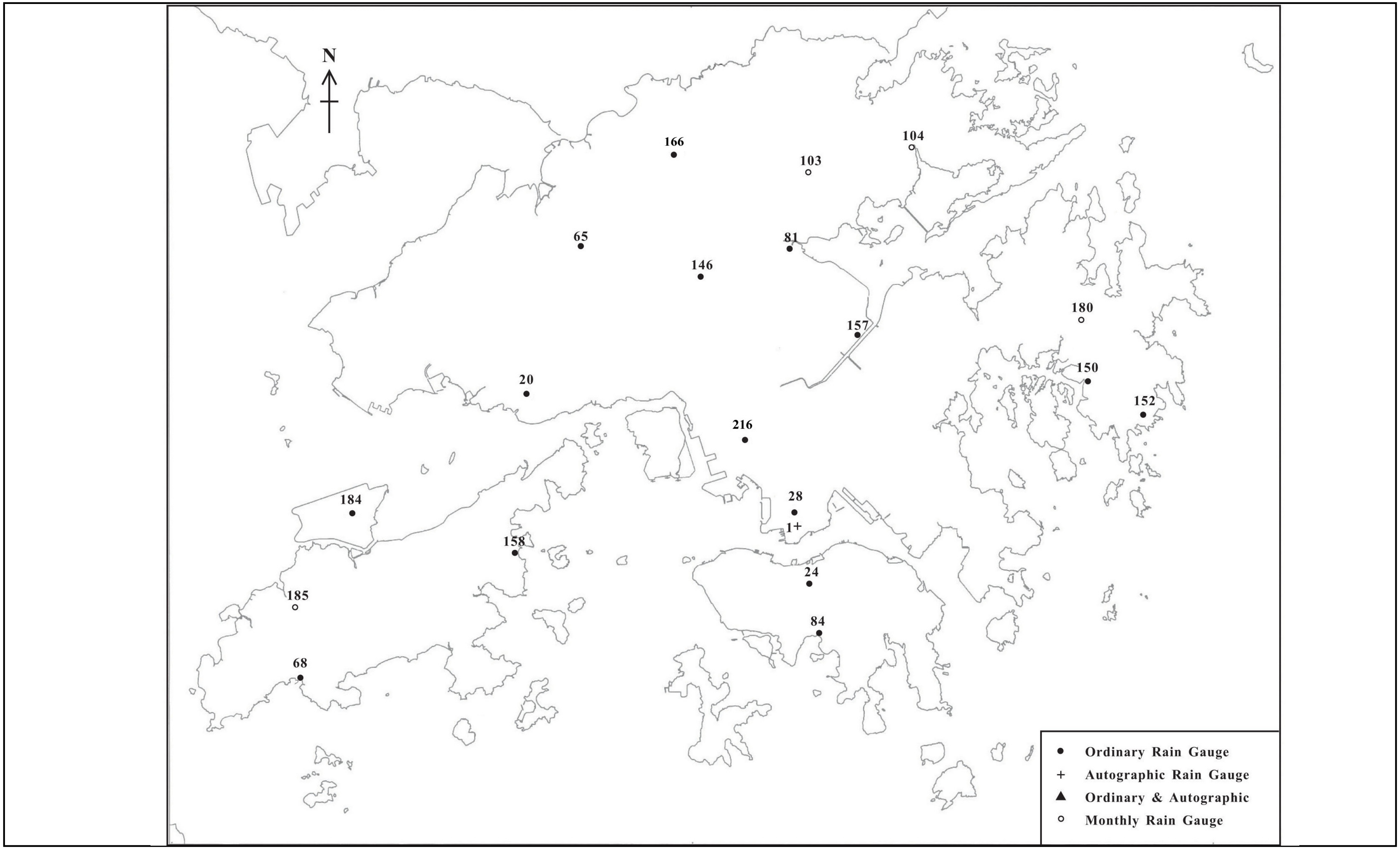


Figure 2b. Locations of HKO Conventional Rain Gauge Stations

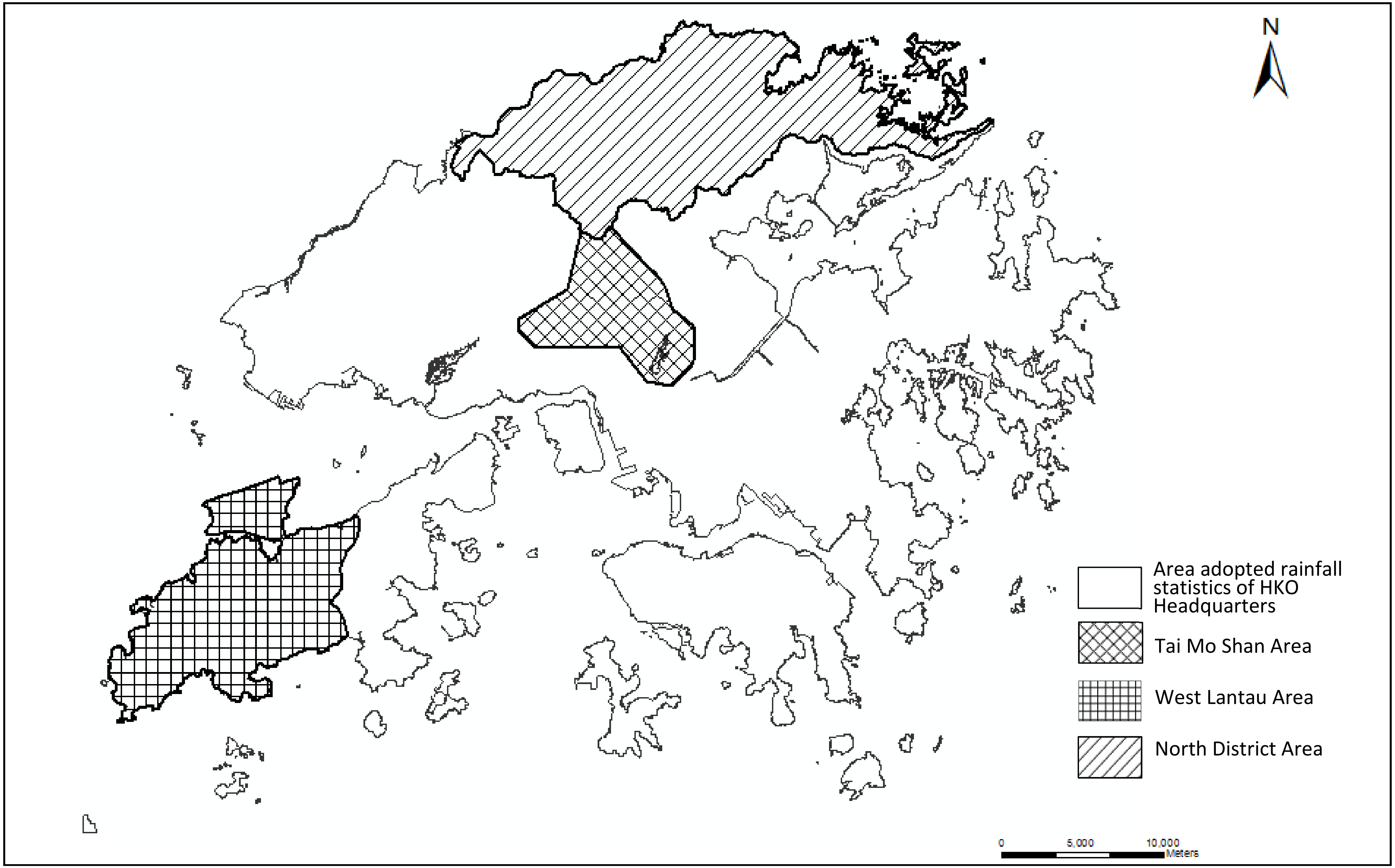


Figure 3 Delineation of Rainfall Zones

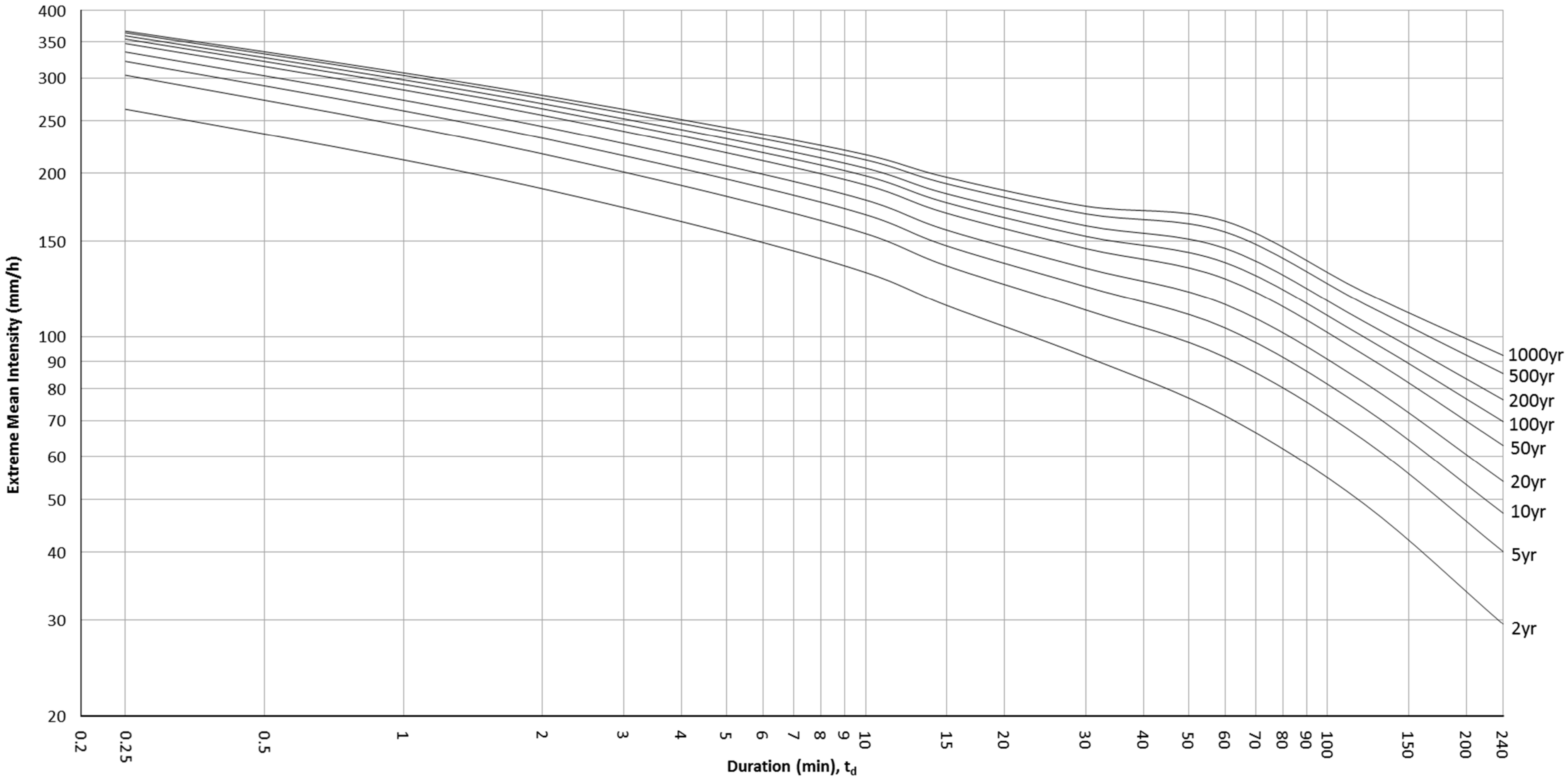


Figure 4a – Intensity-Duration-Frequency Curves of HKO Headquarters
(for durations not exceeding 4 hours)

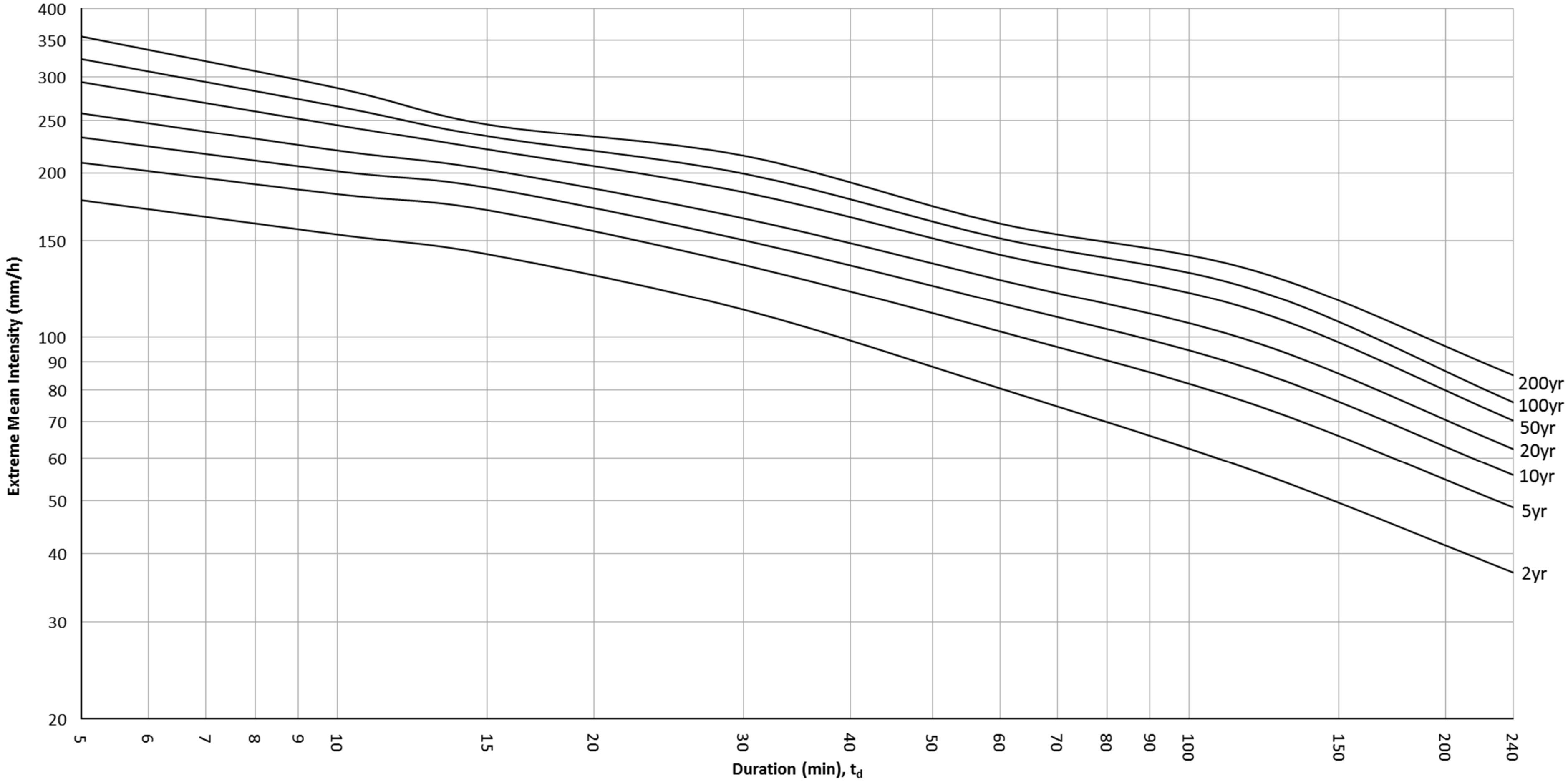


Figure 4b – Intensity-Duration-Frequency Curves of Tai Mo Shan Area
(for durations not exceeding 4 hours)

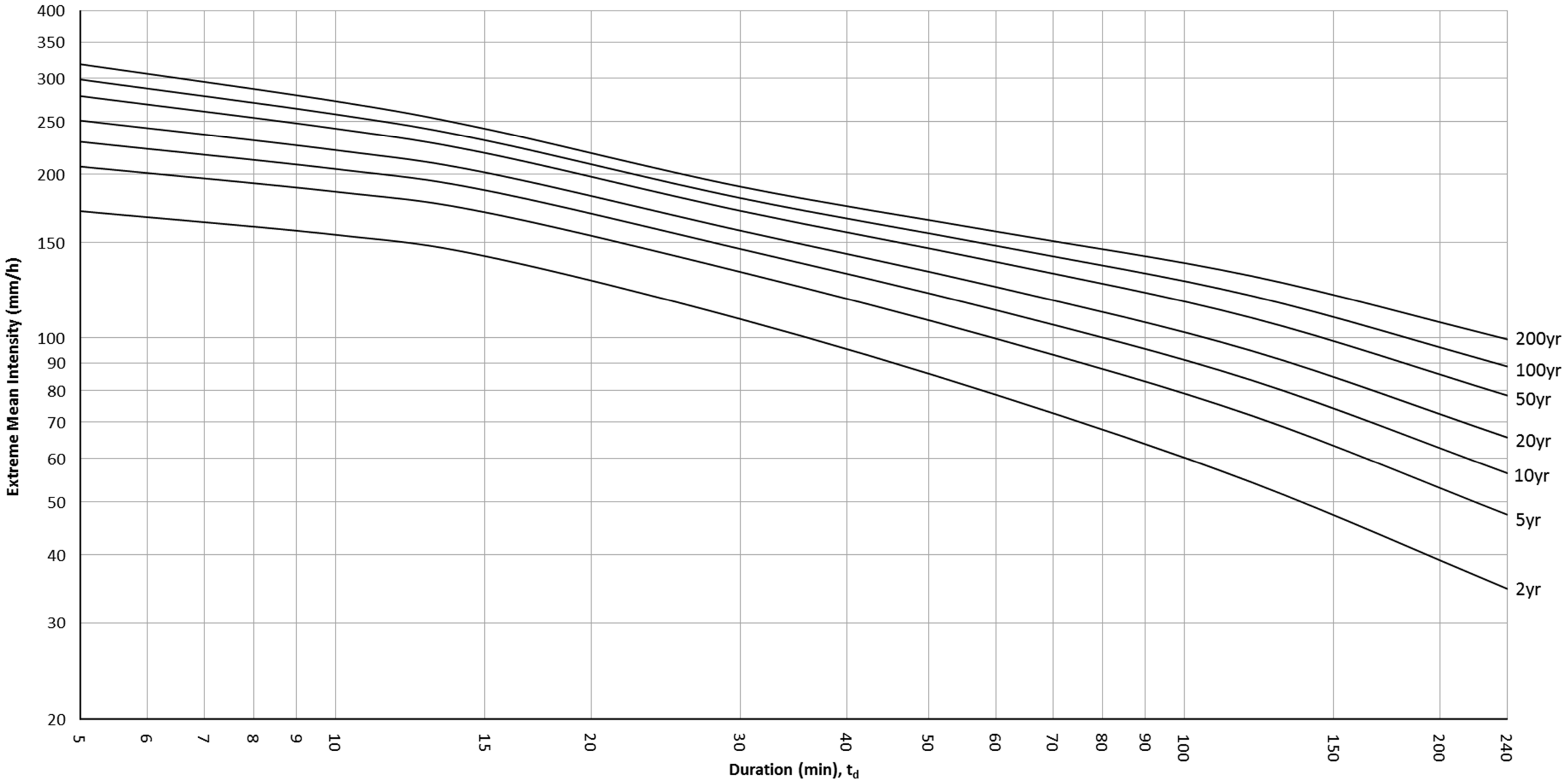


Figure 4c – Intensity-Duration-Frequency Curves of West Lantau Area
(for durations not exceeding 4 hours)

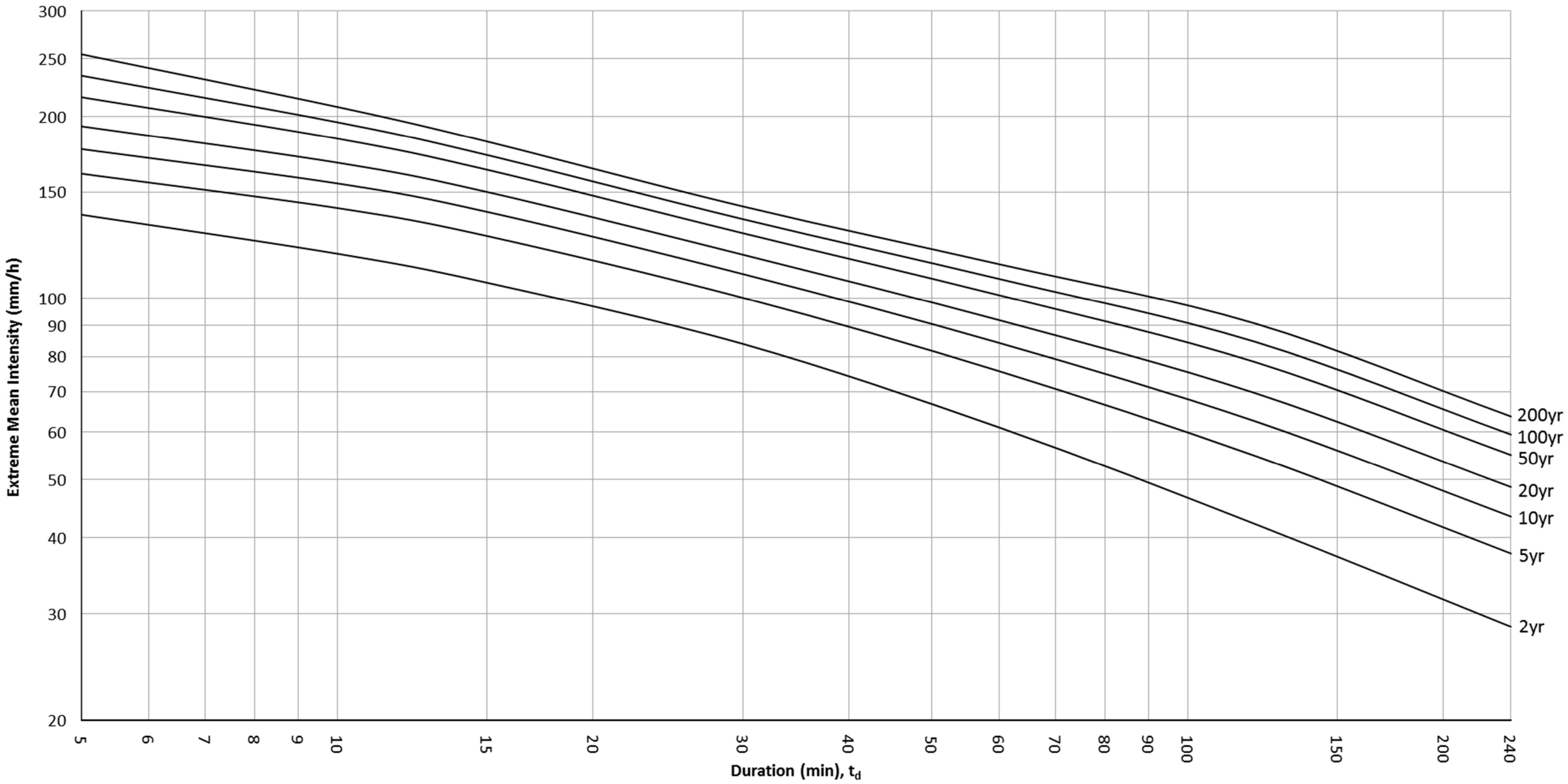


Figure 4d – Intensity-Duration-Frequency Curves of North District Area
(for durations not exceeding 4 hours)

Figure 5a – Synthetic Rainstorm Profiles of HKO Headquarters

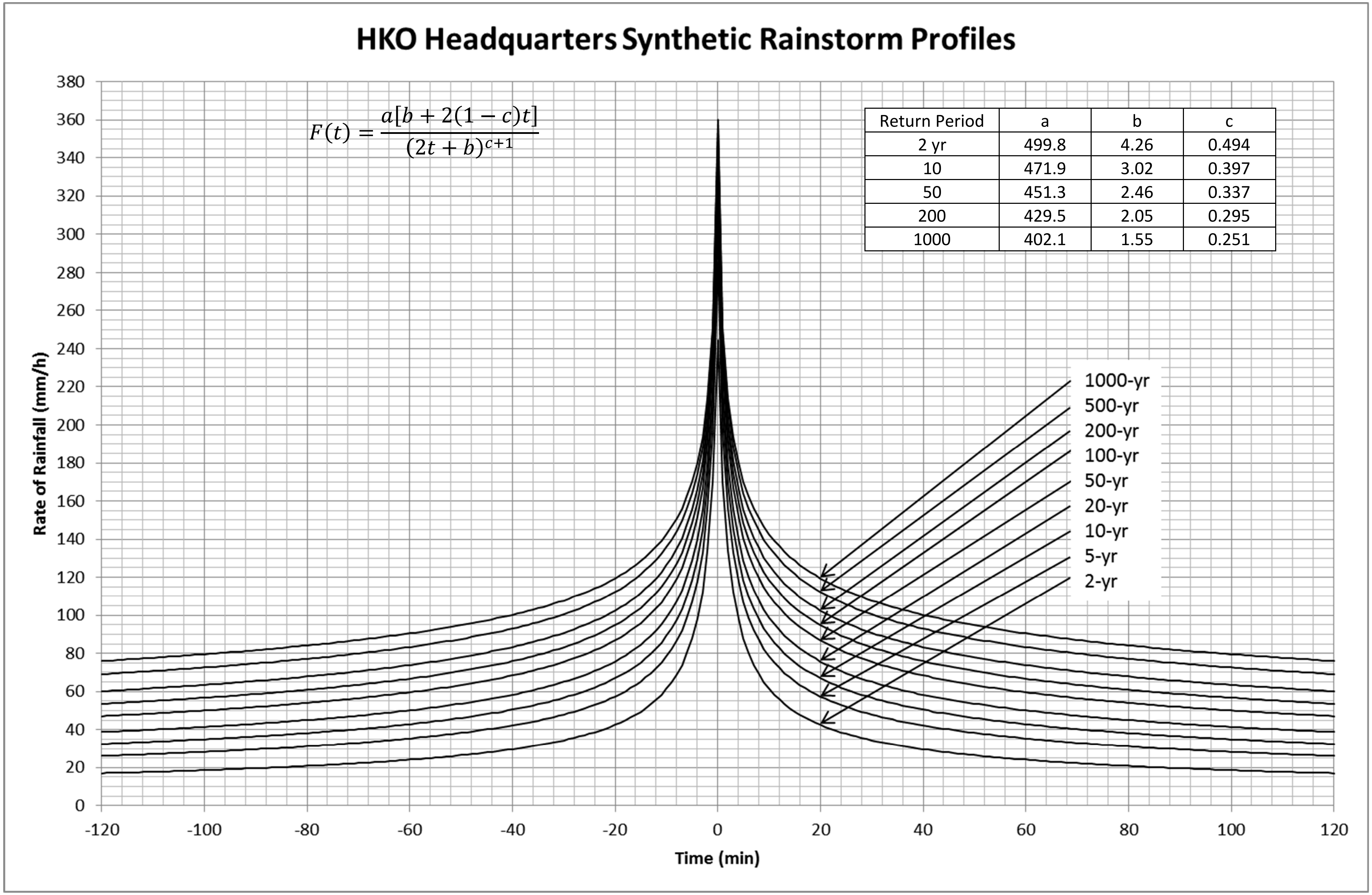
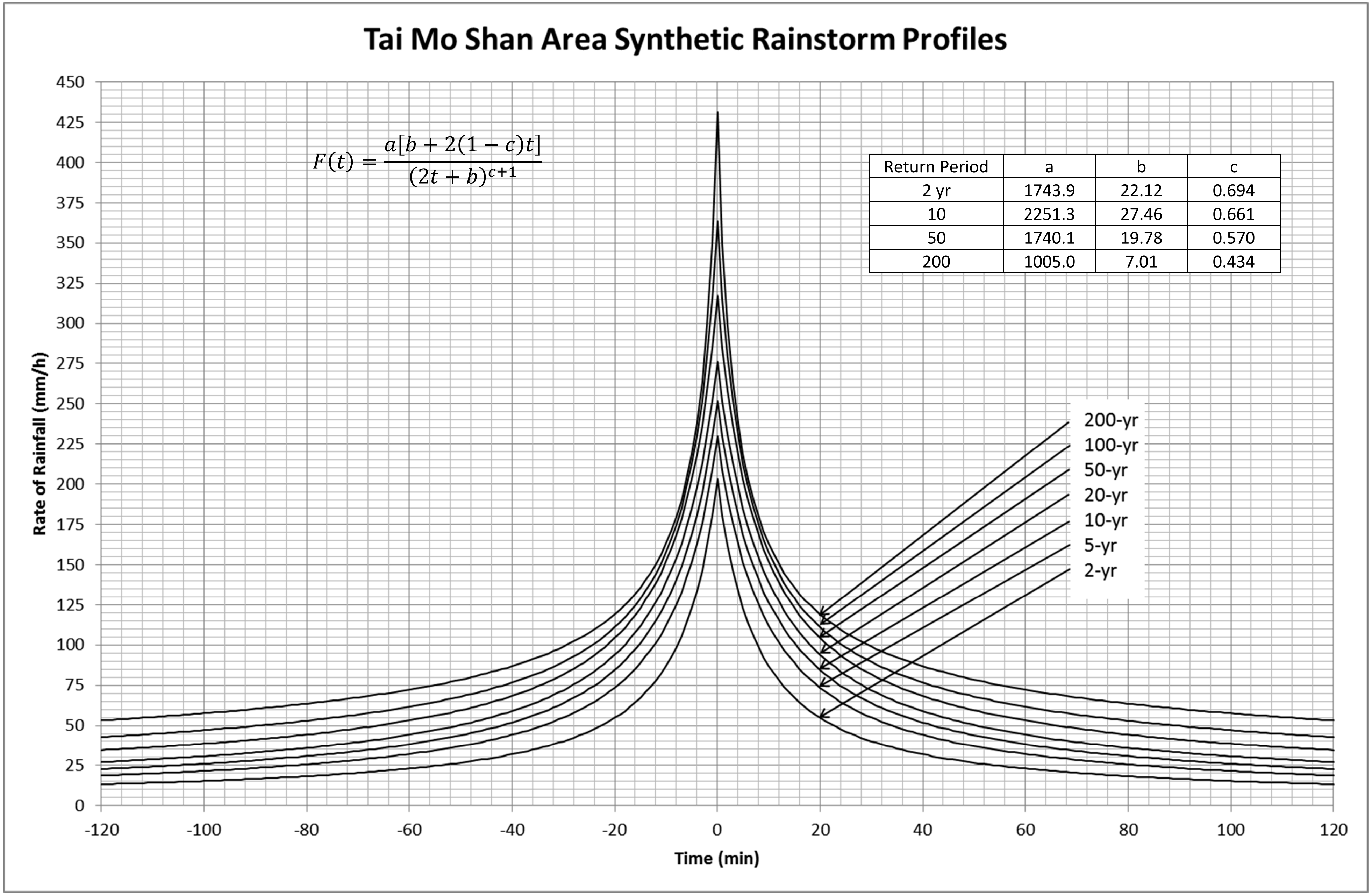


Figure 5b – Synthetic Rainstorm Profiles of Tai Mo Shan Area



$$F(t) = \frac{a[b + 2(1 - c)t]}{(2t + b)^{c+1}}$$

Return Period	a	b	c
2 yr	1743.9	22.12	0.694
10	2251.3	27.46	0.661
50	1740.1	19.78	0.570
200	1005.0	7.01	0.434

Figure 5c – Synthetic Rainstorm Profiles of West Lantau Area

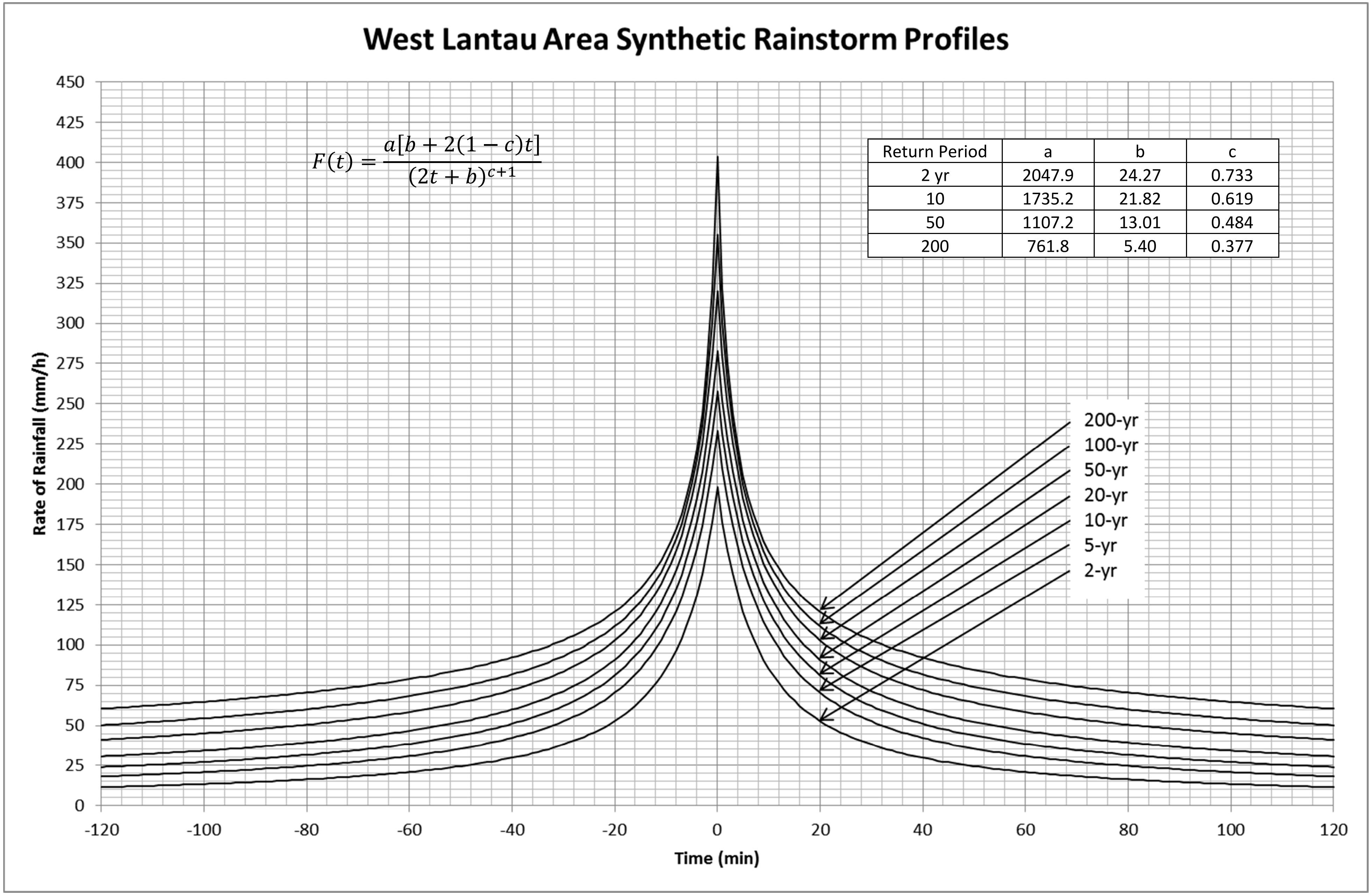
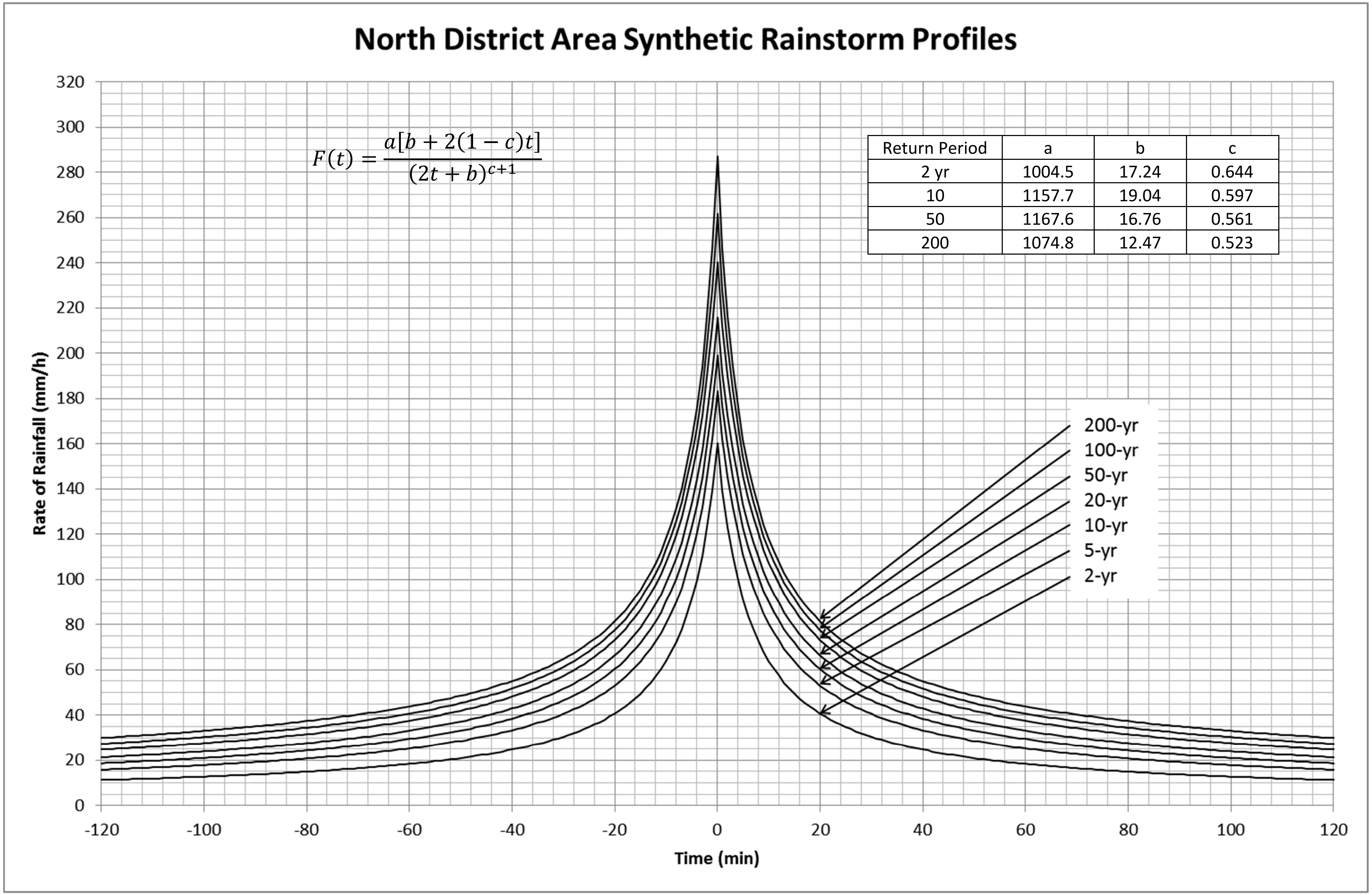
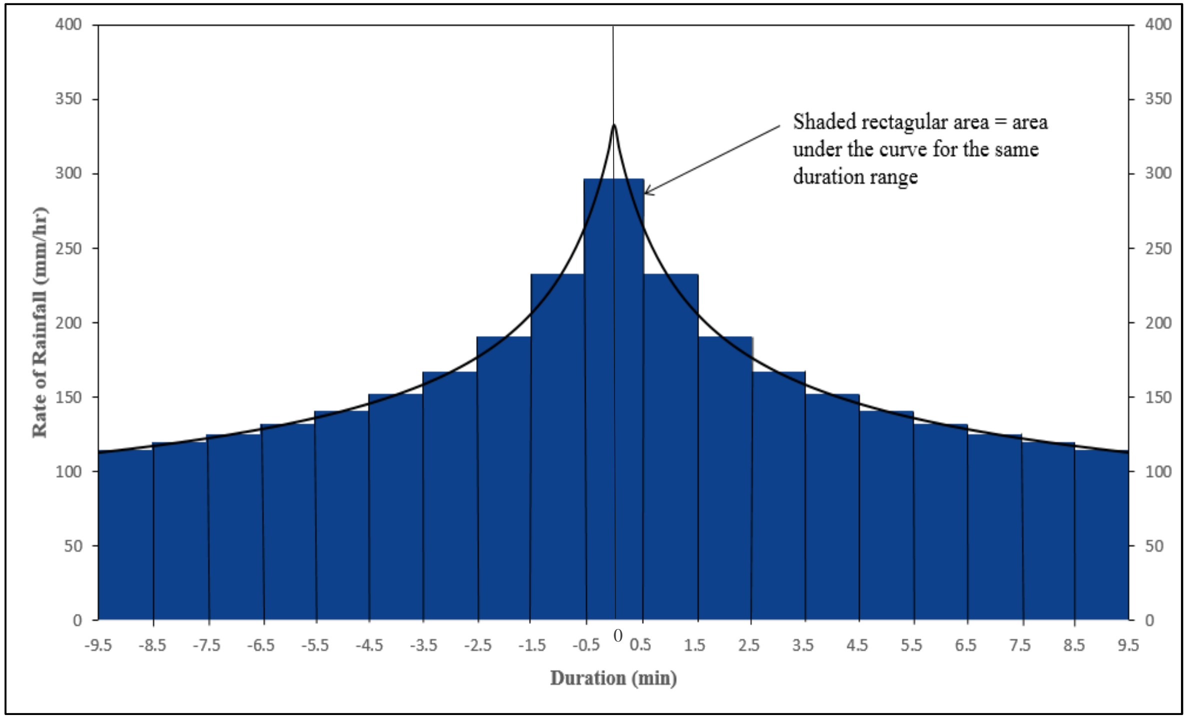


Figure 5d – Synthetic Rainstorm Profiles of North District Area





Example : 5 mins duration at return period = 50 years

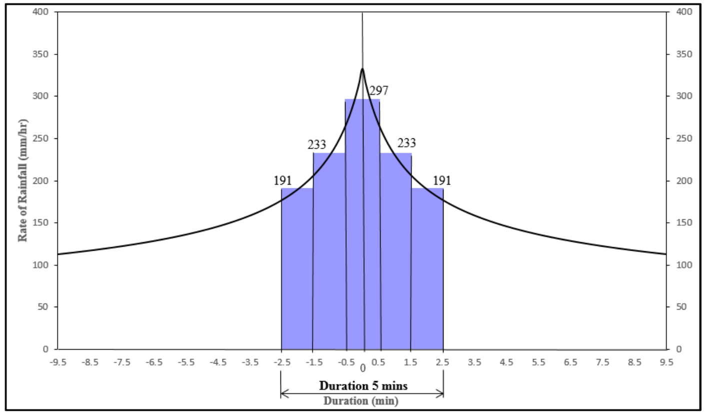


Figure 6. A Schematic Diagram showing a Design Storm Profile corresponding to the Step Function of the Same Return Period

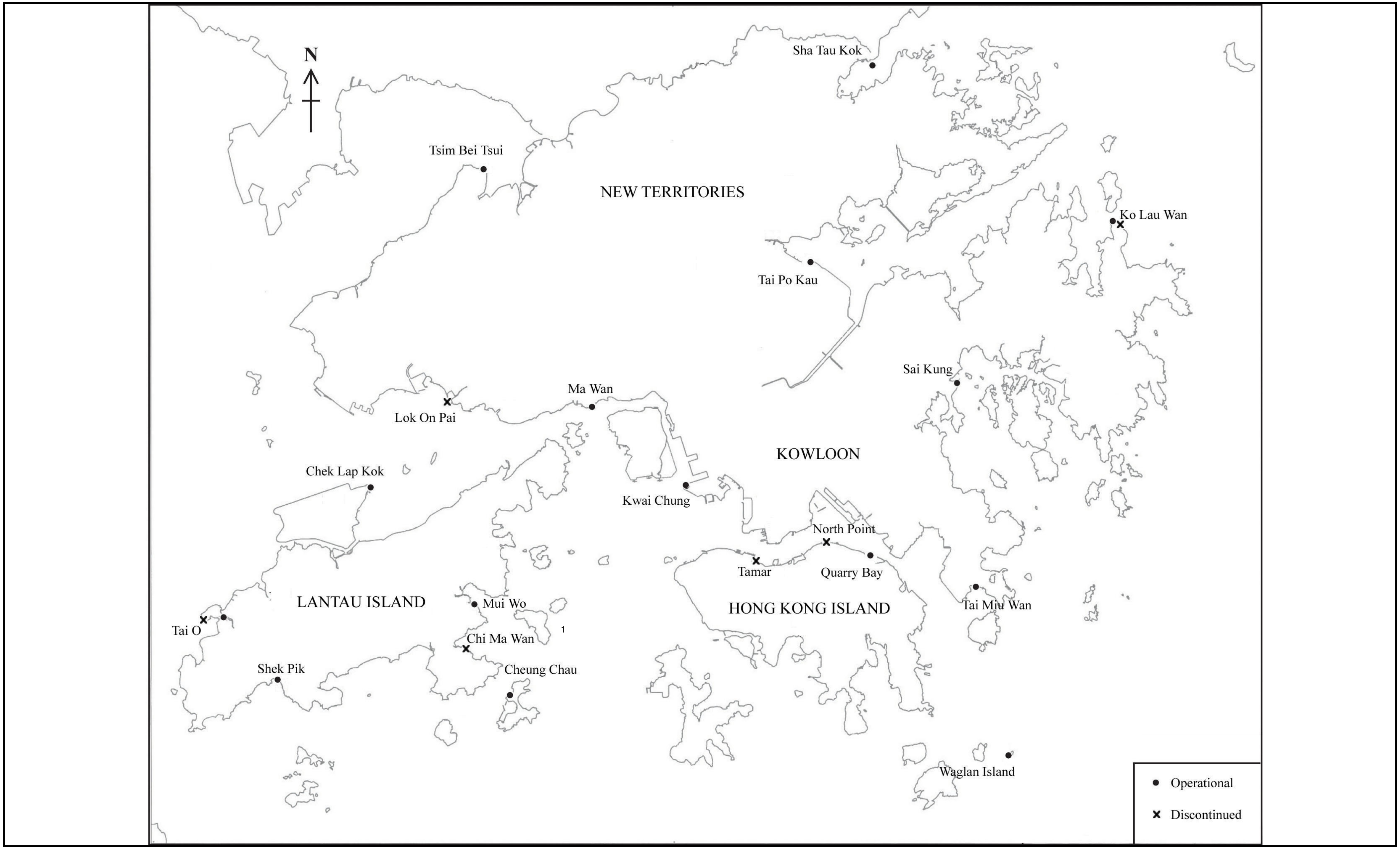


Figure 7. Locations of Tide Gauges

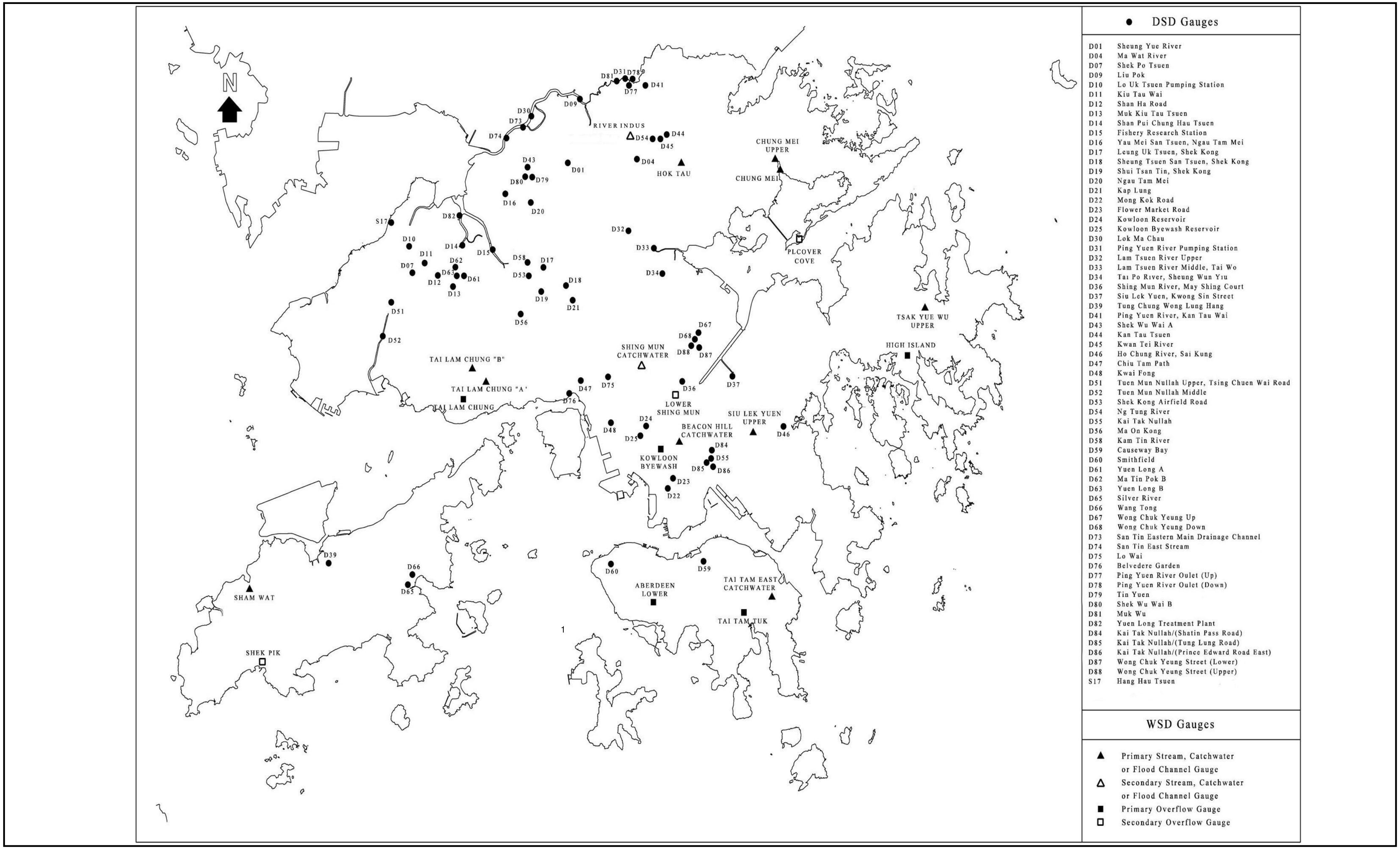


Figure 8. Locations of WSD Stream Flow Measurement Stations and DSD River Stage Gauges

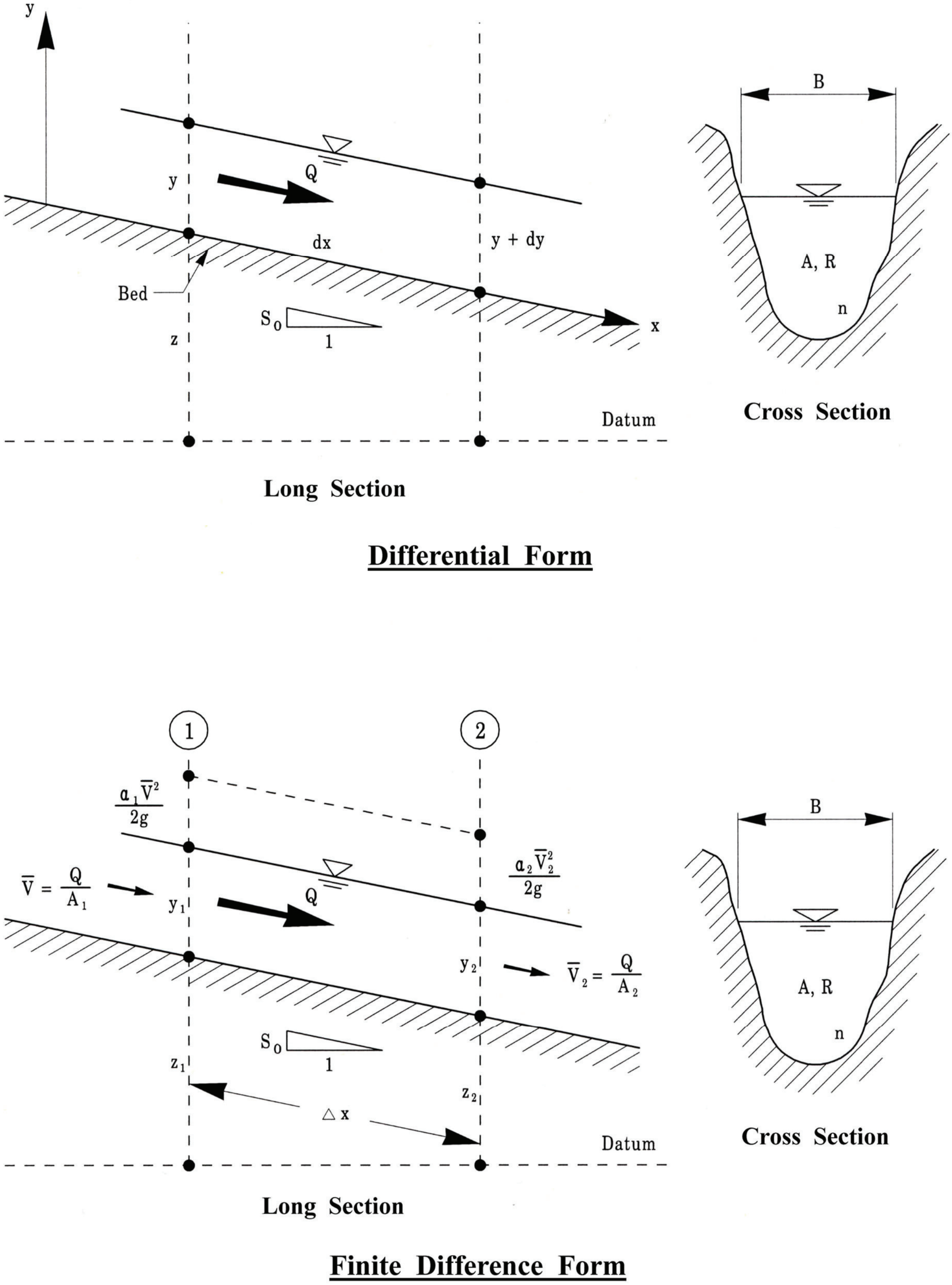


Figure 9. Definition Sketch for Non-uniform Flow

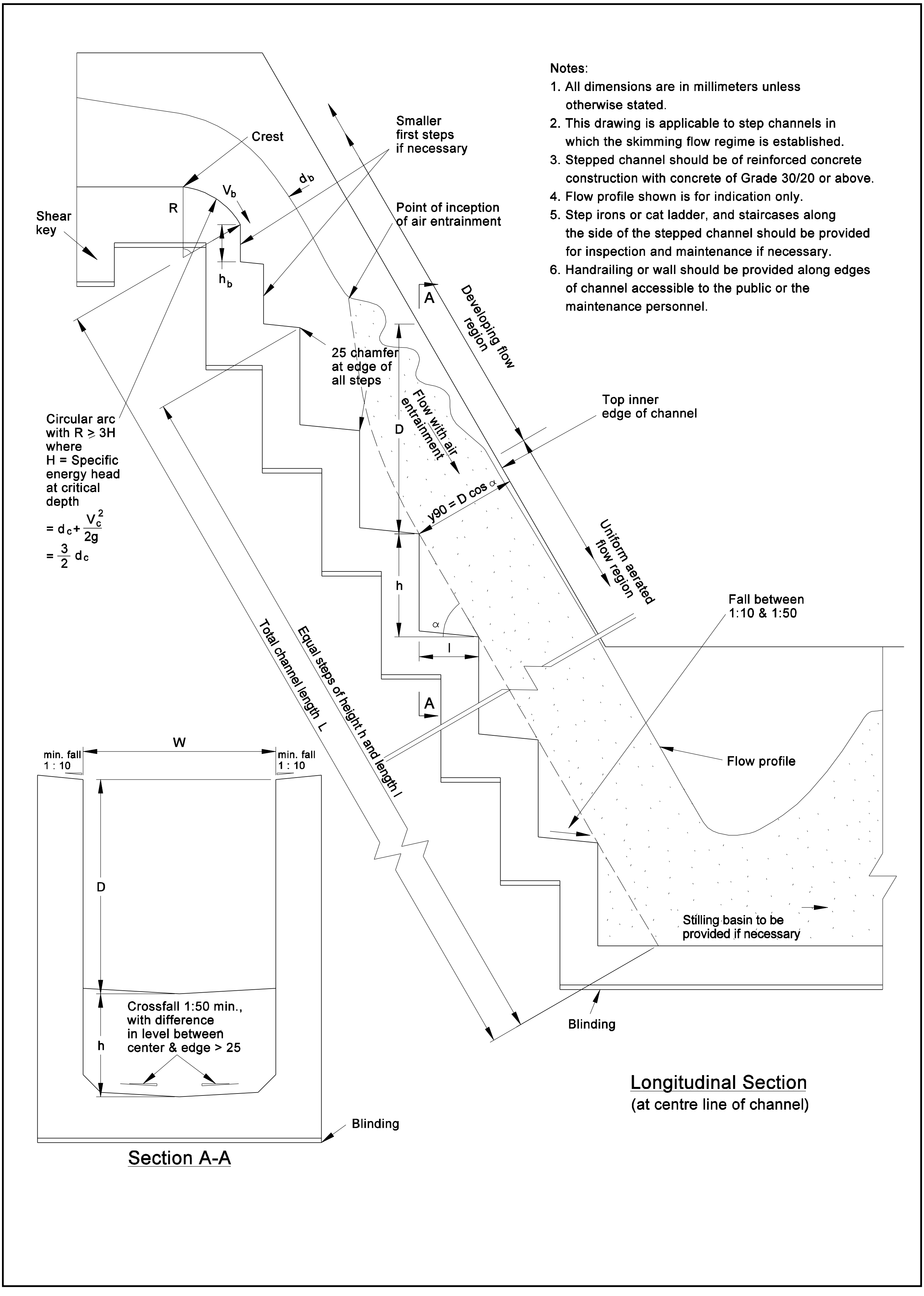


Figure 10. Stepped Channel

Type I Basin (for flow with $2.5 < Fr_1 < 4.5$)

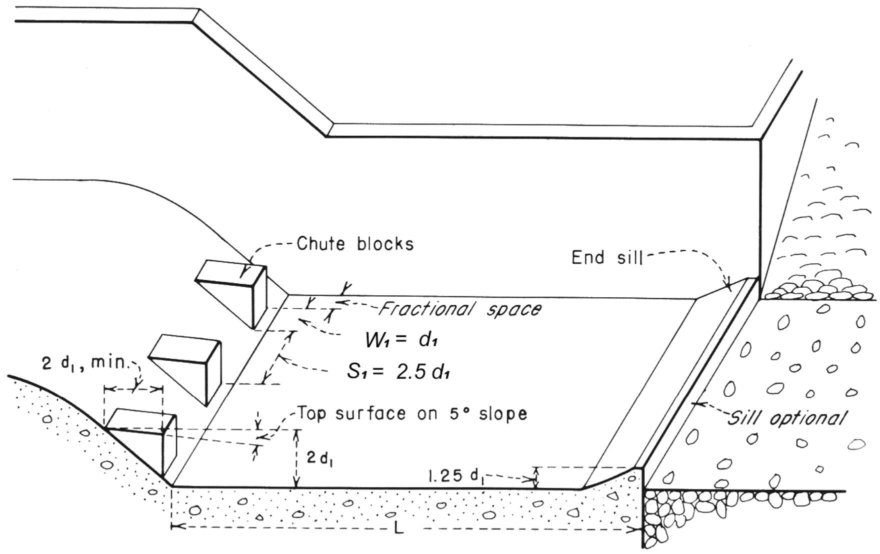


Table A

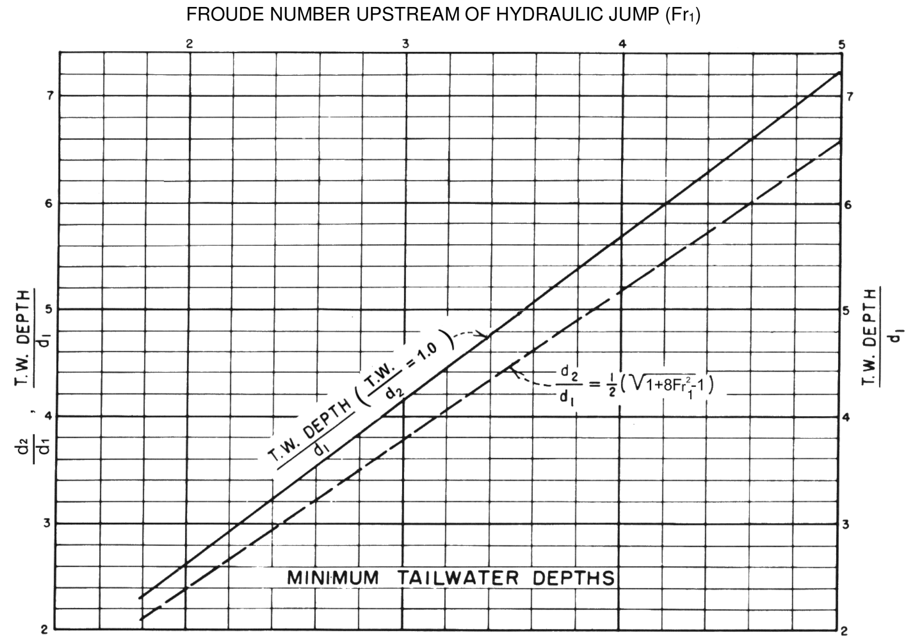
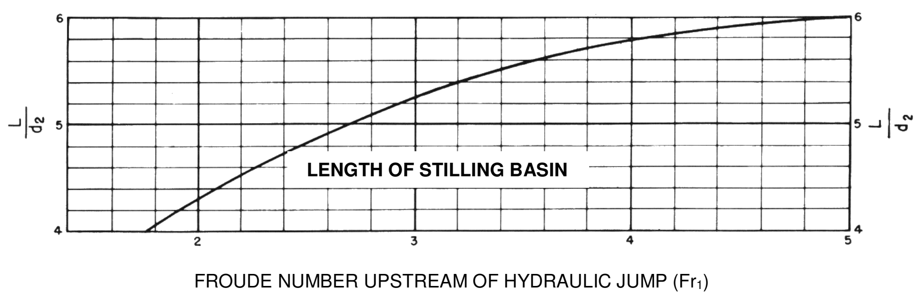


Table C



Notes :

1. See general notes, design procedures & guidelines in sheet 4.

Figure 11. Stilling Basins to Dissipate Energy by means of Hydraulic Jump (Sheet 1 of 4)

Type II Basin (for flow with $Fr_1 \geqslant 4.5$)

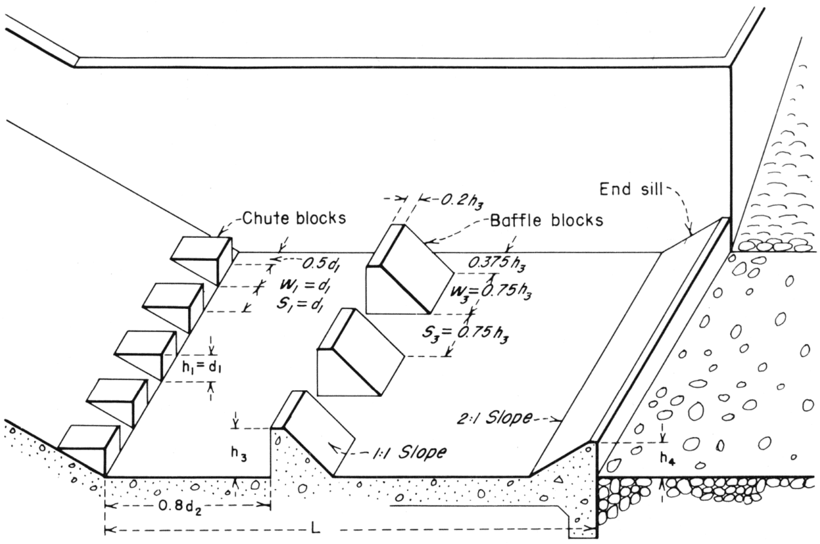


Table A

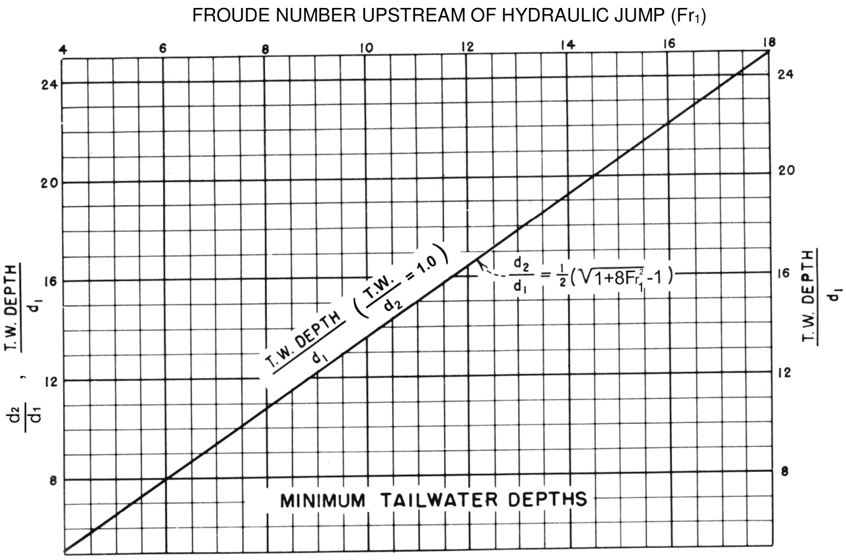


Table B

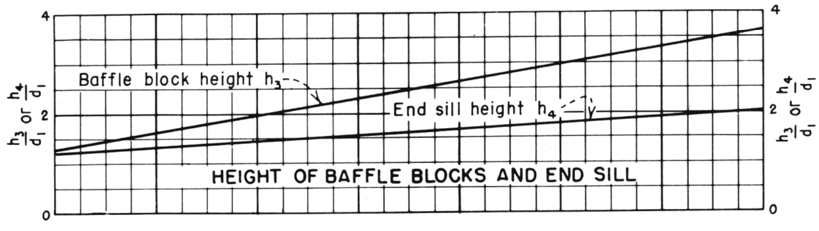
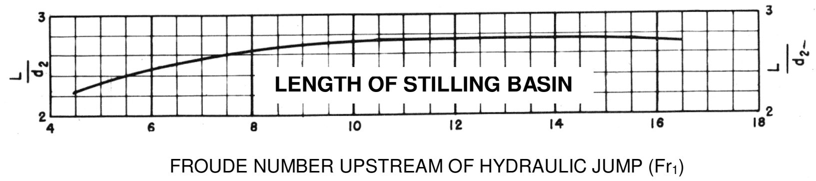


Table C

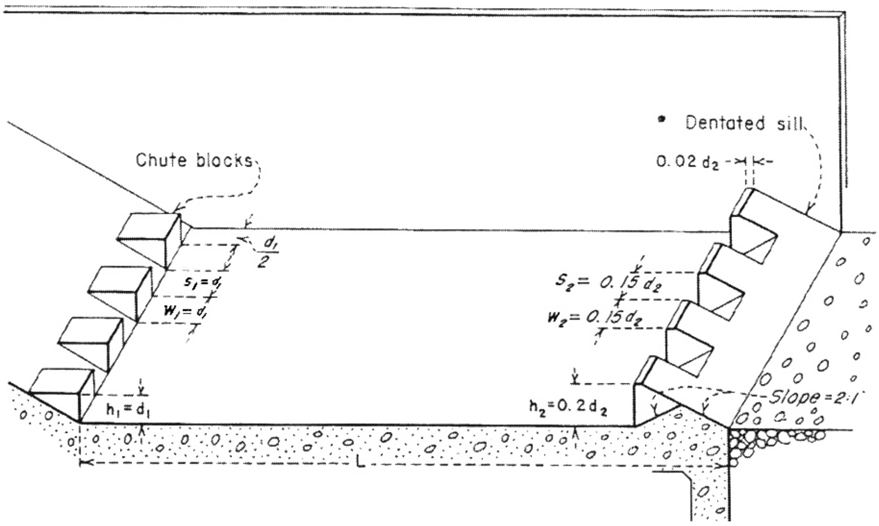


Notes :

1. See general notes, design procedures & guidelines in sheet 4
2. Type II basin is suitable for flow with upstream velocity v_1 not exceeding 15 m/s.

Figure 11. Stilling Basins to Dissipate Energy by means of Hydraulic Jump (Sheet 2 of 4)

Type III Basin (for flow with $Fr_1 \geqslant 4.5$)



FROUDE NUMBER UPSTREAM OF HYDRAULIC JUMP (Fr_1)

Table A

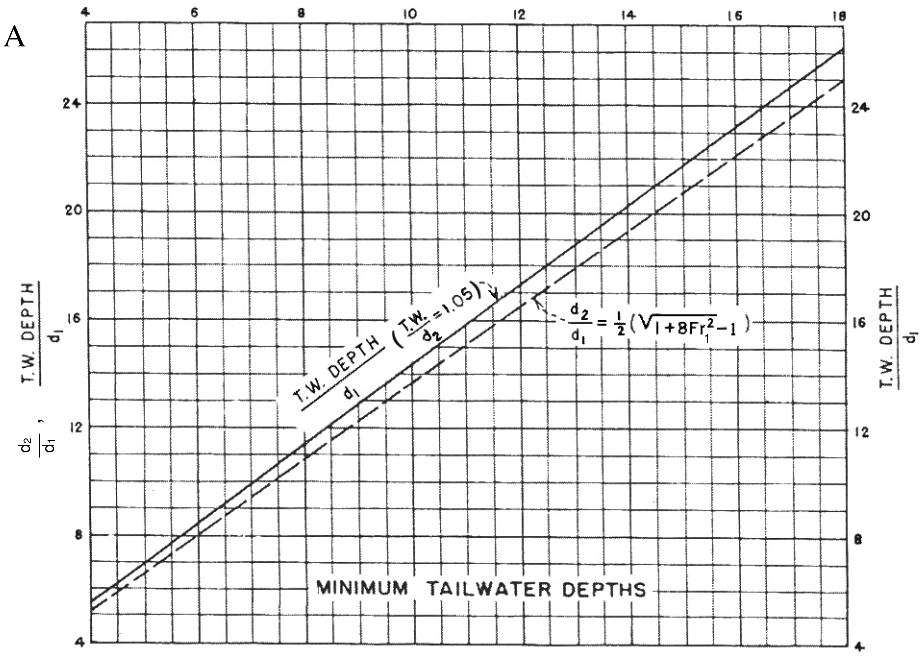
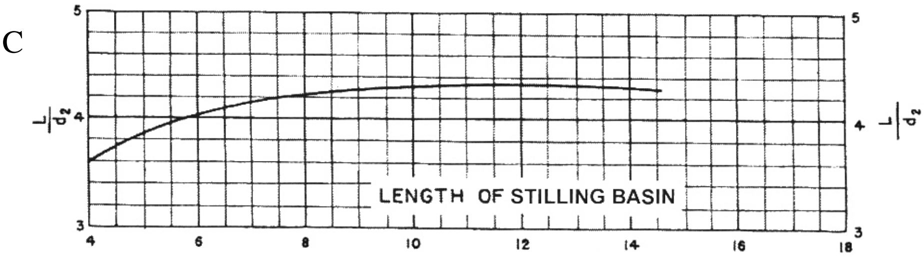


Table C



FROUDE NUMBER UPSTREAM OF HYDRAULIC JUMP (Fr_1)

Notes :

1. See general notes, design procedures & guidelines in sheet 4.
2. Type III basin is suitable for flow with upstream velocity v_1 of all values.

Figure 11. Stilling Basins to Dissipate Energy by means of Hydraulic Jump
(Sheet 3 of 4)

General notes, design procedures and guidelines :

1. The purpose of the stilling basin is to induce hydraulic jump when the slope of a channel changes from steep to gentle by means of chute blocks, baffle blocks and end sill.

   a) The chute blocks, baffle blocks and end sill also have a stabilizing effect on the jump such that the length can be shortened and the risk of sweep-out of the jump due to inadequate tailwater depth can be reduced.

   b) The end sill is also used to deflect any remaining turbulence near the channel bottom to the surface to prevent scouring; this is particularly useful if the downstream channel is lined with riprap or similar.

   c) The walls at the sides of the stilling basin should be vertical.

2. If the Froude no. of the upstream flow (Fr_1) is between 2.5 and 4.5, the hydraulic jump will be an oscillating jump with waves that persist downstream even if Type I stilling basin is used. Auxiliary wave dampeners/wave suppressors may need to be provided in such case. Alternatively, the dimensions of the channel should be revised to increase Fr_1 above 4.5.

3. If stilling basin is provided at the downstream end of a stepped channel, the last step of the stepped channel should be suitably modified to match the design of the chute blocks .

4. List of symbols :

d_1	flow depth upstream of hydraulic jump
v_1	velocity upstream of hydraulic jump
d_2	flow depth downstream of hydraulic jump
v_2	velocity downstream of hydraulic jump
Fr_1	Froude number of flow upstream of hydraulic jump
L	length of stilling basin
w_1	width of chute blocks
h_1	height of chute blocks
s_1	clear space between chute blocks
w_2	width of baffle blocks of dentated end sill
h_2	height of baffle blocks of dentated end sill
s_2	clear space between baffle blocks of dentated end sill
w_3	width of baffle blocks
h_3	height of baffle blocks
s_3	clear space between baffle blocks
h_4	height of end sill
g	acceleration due to gravity

5. Design procedures :

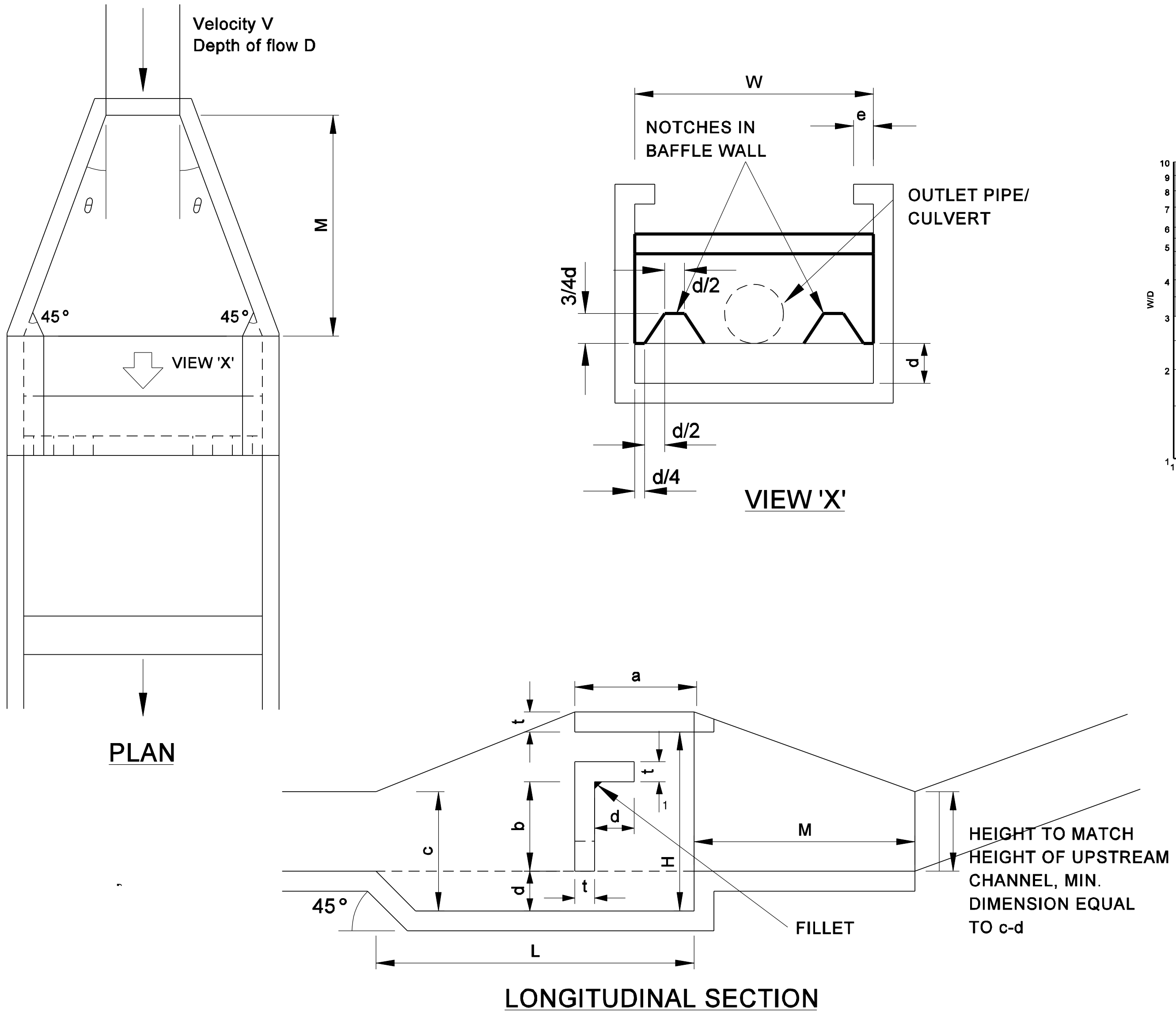
   a) Determine d_1 and v_1, the flow depth and velocity of the upstream flow, and calculate the Froude no. ($\mathrm{Fr_1} = \mathrm{v_1}/\sqrt{\mathrm{gd_1}}$). Choose the type of stilling basin according to the values of Fr_1 and v_1. If $Fr_1 < 2.5$, the hydraulic jump can be allowed to occur by itself provided that the length of the downstream channel (L) exceeds the length given in Table C of sheet 1.

   b) Use Table A and Fr_1 to find d_2/d_1 and "tailwater depth/d_1" (T.W./d_1, equal to d_2/d_1 multiplied by a factor). Calculate d_2 and the required minimum tailwater depth from these ratios. Check whether the tailwater depth exceeds the minimum. If not, a hydraulic control such as a weir should be introduced downstream.

   c) Find the dimensions of the chute block, baffle block and end sill from the figures. For Type II basin, find the ratios h_3/d_1 and h_4/d_1 from Table B of sheet 2, and determine h_3 and h_4.

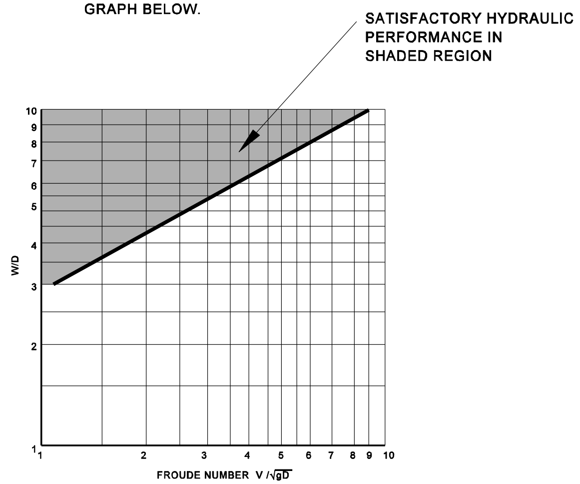
   d) Use Table C and Fr_1 to find L/d_2. Calculate L, the required minimum length of the stilling basin.

Figure 11. Stilling Basins to Dissipate Energy by means of Hydraulic Jump
(Sheet 4 of 4)



NOTES :

1. THE MINIMUM WIDTH OF THE STILLING BASIN (W) SHOULD BE FOUND FROM THE GRAPH BELOW.



THE OTHER DIMENSIONS SHOULD BE
H = 3/4 W, L = 4/3 W, a = 1/2 W,
b = 3/8 W, c = 1/2 W, d = 1/6 W,
e = 1/12 W, t = 1/12 W minimum or as structural design

2. THE CHANNEL IN LENGTH M SHOULD BE HORIZONTAL.
   M ⩾ 3 x WIDTH OF UPSTREAM CHANNEL
3. THIS DESIGN CAN HANDLE FLOWS UP TO $9 m^3/s$ AND VELOCITY UP TO 9m/s. FOR LARGER FLOWS, 2 OR MORE STILLING BASINS MAY BE USED ALONGSIDE EACH OTHER.
4. FOR BEST HYDRAULIC PERFORMANCE, THE TAIL WATER HEIGHT SHOULD APPROACH BUT NOT EXCEED THE MID-HEIGHT OF THE BAFFLE.
5. THE NOTCHES AT THE BAFFLE WALL ARE FOR THE PURPOSE OF PREVENTING DEPOSITION IN THE BASIN THROUGH FLUSHING ACTION OF THE EMITTING JET.
6. HANDRAILS AND LADDER/STEP IRONS SHOULD BE PROVIDED WHERE NECESSARY.

Figure 12. Impact Type Stilling Basin

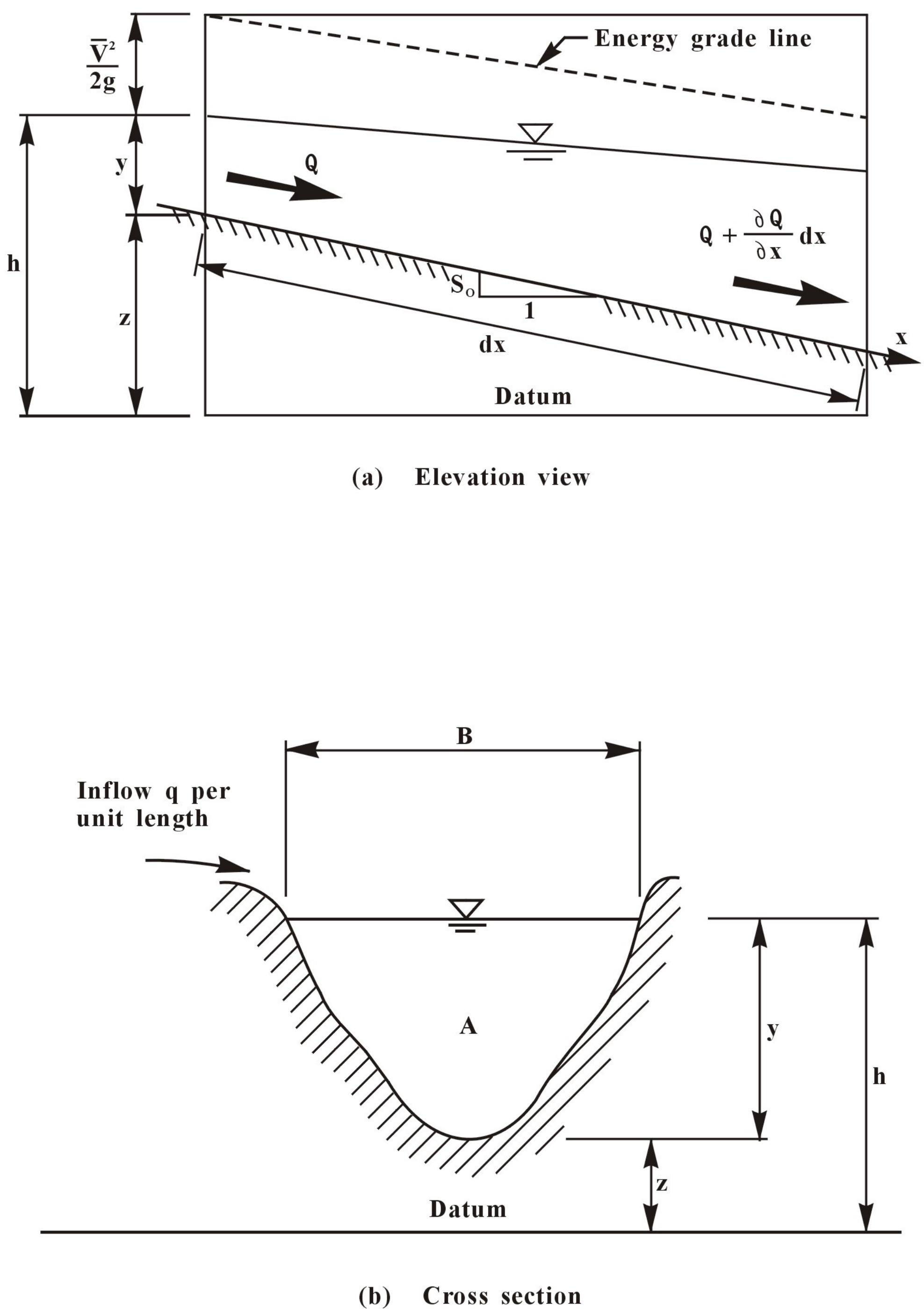
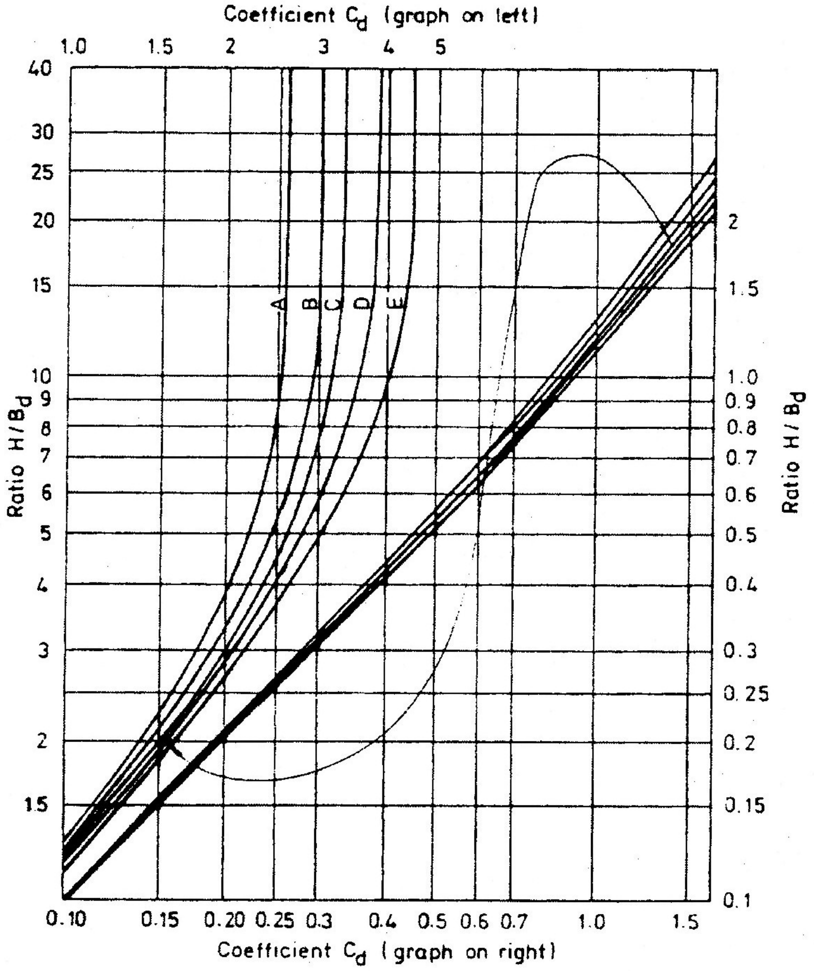
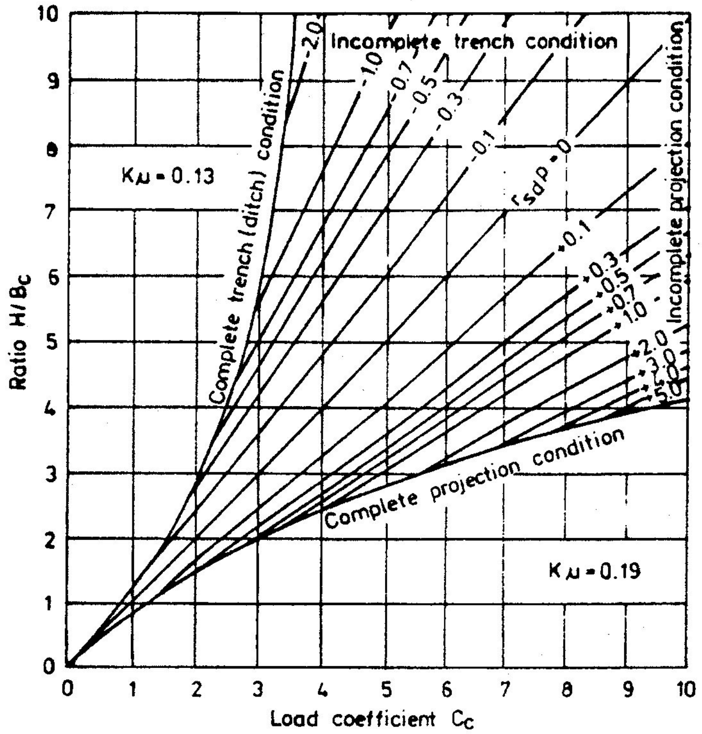


Figure 13. An Elemental Reach of Channel in the Saint Venant Equations



A - C_d for Kμ and Kμ'	=	0.19 for granular materials without cohesion
B - C_d for Kμ and Kμ'	=	0.165 max for sand and gravel
C - C_d for Kμ and Kμ'	=	0.150 max for saturated top soil
D - C_d' for Kμ and Kμ'	=	0.130 ordinary max for soil
E - C_d for Kμ and Kμ'	=	0.110 max for saturated clay

Figure 14. Values of Narrow Trench Coefficient C_d



To extrapolate values of C_c for higher values of H/B use the following equations

Condition	$r_{sd}p$	Equation of curve
Incomplete trench $K\mu = 0.13$	-0.1	$C_c = 0.82\,(H/B_c) + 0.05$
	-0.3	$C_c = 0.69\,(H/B_c) + 0.11$
	-0.5	$C_c = 0.61\,(H/B_c) + 0.20$
	-0.7	$C_c = 0.55\,(H/B_c) + 0.25$
	-1.0	$C_c = 0.47\,(H/B_c) + 0.40$
	-2.0	$C_c = 0.30\,(H/B_c) + 0.91$
Incomplete projection $K\mu = 0.19$	0	$C_c = H/B$
	+0.1	$C_c = 1.23\,(H/B_c) - 0.02$
	+0.3	$C_c = 1.39\,(H/B_c) - 0.05$
	+0.5	$C_c = 1.50\,(H/B_c) - 0.07$
	+0.7	$C_c = 1.59\,(H/B_c) - 0.09$
	+1.0	$C_c = 1.69\,(H/B_c) - 0.12$
	+2.0	$C_c = 1.93\,(H/B_c) - 0.17$
	+3.0	$C_c = 2.08\,(H/B_c) - 0.20$
	+4.0	$C_c = 2.19\,(H/B_c) - 0.21$
	+5.0	$C_c = 2.28\,(H/B_c) - 0.22$
Complete trench	use the formula $C_c = C_d = \dfrac{1 - \exp(-2K\mu\, H/B_c)}{2K\mu}$ and $K\mu = 0.13$	
Complete projection	use the formula $C_c = \dfrac{\exp(2K\mu\, H/B_c) - 1}{2K\mu}$ and $K\mu = 0.19$	

Figure 15. Values of Embankment Coefficient C_c

Figure 1
Class F
Bedding Factor 1.9

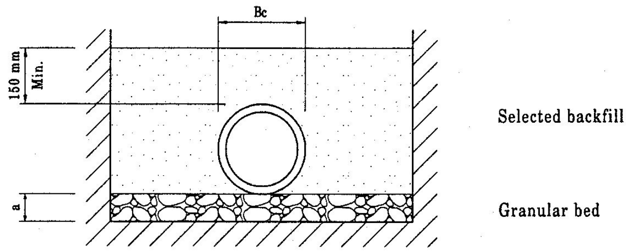


Figure 2
Class B
Bedding Factor 2.5

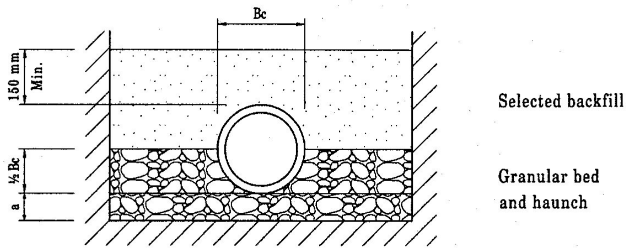
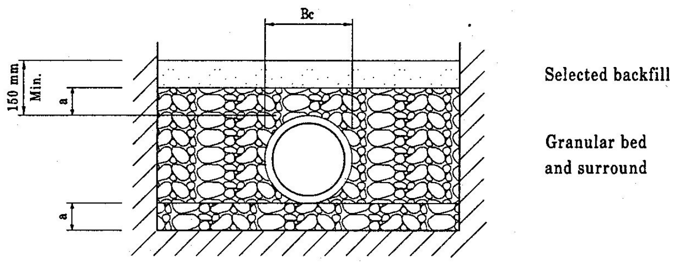


Figure 3
Class S
Bedding Factor 2.5



Normal bore of pipe (mm)	Size (mm)	
	singled - sized	graded
100 - 125	10	--
150 - 200	10 or 14	14 to 5
225 - 300	10, 14 or 20	14 to 5 or 20 to 5
375 - 500	14 or 20	14 to 5 or 20 to 5
exceeding 500	14, 20 or 40	14 to 5 20 to 5 or 40 to 5

All granular material to be sized or graded in accordance with the adjacent table. Aggregates to BS 882, sintered pulverized-fuel ash to BS 3797 and air-cooled blast furnace slags to BS 1047 are suitable.

Dimension a = For sleeve jointed pipes, a minimum of 50mm or 1/6 Bc, whichever is the greater. For socketed pipes a min. of 100mm or 1/6 Bc, whichever is the greater, under barrels and not less than 50 mm under sockets

In rock or material containing hard spots :

a = For sleeve jointed pipes, a minimum of 150mm or ¼ Bc, whichever is the greater. For socketed pipes a min. of 200mm or ¼ Bc, whichever is the greater, under barrels and not less than 150 mm under sockets

Bc = outside pipe diameter

Figure 16. Bedding Factors for Vitrified Clay Pipes

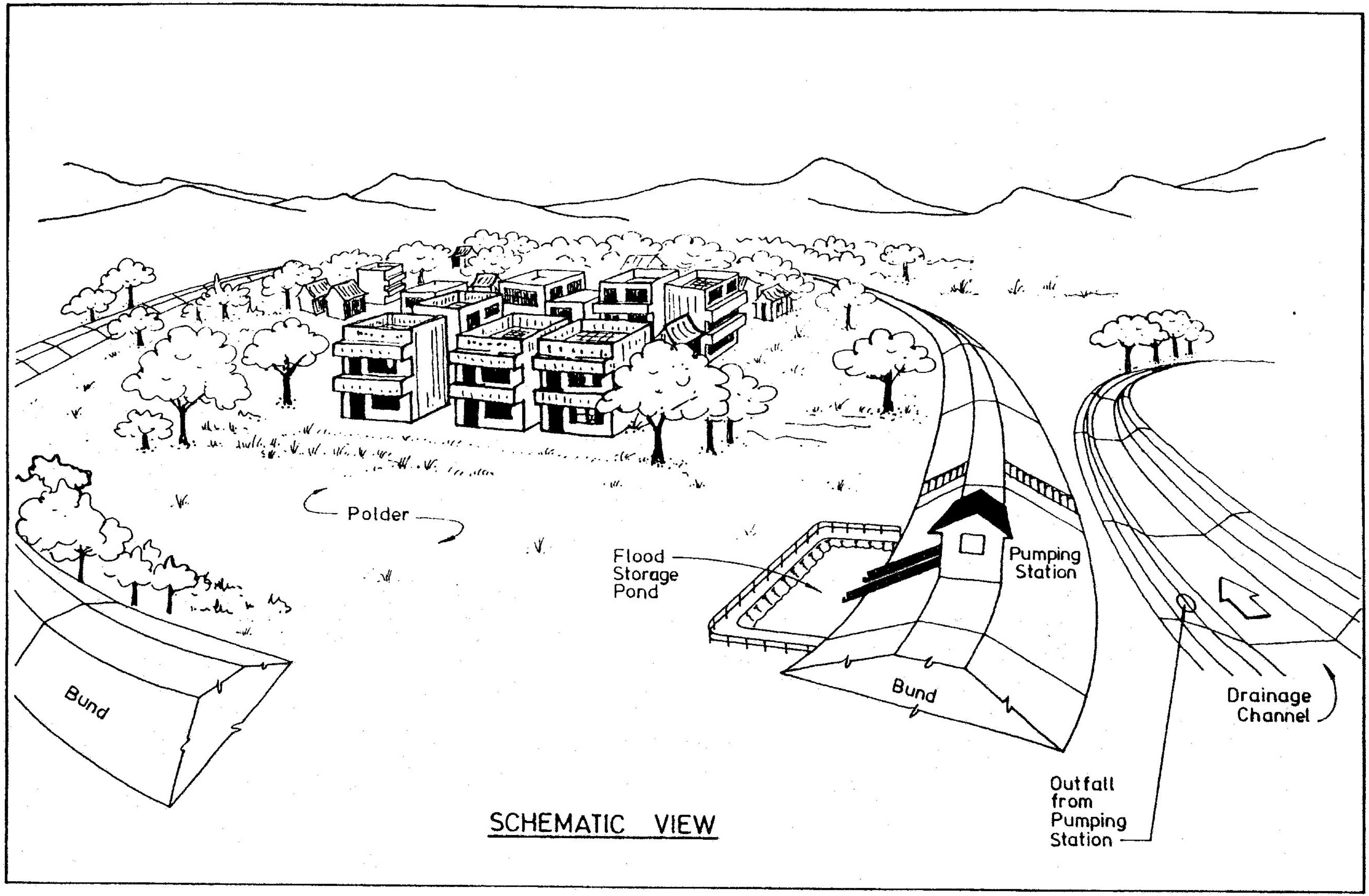
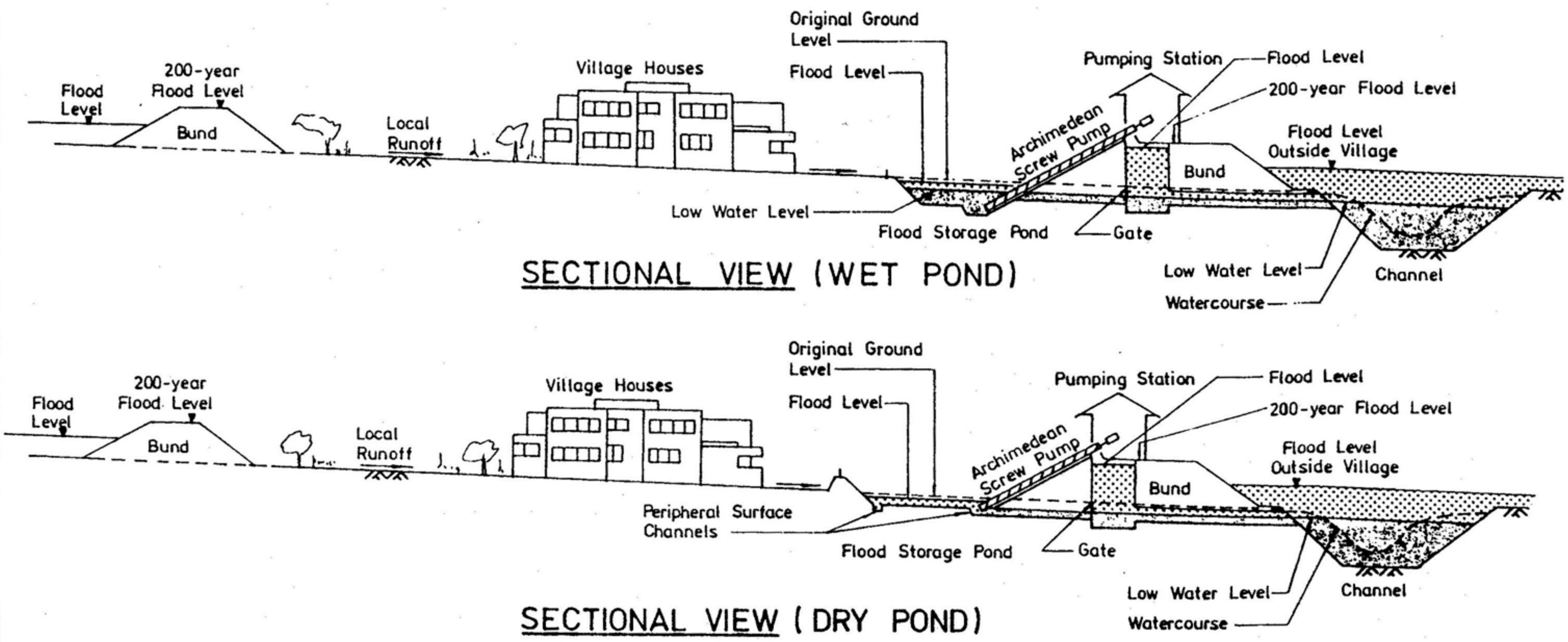


Figure 17. Polder Drainage / Flood Pumping Scheme – Schematic View



NOTES :

Normal condition - flow path

1. Runoff will drain to the pond.
2. The pond will drain over a weir into an outlet pipe.
3. With gate opened, the water will flow to the nearest watercourse.

Flood condition - flow path

1. The system will operate if the water level in either or both the pond and the watercourse exceed a level that would lead to flooding the village (i.e. a water level within the area in the section.)
2. Runoff will drain to the pond as under normal conditions.
3. The gate will close and the water will be pumped from the pond into the watercourse.

Figure 18. Polder Drainage / Flood Pumping Scheme – Sectional View

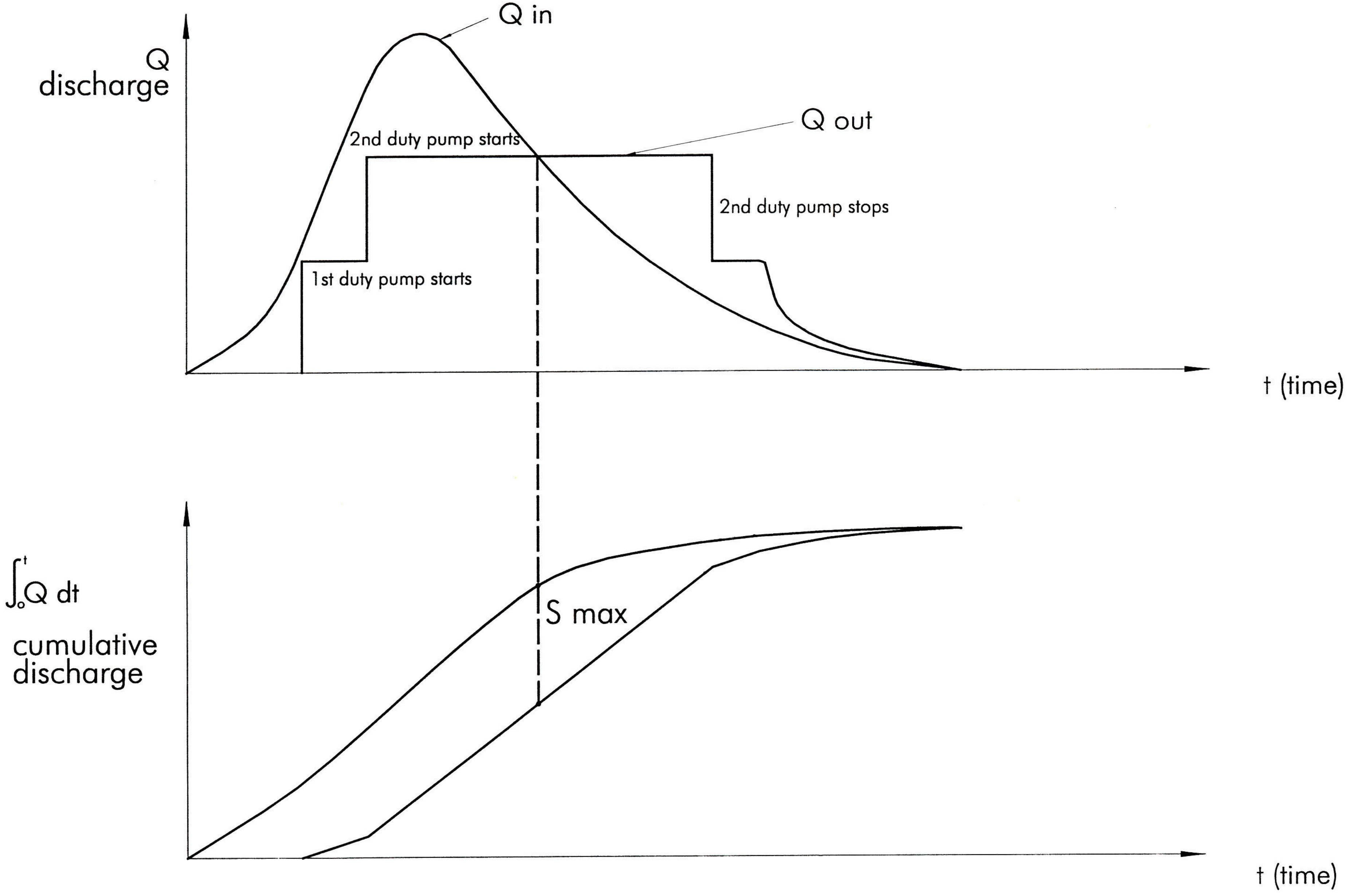


Figure 19. Sizing of Floodwater Storage Pond / Pumps