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Urban Sanitation

Course handout

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Contents

List of Figures

List of Tables

Nomenclature

β, K, α and γ	Factors of *Caquot*	-
$\Delta\theta$	Initial soil water deficit, $\Delta\theta = \theta_s - \theta_i$	-
$\mathcal{C}$	*Chezy* constant	$m^{1/2} \cdot s^{-1}$
$\mathcal{F}$	Frequency of the rainfall, see equation (3.3), page 15	-
$\mathcal{T}$	Return period of a rainfall event, see equation (3.4), page 15	-
$\mathcal{V}_p$	Inflow volume into the storage facility, see equation (5.30), page 110	m^3
$\mathcal{V}_r$	Discharged volume, see equation (5.31), page 110	m^3/s
$\mathcal{V}_s$	Storage volume, see equation (5.32), page 110	m^3/s
$\mathcal{V}_{\max}$	Maximum storage volume of a retention basin, see equation (5.28), page 108	m^3
ψ	Suction head at the wetting front	mm
θ	Recurrence interval of a rainfall event, see equation (3.4), page 15	-
θ_i	Initial uniform water content	-
θ_s	Water content at saturation	-
A	Area	m^2
a	Coefficient depending on the local climate and the calculation frequency	mm
A_a	Active area of the basin, see equation (5.29), page 109	m^2

A_{imp}	impervious area (roofs and paved surfaces)	ha
B	Width	m
b	Coefficient depending on the local climate and the calculation frequency	-
b_w	Length of the weir crest	m
C	Runoff coefficient	-
C_0 C_1 and C_1	Constants, see Table 3.17, page 42	-
C_a	Multiplier coefficient in the modified rational method equation, see Table 3.14, page 36	
C_d	Discharge coefficient	
C_r	Retardation coefficient, see Table 3.10, page 27	-
CN	Curve number, see equation (3.43), page 39	-
D	Standard sewer diameter	m
d	Depression storage, see equation (3.9), page 16	mm
d_0	Computed diameter	m
DHB	Soil moisture content just before the event	-
E	Interception efficiency of an inlet, see equation (5.14), page 90	-
E	Interception efficiency of an inlet, see equation (5.14), page 90	-
E_0	Frontal to total flow ratio, see equation (5.7)	-
F	Cumulative infiltration	mm/h
f	*Darcy-Weisbach* coefficient	-
f_0	Initial infiltration capacity	mm/h
F_a	Pond and swamp adjustment factor, see Table 3.16, page 41	
f_c	Ultimate or equilibrium infiltration capacity	mm/h
f_p	Infiltration capacity	mm/h
f_s	Factor related to surface, see Table 3.7, page 26	-
H	vertical distance	m
h	Amount of rainfall	mm
h_e	Fictitious head height over the overflow	m

I	Interception	mm
i	Average rainfall intensity, see equation (3.5), page 15	mm/h
I_a	Initial losses	mm
k_1, k_2	Coefficients	-
K_B	*Bazin* coefficient, see equation (4.5), page 60	-
K_K	*Kutter* coefficient, see equation (4.6), page 60	-
K_S	*Strickler* coefficient, see equation (4.8), page 62	$m^{1/3}/s$
K_s	Saturated hydraulic conductivity	mm/h
L	Length	m
L_1	Length from the crest to the first inlet, see equation (5.25), page 98	m
L_i	spacing between subsequent inlets, see equation (5.26), page 98	m
L_T	Curb-opening length	m
M	Elongation coefficient, see equation (3.2), page 14	-
m	Multiplier coefficient in the equation of *Caquot*	-
n	*Manning* coefficient, see equation (4.8), page 62	$s/m^{1/3}$
P	Percentage of impervious surfaces	-
P_w	Wetted perimeter	m
PF	Peak factor	-
PR	Percentage runoff from the entire sub-basin, see equation (3.23), page 24	%
Q_*	Relative conductivity, see equation (4.20), page 65	-
Q_b	Bypass flow or carryover flow of an inlet, see equation (5.15), page 90	m^3/s
Q_i	Intercepted flow rate of an inlet, page 90	m^3/s
Q_p	Inflow to the retention basin	m^3/s
Q_r	Outflow from the retention basin	m^3/s
Q_s	Gutter discharge in the section that is not depressed	m^3/s
Q_T	Total flow rate, see equation (3.51), page 44	m^3/s
Q_U	Amount of wastewater discharged, see equation (3.46), page 43	m^3/s

q_u	unit peak discharge, see equation (3.45), page 40	$m^3/s/cm/km^2$
Q_v	Total outflow discharge (drainage or release rate)	m^3/s
Q_w	Gutter discharge in the depressed section	m^3/s
$Q_{avg,d}$	Average daily domestic fresh water consumption discharge	m^3/s
Q_{fs}	Full flow rate, see equation (4.14), page 65	m^3/s
$Q_{overflow}$	Flow rate of the overflow, see equation (5.19), page 101	m^3/s
Q_{si}	Side flow intercepted	m^3/s
Q_{sp}	Wastewater specific flow rate, see equation (3.49), page 43	m^3/s
Q_{Storm}	Stormwater flow rate	m^3/s
Q_{Waste}	Peak wastewater flow rate, see equation (3.47), page 43	m^3/s
R	Runoff depth, see equation (3.39), page 39	mm
r	Roughness coefficient depending on the surface, see Table 3.8, page 27	-
R_f	Ratio of frontal intercepted flow to total frontal flow, see equation (5.18), page 92	-
R_h	Hydraulic radius, see equation (4.3), page 60	m
R_Q	Flow ratio, see equation (4.16), page 65	-
R_s	Ratio of intercepted side flow to total side flow, see equation (5.19), page 92	-
R_V	Velocity ratio, see equation (4.17), page 65	-
R_y	Height ratio, see equation (4.18), page 65	-
S	Maximum potential retention, see equation (3.42), page 39	mm
S_0	Longitudinal slope	-
S_e	Equivalent cross slope, see equation (5.24), page 95	m/m
S_x	Cross slope	m/m
SOL	Geological permeability of the soil	-
T	Top width	m
t	Duration of the rainfall	h
t_c	Time of concentration, see equation (3.25), page 25	min
t_e	Time of entry (or surface runoff time)	min

t_f	Network time of flow (or travel time in conduits)	min
t_m	Duration corresponds to the maximum volume of the retention basin	min
T_p	time to peak on outflow hydrograph	min
TI	Imperviousness rate, see equation (3.24), page 24	%
V	Average flow velocity	m/s
V_0	Splashover velocity	m/s
V_{fs}	Full flow velocity, see equation (4.15), page 65	m/s
W	Width of the depressed gutter or inlet	m
W_p	Lateral distance in m from the pavement crown to the curb	m
y	Water depth	m

Introduction

A sanitation system is designed to transport wastewater and stormwater from various sources to treatment facilities, ensuring safe discharge into the environment.

This handout is intended for third-year students in the new hydraulic engineering bachelor's program (LMD). It is part of the fundamental unit 2 of the sixth semester. The content of this handout includes the syllabus taught in the Civil and Hydraulic Engineering Department of the Institute of Sciences at the Nour Bachir University Center in El-Bayadh.

The aim of this handout is to provide the essential foundations for understanding and calculating sanitation networks. It enables students to acquire knowledge about the basic design of a sanitation network and develop the skills necessary to choose the appropriate structure for housing projects and the proper sizing of the sanitation network.

The handout is structured into five chapters. The first chapter presents the characteristics of discharged waters (wastewater and stormwater). The second chapter covers sanitation systems and various wastewater evacuation schemes. The third chapter explains the methods for evaluating the flows to be collected, detailing concepts such as frequency, intensity, runoff coefficient, and time of concentration. It also discusses methods for calculating stormwater flow (using rational, superficial, and SCS methods) and the procedure for evaluating wastewater flows. The fourth chapter addresses the hydraulic calculation of the drainage network and the necessary conditions for the proper evacuation of discharged waters. It includes the design of combined and separate networks, as well as the calculation of hydraulic parameters. The final chapter is dedicated to ancillary structures that ensure the proper functioning of the network and the transport of polluted effluents to the treatment plant.

The end of each chapter is devoted to review questions and multiple-choice questions aimed at deepening the understanding of the course. Resolved applications and additional exercises are also included, chosen for their practical relevance and diversity. Detailed solutions enable students to master the concepts used and assess their knowledge progress. The appendix section is reserved for a mini-project with data provided for each student.

1. General Characteristics of Wastewater

Wastewater, regardless of its origin, is generally laden with undesirable elements which, depending on their quantity and composition, pose a real danger to receiving environments or their users. In this chapter, we present the types of wastewater and highlight the main physicochemical and bacteriological parameters commonly found in wastewater.

The objectives of this chapter are:

1. Understand the types of wastewater to be evacuated;
2. Know the characteristics of wastewater.

1.1 Definition

Urban wastewater, or simply wastewater, is water loaded with pollutants, either soluble or insoluble, primarily resulting from human activities. This includes domestic and industrial wastewater, as well as a portion of runoff or rainwater.

The purpose of urban sanitation is to ensure the evacuation of all rainwater and wastewater, and their discharge into natural outlets in ways that are compatible with public health and environmental requirements.

1.2 Types of Wastewater

Wastewater can be categorized into three types:

- Runoff water
- Domestic wastewater
- Industrial wastewater

These types of water can be either separate or mixed, giving rise to the concept of urban effluent, which consists of domestic wastewater that is more or less polluted by industrial water and more or less diluted by runoff water. The characteristics of each of these three categories are:

1.2.1 Runoff Water

Runoff water includes rainwater, washing water, and drainage water.

Urban runoff water mainly contains metals such as lead, zinc, copper, nickel, and chrome, as well as some organic pollutants like oils, greases, tannins, lignin, and anions from cleaning products and automobile traffic. They also often contain sand, which is significant in terms of their evacuation.

The pollution level of runoff water varies over time, being higher at the start of precipitation and lower at the end due to the washing of surfaces by the water.

Drainage water, which can come from the rise of a phreatic aquifer, is generally low in pollution.

1.2.2 Domestic Wastewater

Domestic wastewater includes:

- Household water (from kitchens, laundry, showers, etc.): This water is polluted and contains organic matter and various debris (detergents, particles).
- Black water (from toilets, feces, and urine): This water is highly putrescible and malodorous.

1.2.3 Industrial Wastewater

Industrial wastewater comes from various manufacturing or processing plants. It contains a wide range of substances, which can be acidic or alkaline, corrosive or scaling, at high temperatures, often odorous and colored. These waters may require pretreatment at the plant.

1.3 Characteristics of Wastewater

1.3.1 Physical Characteristics of Wastewater

1. Suspended Solids (SS): For purposes related to wastewater treatment, suspended solids are distinguished into those that settle within 2 hours (SSD) and those that do not settle within 2 hours (SSND) due to their very fine granulation, density close to that of water, or colloidal state.
2. Dissolved matter.

1.3.2 Chemical Characteristics of Wastewater

1. Mineral matter: These are represented by the dry residue after heating in a crucible until red hot, of all the materials collected after evaporation. They are not dangerous.
2. Organic matter: These are the materials that volatilize during heating under the same conditions as above. In addition to the primary compounds - C, O, N, H - organic matter contains small quantities or even traces of elements that play an important role in degradation or assimilation processes (for example, S, Fe, Cu, P) (*Gomella et Guerrée*, 1986).
3. Biochemical Oxygen Demand (BOD): This measures the amount of oxygen consumed by chemical oxidation processes and the aerobic degradation of organic matter (excluding nitrogenous compounds) necessary for the breakdown of organic compounds. These two types of oxidation overlap, hence the term "biochemical." In wastewater, biological processes predominate. BOD is expressed in mg of O_2 consumed per liter of effluent. By convention, the oxygen consumption after 5 days (BOD_5) is taken as the standard measure.
4. Chemical Oxygen Demand (COD): This represents the amount of oxygen needed for the chemical breakdown of oxidizable organic and mineral matter in the water.

Table 1.1 gives a summary of the unit concentrations.

Table 1.1: Standard water composition.

Nature	SS				BOD_5		COD	
	Mineral (mg/L)	Organic (mg/L)	Total (mg/L)	Day (g/person)	Average (mg/L)	Day (g/person)	Average (mg/L)	Day (g/person)
SSD	130	270	400	60	130	19	270	40
SSND	70	130	200	30	80	12	140	21
Dissolved matter					150	23	260	39
Total	200	400	600	90	360	54	370	100
Runoff Water			600		100		240	
Wastewater			540	80	400	60	960	140
Combined water			580	85	350	60	800	120

1.3.3 Biological Characteristics of Wastewater

Wastewater contains bacteria, pathogenic microorganisms (protozoa), viruses, and parasites (helminths) that are harmful to human health. It also contains fungi and algae (*Gomella et Guerrée*, 1986).

1.4 Review Questions

1. Cite two sources of wastewater.
2. What are the main sources of water to be discharged in an urban context?
3. What are the physical characteristics of wastewater?
4. What are the chemical characteristics of wastewater?
5. What are the biological characteristics of wastewater?
6. Why is it important to distinguish between stormwater and domestic wastewater?
7. Define the following terms from the perspective of sanitation: runoff water; domestic wastewater.
8. What do the terms BOD and COD mean?
9. What is the objective of sanitation?
10. The pollution of runoff water varies over time, being higher at the beginning of a precipitation event than at the end. Why?

1.5 Objective Questions

Q 1.1 What is sanitation?

a) The treatment of water-related diseases

b) The management of solid waste

c) The control of air quality

d) The disposal of wastewater and excreta

Q 1.2 What is the primary goal of sanitation?

a) Preventing floods

b) Promoting public health

c) Reducing water consumption

d) Saving energy

Q 1.3 What is the main consequence of inadequate sanitation in communities?

a) Increased soil fertility

b) Reduced demand for drinking water

c) Spread of waterborne diseases

d) Increased biodiversity

2. Systems and Schemes of Wastewater Management

The disposal of domestic, industrial, and stormwater must be carried out according to a strategy that takes into account various socio-economic criteria. The evacuation of different types of wastewater follows well-studied paths, forming a hydraulic network designed to ensure that this wastewater is removed from the urban area under good flow conditions and without causing nuisances to the population.

The objectives of this chapter are to:

1. Present the different wastewater evacuation systems.
2. Illustrate the various network schemes encountered in practice.
3. Understand the criteria for choosing the type of evacuation network.

2.1 Network Systems

2.1.1 Fundamental Systems

Two fundamental network systems are distinguished (*Gomella et Guerrée*, 1986):

1. The combined system.
2. The separate system.

Additionally, a mixed system refers to a network that combines, depending on the area, both combined and separate systems. These systems are usually gravity-based but can include pressurized transport.

2.1.1.1 Combined System

In the combined system, all waters, including pre-treated industrial effluents, are collected in a single network that leads to a wastewater treatment plant (WWTP). The occasional installation of overflow weirs allows, in case of storms, the direct discharge of some water into the natural environment.

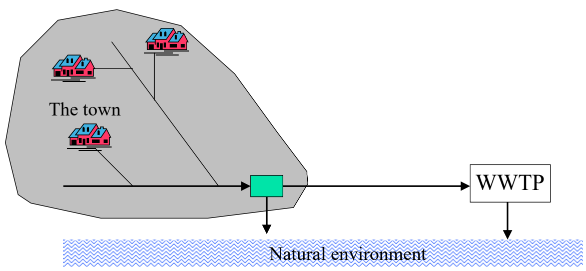


Figure 2.1: Combined System.

Advantages

- Only one network.
- Only one connection per building block or parcel (no risk of connection errors).
- Urban stormwater, polluted after washing roofs and roads, is treated at the WWTP.
- Less expensive than the separate system in terms of investment, operation, and connection management.

Disadvantages

- Requires relatively large infrastructures - sewers and treatment plants - to handle peak runoff; thus, they are often oversized.
- Occasional overflow discharges need careful management.
- Dilution of WWTP waters during rainy periods (highly variable flow rates).

2.1.1.2 Separate System

This system consists of a double parallel network:

- One for domestic wastewater and possibly industrial wastewater leading to the WWTP.
- The other for stormwater, discharged directly into a natural outlet (river) or treated separately (retention basin).

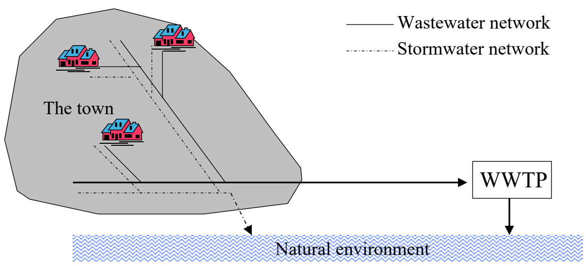


Figure 2.2: Separate System.

Advantages

- Ensures regular operation of the WWTP as the waters to be treated have the lowest and most consistent flow rates.
- The only feasible option when the population is relatively dispersed.
- More reasonable sizing of the WWTP and the wastewater pipes leading to it.
- Better self-cleaning capacity, leading to maintenance cost savings.
- Heavy rains no longer result in wastewater discharge into the natural environment.

Disadvantages

- Multiplication of connections (risk of connection errors).
- Higher overall construction cost, about 1.5 times that of a combined system.
- Higher connection management cost.
- Stormwater is not free of pollutants and sometimes requires specific treatment before final discharge.

2.1.2 Pseudo-Separate System

The term pseudo-separate applies to networks receiving wastewater and some or all runoff from adjacent properties, while runoff from roads is evacuated by gutters and, if necessary, by some stormwater sections (*Gomella et Guerrée*, 1986).

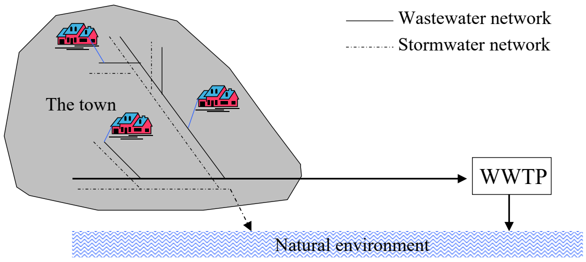


Figure 2.3: Pseudo-Separate System.

Advantages

- No need to separate runoff and wastewater from the same building.
- Can be partially applied upstream of combined networks.
- Economical as long as underground evacuation of surface runoff is not necessary; no need to install a pipe in a road unless runoff exceeds 80 L/s (40 L/s per gutter).
- No risk of connection errors.

Disadvantages

It possesses most of the disadvantages of both the combined and separate systems.

2.1.3 Special Systems

Also known as "assisted transfers," these systems are used whenever topography demands. They avoid the excessive and costly depths required for laying pipes. They are particularly valuable for inter-municipal connections, sometimes following very irregular profiles. The types are:

1. Pressurized sewage networks using pumped water.
2. Pressurized sewage networks using air ejectors: The principle involves pumping using compressed air.
3. Vacuum sewage networks: One or more users are connected by gravity to a transfer manhole, which acts as a buffer overflow. This ensures evacuation by suction to a storage tank located at the vacuum station.
4. Composite systems, based on a separate system, where some particularly polluted runoff is evacuated in the wastewater network for treatment.

2.2 Network Schemes

Drainage networks are essentially free-surface, branched systems that heavily depend on the terrain. If excessive trench depths are to be avoided, the layout of the network will depend on the topography.

Most often, the settlement to be sanitized is located near a river or valley, which allows for final evacuation, after possible treatment, and indicates the general relief guiding the sewer orientation (*Bonnin*, 1977).

2.2.1 Perpendicular Scheme

This layout consists of bringing several collectors perpendicularly to the river, each receiving water from primary sewers (Fig. 2.4). This approach does not allow for the concentration of water towards a single treatment point, effectively prohibiting it. It is suitable when treatment is deemed unnecessary, particularly for stormwater networks. This is the most economical layout, especially if the terrain slope towards the river is mild.

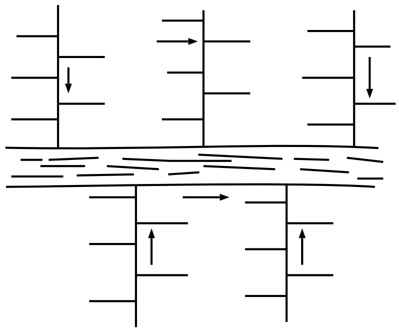


Figure 2.4: Perpendicular network.

2.2.2 Lateral Collector Scheme

The previous layout can be modified to channel all water through a single treatment point E by placing a lateral collector along the river (Fig. 2.5). Two difficulties may arise: insufficient slope of the collector and the hindrance caused by the alluvial aquifer during installation, often necessitating elevation. If the settlement spans both sides of the river, two lateral collectors are required, with one (usually the smaller one) crossing the river to reach the treatment point, typically via a pressure pipe known as a siphon.

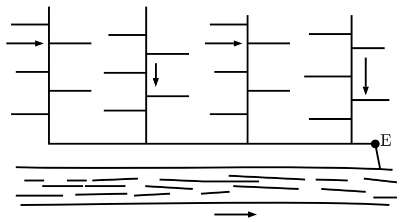


Figure 2.5: Lateral collector network.

2.2.3 Oblique Transverse Collector Scheme

To increase the slope of the collector when the river's slope is insufficient, an oblique layout (Fig. 2.6) can be used to also take advantage of the terrain slope towards the river. In this case, the areas between the collector and the river are less easily sanitized, and the sewer slope will be low, but the discharge to be evacuated there is also less significant. The oblique layout results from an optimal technical and economic balance.

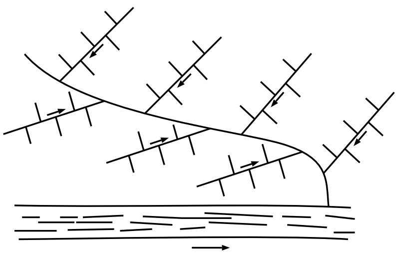


Figure 2.6: Oblique transverse collector network.

2.2.4 Tiered Zones Scheme

The tiered zones scheme (Fig. 2.7) is a variation of the lateral collector scheme, but with multiple longitudinal collectors; it allows unloading the lower collector from inflows from the upper part of the settlement.

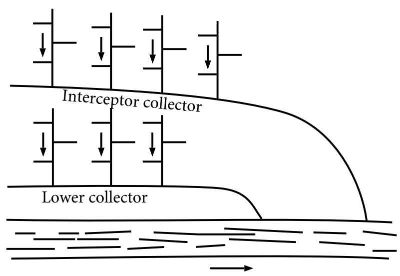


Figure 2.7: Tiered collector network.

2.2.5 Radial Scheme

If the settlement is on a plain (or plateau) without any relief, a slope must be given to the sewers by varying the depth of the trench where they are laid; thus, radial sewers converge towards a point P (figure 2.8). From this point, the wastewater will be pumped and will then flow under pressure to a suitable outlet.
The practical and economical service radius for a pumping station like P in strictly horizontal relief is a few hectometers.

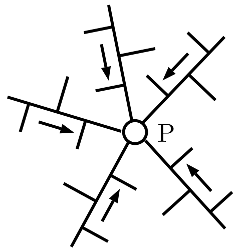


Figure 2.8: Radial network (fan-shaped).

2.2.6 Multi-Radial Scheme

In the case of a city spread out on a horizontal plain, it is necessary to multiply the pumping stations P, considering their practical service radius. The resulting multi-radial network (Fig. 2.9) is very costly in terms of investment and operation.

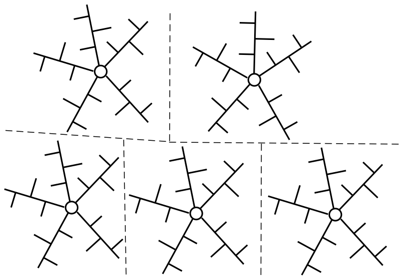


Figure 2.9: Multi-radial network.

2.3 Review Questions

1. What is the main difference between a pseudo-separate sewer system and a combined sewer system?
2. Define the following terms from a sanitation perspective: A combined system; A separate system; A pseudo-separate system.
3. What are the two fundamental systems of wastewater drainage in sanitation? Explain.
4. List the disadvantages of the separate wastewater drainage system.
5. List the advantages and disadvantages of the combined system.
6. List the different sanitation schemes.
7. What is the source of water conveyed by a pseudo-separate system?
8. List three types of urban wastewater collection systems.
9. Why are special wastewater drainage systems needed?
10. What are the four special wastewater drainage systems?

2.4 Objective Questions

Q 2.1 What is the primary function of a sanitation network?

1. To collect wastewater.
2. To collect and transport wastewater.
3. To treat wastewater for later reuse.
4. To harvest rainwater for sustainable use.

■

Q 2.2 What is a sewer system?

a) A system for collecting and transporting rainwater.

b) A system for collecting and transporting groundwater.

c) A system for collecting and transporting wastewater.

d) A system for collecting and transporting potable water.

■

Q 2.3 The sewer that carries domestic wastewater and stormwater is called

a) combined sewer

b) separate sewer

c) pseudo-separate sewer

d) composite sewer

■

Q 2.4 What is the main difference between a combined sewer system and a separate sewer system?

a) The method of collecting rainwater

b) The amount of water used in households

c) The treatment of industrial wastewater

d) The separation of domestic wastewater and stormwater

■

Q 2.5 The network scheme shown in figure Q2.5 is:

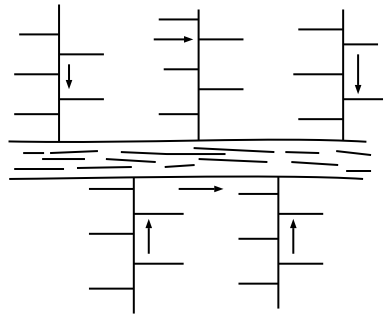


Figure Q2.5

a) Lateral collector scheme

b) Radial scheme

c) Tiered zone scheme

d) Perpendicular scheme

■

3. Assessment of Collected Flows

The knowledge of wastewater flows is crucial for the proper design of collection networks. This knowledge ensures correct network sizing, calculations for network resistance to exceptional inputs, cost rationalization (both investment and operational), and the safety of maintenance personnel and neighboring users. Taking into account the spatial and demographic evolution forecasts of the concerned locality is challenging, but it helps avoid the risks of over- or under-sizing.

The establishment of an urban area's sanitation networks must meet two main objectives:

1. Proper evacuation of stormwater to prevent the flooding of urban areas and avoid water stagnation, particularly in low-lying areas of the city.
2. Collection and evacuation of all types of wastewater (blackwater, greywater, industrial water), ensuring their transport as quickly as possible to the treatment plant (sewage treatment plant).

The sizing of a sanitation network requires knowledge of three main parameters. These parameters are: the flow rate of the water to be evacuated, the geometric slope of the pipes, and the absolute roughness of the inner pipe wall. This chapter focuses on addressing the first parameter, which is the assessment of the flow rates of the water to be evacuated.

The objectives of this chapter are:

1. Understand the concepts of frequency, intensity, runoff coefficient, and time of concentration;
2. Learn how to calculate stormwater flow rates using the rational, superficial, and SCS methods;
3. Know the procedure for assessing water flow rates.

3.1 Evaluation of Stormwater Flows

Stormwater is generated by precipitation, typically rainfall, and consists of that proportion that runs off from urban surfaces (see Figure 3.1)). Hence, the properties of stormwater, in terms of quantity and quality, are intrinsically linked to the nature and characteristics of both the rainfall and the catchment (*Butler et al.*, 2018).

A specific structure must be able to discharge, under satisfactory conditions, the volume of water resulting from rainfall over the basin it serves.

Structures are not designed for the highest known rainfall, which would lead to excessive costs, but for rainfall with a determined probability.

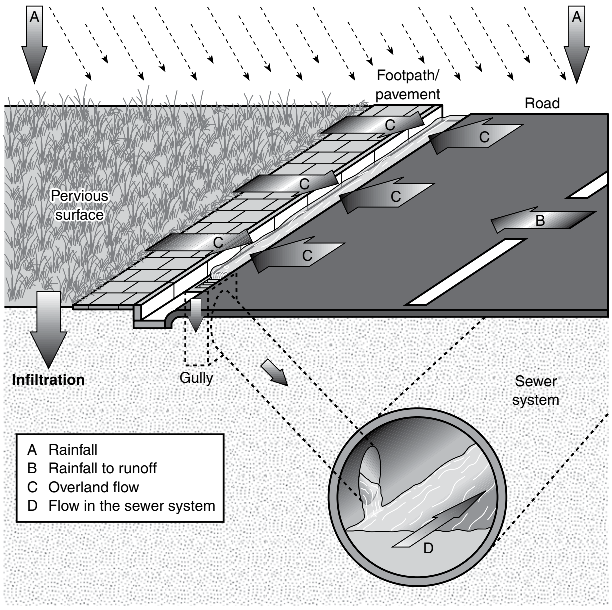


Figure 3.1: Runoff generation process.

3.1.1 Watershed

A watershed is the land area that drains water to a single point, the discharge (*Brière*, 2012). The characteristics of watersheds are:

a) Area, A (in ha).

b) Length, L, is the longest hydraulic path composed of n segments of horizontal lengths L_j.

c) The average slope is equal to:

- The difference in altitude between the beginning and end of the hydraulic path, divided by its length, if the path is formed of a single segment.
- The hydraulic (or weighted) slope S_0, if the path is formed of n segments of horizontal lengths L_j and slopes S_{0_j}, given by:

$$S_0 = \left(\frac{\sum L_j}{\sum \left(L_j / \sqrt{S_{0_j}} \right)} \right)^2 \tag{3.1}$$

d) The elongation ratio of a basin M, is the ratio of the longest hydraulic path L (in m) to the side of the square whose area is equivalent to that of the basin A (in m^2), as follows (*Gomella et Guerrée*, 1986):

$$M = \frac{L}{\sqrt{A}} \tag{3.2}$$

- If M tends towards 1, the basin is said to be compact.
- If M is significantly greater than 1, the basin is said to be elongated.

3.1.2 Frequency, Recurrence Interval, Intensity

Let $h(t)$ be the amount of rainfall (in mm) during the time interval t (in h). If this rainfall has been recorded n times over a period of N years, the frequency of this rainfall is given by:

$$\mathcal{F} = \frac{n}{N} \tag{3.3}$$

This rainfall is said to have an annual (one year), biennial (two years), five-year, ten-year (decadal), etc., frequency if it occurs on average once every year, every two years, every five years, ten years, etc.

The return period $\mathcal{T}$ or recurrence interval θ of a rainfall event is defined as the inverse of its frequency.

$$\mathcal{T} = \theta = \frac{1}{\mathcal{F}} \tag{3.4}$$

The average intensity (in mm/h) is defined as the ratio of the rainfall amount P to the corresponding duration t (*Bourrier et Claudon*, 1981):

$$i = \frac{h}{t} \tag{3.5}$$

The IDF curves (Intensity-Duration-Frequency curves) graphically represent the variation of average rainfall intensity as a function of its duration for short-duration rains (< 3 hours, often < 1 hour) and for various recurrence intervals (*Brière*, 2012). Figure 3.2 shows the IDF curves for the Boukerdane region (Tipasa, Algeria); the most common recurrence intervals are 100 years, 50 years, 20 years, 10 years, 5 years, and 2 years.

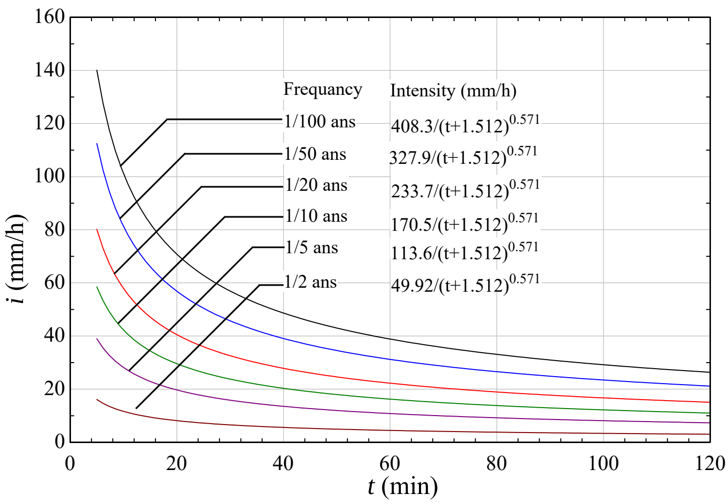


Figure 3.2: IDF curves for the Boukerdane station (Tipasa, Algeria).

The equations used to describe the IDF curves are of the types shown in Table 3.1: where a (in mm), b, and n are coefficients depending on the local climate and the calculation frequency. t is the rainfall duration in min

A return period of $\mathcal{T} = 10$ years is frequently adopted. For $\mathcal{T} \neq 10$ years, the corresponding discharge can be obtained by multiplying the discharge calculated for the 10-year period by a coefficient f defined as follows (*Gomella et Guerrée*, 1986):

- $f = 1.25$ for $\mathcal{T} = 20$ years,
- $f = 1.60$ for $\mathcal{T} = 50$ years,
- $f = 2.00$ for $\mathcal{T} = 100$ years.

Table 3.1: Formulas for calculating the maximum rainfall intensity.

No.	Name	Unit	Formula	
1	*Montana*	mm/min	$i = at^b$	(3.6)
2	*Talbot*	mm/min	$i = \dfrac{a}{t+b}$	(3.7)
3	*Keifer* and *Chu*	mm/min	$i = \dfrac{a}{(t+b)^n}$	(3.8)

3.1.3 Losses

Net rainfall represents the amount of rain that runs off directly over the land surface during a storm. Net rainfall is derived from the total rainfall, reduced by losses (*Musy et Higy*, 2004).

Two types of losses must be considered for establishing the excess rainfall contributing to runoff:

3.1.3.1 Initial Losses

Including:

- Interception, I, is the amount of precipitation that can be stored on vegetation within the watershed, to be later evaporated into the atmosphere. The interception loss for impervious areas is low (< 1 mm) and is normally neglected or combined with depression storage.
- Surface retention accounts for water held in small soil depressions or that fills rills, channels, gutters, and ditches, allowing for the pressure required for surface runoff. This water may subsequently partially evaporate or infiltrate (*Bourrier et Claudon*, 1981). Depression storage d (mm) can be represented by:

$$d = \frac{k_1}{S_0} \tag{3.9}$$

where k_1 is a coefficient dependent on the surface type (0.07 for impervious surfaces and 0.28 for pervious surfaces) (mm), and S_0 is the soil slope (-). Typical values for I_d are 0.5 to 2 mm for impervious areas, 2.5 to 7.5 mm for flat roofs, and up to 10 mm for gardens.

Initial losses ($I_a = I + d$) are generally subtracted from the precipitation at the start of the storm to yield the net rainfall.

Example 3.1 For an urban watershed with an average slope of 1% and an estimated interception loss of 0.5 mm, calculate the net rainfall profile (based solely on initial losses) for the following storm:

Table E3.1

Time (min)	0-10	10-20	20-30	30-40
Rainfall intensity (mm/h)	6	12	18	6

Assume $k_1 = 0.1$ ■

Solution 3.1
Interception loss = 0.5 mm
Depression storage from eq. (3.9): $d = 1/\sqrt{0.01} = 1$ mm

Precipitated water depth = Rainfall intensity $\times \Delta t$ = Rainfall intensity $\times 10/60$
Net rainfall depth = Precipitated water depth - initial losses = Precipitated water depth - 1.5 (once, at the beginning of the storm)
Net rainfall = Net rainfall depth $/\Delta t$ = Net rainfall depth $\times 60/10$

Table S3.1

Time (min)	0-10	10-20	20-30	30-40
Rainfall intensity (mm/h)	6	12	18	6
Precipitated water depth (mm)	1	2	3	1
Net rainfall depth (mm)	0	1.5	3	1
Net rainfall (mm/h)	0	9	18	6

■

3.1.3.2 Continuing losses

Including:

- Evapotranspiration is the evaporation of water from plants and open water surfaces. Its effect during short-duration rainfall events is negligible.
- Infiltration is the portion of precipitation absorbed by the soil and directed toward lower layers. The infiltration capacity of a soil is defined as the rate at which water infiltrates it. Infiltration depends on factors such as soil type, structure and compaction, initial moisture content, surface cover, topography, and morphology, and the supply rate (intensity). The infiltration rate tends to be high initially but decreases exponentially to a nearly constant final rate as the soil's upper zone becomes saturated. Urban hydrology infiltration modeling has evolved in two main directions: an empirical or hydrological approach (with NRCS (formerly SCS) and *Horton* models) or an approach based on approximations of physical models (the *Green-Ampt* model).

Infiltration is limited by the rainfall rate:

1. If $i \leq K_s$ (K_s is the saturated hydraulic conductivity), ensuring complete and immediate infiltration of the precipitation as shown by line I in figure 3.3a.
2. If $i > K_s$ represented by curve II in figure 3.3a, as water infiltrates, the surface water content will increase to saturation after a time, t_p. After this time t_p, the infiltration capacity becomes less than the rainfall rate, i. Surface ponding begins, resulting in depression storage and runoff.
3. For a rainfall intensity that exceeds infiltration capacity, water continues to accumulate on the surface. In this case, the infiltration rate is controlled only by soil-related factors (it varies over time). This rate, represented by curve III in figure 3.3a, is noted as f_p (*Gupta*, 2016).

Horton Model

Horton (1939) established a three-parameter equation expressed as follows:

$$f_p = f_c + (f_0 - f_c)e^{-k_2 t} \tag{3.10}$$

With:

- f_p: infiltration capacity (mm/h),
- f_c: ultimate or equilibrium infiltration capacity (mm/h),
- f_0: initial infiltration capacity (mm/h),

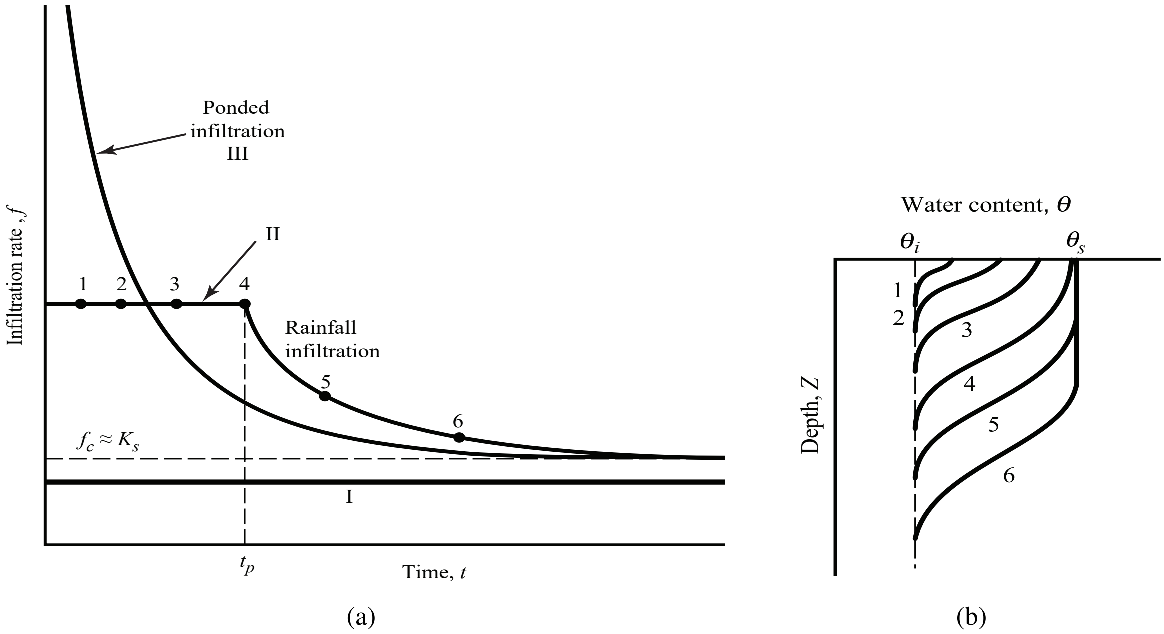


Figure 3.3: Infiltration behavior under different precipitation conditions.

- t: time since the beginning of precipitation (h),
- k_2: a constant representing the decay rate in infiltration capacity (h^{-1}).

This equation is graphically represented in Figure 3.4. The area under the curve in Figure 3.4 indicates, for each time step, the amount of water infiltrated during that interval.

The integration of equation (3.10) gives the cumulative infiltration:

$$F = f_c t + \left[\frac{(f_0 - f_c)(1 - e^{-k_2 t})}{k_2}\right] \tag{3.11}$$

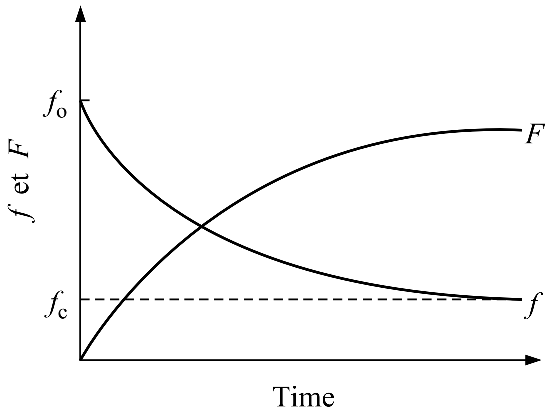


Figure 3.4: Infiltration curves.

Table 3.2: Indicative values of the parameters in Horton's equation for different soil types.

Soil Type	f_0 (mm/h)	f_c (mm/h)	k_2 (h^{-1})
Bare standard	280	6-220	96
Agricultural with peat	900	20-290	48
Peat	320	2-20	108
Fine bare sand	210	2-25	120
Clay with peat	670	10-30	84

Example 3.2 A watershed has an initial infiltration rate of 210 mm/h and a final rate of 10 mm/h. The soil depletion constant is 2 min^{-1}. Use Horton's equation to find (a) the values of f_p at t=10 s, 1 min, 3 min, and 15 min, and (b) the total infiltration volume over the 15 minute period. ■

Solution 3.2

a) The values of f_p at t=10 s, 1 min, 3 min, and 15 min

$f_0 = 210$ mm/h, $f_c = 10$ mm/h and $k_2 = 2 \times 60 = 120$ h^{-1}

From eq. (3.10): $f_p = f_c + (f_0 - f_c)e^{-k_2 t} = 10 + (210 - 10)e^{-120t}$

The results for different values of t are shown in Table S3.3

Table S3.3

t (h)	f_p (mm/h)
10/3600	153.4
1/60	36.9
3/60	10.5
15/60	10.0

b) The total infiltration volume over the 15 minute period

From eq. (3.11): $F = f_c t + \left[\dfrac{(f_0 - f_c)\left(1 - e^{-k_2 t}\right)}{k_2}\right] = 10 \times \dfrac{15}{60} + \left[\dfrac{(210 - 10)(1 - e^{-120\frac{15}{60}})}{120}\right]$

$\boxed{F_{15\text{ min}} = 4.2 \text{ mm}}$.

Green-Ampt Model

The application of *Darcy's* law leads to the following form of the equation by *Green et Ampt* (1911):

$$f_p = K_s \left(\frac{\psi \Delta\theta}{F} + 1\right) \tag{3.12}$$

where

- f_p: infiltration capacity (mm/h),
- K_s: saturated hydraulic conductivity (mm/h),
- ψ: suction head at the wetting front (mm),
- $\Delta\theta$: initial soil water deficit, $\Delta\theta = \theta_s - \theta_i$,
- θ_i: initial uniform water content,
- θ_s: water content at saturation,
- F: cumulative infiltrated water volume, $F = \Delta\theta L$,
- L: depth to the wetting front (see Fig. 3.5. In the figure, H represents the surface ponding depth).

Since $f_p = dF/dt$, integrating eq. (3.12) gives the cumulative infiltration as:

$$K_s t = F - \Delta\theta\psi \ln\left(1 + \frac{F}{\Delta\theta\psi}\right) \tag{3.13}$$

Equations (3.12) and (3.13) apply when there is ponding ($i \geq f_p$) from the beginning. If $i < f_p$, surface ponding will not occur until time t_p. Under these conditions, for steady rainfall, the actual infiltration rate, f, can be summarized as follows (*Gupta*, 2016):

1. For $t < t_p$,

$$f = i \tag{3.14}$$

2. For $t = t_p$,

$$f = f_p = i \tag{3.15}$$

The volume of infiltrated water required to bring the soil surface to saturation:

$$F_p = K_s \frac{\psi \Delta\theta}{i - K_s} \tag{3.16}$$

The time associated with reaching saturation:

$$t_p = \frac{F_p}{i} \tag{3.17}$$

All the rainfall infiltrates from the start of the storm until t_p, for a total F_p.

3. For $t > t_p$, the infiltration rate will decrease with time as indicated in eq. (3.12)

$$f = f_p = K_s \left(\frac{\psi \Delta\theta}{F} + 1 \right) \tag{3.18}$$

Cumulative infiltration can be found by solving the equation:

$$K_s(t - t_p + t'_p) = F - \Delta\theta \psi \ln \left(1 + \frac{F}{\Delta\theta \psi} \right) \tag{3.19}$$

where t'_p is the equivalent time to infiltrate F_p under surface ponding conditions from the beginning, obtained from eq. (3.13).

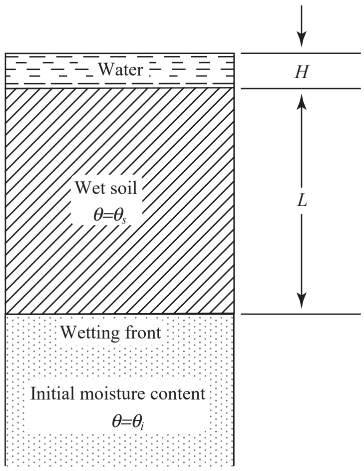


Figure 3.5: Simplified wetting front in the *Green-Ampt* model.

Table 3.3: *Green-Ampt* Infiltration Parameters

Soil Type	θ_s	ψ (cm)	K_s (cm/h)
Sand	0.437	4.95	11.78
Loamy Sand	0.437	6.13	2.99
Sandy Loam	0.453	11.01	1.09
Sandy Clay Loam	0.398	21.85	0.15
Loam	0.463	8.89	0.66
Silt Loam	0.501	16.68	0.34
Clay Loam	0.464	20.88	0.10
Clay	0.475	31.63	0.03
Sandy Clay	0.430	23.90	0.06
Silty Clay Loam	0.471	27.30	0.10
Silty Clay	0.479	29.22	0.05

Example 3.3 A watershed consists almost entirely of loamy sand with an initial uniform water content of 0.076. You are designing a stormwater management system to handle runoff from this area, using the one-hour storm given in Table E3.3 as the basis for your design. Determine the runoff over time for average initial moisture conditions and compare the amount of rainfall with the amount of runoff. Use the *Green-Ampt* method for your calculations and assume negligible interception and depression storage. ■

Table E3.3

Interval (min)	Average Precipitation (mm/h)
0-10	10
10-20	20
20-30	80
30-40	100
40-50	80
50-60	10

Solution 3.3

In this case, $K_s = 29.9$ mm/h; $\theta_i = 0.076$; $\theta_s = 0.437$; $\Delta\theta = 0.437 - 0.076 = 0.361$ and $\psi = 61.3$ mm.

t=0 to 10 min

Since $i < K_s$, there is no surface accumulation and all the rain infiltrates.
The cumulative infiltration, $F_1 = 10(10/60) = 1.7$ mm.

t=10 to 20 min

Again, $i < K_s$, there is no surface accumulation and all the rain infiltrates.
The cumulative infiltration, $F_2 = 1.7 + 20(10/60) = 5.0$ mm.

t=20 to 30 min

$i > K_s$, surface accumulation is possible.
The cumulative infiltration required to saturate the soil surface:
$F_p = K_s \dfrac{\psi\Delta\theta}{i - K_s} = 30 \dfrac{61.3 \times 0.361}{80 - 30} = 13.2$
The incremental cumulative infiltration: $\Delta F_p = 13.2 - 5 = 8.2$ mm
The time associated with saturation: $t_p = \dfrac{20}{60} + \dfrac{8.2}{80} = 0.435$ h $= 26.1$ min
The next step is to find the time, t'_p, needed for 13.2 mm to infiltrate from $t = 0$. From eq. (3.13):
$29.9t'_p = 13.21 - 0.361 \times 61.3 \ln\left(1 + \dfrac{13.2}{0.361 \times 61.3}\right) = 0.095$ h $= 5.7$ min
From eq. (3.19): $29.9(t - 0.435 + 0.095) = F - 0.361 \times 61.3 \ln\left(1 + \dfrac{F}{0.361 \times 61.3}\right)$

$$29.9(t - 0.34) = F - 22.13 \ln\left(1 + \frac{F}{22.13}\right) \qquad \text{(S3.3a)}$$

The curve of F as a function of t is shown in figure S3.3

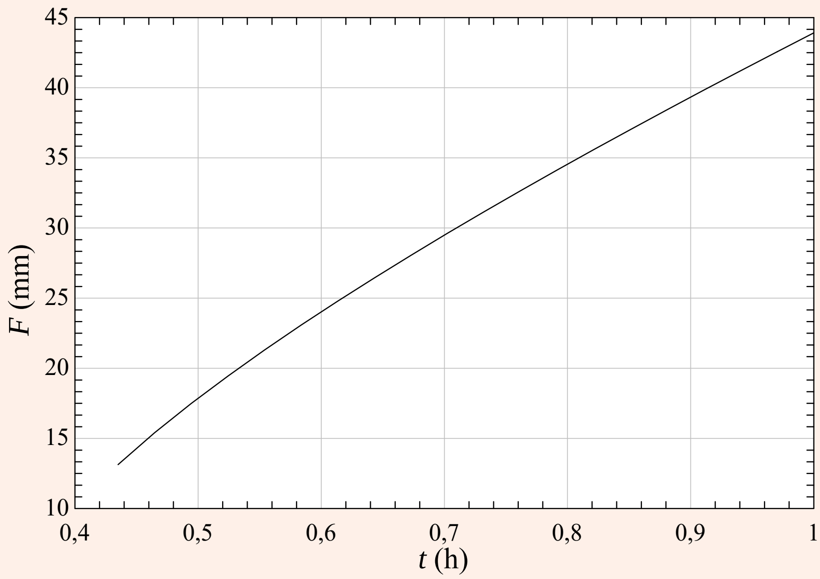


Figure S3.3

At the end of this period, when $t = 30$ min or 0.5 h, $F = 17.9$ mm (from figure S3.3)
Precipitation during this period: $i\Delta t = 80 \times 10/60 = 13.3$ mm
The amount infiltrated during this period: ΔF = 17.9 mm - 5 mm = 12.9.
The amount of precipitation that does not infiltrate: $i\Delta t - \Delta F$ = 13.3 mm - 12.9 mm = 0.4 mm.
Depression storage = 0,
Runoff = 0.4 mm.

t = 30-40 min
$i > K_s$, surface accumulation continues.
At the end of this period, when $t = 40$ min or 0.667 h, $F = 27.7$ mm (from figure S3.3)
Precipitation during this period is $i\Delta t = 100 \times 10/60 = 16.7$ mm
The amount infiltrated during this period ΔF = 27.7 mm - 17.9 mm = 9.8 mm.
The amount of rain that does not infiltrate: $i\Delta t - \Delta F$ = 16.7 mm - 9.8 mm = 6.9 mm.
Depression storage = 0
Runoff = 6.9 mm.
t = 40-50 min
$i > K_s$, surface accumulation continues.
At the end of this period, when $t = 50$ min or 0.833 h, $F = 36.2$ mm (from figure S3.3)
Precipitation during this period is $i\Delta t = 80 \times 10/60 = 13.3$ mm
The amount infiltrated during this period ΔF = 36.2 mm - 27.7 mm = 8.5 mm.
The amount of rain that does not infiltrate: $i\Delta t - \Delta F$ = 13.3 mm - 8.5 mm = 4.8 mm.
Depression storage = 0
Runoff = 4.8 mm.
t = 50-60 min
$i < K_s$, Under these circumstances, if there is sufficient infiltration capacity, some of the rain may also infiltrate.
At the end of this period, when $t = 60$ min or 1 h, $F = 43.9$ mm (from figure S3.3)
Precipitation during this period is $i\Delta t = 10 \times 10/60 = 1.7$ mm
The amount infiltrated during this period ΔF = 43.9 mm - 36.2 mm = 7.7 mm.
The amount of rain that does not infiltrate: $i\Delta t - \Delta F$ = 1.7 mm - 7.7 mm = - 6.0 mm.
Depression storage = 0
Runoff = 0.0 mm.

Table S3.3: Solution of example 3.3

Time		F	Δt	ΔF	i	$i\Delta t$	$i\Delta t - \Delta F$	Runoff
(min)	(h)	(mm)	(h)	(mm)	(mm/h)	(mm)	(mm)	(mm)
1	2	3	4	5	6	7	8	9
0	0.000	0.0						
			0.167	1.7	10	1.7	0.0	0.0
10	0.167	1.7						
			0.167	3.3	20	3.3	0.0	0.0
20	0.333	5.0						
			0.167	12.9	80	13.3	0.4	0.4
30	0.500	17.9						
			0.167	9.8	100	16.7	6.9	6.9
40	0.667	27.7						
			0.167	8.5	80	13.3	4.8	4.8
50	0.833	36.2						
			0.167	7.7	10	1.7	-6.0	0.0
60	1.000	43.9						

■

3.1.4 Volumetric Runoff Coefficient

The runoff coefficient (C) for a given surface is the ratio of the volume of water that runs off the surface to the volume of water that falls on it.

This coefficient globally accounts for runoff losses, which consist of:

- evaporation, which varies with climate and season;
- wetting and infiltration, which vary with soil type;
- depression storage.

Table 3.4 presents the runoff coefficients for various surfaces, and Table 3.5 shows the runoff coefficients for various neighborhoods.

Table 3.4: Runoff coefficients for various surfaces (*Brière*, 2012).

Surface	C
Concrete or asphalt pavement	0.70-0.95
Brick pavement	0.70-0.85
Roof	0.75-0.95
Grassland, sandy soil	
- flat (slope $< 2\%$)	0.05-0.10
- moderate slope (2 to 7%)	0.10-0.15
- steep slope ($> 7\%$)	0.15-0.20
Grassland, dense soil	
- flat (slope $< 2\%$)	0.13-0.17
- moderate slope (2 to 7%)	0.18-0.22
- steep slope ($> 7\%$)	0.25-0.35
Gravel driveway	0.15-0.30

Table 3.5: Runoff coefficients for various neighborhoods (*Brière*, 2012).

Neighborhood	C
Commercial	
-downtown	0.70-0.95
-suburban	0.50-0.70
Residential	
-single-family homes	0.30-0.50
-detached multi-family homes	0.40-0.60
-row houses	0.60-0.75
-lightly developed suburban	0.10-0.25
-suburban	0.25-0.40
-apartment buildings	0.50-0.70
Industrial	
-moderate density	0.50-0.80
-high density	0.60-0.90
Parks, cemeteries, meadows	0.10-0.25
Playgrounds	0.25-0.35
Vacant land	0.10-0.30

In general, the drained areas are divided into partial areas, A_j (roofs, pavements, etc.) to which the corresponding coefficients, C_j, are applied. The equivalent runoff coefficient is then calculated using the following equation:

$$C = \frac{\sum C_j A_j}{\sum A_j} \tag{3.20}$$

The runoff coefficient can vary with the duration of the storm: it is lower at the beginning of the rain because some of the water infiltrates into the soil and some fills surface depressions. One can assume a coefficient C that varies with time $C = C(t)$ and depends on the intensity $i(t)$ (*Bourrier et Claudon*, 1981).

$$C = \frac{0,98t}{4,53+t}P + \frac{0,78t}{31,17+t}(1-P) \tag{3.21}$$

where:

- C: runoff coefficient,
- t: time elapsed from the start of precipitation,
- P: percentage of impervious surfaces (Values of coefficient C from Table 3.4).

For urban watersheds, the runoff coefficient can be estimated using equation (3.22), initially developed in the Wallingford procedure (*Butler et al.*, 2018).

$$C = \frac{PR}{TI} \tag{3.22}$$

where:

- PR: percentage runoff from the entire sub-basin (%),

$$\begin{array}{ll} PR = 0,829\,TI + 25,0\,SOL + 0,078\,DHB - 20,7 & \text{if } PR > 0,4\,TI \\ PR = 0,4\,TI & \text{if } PR \leq 0,4\,TI \end{array} \tag{3.23}$$

- TI: imperviousness rate (%),

$$TI = \frac{A_{imp}}{A} \times 100 \tag{3.24}$$

- A_{imp}: impervious area (roofs and paved surfaces) (ha),
- A: total area of the watershed (ha),
- SOL: geological permeability of the soil, $SOL = 0.15$ for very permeable soil (sand) and $SOL = 0.50$ for very clayey soil,
- DHB: soil moisture content just before the event, $DHB = 20$ for very dry soil, and $DHB = 140$ for very wet soil.

Using eq. (3.23) would require adjustment of the numerical parameters.

Example 3.4 A drainage area consists of 42% lawn ($C = 0.3$) and 58% paved surface ($C = 0.9$). The point under design has a time of concentration of 20 minutes. The total storm duration is 3 hours. Determine the adjusted value of C for prior rainfall. ■

Solution 3.4

The runoff coefficient varies here with the duration of the storm.

The composite (equivalent) runoff coefficient:

$C = \frac{\sum C_j A_j}{\sum A_j} = \frac{0,42 \times 0.30 + 0,58 \times 0.9}{0,42+0,58} = 0.65.$

The rise time $= \frac{3}{2} = 1.5$ h or 90 min.

Start time of project rainfall of 20 minutes from the start of the 3-hour storm, $t = 90 - \frac{1}{2}20 = 80$ min.

End time of project rainfall of 20 minutes from the start of the 3-hour storm, $t = 90 + \frac{1}{2}20 = 100$ min.

From eq.(3.21) : $C = \frac{0.98t}{4.53+t}P + \frac{0.78t}{31.17+t}(1-P)$

For $t = 80$ min : $C = \frac{0.98 \times 80}{4.53+80}0.65 + \frac{0.78 \times 80}{31.17+80}(1-0.65) = 0.80$

For $t = 100$ min $C = \frac{0.98 \times 100}{4.53+100}0.65 + \frac{0.78 \times 100}{31.17+100}(1-0.65) = 0.82$

The average gives $\boxed{C = 0.81}$.

Example 3.5 Compute the net rainfall for the storm in example 3.1. The rain falls on a basin that is 78% impermeable, with a geological permeability degree of 0.25 and a soil moisture content just before the event of around 40.

Solution 3.5

We have: $TI = 78\%$, $SOL = 0.25$ and $DHB = 40$

Applying eq. (3.23) gives:

$PR = 0.829\ TI + 25.0\ SOL + 0.078\ DHB - 20.7$

$PR = 0.829 \times 78 + 25.0 \times 0.25 + 0.078 \times 40 - 20.7 = 53\%$

The equation is valid since $[PR = 53] > [0.4 \times 78 = 31]$

Total net precipitation height = 5.5 mm (= the sum of net rainfall heights from table S3.1).

Runoff height $= 0.53 \times 5.5 = 2.9$ mm

Runoff loss = 5.5–2.9 = 2.6 mm

Continuous loss $= 2.6/0.5 = 5.2$ mm/h (over 30 minutes)

The hyetograph of effective rainfall is as follows:

Table S3.5

Time (min)	0–10	10–20	20–30	30–40
Net rainfall (mm/h)	0	9	18	6
Effective rainfall (direct runoff) (mm/h)	0	3.8	12.8	0.8

R Runoff height = (3.8 + 12.8 + 0.8)10/60 = 2.9 mm.

3.1.5 Time of Concentration

The time of concentration, t_c, for a watershed is the longest time it takes for runoff water from the watershed to reach the outlet (*Brière*, 2012). It is given by:

$$t_c = t_e + t_f \tag{3.25}$$

where t_e is the time of entry (or surface runoff time) and t_f is the network time of flow (or travel time in conduits).

3.1.5.1 Time of Entry

The time of entry, t_e, for an urban sub-watershed is the longest time it takes for runoff water from the watershed to reach the storm drain. The time of entry value depends on (*Brière*, 2012):

- the average slope of the ground surface towards the storm drain;
- the distance the water must travel on the surface to reach the storm drain;

- the nature of the surface over which the water must flow.

The most popular equations for calculating time of entry are shown in Table 3.6.

Table 3.6: Entry time calculation formulas.

N°	Name	Indications	Formula	
1	*Kirpich* (1940)	f_s: factor related to surface (Table 3.7)	$t_e = 0.0195 f_s \frac{L^{0.77}}{S_0^{0.385}}$	(3.26)
2	*Kerby* (1959)	Applicable if $L < 365$ m r: roughness coefficient depending on the surface (Table 3.8)	$t_e = 1.44 \left(\frac{L \cdot r}{S_0^{0.5}}\right)^{0.467}$	(3.27)
3	Kinematic Wave	n: *Manning* roughness coefficient (Table 3.9)	$t_e = 6.99 \frac{(L \cdot n)^{0.6}}{i^{0.4} S_0^{0.3}}$	(3.28)
4	*Izzard* (1944)	Applicable if $iL < 3.9$ m^2/h $K = \frac{2.8 \times 10^{-6} i + C_r}{S_0^{1/3}}$ C_r: retardation coefficient (Table 3.10)	$t_e = \frac{530 K L^{1/3}}{i^{2/3}}$	(3.29)
5	*Williams* (1922)		$t_e = 0.023 \frac{L}{A^{0.1} S_0^{0.2}}$	(3.30)

where t_e = time of entry (min), i = rainfall intensity (mm/h), L = maximum distance traveled on the surface (m), S_0 = average slope of the water path (m/m), and A = drained area (ha).

Table 3.7: Values of f_s in the *Kirpich* equation, eq. (3.26) (*Brière*, 2012).

Surface	f_s
Graded soil with a flat surface (rural basin)	1.0
Grass surface	2.0
Concrete or asphalt surface	0.4
Well-maintained grass shoulders	1.0
Runoff in a concrete channel	0.2

3.1.5.2 Time of Flow

In a pipe of length L (m) and diameter d (m), the time of flow t_f is obtained after calculating the velocity (V) using the *Manning-Strickler* method:

$$t_f = \frac{L}{V} \tag{3.31}$$

Table 3.8: Values of r in the *Kerby* equation, eq. (3.27) (*Chin*, 2013b).

Surface	r
Smooth pavement	0.02
Asphalt/concrete	0.05 - 0.15
Smooth bare packed soil, free of stones	0.10
Grass	0.20
Pasture	0.40
Dense grass/turf	0.17-0.80
Deciduous timberland	0.60
Deciduous timberland (w/ deep forest litter)	0.80

Table 3.9: Values of n in the kinematic wave equation, eq. (3.28) (*Chin*, 2013b).

Surface Type	n	Range
Smooth concrete	0.011	0.01-0.014
Bare sand	0.01	0.01-0.016
Asphalt	0.012	0.010-0.018
Bare clay	0.012	0.010-0.016
Smooth earth	0.018	0.015-0.021
Graveled surface	0.02	0.012-0.030
Grass (short prairie)	0.15	0.10-0.25
Light turf	0.20	
Lawns	0.25	0.20-0.30
Dense turf	0.35	0.30-0.35

Table 3.10: Values of C_r in the *Izzard* equation, eq. (3.30) (*Chin*, 2013b).

Surface	C_r
Very smooth asphalt	0.0070
Tar and sand pavement	0.0075
Tar and gravel pavement	0.017
Concrete	0.012
Closely clipped sod	0.016

Example 3.6 Consider the urbanized watershed illustrated in Figure E3.6. Determine the time of concentration at point C using different methods. The average flow velocity in the channel is 0.75 m/s and the intensity $i = 75$ mm/h. ■

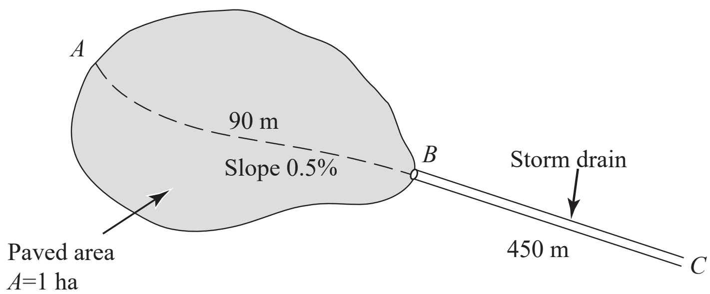


Figure E3.6

Solution 3.6

a) Time of Entry:

1. *Kirpich* Formula

Given $L = 90$ m, $S_0 = 0,005$, and $f_s = 0,4$ (asphalt surface, Table 3.7).

$t_e = 0,0195 f \dfrac{L^{0,77}}{S_0^{0,385}} = 0,0195 \times 0,4 \dfrac{90^{0,77}}{0,005^{0,385}}$, $\boxed{t_e = 1,9 \text{ min}}$.

2. *Kerby* Formula

Given $L = 90$ m < 365 m, hence *Kerby* equation is applicable, $S_0 = 0,005$, and $r = 0,05$ (asphalt surface, Table 3.8).

$t_e = 1,44 \left(\dfrac{L \cdot r}{S_0^{0,5}}\right)^{0,467} = 1,44 \left(\dfrac{90 \times 0,05}{0,005^{0,5}}\right)^{0,467}$, $\boxed{t_e = 10 \text{ min}}$.

3. Kinematic Wave Formula

Given $L = 90$ m, $S_0 = 0,005$, $i = 75$ mm/h, and $n = 0,012$ (asphalt surface, Table 3.9).

$t_e = 6,99 \dfrac{(L \cdot n)^{0,6}}{i^{0,4} S_0^{0,3}} = 6,99 \dfrac{(90 \times 0,012)^{0,6}}{75^{0,4} 0,005^{0,3}}$, $\boxed{t_e = 6,4 \text{ min}}$.

4. *Izzard* Formula

Given $L = 90$ m, $S_0 = 0,005$, and $i = 75$ mm/h. Since $iL = 0,075 \times 90 = 6,75 > 3,9$ m^2/h, *Izzard* equation is not applicable.

5. *Bransby-Williams* Formula

Given $L = 90$ m, $S_0 = 0,005$, and $A = 1$ ha.

$t_e = 0,023 \dfrac{L}{A^{0,1} S_0^{0,2}} = 0,023 \dfrac{90}{1^{0,1} \times 0,005^{0,2}}$, $\boxed{t_e = 6 \text{ min}}$.

b) Time of Flow :

$t_f = \dfrac{L}{V} = \dfrac{450}{0,75} = 600$ s, $\boxed{t_f = 10 \text{ min}}$.

Therefore, the time of concentration $t_c = t_e + t_f$ varies between 11.9 and 20 minutes depending on the calculation method. ■

3.1.6 Calculation Methods

The problem of evaluating stormwater runoff in watersheds has long been the subject of numerous theoretical studies, particularly concerning the practices of urban hydrology engineers. Three main methods are commonly used:

- Rational Method;
- Superficial Method;
- NRCS (SCS) TR-55 Method.

3.1.6.1 Rational Method

For a rainfall event with intensity i (mm/h) over an area A (ha) with a weighted runoff coefficient C, the resulting runoff flow rate (in m^3/s) is expressed by the equation (*Gomella et Guerrée*, 1986):

$$Q_{Storm} = \frac{1}{360} C \cdot i \cdot A \tag{3.32}$$

Assumptions

- The rainfall intensity is constant throughout the entire duration of the event;
- The rainfall intensity is uniform over the entire watershed;
- The rainfall occurs over the entire watershed for its entire duration;

- The runoff coefficient is constant throughout the entire duration of the rainfall;
- For a given recurrence interval, the maximum area on which rainfall occurs has a time of concentration equal to the duration of the rainfall;
- For a given recurrence interval, the maximum runoff flow rate corresponds to precipitation whose duration is equal to the time of concentration of the watershed;
- The recurrence of the maximum flow rate is the same as that of the rainfall used for the calculations;
- Losses vary linearly with precipitation and are incorporated into the runoff coefficient (*Brière*, 2012).

Validity Limits

Table 3.11: Validity limits of the rational method.

Parameter	Minimum	Maximum
Slope (m/m)	0.002	0.050
Runoff coefficient	0.2	1.0
Watershed area (ha)	0	5

Example 3.7

For the drainage system illustrated in Figure E3.7, determine the design flow rate for each sewer section using the rational method. The rainfall intensity (mm/min) is given by $i = 52.42/(t_c + 300)^{0.58}$, where t_c is in minutes. The flow from each area is indicated by an arrow. The average velocity in the pipes is 1.20 m/s.

Table E3.7.1

Catchment	Area (ha)	C	t_e (min)
A	0.4	0.7	10
B	0.5	0.7	10
C	0.3	0.80	5
D	0.3	0.80	5
E	0.4	0.65	10
F	0.5	0.60	10
G	0.6	0.60	10

Table E3.7.2

Pipe	Length (m)	Slope (m/m)
1-3	150	0.008
2-3	460	0.01
3-5	180	0.0075
4-5	230	0.01
5-6	180	0.0075

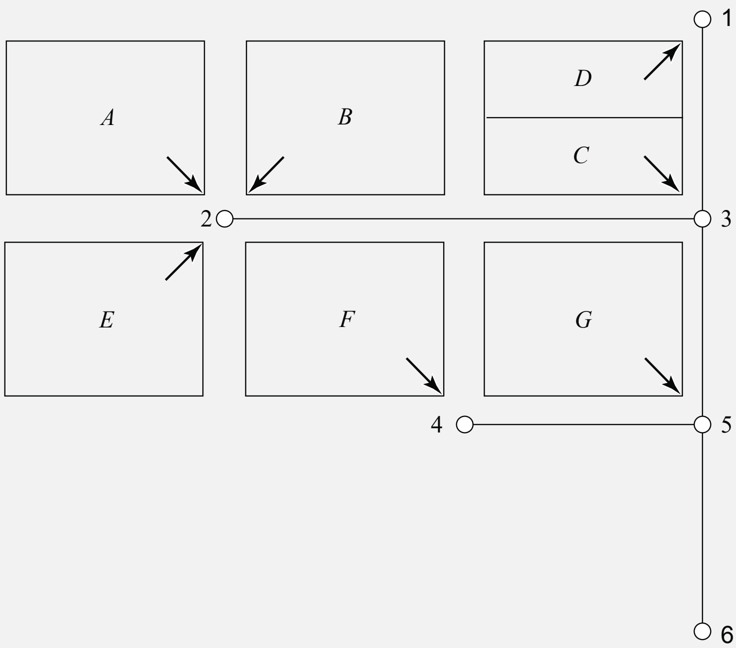


Figure E3.7

■

Solution 3.7

Node 1: Pipe 1-3

$A_D = 0.3$ ha, $C_D = 0.8$, so $C_D A_D = 0.8 \times 0.3 = 0.24$ ha.

$t_c = t_e = 5$ min

$$i = \frac{52.42}{(t_c + 300)^{0.58}} = \frac{52.42}{(5+300)^{0.58}} = 1.899 \text{ mm/min.}$$

$$Q = \frac{CAi}{360} = \frac{0.24(1.899 \times 60)}{360}, \boxed{Q = 0.076 \text{ m}^3/\text{s}}.$$

$$L = 150 \text{ m and } V = 1.20 \text{ m/s, so } t_f = \frac{L}{60V} = \frac{150}{60 \times 1.20} = 2.08 \text{ min.}$$

$t_c = t_e + t_f = 5 + 2.08 = 7.08$ min at downstream node 3.

Node 2: Pipe 2-3

$\sum CA = C_A A_A + C_B A_B + C_E A_E = 0.7 \times 0.4 + 0.7 \times 0.5 + 0.65 \times 0.4 = 0.890$ ha,

$t_c = \max(t_{eA}; t_{eB}; t_{eE}) = 10$ min.

$$i = \frac{52.42}{(t_c + 300)^{0.58}} = \frac{52.42}{(10+300)^{0.58}} = 1.882 \text{ mm/min.}$$

$$Q = \frac{(\sum CA)i}{360} = \frac{0.890(1.882 \times 60)}{360}, \boxed{Q = 0.279 \text{ m}^3/\text{s}}.$$

$$L = 460 \text{ m and } V = 1.20 \text{ m/s, so } t_f = \frac{L}{60V} = \frac{460}{60 \times 1.20} = 6.39 \text{ min.}$$

$t_c = t_e + t_f = 10 + 6.39 = 16.39$ min at downstream node 3.

Node 3: Pipe 3-5

$\sum CA = C_D A_D + 0.890 + C_C A_C = 0.24 + 0.890 + 0.80 \times 0.3 = 1.37$ ha,

$t_c = \max(7.08; 16.39; 5) = 16.39$ min.

$$i = \frac{52.42}{(t_c + 300)^{0.58}} = \frac{52.42}{(16.39+300)^{0.58}} = 1.859 \text{ mm/min.}$$

$$Q = \frac{(\sum CA)i}{360} = \frac{1.37(1.859 \times 60)}{360}, \boxed{Q = 0.425 \text{ m}^3/\text{s}}.$$

$$L = 180 \text{ m and } V = 1.20 \text{ m/s, so } t_f = \frac{L}{60V} = \frac{180}{60 \times 1.20} = 2.50 \text{ min.}$$

$t_c = t_e + t_f = 16.39 + 2.50 = 18.89$ min at downstream node 5.

Node 4: Pipe 4-5

$A_F = 0.5$ ha, $C_F = 0.6$, so $C_F A_F = 0.6 \times 0.5 = 0.30$ ha.

$t_c = t_e = 10$ min

$$i = \frac{52.42}{(t_c + 300)^{0.58}} = \frac{52.42}{(10+300)^{0.58}} = 1.882 \text{ mm/min.}$$

$$Q = \frac{CAi}{360} = \frac{0.30(1.882 \times 60)}{360}, \boxed{Q = 0.094 \text{ m}^3/\text{s}}.$$

$$L = 230 \text{ m and } V = 1.20 \text{ m/s, so } t_f = \frac{L}{60V} = \frac{230}{60 \times 1.20} = 3.19 \text{ min.}$$

$t_c = t_e + t_f = 10 + 3.19 = 13.19$ min at downstream node 5.

Node 5: Pipe 5-6

$\sum CA = C_F A_F + 1.37 + C_G A_G = 0.3 + 1.37 + 0.60 \times 0.6 = 2.03$ ha,

$t_c = \max(18.89; 10; 13.19) = 18.89$ min.

$$i = \frac{52.42}{(t_c + 300)^{0.58}} = \frac{52.42}{(18.89+300)^{0.58}} = 1.851 \text{ mm/min.}$$

$$Q = \frac{(\sum CA)i}{360} = \frac{2.03(1.851 \times 60)}{360}, \boxed{Q = 0.626 \text{ m}^3/\text{s}}.$$

$$L = 180 \text{ m and } V = 1.20 \text{ m/s, so } t_f = \frac{L}{60V} = \frac{180}{60 \times 1.20} = 2.50 \text{ min.}$$

$t_c = t_e + t_f = 18.89 + 2.50 = 21.39$ min at downstream node 6.

The previous calculations are summarized in Table S3.7. ■

Table S3.7: Solution to Exercise 3.7 using the rational method.

Node	A	C	CA	$\sum CA$	Path	time			i	Q
						t_e	t_f	t_c		
	(m^2)		(m^2)	(m^2)		(min)	(min)	(min)	(mm/min)	(m^3/s)
1	0.3	0.80	0.24	0.24	D-1	5	0	5	1.899	0.076
2	0.4	0.70	0.28	0.28	A-2	10	0	10		
	0.5	0.70	0.35	0.63	B-2	10	0	10		
	0.4	0.65	0.26	0.89	E-2	10	0	10	1.882	0.279
3	0.3	0.80	0.24	1.37	C-3	5	0	5		
					1-3	5	2.08	7.08		
					2-3	10	6.39	**16.39**	1.859	0.425
4	0.5	0.60	0.30	0.30	F-4	10	0	10	1.882	0.094
5	0.6	0.60	0.36	2.03	G-5	10	0	10		
					3-5	16.39	2.50	**18.89**	1.851	0.626
					4-5	10	3.19	13.19		

3.1.6.2 *Caquot* Method (superficial)

The *Caquot* method represents an evolution of the rational method by avoiding limitations related to the estimation of the time of concentration and by accounting for the water storage possibilities in the catchment area. The superficial formula for the frequency of exceedance flow rate, F, is given as follows (*Deutsch et Tassin*, 2000):

$$Q_{Storm} = m \cdot K \cdot S_0^{\alpha} \cdot C^{\beta} \cdot A^{\gamma} \tag{3.33}$$

With

- Q_{Storm} is the elementary discharge of the catchment (in m^3/s);
- S_0 is the average slope of the collector of the considered catchment (in m/m);
- C is the runoff coefficient;
- A is the catchment area (in hectares);
- m is a multiplicative coefficient.

$$m = \left(\frac{M}{2}\right)^{0.84 \times b \times \beta} \tag{3.34}$$

M is the elongation coefficient:

$$M = \max\left(0.8; \frac{L}{\sqrt{A}}\right) \tag{3.35}$$

β, K, α, and γ are the *Caquot* coefficients.

$$\beta = \frac{1}{1 + 0.287 \times b} \tag{3.36a}$$

$$K = \left[\frac{a \times 0.5^b}{6.6}\right]^{\beta} \tag{3.36b}$$

$$\alpha = -0.41 \times b \times \beta \tag{3.36c}$$

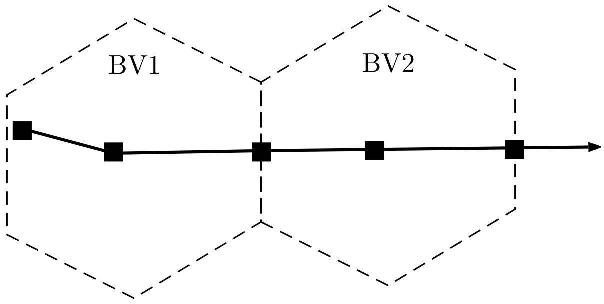
$$\gamma = (0.95 + 0.507 \times b)\,\beta \tag{3.36d}$$

Validity Limits

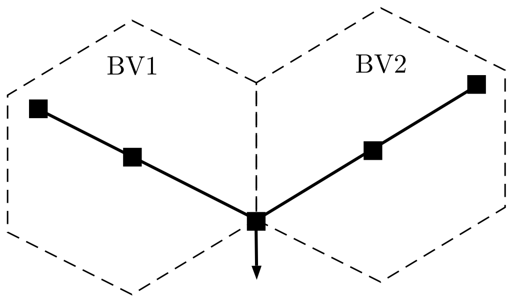
The validity limits of the *Caquot* formula are given in the following table:

Table 3.12: Validity limits of the *Caquot* method (*Deutsch et Tassin*, 2000).

Parameter	Minimum	Maximum
Slope (m/m)	0.002	0.050
Runoff coefficient	0.2	1.0
Catchment area (ha)	5	200



(a) In series



(b) In parallel

Figure 3.6: Catchment assemblage.

Catchment Assemblage

Once the elementary catchments have been determined with all their characteristics, they need to be assembled either in series or in parallel. The catchments will thus be assembled in pairs from upstream to downstream to determine the flow rates generated by certain parts and by the entire system.

Assemblages in series occur when two catchments follow each other on the same hydraulic path (Fig. 3.6a). Parallel assemblages occur when two catchments converge at the same outlet but have different hydraulic paths (Fig. 3.6b).

Regardless of the chosen flow calculation formula, grouping heterogeneous sub-catchments requires the use of equivalence formulas (see Table 3.13) for the parameters A (area), C (runoff coefficient), S_0 (slope), and M (elongation) (*Deutsch et Tassin*, 2000).

From upstream to downstream, sub-catchments are grouped to ultimately form an overall catchment, allowing the calculation of the flow rate at the outlet (Table 3.13).

In parallel assemblages, the equivalent flow rate must be compared to the sum of the two involved catchments, and the lower of the two values should be adopted.

Table 3.13: Equivalent parameters for catchment grouping (*Deutsch et Tassin*, 2000).

Equivalent Parameters	A_{eq}	C_{eq}	$S_{0_{eq}}$	M_{eq}
Catchments in series	$\sum A_j$	$\frac{\sum C_j A_j}{\sum A_j}$	$\left[\frac{\sum L_j}{\sum \frac{L_j}{\sqrt{S_{0_j}}}}\right]^2$	$\frac{\sum L_j}{\sqrt{\sum A_j}}$
Catchments in parallel	$\sum A_j$	$\frac{\sum C_j A_j}{\sum A_j}$	$\frac{\sum S_{0_j} Q_j}{\sum Q_j}$	$\frac{L(Q_{\max})}{\sqrt{\sum A_j}}$

$L(Q_{\max})$ =length of the longest path in the catchment with the highest individual peak flow rate.

Example 3.8 An industrial zone is under design, and the aim is to determine the rainfall water flow rates for the sub-catchments (1 to 4) and at the outlet. The *Caquot* method will be applied for a ten-year return period. The coefficients a and b of the *Montana* law are assumed as: $a = 3.16$ and $b = -0.64$.

The characteristics of the sub-catchments are summarized in Table E3.8.

Table E3.8

Catchment	A (ha)	C	S_0(m/m)	L_{max} (m)
A	9	0.60	5×10^{-3}	360
B	7	0.60	5×10^{-3}	360
C	15	0.60	5×10^{-3}	340
D	20	0.60	4×10^{-3}	350

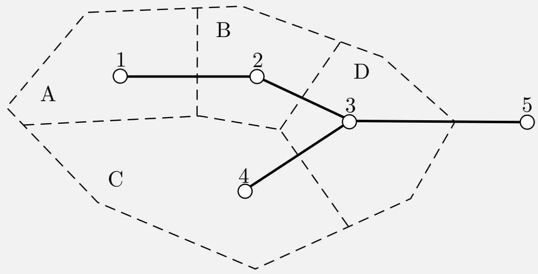


Figure E3.8

Solution 3.8

With $a = 3.16$ and $b = -0.64$, using equations (3.36), we calculate the *Caquot* coefficients:

$$\beta = \frac{1}{1+0.287\times b} = \frac{1}{1+0.287(-0.64)} = 1.225.$$

$$K = \left[\frac{a\times 0.5^b}{6.6}\right]^\beta = \left[\frac{3.16\times 0.5^{-0.64}}{6.6}\right]^{1.225} = 0.7.$$

$$\alpha = -0.41\times b\times\beta = -0.41(-0.64)1.225 = 0.321.$$

$$\gamma = (0.95+0.507\times b)\,\beta = (0.95+0.507(-0.64))\,1.225 = 0.766.$$

The multiplier coefficient: $m = \left(\frac{M}{2}\right)^{0.84\times b\times\beta} = \left(\frac{M}{2}\right)^{0.84(-0.64)1.225} = \left(\frac{M}{2}\right)^{-0.659}$

where: $M = \max\left(0.8; L/\sqrt{A}\right)$

from eq. (3.33): $Q_{Storm} = m\cdot K\cdot S_0^\alpha\cdot C^\beta\cdot A^\gamma = 0.7m\cdot S_0^{0.321}\cdot C^{1.225}\cdot A^{0.766}$

Catchment A: pipe 1-2

$A_A = 9$ ha, $C_A = 0.60$, $S_{0_A} = 5\times10^{-3}$ and $L_{max_A} = 360$ m

$$M_A = \max\left(0.8; 360/\sqrt{9\times10^4}\right) = \max(0.8; 1.2) = 1.2$$

$$m_A = \left(\frac{1.2}{2}\right)^{-0.659} = 1.4$$

$$Q_A = 1.4\times0.7\times0.005^{0.321}\times0.6^{1.225}\times9^{0.766},$$

$\boxed{Q_A = 0.515\ \mathrm{m^3/s}}$ in pipe 1-2.

Catchment AB: pipe 2-3

Both catchments A and B are in series to form catchment AB. Applying the equations from Table 3.13 on page 33, we obtain:

$$A_{AB} = A_A + A_B = 9+7 = 16\text{ ha.}$$

$$C_{AB} = \frac{C_A A_A + C_B A_B}{A_A + A_B} = \frac{0.6\times9+0.6\times7}{9+7} = 0.6.$$

$$S_{0_{AB}} = \left[\frac{L_A+L_B}{\frac{L_A}{\sqrt{S_{0_A}}}+\frac{L_B}{\sqrt{S_{0_B}}}}\right]^2 = \left[\frac{360+360}{\frac{360}{\sqrt{5\times10^{-3}}}+\frac{360}{\sqrt{5\times10^{-3}}}}\right]^2 = 0.005\ \mathrm{m/m}$$

$$M_{AB} = \max\begin{cases} 0.8 \\ \dfrac{L_A+L_B}{\sqrt{A_A+A_B}} = \dfrac{360+360}{\sqrt{16\times10^4}} = 1.8 \end{cases},\ M_{AB} = 1.8$$

$m_{AB} = \left(\dfrac{1.8}{2}\right)^{-0.659} = 1.07$

$Q_{AB} = 1.07 \times 0.7 \times 0.005^{0.321} \times 0.6^{1.225} \times 16^{0.766}$,

$\boxed{Q_{AB} = 0.612 \text{ m}^3/\text{s}}$ in pipe 2-3.

Catchment C: pipe 4-3

$A_C = 15$ ha, $C_C = 0.60$, $S_{0_C} = 5 \times 10^{-3}$ and $L_{\max_C} = 340$ m

$M_C = \max\left(0.8; 340/\sqrt{15 \times 10^4}\right) = \max(0.8; 0.88) = 0.88$

$m_C = \left(\dfrac{0.88}{2}\right)^{-0.659} = 1.72$

$Q_C = 1.72 \times 0.7 \times 0.005^{0.321} \times 0.6^{1.225} \times 15^{0.766}$,

$\boxed{Q_C = 0.936 \text{ m}^3/\text{s}}$ in pipe 4-3.

Catchment ABCD: pipe 3-5

For this pipe, we need to series-connect catchment ABC and catchment D. Given that catchments ABC are composed of catchments AB and C arranged in parallel.

Catchment ABC

Applying the equations from Table 3.13 on page 33, we obtain:

$A_{ABC} = A_{AB} + A_C = 16 + 15 = 31$ ha.

$$C_{ABC} = \frac{C_{AB}A_{AB} + C_C A_C}{A_{AB} + A_C} = \frac{0.6 \times 16 + 0.6 \times 15}{16 + 15} = 0.6.$$

$$S_{0_{ABC}} = \frac{S_{0_{AB}} Q_{AB} + S_{0_C} Q_C}{Q_{AB} + Q_C} = \frac{0.005 \times 0.612 + 0.005 \times 0.936}{0.612 + 0.936} = 0.005 \text{ m/m}$$

$$M_{ABC} = \max \begin{cases} 0.8 \\ \dfrac{L(Q_{\max})}{\sqrt{A_{ABC}}} = \dfrac{340}{\sqrt{31 \times 10^4}} = 0.61 \end{cases}, M_{ABC} = 0.8$$

$$m_{ABC} = \left(\frac{M_{ABC}}{2}\right)^{-0.659} = \left(\frac{0.8}{2}\right)^{-0.659} = 1.83$$

$Q_{ABC} = 1.83 \times 0.7 \times 0.005^{0.321} \times 0.6^{1.225} \times 31^{0.766} = 1.732$,

When parallel assemblies, it is necessary to compare this last value to that resulting from the sum of the two interested catchments and to adopt the lower of the two.

We have $Q_{AB} + Q_C = 0.612 + 0.936 = 1.548 < 1.732$.

Adopt $\boxed{Q_{ABC} = 1.548 \text{ m}^3/\text{s}}$.

Catchment ABCD

Catchments ABC and D are in series to form catchment ABCD. Applying the equations from Table 3.13 on page 33, we obtain:

$A_{ABCD} = A_{ABC} + A_D = 31 + 20 = 51$ ha.

$$C_{ABCD} = \frac{C_{ABC}A_{ABC} + C_D A_D}{A_{ABC} + A_D} = \frac{0.6 \times 31 + 0.6 \times 20}{31 + 20} = 0.6.$$

Conduit 3-5 is supplied by catchment D as well as by the peak flows from both branches (AB and C). If we consider that the peak flows arrive at the same time, pipe 3-5 is oversized. Since the two peak flows have a low probability of occurring simultaneously, we calculate the equivalent slope of catchment ABCD following the path of the strongest flow.

We have: $Q_{AB} < Q_C$, so the value of $S_{0_{ABCD}}$ to consider is calculated based on the series assembly of catchments C and D

$$S_{0_{ABCD}} = \left[\frac{L_C + L_D}{\frac{L_C}{\sqrt{S_{0_C}}} + \frac{L_D}{\sqrt{S_{0_D}}}}\right]^2 = \left[\frac{340 + 350}{\frac{340}{\sqrt{5 \times 10^{-3}}} + \frac{350}{\sqrt{4 \times 10^{-3}}}}\right]^2 = 0.0044 \text{ m/m}$$

$$M_{ABCD} = \max\begin{cases} 0.8 \\ \dfrac{L_C + L_D}{\sqrt{A_{ABCD}}} = \dfrac{340+350}{\sqrt{51\times10^4}} = 0.966 \end{cases}, M_{AB} = 0.966$$

$$m_{ABCD} = \left(\frac{0.966}{2}\right)^{-0.659} = 1.615$$

$Q_{AB} = 1.615 \times 0.7 \times 0.0044^{0.321} \times 0.6^{1.225} \times 51^{0.766}$,

$\boxed{Q_{AB} = 2.153\ \mathrm{m^3/s}}$ in pipe 3-5.

The above calculations are summarized in Table S3.8.

Table S3.8: Calculation of flow rates for Exercise 3.8.

Assembly	A (ha)	C	S_0 (m/m)	Q_{Storm} ($\mathrm{m^3/s}$)	Observation	Adopted Q_{Storm} ($\mathrm{m^3/s}$)	Section of application
A	9	0.6	0.005	0.515		0.515	1-2
AB	16	0.6	0.005	0.612		0.612	2-3
C	15	0.6	0.005	0.936		0.936	4-3
ABC	31	0.6	0.005	1.732	Adopt $Q_{AB} + Q_C$	1.548	
ABCD	51	0.6	0.0044	2.153		2.153	3-5

3.1.6.3 Modified Rational Method

This procedure extends the rational method to the development of runoff hydrographs. Runoff hydrographs are used to calculate the dimensions of detention/retention structures for specified return intervals and simultaneous discharge rates.

The Modified Rational Method (MRM) is based on the same assumptions as the rational method. An additional important assumption is that the average duration of rainfall intensity used in the MRM is equal to the duration of the storm. In the rational method, this assumption means that precipitation and runoff generated by rainfall events occurring before or after the period used to calculate average precipitation are not considered.

The Modified Rational Method (MRM) recommends including a preceding precipitation factor (C_A) in the rational formula, so it becomes:

$$Q_{Storm} = \frac{1}{360} C C_A i A \tag{3.37}$$

where C is the volumetric flow coefficient given by Eq. (3.22).

Recommended values for C_A are listed in Table 3.14. The product of $C \times C_A$ should never exceed 1.

Table 3.14: Recommended values of the coefficient C_A (*Strom et al.*, 2013).

Frequency	2 to 10	25	50	100
C_A	1.0	1.1	1.2	1.25

As illustrated in Figure 3.7, three types of hydrographs can be developed for a watershed using the MRM procedure. The type of hydrograph depends on the average duration of the storm, t, relative to the watershed's time of concentration. The following three types are possible (*Walesh*, 1991):

1. Type A: $t = t_c$; The resulting triangular hydrograph has a maximum flow rate given by Eq. (3.37). The rising and falling linear limbs each have a duration equal to t_c.

2. Type B: $t > t_c$; The resulting trapezoidal hydrograph has a maximum flow rate given by Eq. (3.37). This flow rate is reached over a duration equal to the time of concentration but is lower than

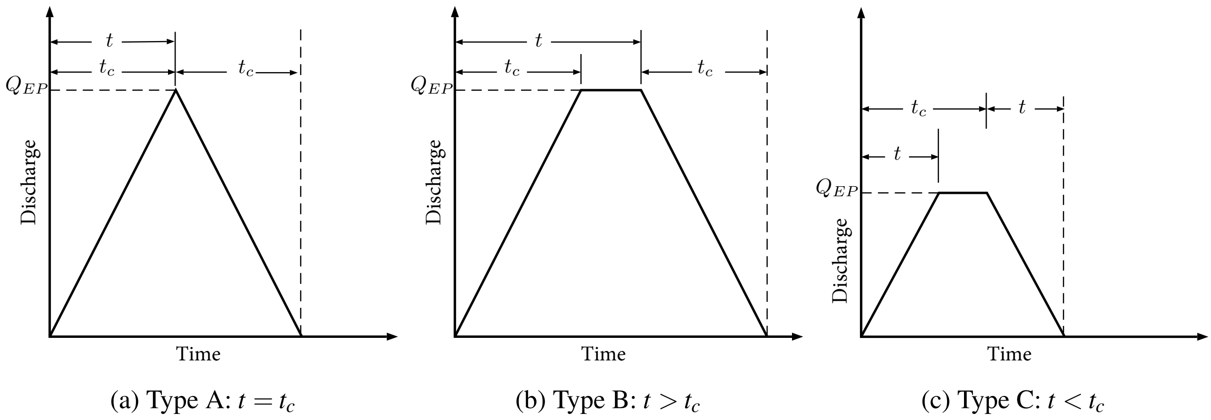


Figure 3.7: Hydrographs Types for the Modified Rational Method.

that of the Type A hydrograph, as the intensity will be less for a longer duration. Runoff flow remains constant until the end of the storm duration. The recession time is equivalent to the time of concentration.

3. Type C: $t < t_c$; The resulting trapezoidal hydrograph has a uniform maximum flow rate

$$Q_{Storm} = CC_A iA(t/t_c) \tag{3.38}$$

The maximum runoff flow rate (Q_{Storm}) is reached at the end of the storm duration. It remains constant until the end of the time of concentration. The recession time corresponds to t. The intensity of the storm is higher than that used for the Type A hydrograph. However, since the drainage area has not reached its time of concentration and maximum potential runoff rate, the reduction factor (t/T_c) is included in the equation.

Example 3.9 Establish hydrographs for 10, 25, and 50 minutes for a drainage area of 6 ha, with a volumetric runoff coefficient $C = 0.3$ and a time of concentration $t_c = 25$min. A 100-year frequency is desired. The intensity of precipitation (in mm/h) is given by the formula: $i = 674.7t^{-0.52}$ (where t is in minutes). ■

Solution 3.9

a) 10-minute Storm

For $t = 10\text{min} < t_c = 25\text{min}$, the hydrograph is of Type C.

At $t = 10$min, $i = 674.7 \times 10^{-0.52} = 203.8$mm/h

For a 100-year frequency, $C_A = 1.25$

$$Q_{Storm} = \frac{1}{360} CC_A iA\left(\frac{t}{t_c}\right) = \frac{1}{360} \times 0.3 \times 1.25 \times 203.8 \times 6 \times \left(\frac{10}{25}\right), \boxed{Q_{Storm} = 0.509\text{m}^3/\text{s}}.$$

R Although the duration of the storm is only 10 minutes, the runoff duration is 35 minutes $(t_c + t)$ (Fig. E3.9a).

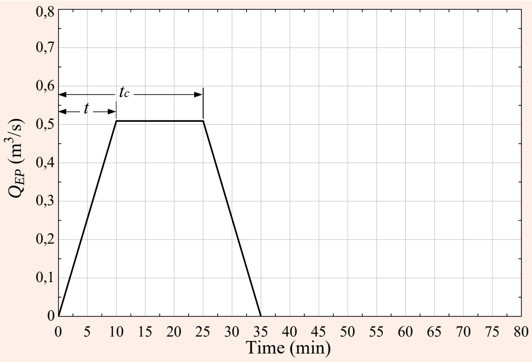


Figure E3.9a

b) 25-minute Storm

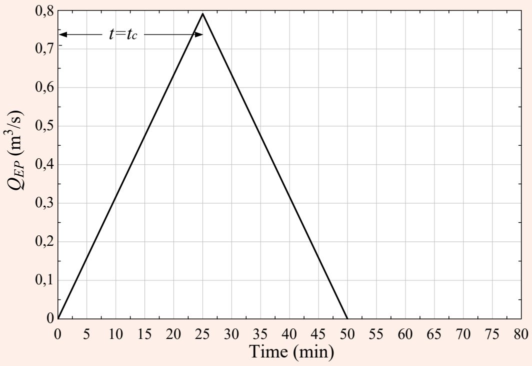


Figure E3.9b

For $t = 25\text{min} = t_c$, the hydrograph is of Type A (Fig. E3.9b)
At $t = 25\text{min}$, $i = 674.7 \times 25^{-0.52} = 126.5\text{mm/h}$

$$Q_{Storm} = \frac{1}{360} C C_A i A = \frac{1}{360} \times 0.3 \times 1.25 \times 126.5, \boxed{Q_{Storm} = 0.791\text{m}^3/\text{s}}.$$

c) 50-minute Storm

For $t = 50\text{min} > t_c$, the hydrograph is of Type B (Fig. E3.9c)
At $t = 25\text{min}$, $i = 674.7 \times 50^{-0.52} = 88.2\text{mm/h}$

$$Q_{Storm} = \frac{1}{360} C C_A i A = \frac{1}{360} \times 0.3 \times 1.25 \times 88.2, \boxed{Q_{Storm} = 0.551\text{m}^3/\text{s}}.$$

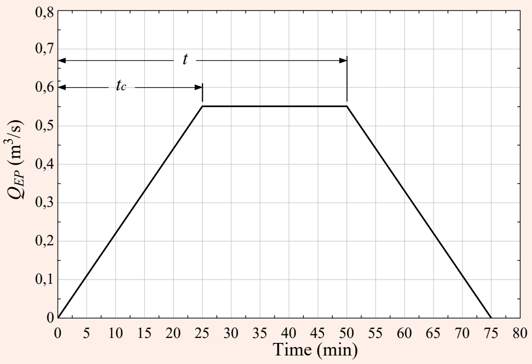


Figure E3.9c

3.1.6.4 The NRCS (SCS) TR-55 Method

Based on tests conducted on small agricultural plots, the SCS[1] (currently NRCS[2]) developed a method to establish a relationship between precipitation and runoff (*Cronshey*, 1986). This method was presented in the TR-55 report, initially published in 1975 and revised in 1986. The method, which is based on 24-hour rainfall events and time of concentrations, t_c, ranging from 0.1 h to 10 h, is applied to small and medium-sized watersheds up to 13 km^2 (*Pazwash*, 2011). This method, relying on the rational method, allows for the transformation of rainfall into peak discharge for ungauged rural watersheds, based on the runoff depth estimated by the following empirical equation (*Bourrier et Claudon*, 1981):

$$R = \frac{(P - I_a)^2}{P - I_a + S} \tag{3.39}$$

where:

- R (referred to as Q in TR-55) is the runoff depth in mm,
- P is the precipitation depth in mm of a design storm of duration d and return period $\mathcal{T}$,
- I_a is the initial abstraction in mm before runoff occurs.

$$I_a = 0.2S \tag{3.40}$$

where:

- S is the maximum potential retention in mm.

Thus, the relationship becomes:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{3.41}$$

Determining the maximum potential infiltration depth S for each sub-watershed involves characterizing the land use and soil permeability, assessed overall by the following relationship:

$$S = 25.4\left(\frac{1000}{CN} - 10\right) \quad \text{(in mm)} \tag{3.42}$$

linked to the curve number value:

$$CN = \frac{1000}{10 + S/25,4} \tag{3.43}$$

For runoff calculation, the SCS classifies soils into four hydrologic groups. These groups range from A to D, with group A being the most permeable and group D the least permeable. The following describes these soil groups and their permeability (*Anctil et al.*, 2012):

- Group A: Low runoff potential. High infiltration rates even when thoroughly wetted (Minimum infiltration rate > 7.6 mm/h). Excessively well-drained soils with high hydraulic conductivity. Less than 10% clay, more than 90% sand (depth). Examples: Sand, sandy loam, or loamy sand.
- Group B: Moderate infiltration rates when wetted (between 3.8 and 7.6 mm/h). Well-drained soils with fine to coarse texture and moderate hydraulic conductivity. Generally 10 to 20% clay, 50 to 90% sand. Examples: Loams, silty loams, and shallow well-aggregated sandy soils.
- Group C: Low infiltration rates when wetted (between 1.3 and 3.8 mm/h). Fine to very fine texture soils with low hydraulic conductivity. Generally 20 to 40% clay, less than 50% sand. Examples: Sandy clay loam.

[1] Soil Conservation Service
[2] Natural Resources Conservation Service

- Group D: High runoff potential. Very low infiltration rates when wetted (< 1.3 mm/h). Predominantly clay soils with very low hydraulic conductivity. Primarily more than 40% clay, clayey textures. Includes all soils with an impervious layer within 50 cm of the surface, and all soils with a high water table within 60 cm of the surface. Examples: Clay loam, silty clay loam, sandy clay, silty clay, or clay.

The SCS links the curve number to land cover and hydrologic soil groups. Table 3.15 presents the curve numbers for urban areas.

Table 3.15: Curve number for urban areas according to SCS (*Anctil et al.*, 2012).

Soil Description	A	B	C	D
Open spaces, lawns, golf courses, cemeteries				
Good condition: grass cover more than 75%	39	61	74	80
Fair condition: grass cover between 50% and 75%	49	69	79	84
Poor condition: grass cover less than 50%	68	79	86	89
Commercial areas (85% impervious)	89	92	94	95
Industrial areas (70% impervious)	81	88	91	93
Residential districts				
500 m^2 lots (65% impervious)	77	85	90	92
1000 m^2 lots (38% impervious)	61	75	83	87
2000 m^2 lots (25% impervious)	54	70	80	85
4000 m^2 lots (20% impervious)	51	68	79	84
Parking lots, roofs, and driveways (paved surfaces)	98	98	98	98
Roads				
Paved, with curbs and storm sewers	98	98	98	98
Gravel roads	76	85	89	91
Dirt roads	72	82	87	89
Forest land				
Sparse ground cover, little organic material	45	66	77	83
Good vegetative cover	25	55	70	77

In the TR-55 method, the peak discharge, Q_{Storm}, can be calculated using the following equation (*Gupta*, 2016; *Pazwash*, 2011):

$$Q_{Storm} = F_a q_u A R \tag{3.44}$$

where

- q_u = unit peak discharge (m^3/s/cm/km^2: m^3/s per cm of runoff per km^2 of watershed),
- A = drainage area (km^2),
- R = runoff depth over a 24-hour period (cm),
- F_a = pond and swamp adjustment factor (dimensionless) obtained from Table 3.16. The area of ponds and swamps creates conditions for flood attenuation.

The unit peak discharge is estimated using the following formula:

$$\log(q_u) = C_0 + C_1 \log_{10} t_c + C_2 (\log_{10} t_c)^2 - 2.366 \tag{3.45}$$

where C_0, C_1, and C_2 are constants obtained from Table 3.17, and t_c is expressed in hours. The values of C_0, C_1, and C_2 are functions of I_a/P and the type of precipitation (see Table 3.17). The four types of

24-hour precipitation distributions by the NRCS are presented in Figure 3.8 as the ratio of accumulated precipitation to total precipitation over time. This figure shows that Type II is the most intense and Type IA is the least intense distribution. The maximum precipitation intensities for Types II and III occur in about 12 hours, while for Types I and IA, they occur in about 8 and 10 hours, respectively. The NRCS Type II and Type III distributions are very similar, concentrating 45% and 40% of the total precipitation height within a maximum of 1 hour, respectively. Type II distribution is applicable for the most intense cases and is commonly used in many countries, even in arid and semi-arid regions. The use of SCS Type III is recommended for areas affected by hurricanes.

Table 3.16: Adjustment factor (F_a) for pond and swamp areas that are spread throughout the watershed (*Gupta*, 2016)

Percentage of pond and swamp areas	F_a
0	1.0
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

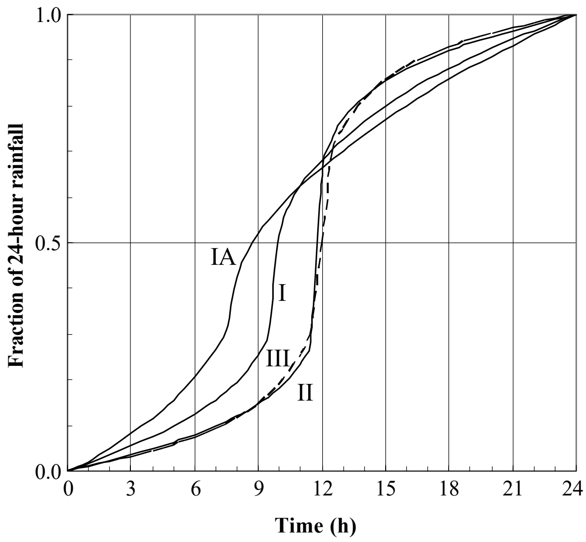


Figure 3.8: NRCS 24-hour rainfall distributions (*Cronshey*, 1986).

Example 3.10 A watershed is composed of 30% lots of 2000 m^2 with soil type B and 40% lots of 2000 m^2 with soil type C. The remaining area of the watershed consists of scattered open spaces in good hydrological condition and with soil type C. If the total area of the watershed is 1 km^2 and the time of concentration is $t_c = 1.53$ h, calculate the peak discharge for a Type II rainfall of 152.4 mm. ■

Solution 3.10
Using Table 3.15:

- for lots of 2000 m^2 with soil type B, $CN = 70$
- for lots of 2000 m^2 with soil type C, $CN = 80$
- for scattered open spaces in good hydrological condition with soil type C, $CN = 74$

Table 3.17: Parameters used to estimate peak unit discharge, q_u (*Cronshey*, 1986).

Type de précipitation	I_a/P	C_0	C_1	C_2
I	≤ 0.10	2.30550	–0.51429	–0.11750
	0.15	2.27044	–0.50908	–0.10339
	0.20	2.23537	–0.50387	–0.08929
	0.25	2.18219	–0.48488	–0.06589
	0.30	2.10624	–0.45696	–0.02835
	0.35	2.00303	–0.40769	–0.01983
	0.40	1.87733	–0.32274	0.05754
	0.45	1.76312	–0.15644	0.00453
	≥ 0.50	1.67889	–0.06930	0
IA	≤ 0.10	2.03250	–0.31583	–0.13748
	0.15	1.97614	–0.29899	–0.10384
	0.20	1.91978	–0.28215	–0.07020
	0.25	1.83842	–0.25543	–0.02597
	0.30	1.72657	–0.19826	0.02633
	0.35	1.70347	–0.17145	0.01975
	0.40	1.68037	–0.14463	0.01317
	0.45	1.65727	–0.11782	0.00658
	≥ 0.50	1.63417	–0.09100	0
II	≤ 0.10	2.55323	–0.61512	–0.16403
	0.15	2.53125	–0.61698	–0.15217
	0.20	2.50928	–0.61885	–0.14030
	0.25	2.48730	–0.62071	–0.12844
	0.30	2.46532	–0.62257	–0.11657
	0.35	2.41896	–0.61594	–0.08820
	0.40	2.36409	–0.59857	–0.05621
	0.45	2.29238	–0.57005	–0.02281
	≥ 0.50	2.20282	–0.51599	–0.01259
III	≤ 0.10	2.47317	–0.51848	–0.17083
	0.15	2.45395	–0.51687	–0.16124
	0.20	2.43473	–0.51525	–0.15164
	0.25	2.41550	–0.51364	–0.14205
	0.30	2.39628	–0.51202	–0.13245
	0.35	2.35477	–0.49735	–0.11985
	0.40	2.30726	–0.46541	–0.11094
	0.45	2.24876	–0.41314	–0.11508
	≥ 0.50	2.17772	–0.36803	–0.09525

The average CN is $CN = 70 \times 0.3 + 80 \times 0.4 + 74 \times 0.3 = 75.2$

The maximum potential retention: $S = 25.4\left(\frac{1000}{CN} - 10\right) = 25.4\left(\frac{1000}{75.2} - 10\right) = 83.77$ mm

The runoff depth $R = \frac{(P-0.2S)^2}{P+0.8S} = \frac{(152.4-0.2\times 83.77)^2}{152.4+0.8\times 83.77} = 83.86$ mm $= 8.386$ cm

The amount of rain absorbed before runoff: $I_a = 0.2S = 0.2 \times 83.77 = 16.75$ mm

$\frac{I_a}{P} = \frac{16.75}{152.4} = 0.11$. From Table 3.17 and for a Type II rainfall, C_0, C_1, and C_2 are obtained by interpolation as follows:

$$C_0 = 2.53125 - \frac{(0.15-0.11)(2.53125-2.55323)}{0.15-0.10} = 2.5488,$$
$$C_1 = -0.61698 - \frac{(0.15-0.11)(-0.61698+0.61512)}{0.15-0.10} = -0.6155$$
$$C_2 = -0.15217 - \frac{(0.15-0.11)(-0.15217+0.16403)}{0.15-0.10} = -0.1617$$

Applying eq. (3.45): $\log_{10}(q_u) = C_0 + C_1 \log_{10} t_c + C_2 (\log_{10} t_c)^2 - 2.366$

$\log_{10}(q_u) = 2.5488 - 0.6155 \log_{10}(1.53) - 0.1617(\log_{10}(1.53))^2 - 2.366 = 0.0636$

$\therefore q_u = 10^{0.0636} = 1.158 \text{ m}^3/\text{s/cm/km}^2$

From eq. (3.44) and taking $F_a = 1$:

$Q_{Storm} = F_a q_u AR = 1 \times 1.158 \times 1 \times 8.386$; $\boxed{Q_{Storm} = 9.71 \text{ m}^3/\text{s}}$

3.2 Evaluation of Wastewater Flows

The amount of wastewater discharged by the population and various facilities represents approximately 80% of the water supplied for their consumption.

$$Q_U = 0.8 Q_{avg,d} \tag{3.46}$$

3.2.1 Determination of Peak Flow

This is the maximum discharged flow rate, calculated to size the sewage system. The following relation is used:

$$Q_{Waste} = PF \cdot Q_U \tag{3.47}$$

Where Q_{Waste} is the peak discharge flow, PF is the peak factor, and Q_U is the average daily discharge flow.

- If $Q_U < 2.8$ L/s then $PF = 3$
- If $Q_U > 2.8$ L/s then

$$PF = 1.5 + \frac{2.5}{\sqrt{Q_U}} \tag{3.48}$$

3.2.2 Calculation of Flows Discharged by Catchment Area

3.2.2.1 Calculation of Specific Flow rate

The specific flow rate is calculated using the following relation:

$$Q_{sp} = \frac{Q_{UE}}{\sum_{i=1}^{n_b} A_i} \tag{3.49}$$

- Q_{sp}: specific wastewater flow ($\text{m}^3/\text{d/ha}$);
- Q_{UE}: peak discharge flow in the target year (m^3/d);
- A_i: area of basin i;
- n_b: number of catchment basins.

3.2.2.2 Calculation of Elementary Wastewater Flows

The elementary flow of a catchment basin is calculated using the following formula:

$$Q_{Waste} = Q_{sp} \cdot A_i \tag{3.50}$$

3.3 Calculation of Total Flow

For each elementary catchment basin, the total flow (Q_T) is the sum of the wastewater flow (Q_{Waste}) and the stormwater flow (Q_{Storm}):

$$Q_T = Q_{Waste} + Q_{Storm} \tag{3.51}$$

Example 3.11 Consider a town with a population of 3000 inhabitants.

- Compute the average discharge flow if the discharge coefficient is 60% and the water supply allocation is 150 l/d/person.
- Compute the peak factor.
- Compute the maximum (peak) discharge flow.

Solution 3.11

The average discharge flow: $Q_U = 0.6 \times 150 \times 3000 = 270000$ L/d, $\boxed{Q_U = 3.125 \text{ L/s}}$.

The peak factor: Since $Q_U > 2.8$ L/s then $PF = 1.5 + \frac{2.5}{\sqrt{Q_U}} = 1.5 + \frac{2.5}{\sqrt{3.125}}$, $\boxed{PF = 2.91}$.

The maximum (peak) discharge flow: $Q_{Waste} = PF\,Q_U = 2.91 \times 3.125$, $\boxed{Q_{Waste} = 9.11 \text{ L/s}}$.

3.4 Review Questions

1. Define the time of concentration.
2. Consider a rainfall with a water height, P (in mm), occurring over a duration t (in h). Define: Frequency, Return period, Average intensity, Time of concentration.
3. List the different types of losses to be considered for establishing the excess rainfall contributing to runoff.
4. When using the rational equation, can you use rainfall with a duration, t, shorter than the time of concentration, t_c, of the watershed, considering a given recurrence period? Comment.
5. When using the rational equation, can you use rainfall with a duration, t, longer than the time of concentration, t_c, of the watershed, considering a given recurrence period? Comment.
6. Clearly state the fundamental assumptions of the rational equation regarding the runoff coefficient.
7. Describe the rational method for estimating the rainfall runoff to be collected.
8. What parameters are considered in the rational method for calculating rainfall runoff?
9. What does the intensity-duration-frequency (IDF) curve represent?
10. What is the difference between frequency and recurrence interval?
11. What factors influence rainfall intensity?
12. Define a drainage basin.
13. What is the validity domain of the *Caquot* method (usage limits)?
14. What are the parameters characterizing a drainage basin?

15. What does the runoff coefficient signify?
16. What factors influence the volumetric runoff coefficient?
17. Explain how the type of surface, land slope, and land use affect the volumetric runoff coefficient.
18. What is the relationship between the volumetric runoff coefficient and the imperviousness rate of an area?
19. How is the volumetric runoff coefficient used in the design of urban drainage systems and sanitation infrastructure?
20. Demonstrate that a rainfall intensity of $0.015\ \mathrm{L/s/m^2}$ is equivalent to $54\ \mathrm{mm/h}$.
21. Explain the concept of the rational method. What are its main limitations?
22. What factors influence the time of concentration of a watershed?
23. Explain how topography, watershed size, and terrain roughness affect the time of concentration.
24. Why is the time of concentration important in the design of urban drainage systems?
25. What are the common methods used to calculate the time of concentration of a watershed?
26. Explain the difference between the time of entry and the time of flow.
27. Concerning urban runoff, explain the difference between a constant runoff coefficient and a time-varying runoff coefficient.
28. Describe the main steps of the rational method for calculating rainfall runoff.
29. What parameters are considered in the rational method for estimating rainfall runoff?
30. What is the general formula used in the rational method to calculate peak discharge?
31. How is the area of a watershed determined in the context of the rational method?
32. What factors can influence the accuracy of results obtained with the rational method?
33. How is the rational method used in the design of urban drainage networks?
34. What are the alternatives to the rational method for estimating rainfall runoff, and in what situations are they preferred?
35. What is the *Caquot* method in sanitation, and how does it differ from the traditional rational method?
36. Explain the basic principles of the *Caquot* method for calculating rainfall runoff.
37. What steps should be followed to apply the *Caquot* method in peak discharge estimation?
38. What hydraulic parameters are considered in the *Caquot* method for determining rainfall runoff?
39. How does the *Caquot* method account for watershed characteristics in calculating rainfall runoff?
40. What are the advantages of the *Caquot* method over the traditional rational method?
41. What is the NRCS (SCS) TR-55 method, and what is its main objective?
42. What are the main differences between the TR-55 method and the rational method in estimating rainfall runoff?
43. What types of data are required to use the TR-55 method in estimating rainfall runoff?

44. Explain the process of calculating rainfall runoff using the TR-55 method.
45. What are the advantages of the TR-55 method over other methods of estimating rainfall runoff?
46. Describe the various steps involved in evaluating wastewater discharge.
47. What are the assumptions of the rational method that limit its application to large geographic areas?
48. When the values of the runoff coefficient, C, approach 1, do the areas become more or less permeable?
49. The intensity of rainfall, i, corresponds to two factors of a rain event, which are they?

3.5 Objective Questions

Q 3.1 What is frequency in the context of recurring events?

a) The elapsed time between two successive events
b) The total number of events over a given period
c) The intensity of each individual event
d) The total duration of all combined events

■

Q 3.2 How is frequency generally measured?

a) In years
b) In days
c) In hours
d) In occurrences per unit of time

■

Q 3.3 What is the recurrence interval?

a) The elapsed time between two consecutive events
b) The total duration of all events
c) The number of events over a given period
d) The average intensity of events

■

Q 3.4 What is the relationship between frequency and recurrence interval?

a) They are identical and can be used interchangeably.
b) Frequency is the inverse of the recurrence interval.
c) Frequency equals the recurrence interval multiplied by intensity.
d) There is no direct relationship between frequency and recurrence interval.

■

Q 3.5 What is the intensity of a recurring event?

a) The ratio of the water depth to the corresponding fall time
b) The ratio of the water volume to the corresponding fall time
c) The ratio of the water depth to the watershed area
d) The ratio of the water volume to the watershed area

■

Q 3.6 What is the relationship between frequency and intensity?

a) Higher frequency corresponds to lower intensity.
b) Higher frequency corresponds to higher intensity.
c) Frequency and intensity are not related.
d) Frequency and intensity have an inverse relationship.

■

Q 3.7 What is net rainfall?

a) The total amount of precipitation over a given period
b) The amount of precipitation that strictly runs off the ground surface

c) The amount of precipitation that evaporates before reaching the ground

d) The amount of precipitation that infiltrates into the soil and joins the groundwater

Q 3.8 What is interception?

a) The process by which plants absorb precipitation water

b) The process by which precipitation water is stored in reservoirs

c) The process by which precipitation water is retained on the surface of leaves and evaporated

d) The process by which precipitation water infiltrates the soil

Q 3.9 What is retention in the context of precipitation?

a) The process by which precipitation water flows down slopes

b) The process by which precipitation water is stored in reservoirs

c) The process by which precipitation water is evaporated from land surfaces

d) The process by which precipitation water is retained on the soil surface or structures

Q 3.10 What is the main difference between interception and retention of precipitation?

a) Interception refers to the amount of water retained by vegetation, while retention refers to the amount of water stored on the soil surface.

b) Interception refers to the amount of water stored on the soil surface, while retention refers to the amount of water retained by vegetation.

c) Interception and retention refer to the same thing and can be used interchangeably.

d) Interception and retention refer to the total amount of water reaching the ground from precipitation.

Q 3.11 What is the volumetric runoff coefficient?

a) The percentage of the watershed area that is impervious

b) The ratio of runoff volume to the total precipitation volume

c) The ratio of infiltration volume to the total precipitation volume

d) The percentage of precipitation that evaporates

Q 3.12 What is the runoff coefficient used for in urban drainage systems?

a) To determine the total volume of precipitation

b) To calculate the peak discharge from rainfall

c) To design the capacity of storage reservoirs

d) To assess the efficiency of infiltration systems

Q 3.13 What is the time of concentration?

a) The longest time for precipitation to reach the ground.

b) The longest time for water to flow through a drainage system.

c) The longest time for runoff water to reach the outlet of the watershed.

d) The longest time for water to infiltrate into the ground.

Q 3.14 The *Kerby* equation for calculating the time of entry (or surface runoff time); t_e is [Rat:2022/2023]:

a) $t_e = 0,0195 f \frac{L^{0,77}}{S_0^{0,385}}$

b) $t_e = 1,44 \left(\frac{L \cdot r}{S_0^{0,5}} \right)^{0,467}$

c) $t_e = 6,99 \frac{(L \cdot n)^{0,6}}{i^{0,4} S_0^{0,3}}$

d) $t_e = 0,023 \frac{L}{A^{0,1} S_0^{0,2}}$

Q 3.15 The method used to estimate stormwater discharge without being limited by the estimation of the time of concentration, while also accounting for the storage capabilities of the watershed, is [Rat:2022/2023]

a) The rational method

b) The *Caquot* method (surface)

c) The runoff coefficient method

d) The Manning equations method

■

Q 3.16 The recommended return period for the design of the drainage network is:

a) 10 years

b) 30 years

c) 50 years

d) 100 years

■

Q 3.17 In the rational method, calculating the time of concentration is necessary to

a) Determine the intensity of the precipitation

b) Determine the runoff coefficient

c) Determine the depth of the precipitation

d) All of the above

■

Q 3.18 The amount of runoff water reaching a storm sewer depends on the following factors

a) The intensity and duration of the storm/precipitation

b) The characteristics of the watershed

c) The impermeability of the watershed

d) All of the above factors

■

Q 3.19 Regarding the runoff coefficient, which of the following statements is correct?

a) The runoff coefficient remains constant and is independent of the duration of the rain event.

b) For permeable soils, the runoff coefficient decreases with increasing duration of the rain event.

c) The runoff coefficient decreases with increasing duration of the rain event.

d) None of the above statements is correct.

■

3.6 Problems

P 3.1 The infiltration capacity curve for a watershed is given by : $f_p = (296,1 - 21,1)e^{-0,07t} + 21,1$ where t is in min et f_p in mm/h. The storm pattern is given in Table P3.1:

Table P3.1

t, (min)	Intensity, (mm/h)
0-10	88.9
10-20	76.2
20-30	203.2
30-40	127.0
40-50	38.1
50-60	61.0
60-70	38.1

Determine the rainfall excess for the successive 10-min period. ■

P 3.2 In a residential area of 250,000 m^2 (25 ha), the percentage of impervious surfaces is 35. Given that the time of concentration of this catchment area is 35 minutes and the absorptivity of the permeable soil is low, calculate the maximum runoff rate for a rainfall event with a recurrence interval of 5 years ($i = 2184.4/(t_c + 12)$ in mm/h with t_c in minutes). Consider both cases: (a) the runoff coefficient, C is constant, and (b) C varies over time. ■

P 3.3 Compute the runoff coefficient of a commercial district with a time of concentration (or time of entry in this case) of 30 min. The soil in this area is mostly clay and the water table is not deep (absorptivity of the permeable surfaces is low). The different elements of the watershed's surface and the percentages of the total area used by these elements are:

Roofs:	40%
Paved streets:	18%
Parking lots:	25%
Lawns with steep slope (dense soil):	6%
Sidewalks:	6%
Driveways:	5%

■

P 3.4 In a rectangular catchment area with a time of concentration (or time of entry, in this case) of 40 minutes, the runoff flows parallel to the widths. When it reaches the lowest length, it flows into channels that direct it to the outfall located in the middle of this length. If the water flow velocity is constant up to the outfall and if the runoff coefficient is constant and uniform, calculate the maximum flow rate caused by each of the following rainfall events:

a) 100 mm/h for 10 minutes over the entire basin area;

b) 25 mm/h for 50 minutes over the entire basin area.

■

P 3.5 The time of concentration (which is also the time of entry) for a rectangular basin with a length equal to 4 times the width is 30 minutes. The runoff flows from the highest width to the other width, at the middle of which is the outfall. If the runoff coefficient is constant, calculate the maximum flow rate at the basin's outfall resulting from the following precipitations:

a) 100 mm/h for 10 minutes over the entire basin area;

b) 25 mm/h for 40 minutes over the entire basin area.

■

P 3.6 A rectangular concrete parking lot ($n = 0.013$) measuring 30 m by 50 m is serviced by a single storm drain located at one of its interior corners. The corners of the higher short side of the lot are at an elevation of 100 m, and the other two corners are at an elevation of 99.5 m. Using the intensity equation $i = 2184.4/(t_c + 12)$ (i in mm/h and t_c in minutes),

a) At what level should the storm drain be placed, given that the surface it serves must converge radially towards it (like a cone) with a slope as gentle as possible but never less than 2%?

b) What is the time of entry? (Use *Kerby's* equation).

c) What is the maximum rainfall rate the drain will receive, given that the rainfalls have a recurrence interval of 5 years and the runoff coefficient of the parking lot is 0.85?

d) If the capacity of a storm drain is 30 L/s, will the proposed drain meet the needs? If not, propose an alternative option. Several iterative calculations are required; do not perform more than three relevant ones.

■

P 3.7 Figure P3.7 shows a terrain with part asphalted ($C = 0.8$) and part grassed ($C = 0.18$). Compute the time of entry, t_e, for each of the two sub-basins (i.e., t_e from A to C, and t_e from B to C):

a) using the *Kerby* model (see table 3.8 for values of r);

b) using the *Kirpich* model (see table 3.7 for values of f_s).

The value of the *Manning* coefficient n for the concrete ditch is 0.02.

Indicate, with comments, which time of entry should be used to calculate the dimensions of the storm sewer at the outlet of this watershed.

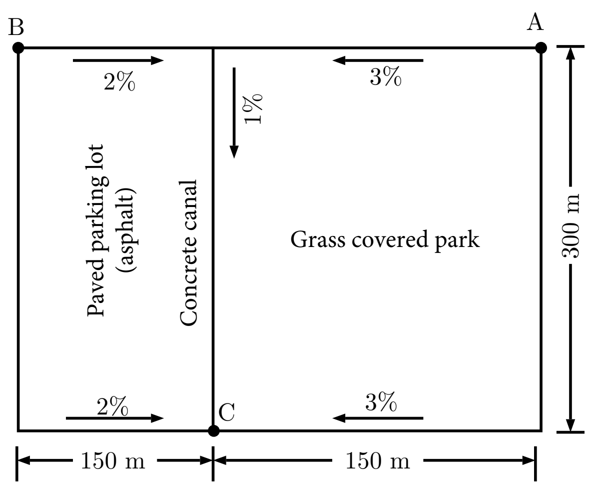


Figure P3.7

■

P 3.8 For the watershed in P 3.7, if rainfall with a recurrence interval of 10 years occurs, calculate the maximum normal discharge at the outlet of each of the two sub-basins after evaluating the two time of entrys, t_e, using the *Kerby* model. Indicate which of these time of entrys is critical and justify. Use the following equation for intensity calculation: $i = 2743.2/(t_c + 14)$ (i in mm/h and t_c in minutes). ■

P 3.9 Solve P 3.8 for rainfall with a recurrence interval of 5 years, with intensity given by: $i = 2184.4/(t_c + 12)$ (i in mm/h and t_c in minutes). ■

P 3.10 A sub-basin of 50000 m^2 (5 ha) is part of a neighborhood consisting only of single-family homes ($C = 0.4$ and $n = 0.1$). If the average slope of this sub-basin towards its outlet is 5% and the runoff water must travel a distance of 150 m to reach the storm drain (outlet of the sub-basin), calculate the time of concentration (time of entry) for this urban sub-basin using the *Kerby* equation. Evaluate the maximum normal discharge generated by this sub-basin after rainfall with a recurrence interval of 5 years:

1. if the runoff coefficient is constant;
2. if the runoff coefficient varies with time.

Impermeable surfaces cover 30% of the basin and the absorptivity of permeable surfaces is average. Use the following equation for intensity calculation: $i = 2743.2/(t_c + 14)$ (i in mm/h and t_c in minutes). Provide recommendations. ■

P 3.11 On a 400,000 m^2 (40 ha) site, single-family homes will be built on half of the area and row houses on the other half. The time of concentration for this basin is 40 minutes, its constant runoff coefficient is 0.55, and its *Manning* coefficient n is 0.1. The storm sewer located at the outlet of this watershed can handle a maximum discharge of 1.27 m^3/s. Is the capacity of this sewer sufficient to collect the runoff from rainfall with a recurrence period of 5 years? Discuss the results. Use the following equation for intensity calculation: $i = 2743.2/(t_c + 14)$ (i in mm/h and t_c in minutes). ■

P 3.12 A watershed of 5,000 m^2 (0.5 ha), with a runoff coefficient of 0.5 and a time of concentration of 22 minutes. Using the rational equation and considering an intensity $i = \dfrac{2743.2}{t+12}$; where i is in (mm/h) and t is in minutes, calculate, in cubic meters per second: a) the maximum normal runoff discharge at the outlet of this basin; b) the discharge 10 minutes after the start of rainfall; c) the discharge 30 minutes after the start of rainfall; d) the discharge 45 minutes after the start of rainfall; e) the maximum discharge if a 10-minute rainfall (from the start of rainfall, the precipitation front reaches the basin outlet) were considered; f) the discharge 4 minutes after the start of a 10-minute rainfall; g) the discharge 12 minutes after the start of a 10-minute rainfall; h) the maximum discharge if a 40-minute rainfall were considered; i) the discharge 20 minutes after the start of a 40-minute rainfall; j) the discharge 42 minutes after the start of a 40-minute rainfall. ■

P 3.13 We want to equip a newly constructed watershed with a separate sewerage system. You are asked to calculate the peak flow rate required to design the stormwater sewer network. Data:

- Entry time into the network is 4 minutes.
- Average water velocity in the pipe is 1.5 m/s.
- Length of the longest pipe is 1350 m.
- Size of the watershed is 1200×900 m.
- Impervious surface area is estimated at 30%.
- Parameters a and b of the Montana law for a return period of 10 years are: $a = 2.62$ and $b = -0.48$.

■

P 3.14 An zone is located on a total area of 9.3 ha, divided into four basins as shown in the figure below. Using the rational method, determine the rainfall water flows at points 1, 2, and 3, knowing that parameters a and b of the Montana law for a return period of 10 years are: $a = 3.16$ and $b = -0.64$. Use the Kerby formula to evaluate the entry time t_e.

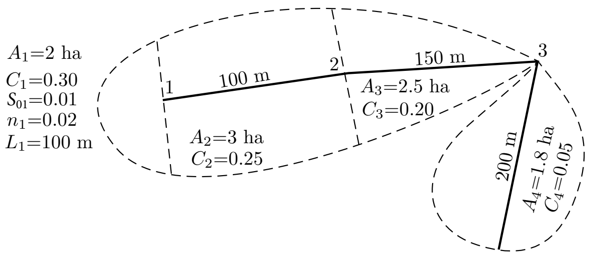


Figure P3.14

■

P 3.15 A storm drainage system consists of four watersheds as shown in Figure P3.15. The characteristics of the watersheds are summarized in Table P3.15. Determine the design flow rate for each section of the drainage system. The average velocity in the sewers is 1.37. The rainfall intensity (in mm/min) is given by the formula: $i = 5.68t_c^{-0.64}$; where t_c is in minutes.

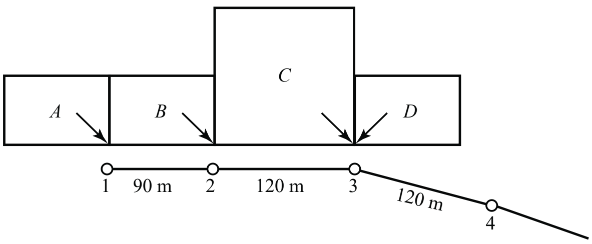


Figure P3.15

Table P3.15

Watershed	A (ha)	C	t_e (min)
A	4.85	0.8	10
B	4.85	0.8	10
C	13.76	0.6	28
D	4.05	0.9	8

P 3.16 A simple storm sewer network is illustrated in Figure P3.16. Relevant rainfall data is provided in Table P3.16a and network data in Table P3.16b. Assume fixed pipe gradients. Compute the stormwater flows using the modified rational method for a 2-year return period storm using the volumetric runoff coefficient from Example 3.5 and an inlet time of 5 min.

Table P3.16a: Rainfall intensities (mm/h) at the network site of Example 3.16.

Duration (min)	Return Period (years) 1	2	5
6.0	50.0	61.7	81.6
6.2	49.3	60.7	80.4
6.4	48.5	59.8	79.2
6.6	47.8	58.9	78.1
6.8	47.1	58.0	77.0
7.0	46.4	57.2	75.9
7.2	45.8	56.4	74.9
7.4	45.2	55.6	73.9
7.6	44.5	54.8	72.9
7.8	44.0	54.1	71.9
8.0	43.4	53.4	71.0
8.2	42.8	52.7	70.1
8.4	42.3	52.0	69.3
8.6	41.8	51.4	68.4
8.8	41.2	50.7	67.6
9.0	40.8	50.1	66.8

Table P3.16b: Pipe characteristics of Example 3.16.

Pipe No.	L (m)	A (ha) (ha)	t_f (min)
1.1	120	0.4	1.8
2.1	100	0.6	1.3
1.2	150	0.8	2.1

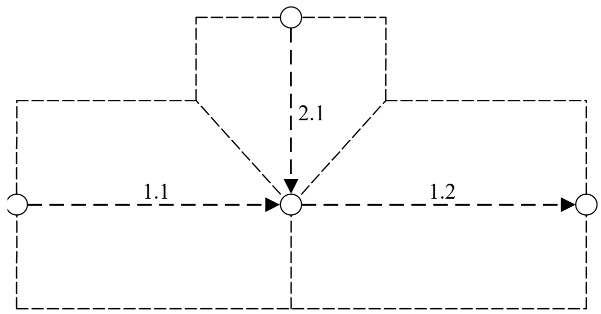


Figure P3.16

P 3.17 Table P3.17 provides the instantaneous cumulative rainfall values.

1. Determine the heights and corresponding rainfall intensities. Provide a detailed calculation example for the first time step and summarize all results in a table.

2. Determine the maximum average intensity over the time interval $t_1 = 12.5$ min.

Table P3.17

t (min)	0	5	10	15	20
P (mm)	0	6	10	15	24
t (min	25	30	35	40	45
P (mm)	32	37	37	44	48

■

P 3.18 Considering Figure P3.18:

1. Determine, using the rational method, the peak flow Q_{Storm} generated by the watershed at point E. Use the *Kirpich* formula to calculate t_e.

2. Assuming $Q_{Storm} = 0.8\ \text{m}^3/\text{s}$, size the pipe E-F using the *Manning-Strickler* formula.

DN sizes are given as: 300; 400; 500; 600; 800; 1000; 1200; 1400; 1500; ...

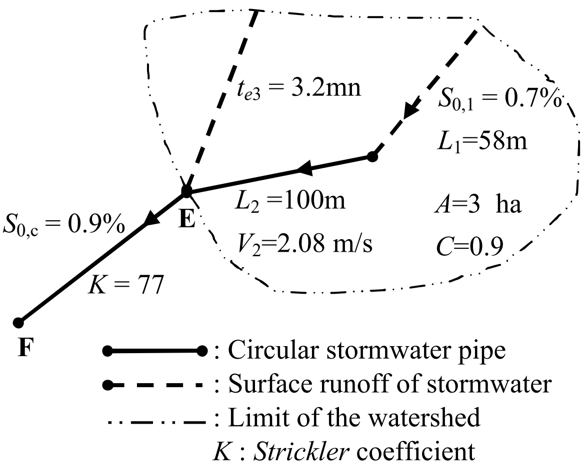


Figure P3.18

■

P 3.19 In Figure P3.19, BV1 is an existing urban watershed and BV2 is a planned extension watershed.

1. Estimate the area A_1 (in m^2) and hydraulic slope $S_{0,1}$ (in %) of BV1. Given are the maximum flow Q_{p1}, elongation M_1, and length L_1 of BV1 (see Figure P3.19). For subsequent calculations, take $A_1 = 5$ ha and $S_{0,1} = 2.5\%$.

2. Given that the maximum possible flow at E is $Q_{pE} = 1.022\ \text{m}^3/\text{s}$, determine the maximum surface (A_2) of BV2.

3. Assuming $A_2 = 1.5$ ha; determine the actual maximum flow Q_{pE} to consider at E.

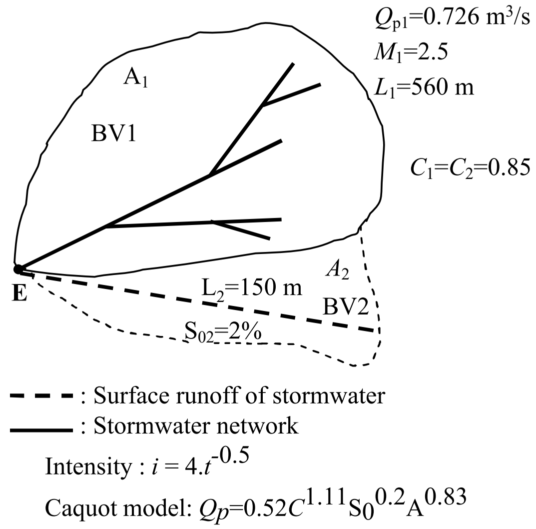


Figure P3.19

■

P 3.20 A watershed with characteristics A_1, C_1, $t_{c,1}$. During a ten-year storm $i = 4 \cdot t^{-0.5}$ this watershed produces at its outlet a peak flow Q_{p1}. This watershed is drained by a pipe capable of handling a maximum flow Q_p. We want to extend this watershed upstream with an urban development having characteristics A_2, C_2, $t_{c,2}$. Given: $A_1 = 2.2$ ha, $C_1 = 0.7$, $C_2 = 0.5$, $t_{c,2} = 30$ min, $Q_p = 0.15\ \text{m}^3/\text{s}$, $Q_{p,2} = 0.13\ \text{m}^3/\text{s}$.

1. What should be the maximum value of A_2 to not exceed Q_p?

2. Assuming $A_2 = 0.66$ ha, calculate Q_p.

■

P 3.21 Two parallel watersheds as shown in Figure P3.21 with characteristics summarized in Table P3.21. Determine by the *Caquot* method the peak flow at outlet E for a ten-year rain: $i = 4 \cdot t^{-0.5}$.

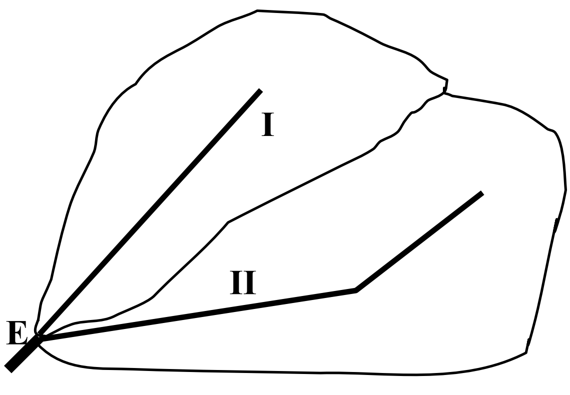


Figure P3.21

Table P3.21

Watershed	I	II
A (ha)	7.3	16.7
C	0.55	0.35
S_0 (m/m)	0.01	0.005
L (m)	605	575

P 3.22 For the drainage system shown in Figure E3.22, determine the design flow for each sewer section by the rational method. The IDF curve for a 1/10-year frequency is given by: $i = 4 \cdot t^{-0,5}$.

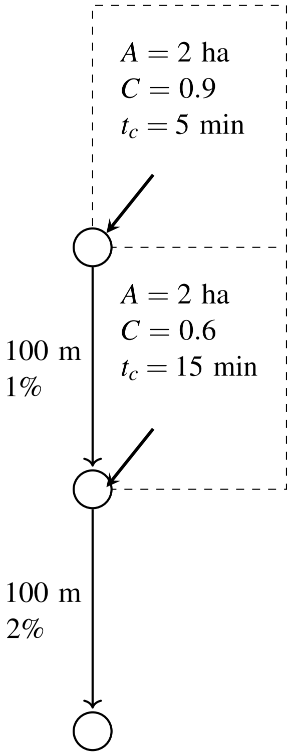


Figure E3.22

P 3.23 Consider a watershed composed of 5 sub-basins (Fig. P3.23). We aim to size the storm sewer network of this basin. The characteristics of the watersheds and sections are represented in Tables P3.23a and P3.23b, respectively.

1. Compute the peak flows at all nodes using the rational method. Use the *Kirpich* formula to calculate t_e.
2. Compute the peak flows at all nodes using the *Caquot* method.
3. Compare the results of both methods.

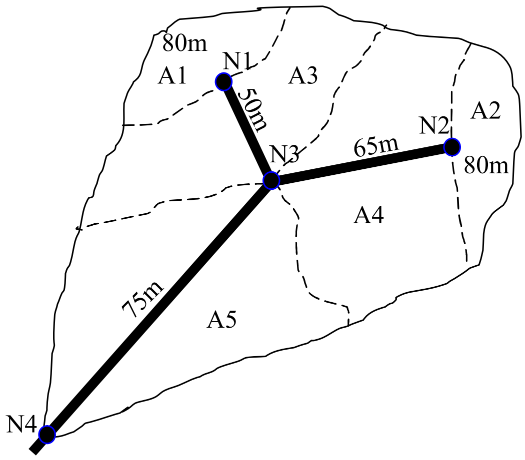


Figure P3.23

Table P3.23a: Characteristics of the watersheds.

Sous-bassin	A(ha)	C	S_0 (%)	L (m)
A1	0.3	0.55	1	80
A2	1.4	0.23	0.8	135
A3	0.4	0.6	0.65	80
A4	1.55	0.35	0.7	140
A5	2.5	0.4	0.72	166

Table P3.23b: Characteristics of the pipes.

Section	S_0 (%)	L (m)
gutter	0.95	80
N1-N2	0.9	50
gutter	0.8	80
N2-N3	0.9	65
N3-N4	1	75

P 3.24 Consider a locality with 20,000 inhabitants, where 90% are connected to the potable water network,

1. Compute the water consumption in m^3/d, knowing that the unit allocation is 70 liters of water consumed per inhabitant per day.
2. Compute the average wastewater discharge in m^3/d produced by this locality, knowing that the connection rate to the wastewater network is 75% and the return rate to the sewer is 80%.
3. Compute the peak flow rate for this locality.

P 3.25 Compute the runoff coefficient for an urban residential area of 142 ha. The various elements that make up the basin surface, their percentage of the total area, and their runoff coefficients are given in Table P3.25:

Table P3.25: Characteristics of the surfaces for example 3.25.

Surface type	Percentage	C
Roofs	25%	0.90
Asphalt roads	14%	0.85
Concrete sidewalks	5%	0.90
Gravel driveways	7%	0.25
Grass lawns with low slope	49%	0.15

Given that the time of concentration for this watershed is 35 minutes, calculate the maximum runoff rate at the basin's outlet for a storm with a recurrence interval of 5 years ($i = 2184.4/(t_c + 12)$ in mm/h with t_c in minutes). ■

P 3.26 In a high-density industrial area, the time of entry of an urban sub-basin of 0.5 ha is estimated at 6.5 minutes. The absorptivity of the permeable soil is low, and the percentage of impervious surfaces is 100. Compute the maximum runoff flow at the storm drain for a rain with a recurrence interval of 10 years, where $i = \dfrac{2743.2}{t+14}$; i is in (mm/h) and t is in minutes. ■

P 3.27 In a residential area of 25 ha, the percentage of impervious surfaces is 35. Given that the time of concentration of this watershed is 35 minutes and that the absorptivity of the permeable soil is low, calculate the maximum runoff flow at the outlet of this basin for a rain with a recurrence interval of 5 years. $i = \dfrac{2743.2}{t+12}$; i is in (mm/h) and t is in minutes. ■

P 3.28 A 0.4 ha watershed is shaped like an equilateral triangle and its outlet is at the centre of one side. It has a 19 min time of concentration and its runoff coefficient is 0.85. The rainfall intensity (mm/h) is represented by $i = \dfrac{2184.4}{t_c+12}$; t_c is in minutes.

1. What is the maximum flow at the outlet of this watershed following a precipitation that lasts $0.60t_c$?

2. What is the maximum flow at the outlet of this watershed following a precipitation that lasts $1.0t_c$?

3. What is the maximum flow at the outlet of this watershed following a precipitation that lasts $1.2t_c$?

■

P 3.29 For the drainage system illustrated in Figure P3.29,

1. Determine the flow rate for each sewer section using the rational method,

2. Using the *Bazin* chart, determine the diameter of each section.

The rainfall intensity (mm/h) is represented by $i = 120t_c^{-0.5}$; t_c is in minutes. The flow from each area is indicated by an arrow. The characteristics of the watersheds and sections are shown in Tables P3.29a and P3.29b respectively.

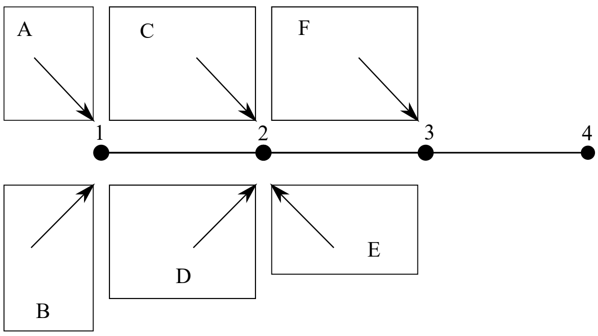


Figure P3.29

Table P3.29a: Characteristics of the watersheds.

Watershed	A (ha)	C	t_e (min)
A	0.5	0.8	8
B	0.8	0.7	9
C	1.3	0.5	10
D	1.5	0.6	12
E	1.2	0.8	15
F	1.6	0.6	18

Table P3.29b: Characteristics of the sections.

Pipe	L (m)	S_0 (m/m)	V (m/s)
1-2	150	0.018	1.23
2-3	160	0.020	1.70
3-4	200	0.010	1.40

■

P 3.30 Consider two residential areas:

- $S1 = 10$ ha with a density $d_1 = 80$ dwellings/ha
- $S2 = 15$ ha with a density $d_2 = 30$ dwellings/ha

If we consider

- that the average occupancy density is 3.1 inhabitants/dwelling;
- that the average needs per inhabitant (in the absence of information provided by metering) are 250 L/inhabitant/day for sector $S1$ and 200 L/inhabitant/day for sector $S2$;
- that losses for private garden and public space irrigation, including network losses, are approximately 30% of the required values,

Compute the peak values applicable to each differentiated sector and to the sum of the combined flows. ■

P 3.31 The small separate storm sewer network (Fig. P3.31) has the characteristics shown in Table P3.31.

Table P3.31: Characteristics of the catchment areas.

Section	Length (m)	Area (3^2)
1.1	180	2000
2.1	90	6000
3.1	90	9000
1.2	90	4000

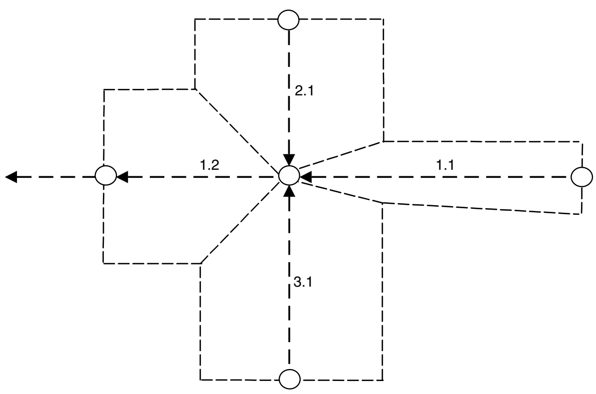


Figure P3.31

Use the rational method to determine the flow rate of each pipe in the network. Assume a time of entry of 4 minutes, that the full-section velocity in each pipe is 1.5 m/s, and that the design rainfall intensities can be determined from the equation $i = \dfrac{2743.2}{t+14}$; where i is in (mm/h) and t is in min. ■

P 3.32 Figure P3.37 schematically shows an urban watershed for which we want to calculate the flow rates of various conduits. The characteristics of the sub-basins and the lengths of the conduits are shown in Tables E3.37a and E3.37b, respectively. Assume that the design rainfall intensities can be determined from the equation $i = \dfrac{2700}{t+35}$; where i is in (mm/h) and t is in minutes.

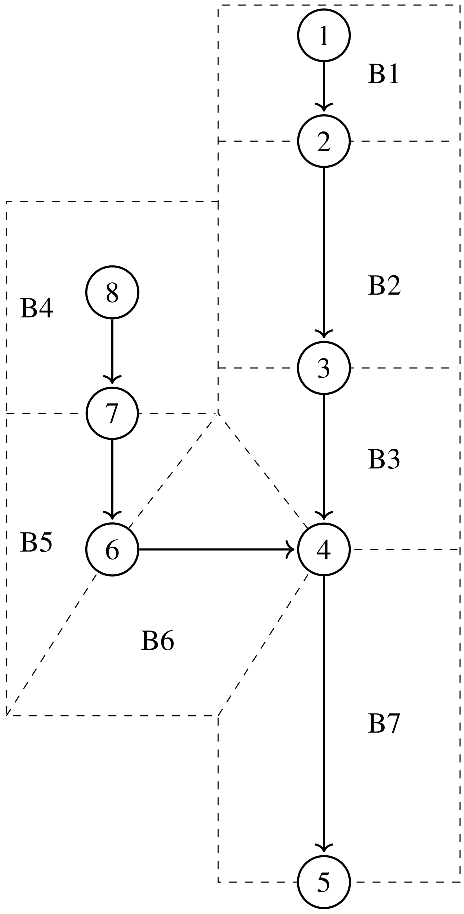


Figure E3.37

Table E3.37a: Caractéristiques des conduites

Segment	Ground Elevation Ups.	Dow.	L (m)	Q (L/s)
1-2	113.03	111.50	90	93.2
2-3	111.50	109.00	100	270.5
3-4	109.00	107.60	100	433.0
8-7	109.19	108.00	70	187.6
7-6	108.00	107.51	70	273.9
6-4	107.51	107.60	110	526.5
4-5	107.60	107.20	125	1279.0

Table E3.37b: Characteristics of the sub-basins.

Sub-basins	A (ha)	t_e (min)	C
B1	1.2	8.5	0.45
B2	2.2	9.0	0.48
B3	2.1	9.0	0.46
B4	1.5	7.0	0.70
B5	1.0	7.4	0.50
B6	2.4	9.0	0.65
B7	4.2	10.0	0.50

P 3.33 Calculate the wastewater flow rates for the neighborhood shown in Figure P3.37, assuming a population density of 600 inhabitants per hectare and a daily water allocation of 150 l/person/day. We will assume that all sanitary wastewater from a sub-basin is collected at the manhole located upstream of this sub-basin.

P 3.34 Three basins are designed to drain into a detention system in Figure P3.34. The watershed parameters for these three subareas are summarized in Table P3.34. Determine the 10-year peak discharge at point B.

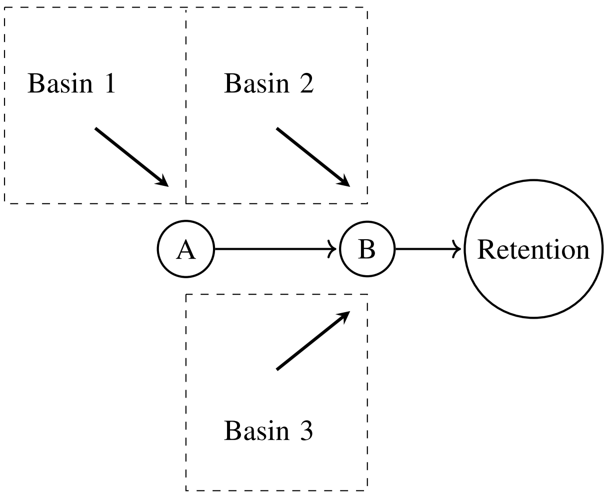


Figure P3.34: Layout for multiple subareas

Table P3.34: Parameters for Multiple Subareas

Subarea	A (ha)	C	t_c (min)
1	0.81	0.55	15
2	2.02	0.65	22
3	0.61	0.81	12

P 3.35 Figure P3.35 presents a street drainage condition. The 10-year rainfall IDF in mm/h can be calculated by : $i = \dfrac{1145}{(10+t_c)^{0.789}}$, in which t_c = time of concentration in minutes. Determine the 10-year peak discharge at Point C and D

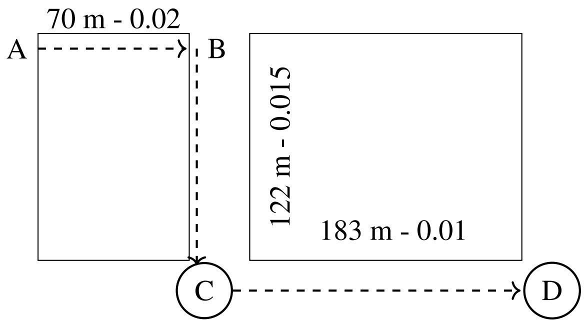
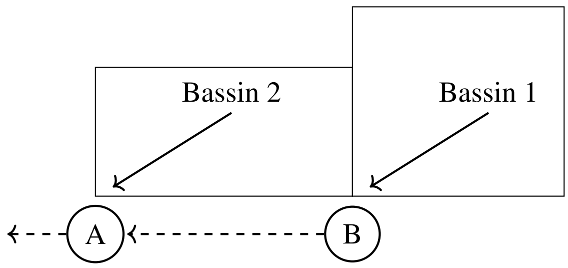


Figure P3.35

P 3.36 Consider the storm runoff for the two catchments shown in Figure P3.36. Determine the design discharge at Point A if the flow time through sewer AB is 2.5 min and $i = \dfrac{977.265}{(10+t_c)^{0.789}}$, in which t_c = time of concentration in minutes.



Catchment	2	1
A (ha)	5.2	10.5
t_e (min)	10.3	23
C	0.75	0.51

Figure P3.36: Layout for multiple subareas

P 3.37 Consider the two inlets and two pipes shown in Figure P3.37. Catchment A has an area of 1 ha and is 50% impervious, and catchment B has an area of 2 ha and is 10% impervious. All impervious areas are directly connected to the storm-sewer inlets. The runoff coefficient, C; length of overland flow, L; roughness coefficient, n; and average slope, S_0, of the pervious and impervious surfaces in both catchments are given in Table P3.37. The design storm has a return period of 10 years, and the 10-year IDF curve can be approximated by $i = \dfrac{7620}{t+36}$

Table E3.38a

Catchment draining to manhole	Ground elevation at manhole (m)	A (ha)	C	t_e (min)	Length of outflow pipe from the manhole (m)
51	219.49	0.506	0.70	10.30	70.1
52	219.83	0.283	0.65	11.80	21.3
53	219.54	0.607	0.55	17.60	39.6
61	219.30	0.243	0.75	9.00	48.8
71	218.05	0.931	0.70	12.00	73.2
81	217.96				

where i is the average rainfall intensity in mm/h and t is the duration of the storm in minutes. Calculate the peak flow rates to be handled by the inlets and pipes.

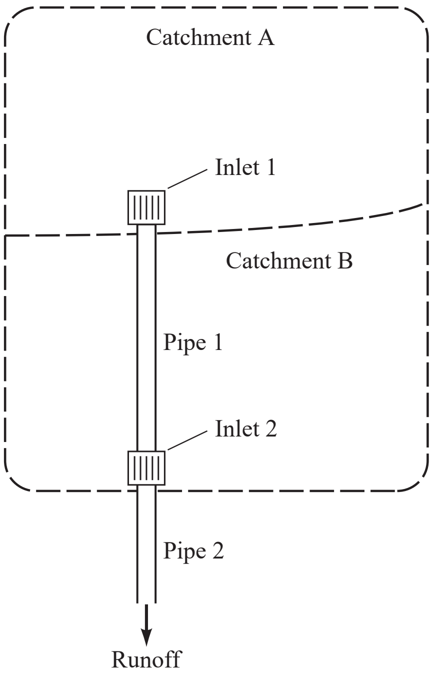


Figure P3.37

Table P3.37: Catchment Characteristics

Catchment	Surface	C	L (m)	n	S_0
A	Pervious	0.2	80	0.2	0.01
	Impervious	0.9	60	0.1	0.01
B	Pervious	0.2	140	0.2	0.01
	Impervious	0.9	65	0.1	0.01

P 3.38 Determine the discharges for pipes 5.1, 5.2, 5.3, 6.1, and 7.1 shown in Fig. P3.38. The catchment characteristics are listed in Table E3.38a. The capacities of sewers in the system should handle surface runoff generated from a 2-year storm. The rainfall-intensity-duration relationship for a 2-year return period is also given in Table P3.38b. A 14.65 ha drainage area with $\sum CA = 10.52$ ha and a time of concentration $t_c = 15.2$ min flows into the upstream end pipe pipe 5.1 through a pipe that is not shown.

Table P3.38b: Rainfall IDF relationship

t_c (min)	5	10	15	20
i (mm/h)	5.4	4.18	3.51	3.1

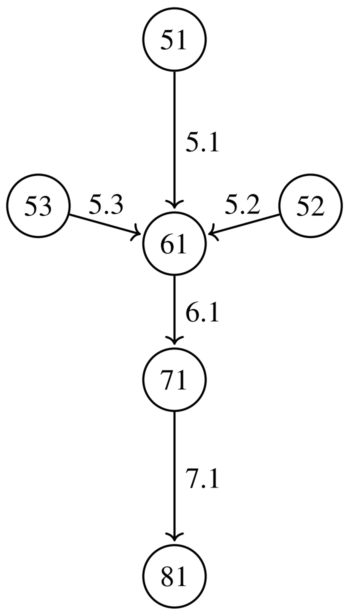


Figure P3.38

P 3.39 The node-link system in Figure P4.10 represents a sewer system. The system has 8 sewers and 9 manholes. Manhole 16 is a confluence in the system. Manhole 99 is the system outlet. Table

Table E3.39

Manhole ID Number	Ground Elevation (m)	Tributary Area (ha)	Runoff Coeff.	Overland Slope (%)	Overland Length (m)	Channel Slope (%)	Channel Length (m)
35	33.8	1	0.9	0.15	76.2	0.49	45.7
12	33.2	2.61	0.85	0.25	54.9	1.00	137.2
23	33.5	2	0.9	1.00	83.8	1.00	137.2
16	30.9	0	0	0.00	0.0	0	0.0
15	31.7	2	0.85	0.5	86.9	2.25	137.2
47	30.2	1	0.8	0.4	76.2	1.56	77.7
99	29.7	0	0	0	0.0	0	0.0
17	30.4	0	0.65	0.1	61.0	0.36	91.4
18	30.4	0.5	0.45	0.4	91.4	0	0.0

E3.39 provides the hydrologic parameters of tributary areas. The design rainfall IDF formula is given as: $i = 977.265/(t_c + 10)^{0.786}$ (i in mm/h and t_c in minutes). Determine the design discharges at the manholes in the system.

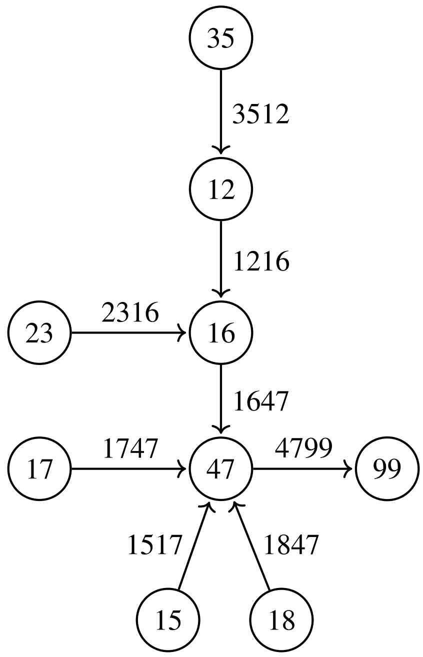
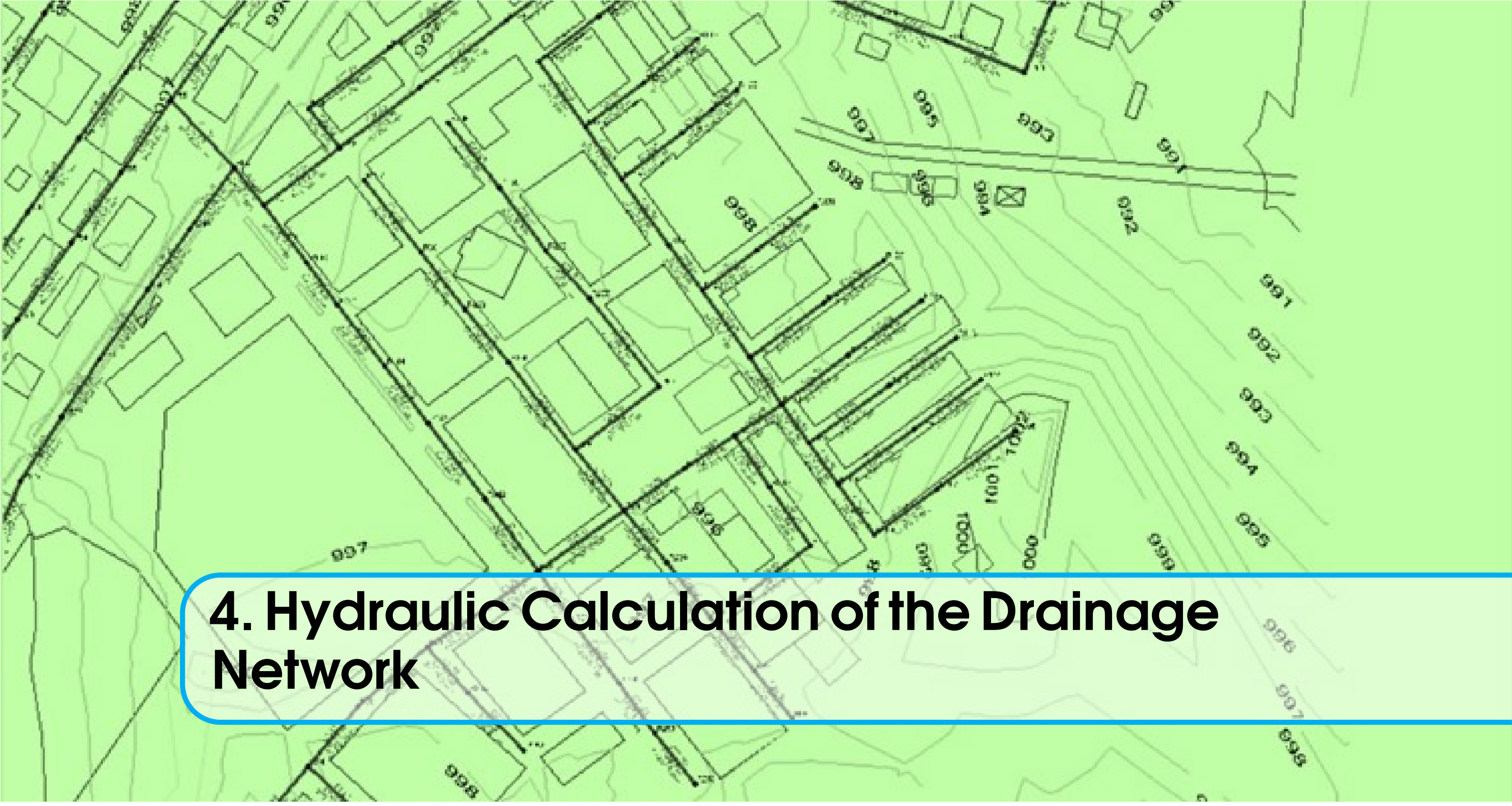


Figure P3.39



4. Hydraulic Calculation of the Drainage Network

Once the total flow rates have been determined, the actual sizing of the structures begins, while adhering to certain flow norms. The sizing of networks is complex due to their structure, which consists of various elements (sections, slopes) interspersed with singularities (connections, junctions, bifurcations, thresholds, drops), differing functions (separate system, combined system), and highly variable inflows and flow conditions over time. These continuously modify the water line profile and transfer capacities. Therefore, calculations must be performed for each hydraulic segment and node.

The design of a hydraulic system is characterized by:

- Physical constraints related to the terrain relief, building connections, and dependence on other networks.
- Hydraulic data: Nature of the effluent, flow rates, and characteristics of the sections.
- Technical provisions, requiring the most economical solutions.

All these design constraints mean that the project of a drainage network is always a unique case, making it difficult to explain all aspects.

Furthermore, without underestimating classic rules, one must be imaginative and find the best solutions adapted to the environment and local data.

The objectives of this chapter are:

1. Differentiate between various equations for calculating flow velocity;
2. Understand the concept of self-cleaning;
3. Size separate and combined networks;
4. Determine flow conditions;
5. Calculate the hydraulic parameters of a drainage network.

4.1 Flow Formulas

The calculation of pipelines is done according to the theory of "open channel flow". The basic formula for open flow:

$$Q_T = VA \tag{4.1}$$

with

- Q_T : Total flow rate (m^3/s),
- A : Wetted area (m^2),
- V : Average flow velocity (m/s),

The flow velocity is given by the *Chezy* equation (*Satin et al.*, 1995):

$$V = \mathcal{C}\sqrt{R_h S_0} \tag{4.2}$$

where

- $\mathcal{C}$: *Chezy* constant ($m^{1/2} \cdot s^{-1}$)
- S_0 : Slope of the pipe (m/m)
- R_h : Hydraulic radius (m);

$$R_h = \frac{A}{P_w} \tag{4.3}$$

- P_w : Wetted perimeter (m)

For calculating the *Chezy* coefficient, there are various formulas, among which are:

1. *Darcy-Weisbach* Formula:

$$\mathcal{C} = \sqrt{\frac{8g}{f}} \tag{4.4}$$

   where f is the *Darcy-Weisbach* coefficient. The definition of this coefficient is similar to that given for pressurized circular pipes. A universal formulation is given by the *Moody* chart (Fig. 4.1).

2. *Bazin* Formula (1897):

$$\mathcal{C} = \frac{87\sqrt{R_h}}{K_B + \sqrt{R_h}} \tag{4.5}$$

   where K_B is the *Bazin* coefficient

   - Combined system: $K_B = 0.46$
   - Separate system (Stormwater EP): $K_B = 0.46$
   - Separate system (Wastewater EU): $K_B = 0.25$

3. *Kutter* Formula (1869):

$$\mathcal{C} = \frac{100\sqrt{R_h}}{K_K + \sqrt{R_h}} \tag{4.6}$$

   where K_K is the *Kutter* coefficient.

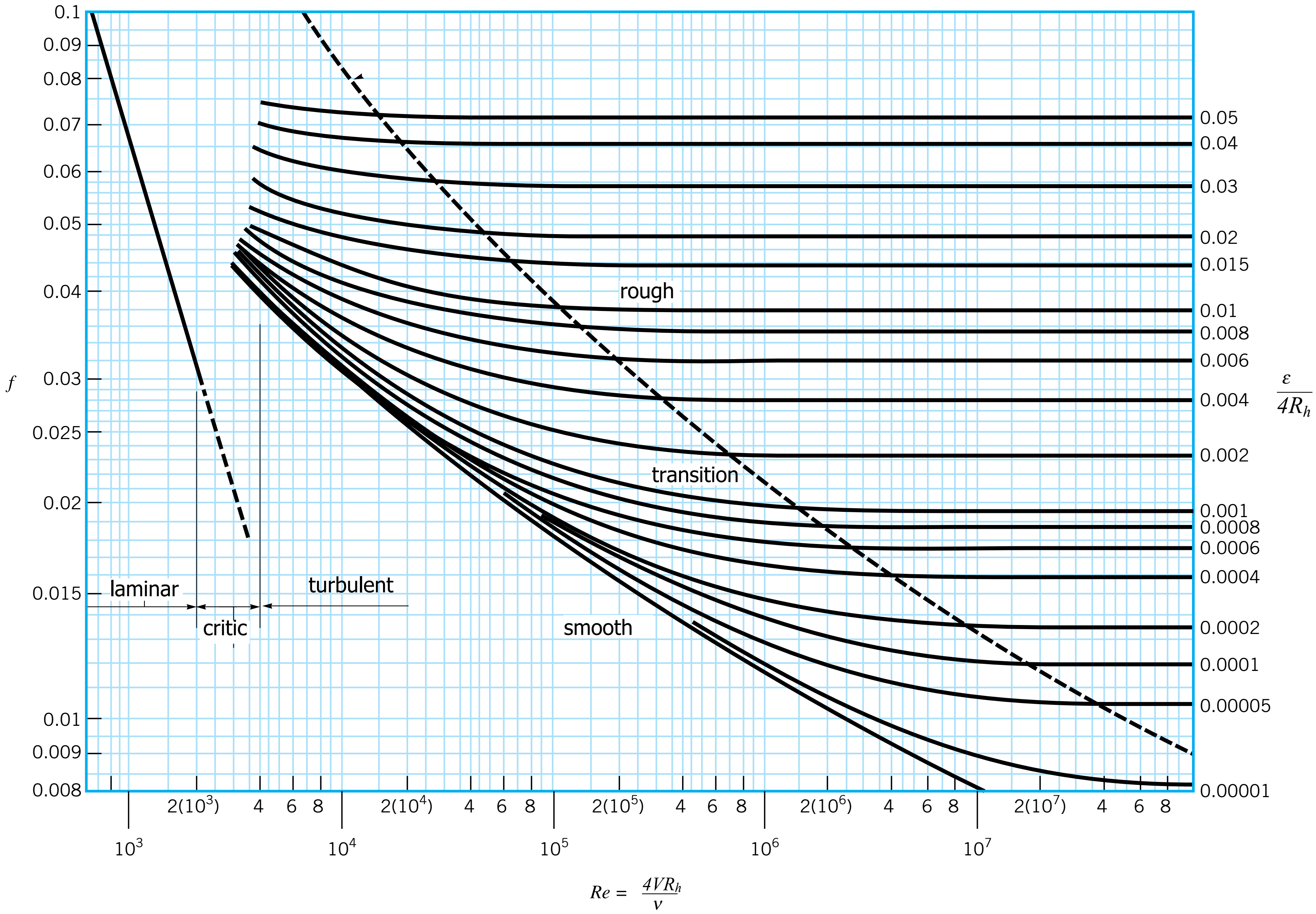


Figure 4.1: Moody chart.

- $K_K = 0.25$ for very smooth-walled sewers, mainly for clean water pipes,
- $K_K = 0.35$ most commonly used for sewers, regardless of the system and material used.

4. *Manning* (1889) and *Strickler* (1891) Formula:

$$\mathcal{C} = \frac{1}{n} R_h^{1/6} = K_S \cdot R_h^{1/6} \tag{4.7}$$

thus, the widely used *Manning-Strickler* formula for velocity:

$$V = \frac{1}{n} R_h^{2/3} S_0^{1/2} = K_S R_h^{2/3} S_0^{1/2} \tag{4.8}$$

with n being the *Manning* coefficient and K_S being the *Strickler* coefficient, whose usual values are given in Table 4.1

Table 4.1: Roughness coefficients K_S from the *Manning-Strickler* formula (*Satin et al.*, 1995).

Natural ditches in very poor condition and low slope	10
Ditches in very poor condition, slope > 3	20
Rough gutters (pebbles, grass...)	30
Clay soil gutter	40
Gutter in large masonry or stabilized soil	50
Coated gutter	60
Concrete gutter	70
Concrete collector with numerous connections	70
Smooth concrete collector, asbestos-cement	80
Large diameter collector	90
Cast iron, metal, PVC collector	100

4.2 Notion of Self-Cleansing

Self-cleansing refers to the ability of a pipeline to clean itself in low-flow conditions, known as dry weather flow. It also includes the capacity to carry away sand and other materials solely due to the flow being driven at a sufficient velocity.

4.3 Design of Combined Sewer Systems

In sanitation, preliminary sizing is done by considering the full participation of the pipeline section, meaning the full section.

4.3.1 Circular Section

When water flows along a partially full conduit, several properties can be defined as shown in Table 4.2.

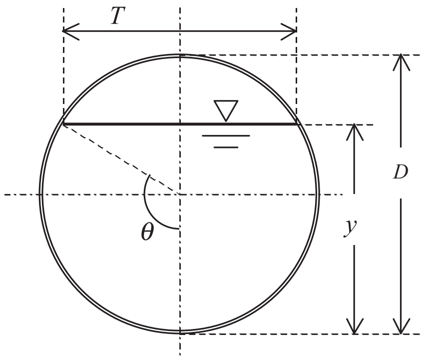
For a full section: $R_h = D/4$.

The *Chezy* equation allows writing for a circular section of initial diameter d_0, which is most often non-standard (non-nominal):

- Using the *Manning-Strickler* formula:

$$d_0 = \left(\frac{3.208 n Q}{S_0^{1/2}} \right)^{0.375} \tag{4.9}$$

Table 4.2: Expressions of geometric elements in a partially full circular pipe



Parameter	Symbol	Expression (θ in $^\circ$)
Wetted Area	A	$\frac{1}{8}\left(\frac{\pi\theta}{90} - \sin 2\theta\right) D^2$
Wetted perimeter	P_w	$\frac{\pi\theta}{180} D$
Hydraulic radius	R_h	$\frac{1}{4}\left(1 - \frac{90\sin 2\theta}{\pi\theta}\right) D$
Top width	T	$D\sin\theta$
Water depth	y	$\frac{D}{2}(1 - \cos\theta)$
Angle	θ	$\theta = 180 - \arccos\left(\frac{2y}{D} - 1\right)$

- Using the *Bazin* formula with $K_B = 0.46$

$$d_0 = \left(\frac{Q}{16.7 S_0^{1/2}}\right)^{4/11} \tag{4.10}$$

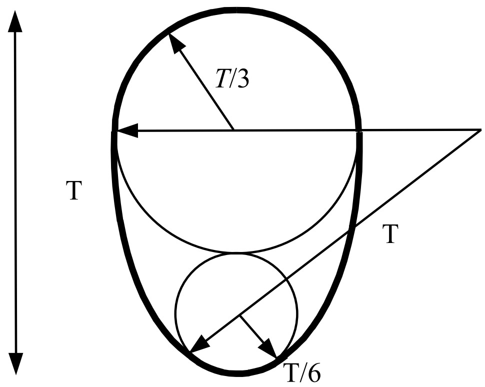
- Using the *Darcy-Weisbach* formula:

$$d_0 = \left(\frac{0.811 f Q^2}{g S_0^{1/2}}\right)^{0.2} \tag{4.11}$$

d_0 must be rounded to the nearest nominal diameter D. In a combined system, the minimum diameter to consider is $D_{min} = 300$ mm (*Gomella et Guerrée*, 1986).

The *Bazin* charts shown in figures 4.3 to 4.5 can also be used to determine the diameter D.

4.3.2 T or Ovoid Section



Ovoid (full section)
$A = 0.510\text{T}^2$
$P_w = 2.643\text{T}$
$R_h = 0.193\text{T}$

Figure 4.2: Geometry of T or ovoid sewers (*Butler et al.*, 2018).

Using the *Bazin* formula with $K_B = 0.46$:

$$\text{T} = 1.5\left(\frac{Q}{27.3 S_0^{1/2}}\right)^{4/11} \tag{4.12}$$

The standard T sections are T100, T130, T150, T180, T200. T100 means that the height is 100 cm.
In practice, beyond $D = 600$ mm, some designers recommend using T sections which provide better flow during dry weather by concentrating the flow during the dry weather period.

4.3.3 Flow Conditions

- Self-cleansing conditions are met by achieving velocities of 0.60 m/s for 1/10 of the full section flow and 0.30 m/s for 1/100 of the same flow. These velocities are obtained with full section velocities around 1.00 m/s in circular pipes and 0.90 m/s in ovoid pipes. The lines $V = 1.00$ m/s and $V = 0.90$ m/s on the charts shown in figures 4.3 and 4.5 respectively define the practical usage limits of these two types of structures in combined sewer systems.

- If self-cleansing conditions are not met, either automatic flushing devices must be installed, or periodic cleaning equipment must be used.

- To prevent wall abrasion and joint degradation in non-accessible structures or ensure the safety of personnel in accessible structures, the water velocity should not exceed 4 m/s. This results in the second usage limit shown on the same charts.

- If the terrain slope is too steep, drops must be installed in manholes.

- Finally, in connecting secondary structures to main structures, a step in the invert should be provided to prevent any slowing backwater from forming in the former (*Gomella et Guerrée*, 1986).

4.4 Design of Separate Sewer Systems

4.4.1 Wastewater Pipes

- Using the *Bazin* formula with $K_B = 0.25$, the initial diameter of the pipe is given by:

$$d_0 = \left(\frac{Q}{21.8 S_0^{1/2}} \right)^{3/8} \tag{4.13}$$

- Only circular sections are used. The minimum diameter of the pipes will be 0.20 m.

- Self-cleaning of the pipes in a separate system is considered ensured if the following three conditions are met:

  a) Full flow velocity V_{fs} greater than 0.7 m/s or, in the extreme, 0.50 m/s;

  b) y/D greater than 0.2 for the average flow;

  c) Actual velocity greater than 0.3 m/s for $y/D = 0.2$;

- The maximum velocity should not exceed 4 m/s.

4.4.2 Stormwater Pipes

- They are calculated similarly to combined sewer systems.

- The minimum diameter of the pipes will be 0.30 m.

- The self-cleaning velocity will be considered equal to 0.3 m/s for a flow passing at 1/10th of the full flow rate.

- The maximum velocity should not exceed 4 m/s.

4.5 Hydraulic Parameters

The full flow rate (Q_{fs}) is given by the equation:

$$Q_{fs} = \frac{\pi D^2}{4} V_{fs} \tag{4.14}$$

where V_{fs} is the full flow velocity given by the equation:

$$V_{fs} = \frac{\mathcal{C}}{2}\sqrt{DS_0} \tag{4.15}$$

The *Bazin* chart can also be used to determine Q_{fs} and V_{fs}.

To determine the actual flow velocity and the filling height for each section, the flow ratio R_Q is calculated:

$$R_Q = \frac{Q_T}{Q_{fs}} \tag{4.16}$$

Using the chart shown in Figure 4.6, the values of R_V (velocity ratio) and R_y (height ratio) can be determined.

The actual flow velocity V in m/s for the flow rate Q_T can then be calculated using the following equation:

$$V = R_V V_{fs} \tag{4.17}$$

and the filling height y in mm using the equation:

$$y = R_y D \tag{4.18}$$

where D is the nominal pipe diameter in mm.

In the case of using the *Manning-Strickler* formula, the simplified relation from *Riabi* (2012) can be used to calculate the ratio of heights:

$$R_y = \frac{2.112}{\pi}\sin^{-1}\left(1.64 Q_*^{0.477}\right) \tag{4.19}$$

where Q_* is the relative conductivity obtained by:

$$Q_* = \frac{nQ}{D^{8/3}\sqrt{S_0}} \tag{4.20}$$

It is also derived that:

$$R_y = \frac{2.112}{\pi}\sin^{-1}\left(0.94 R_Q^{0.477}\right) \tag{4.21}$$

4.6 Examples

Example 4.1 A circular pipe with a nominal diameter of $D = 400$ mm, carrying a flow rate Q with a free surface, has a slope of 1%. Determine the height ratio of the pipe and the corresponding flow rate for an average flow velocity of 2.0 m/s. The *Strickler* coefficient for the channel is given as K_S = 100. ■

Solution 4.1

Given: D=400 mm, $S_0 = 1\% = 0.01$ m/m, $V = 2$ m/s, $K_S = 100$.

According to the *Manning-Strickler* equation, eq. (4.8): $V = K_S R_h^{2/3} S_0^{1/2}$

At full section $R_h = D/4$, the full flow velocity is:

$V_{fs} = K_S (D/4)^{2/3} S_0^{1/2} = 100(0.4/4)^{2/3}(0.01)^{1/2} = 2.15$ m/s

According to eq. (4.17), the velocity ratio is:

$$R_V = \frac{V}{V_{fs}} = \frac{2}{2.15} = 0.93$$

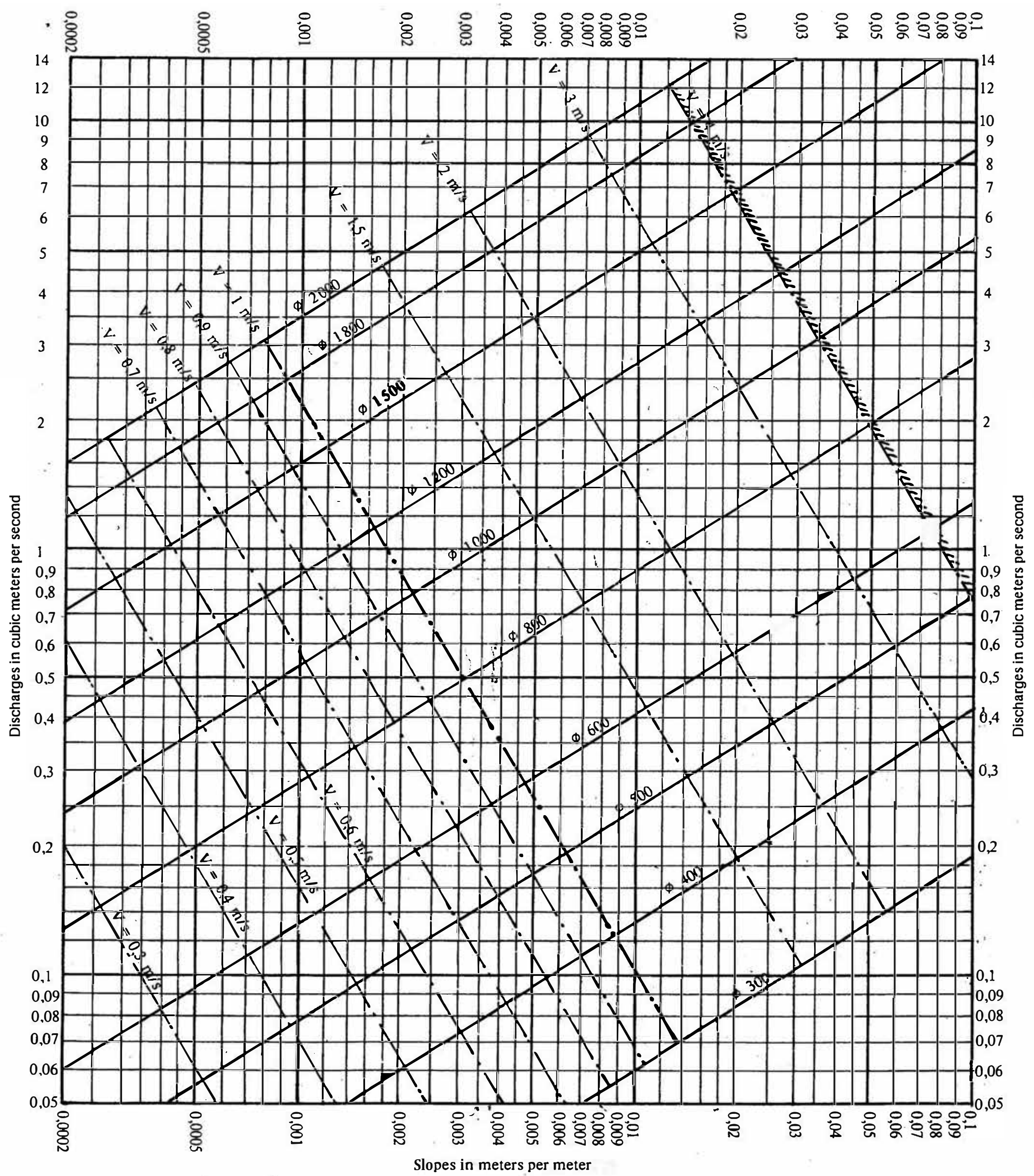


Figure 4.3: *Bazin* Chart for stormwater networks in a combined or separate system (Circular pipes) (*Gomella et Guerrée*, 1986).

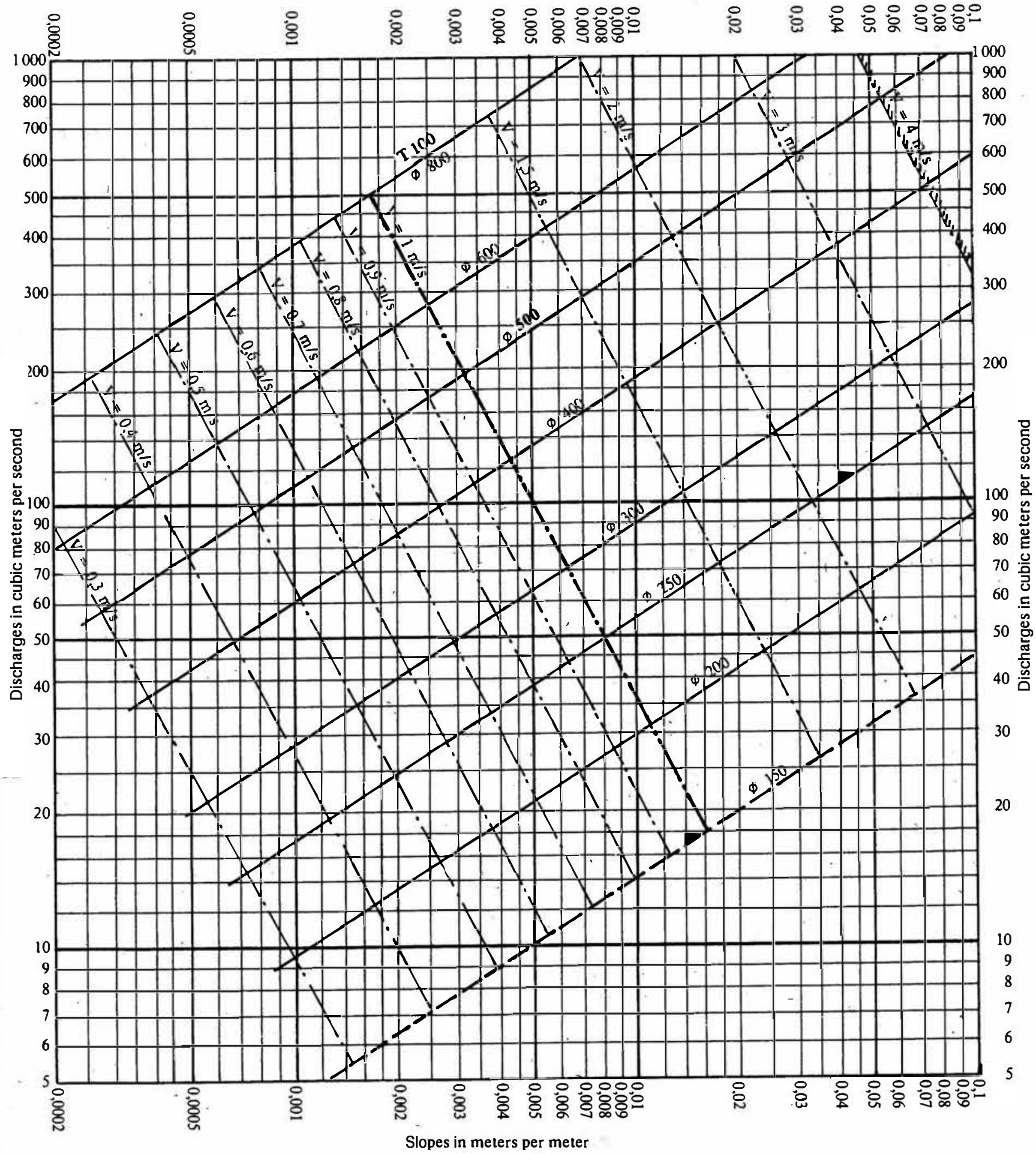


Figure 4.4: *Bazin* Chart for wastewater networks in a separate system (Gomella et Guerrée, 1986).

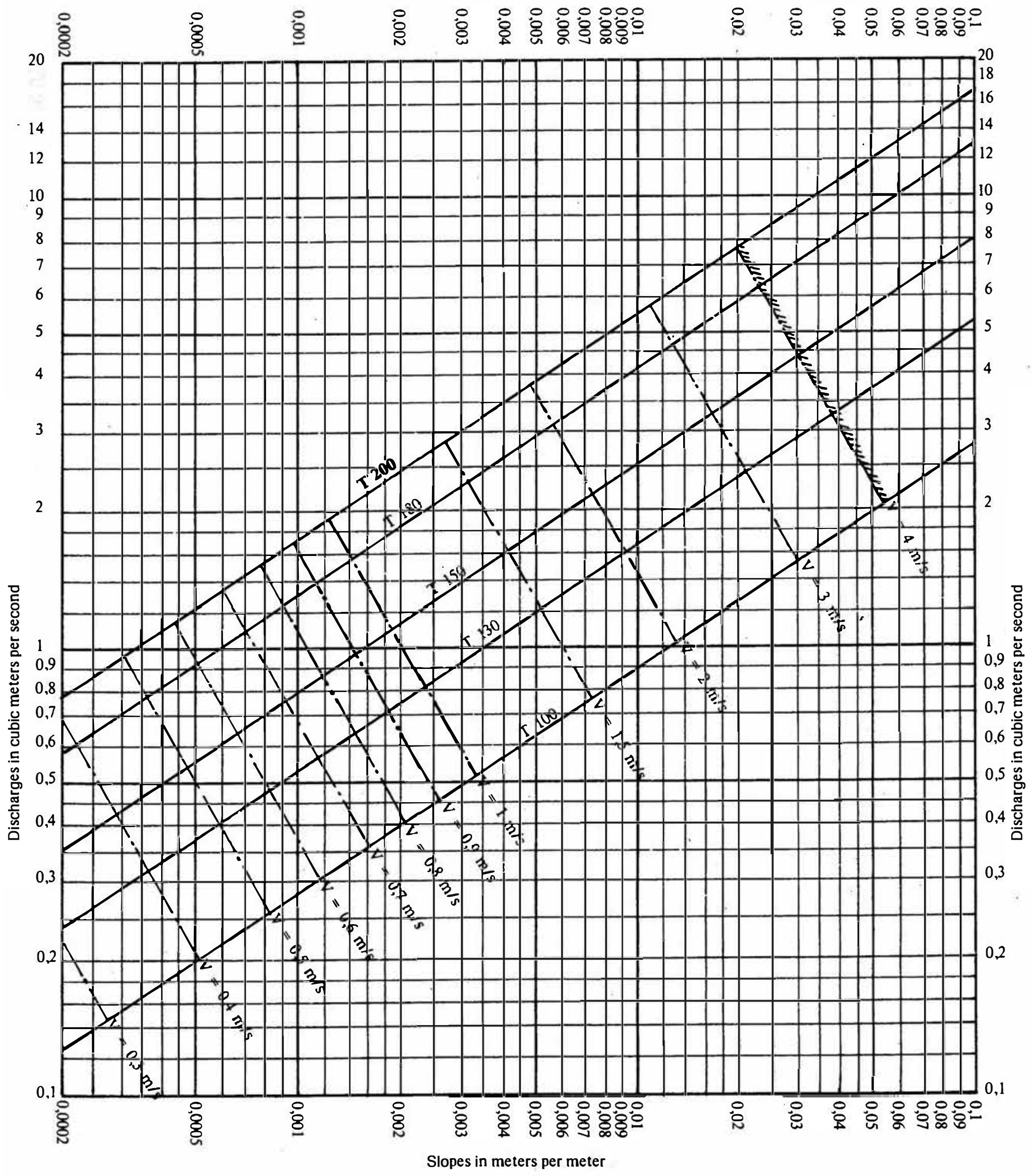


Figure 4.5: *Bazin* Chart for stormwater networks in a combined or separate system (Egg-shaped pipes) (*Gomella et Guerrée*, 1986).

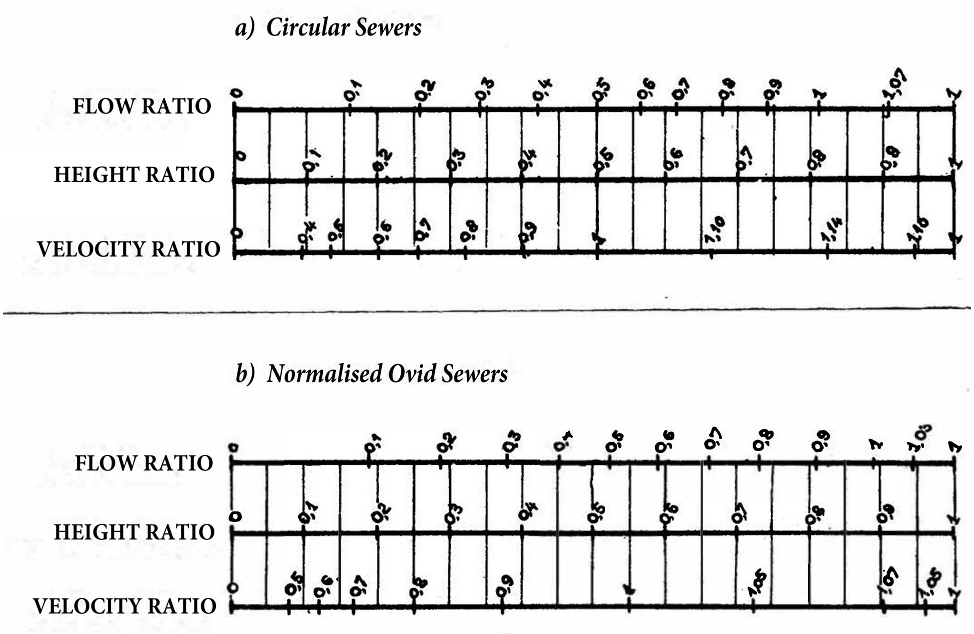


Figure 4.6: Variations of flow rates and velocities as a function of filling height (according to the *Bazin* formula) (*Gomella et Guerrée*, 1986).

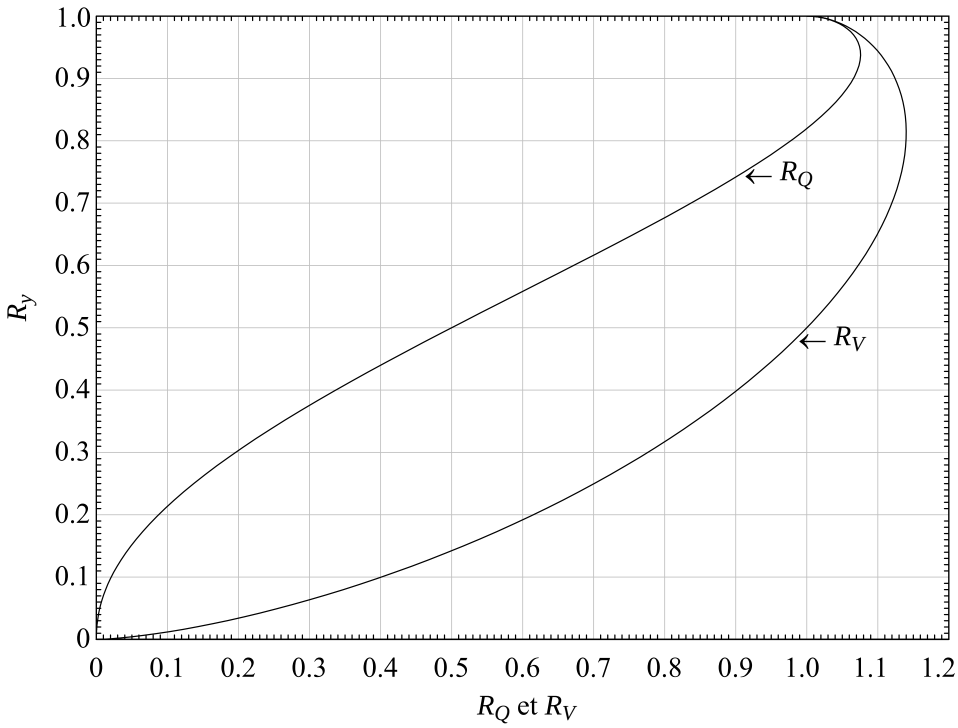


Figure 4.7: Variations of flow rates and velocities as a function of filling height (according to the *Manning-Strickler* formula for circular pipes).

From figure 4.7, and with $R_V = 0.93$, we obtain: $R_y = 0.43$ and $R_Q = 0.38$
The height ratio, according to eq. (4.18), is $y = R_y D = 0.43 \times 400$, $\boxed{y = 172 \text{ mm}}$.
The flow rate, according to eq. (4.16), is $Q = R_Q Q_{fs} = R_Q \frac{\pi D^2}{4} = 0.38 \frac{\pi 0.4^2}{4}$, $\boxed{Q = 0.102 \text{ m}^3/\text{s}}$.

Example 4.2 Consider a sewer pipeline with a circular cross-section, carrying a flow rate $Q = 30$ m^3/h and a slope $S_0 = 0.01$ m/m. Calculate the required diameter of the pipe for height ratios of 0.5, 0.8, and 0.9. The *Strickler* coefficient is given as K_S=70. Calculate the average flow velocity for each height ratio.

Solution 4.2
Given: $Q = 30 \text{ m}^3/\text{h}$, $S_0 = 0.01 \text{ m/m}$, $K_S = 70$
For a height ratio $R_y = 0.5$
From figure 4.7, and with $R_y = 0.5$, we obtain: $R_V = 1$ and $R_Q = 0.5$
The full-section flow rate, according to eq. (4.16), is $Q_{fs} = Q/R_Q = 30/0.5 = 60 \text{ m}^3/\text{h}$
The required diameter is calculated from eq. (4.9) by replacing n with $1/K_S$ and Q with Q_{fs}:

$$D = \left(\frac{3.208 Q_{fs}}{K_S S_0^{1/2}}\right)^{0.375} = \left(\frac{3.208(60/3600)}{70 \times 0.01^{1/2}}\right)^{0.375}, \boxed{D = 0.160 \text{ m}}.$$

The full-section velocity: $V_{fs} = \frac{4Q_{fs}}{\pi D^2} = \frac{4(60/3600)}{\pi \times 0.16^2} = 0.83 \text{ m/s}$,
The velocity, according to eq. (4.17), is $V = R_V V_{fs} = 1 \times 0.83$, $\boxed{V = 0.83 \text{ m/s}}$.

For a height ratio $R_y = 0.8$
From figure 4.7, and with $R_y = 0.8$, we obtain: $R_V = 1.14$ and $R_Q = 0.98$
The full-section flow rate, according to eq. (4.16), is $Q_{fs} = Q/R_Q = 30/0.98 = 30.61 \text{ m}^3/\text{h}$
The required diameter is calculated from eq. (4.9) by replacing n with $1/K_S$ and Q with Q_{fs}:

$$D = \left(\frac{3.208 Q_{fs}}{K_S S_0^{1/2}}\right)^{0.375} = \left(\frac{3.208(30.61/3600)}{70 \times 0.01^{1/2}}\right)^{0.375}, \boxed{D = 0.125 \text{ m}}.$$

The full-section velocity: $V_{fs} = \frac{4Q_{fs}}{\pi D^2} = \frac{4(30.61/3600)}{\pi \times 0.125^2} = 0.69 \text{ m/s}$,
The velocity, according to eq. (4.17), is $V = R_V V_{fs} = 1.14 \times 0.69$, $\boxed{V = 0.79 \text{ m/s}}$.

For a height ratio $R_y = 0.90$
From figure 4.7, and with $R_y = 0.9$, we obtain: $R_V = 1.12$ and $R_Q = 1.07$
The full-section flow rate, according to eq. (4.16), is $Q_{fs} = Q/R_Q = 30/1.07 = 28.15 \text{ m}^3/\text{h}$
The required diameter is calculated from eq. (4.9) by replacing n with $1/K_S$ and Q with Q_{fs}:

$$D = \left(\frac{3.208 Q_{fs}}{K_S S_0^{1/2}}\right)^{0.375} = \left(\frac{3.208(28.15/3600)}{70 \times 0.01^{1/2}}\right)^{0.375}, \boxed{D = 0.121 \text{ m}}.$$

The full-section velocity: $V_{fs} = \frac{4Q_{fs}}{\pi D^2} = \frac{4(28.15/3600)}{\pi \times 0.121^2} = 0.68 \text{ m/s}$,
The flow velocity, according to eq. (**??**), is $V = R_V V_{fs} = 1.12 \times 0.68$, $\boxed{V = 0.76 \text{ m/s}}$.

Example 4.3 A plot of land in a Mediterranean region, with an area of $A = 15$ ha, and a runoff coefficient of $C = 0.8$, is drained by a PVC pipe with a *Manning* coefficient $n = 0.01$ and a slope of $S_0 = 1\%$. The stormwater discharge with a 10-year frequency is estimated using the *Caquot* formula, with the following coefficients: $m = 1$, $K = 0.52$, $\alpha = 0.20$, $\beta = 1.11$, and $\gamma = 0.83$.

a) Determine the discharge to be evacuated by the pipe.

b) Calculate the required diameter of the pipe.

c) Verify the velocity and the self-cleaning condition.

■

Solution 4.3

a) The discharge to be evacuated by the pipe

The drainage system is a separate stormwater system.

Using eq. (3.33): $Q_{Storm} = m \cdot K \cdot S_0^{\alpha} \cdot C^{\beta} \cdot A^{\gamma} = 1 \times 0,52 \times 0,01^{0,20} \times 0,8^{1,11} \times 15^{0,83}$

$\boxed{Q_{Storm} = 1.53 \text{ m}^3/\text{s}}$.

b) The required diameter of the pipe

The initial diameter is calculated using eq. (4.9), replacing Q with Q_{Storm}:

$$d_0 = \left(\frac{3.208 n Q_{Storm}}{S_0^{1/2}}\right)^{0.375} = \left(\frac{3.208 \times 0.01 \times 1.53}{0.01^{1/2}}\right)^{0.375} = 0.766,$$

We take the standardized diameter $\boxed{D = 0.800 \text{ m}}$.

c) Verification of the velocity and self-cleaning condition

At full capacity $R_h = D/4$, the full-section discharge is:

$$Q_{fs} = \frac{\pi D^2}{4n}(D/4)^{2/3} S_0^{1/2} = \frac{\pi 0.8^2}{4 \times 0.01}(0.8/4)^{2/3}(0.01)^{1/2} = 1.72 \text{ m/s}$$

According to eq. (4.16), the flow ratio is: $R_Q = \dfrac{Q_{Storm}}{Q_{fs}} = \dfrac{1.53}{1.72} = 0.89$

From figure 4.7, and with $R_Q = 0.89$, we obtain: $R_V = 1.13$

The full-section velocity: $V_{fs} = \dfrac{4Q_{fs}}{\pi D^2} = \dfrac{4 \times 1.72}{\pi \times 0.8^2} = 3.42 \text{ m/s}$,

The flow velocity, according to eq. (4.17), is $V = R_V V_{fs} = 1.13 \times 3.42$, $\boxed{V = 3.86 \text{ m/s}}$.

$V < 4$ m/s, thus verified

The self-cleaning velocity is considered to be 0.3 m/s for a flow rate of 1/10th of the full-section discharge.

From figure 4.7, and with $R_Q = 1/10$, we obtain: $R_V = 0.63$

The self-cleaning velocity $V_{sc} = R_V V_{fs} = 0.63 \times 3.42$, $\boxed{V_{sc} = 2.15 \text{ m/s}}$.

$V_{sc} > 0.3$, thus verified for the separate stormwater system. ■

Example 4.4 A new 30-m-long pipe segment is to be designed to accommodate a peak flow rate of $1.2 \text{ m}^3/\text{s}$. The new pipe segment is to be an extension of an existing upstream pipe segment that has a diameter of 685 mm. At the upstream end of the new pipe segment, the crown elevation matches the crown of the existing pipe and just meets the minimum-cover requirement of 1.0 m. Local regulations require a minimum full-flow (self-cleansing) velocity of 1.0 m/s, a maximum velocity less than 4.5 m/s, and that reinforced concrete pipe be used with a design *Manning*'s n of 0.013. The ground elevation at the upstream and downstream ends of the pipe segment are 10.04 m and 9.89 m, respectively. Determine the diameters and corresponding slopes that could be used in the new pipe segment. ■

Solution 4.4

Data: $L = 30$ m, $Q = 1.2 \text{ m}^3/\text{s}$, $z_U = 10.04$ m, $z_D = 9.89$ m, $V_{sc} = 1.0$ m/s, $V_{max} = 4.5$ m/s, and $n = 0.013$.

The allowable pipe diameter and slope combinations are determined as follows:

Step 1: Determine the slope to meet the minimum-cover requirement. Since minimum cover already exists at the upstream end of the pipe segment, the slope required to maintain minimum cover is equal to the ground slope, S_{ref1}, given by

$$S_{ref1} = \frac{z_U - z_D}{L} = \frac{10.04 - 9.89}{30} = 0.005$$

Step 2: Assume $D = 685$ mm, which is equal to the diameter of upstream pipe segment.

Step 3: Use the *Manning* equation as the basis for design. For full-flow conditions, the *Manning* equation can be expressed as

$$Q = \frac{1}{n} S R_h^{2/3} S_0^{1/2} = \frac{1}{n} \left(\frac{\pi D^2}{4} \right) \left(\frac{D}{4} \right)^{2/3} S_0^{1/2} = \frac{0.312}{n} D^{8/3} S_0^{1/2}$$

If Q is taken as the design flow rate and D as the given pipe diameter, then the minimum slope, S_{ref2}, required to accommodate the design flow rate is given by

$$S_{ref2} = \left[\frac{nQ}{0.312 D^{8/3}} \right]^2 = \left[\frac{0.013 \times 1.2}{0.312 \times 0.685^{8/3}} \right]^2 = 0.0188$$

Step 4: To determine the slope, S_{ref3}, required to meet the minimum-velocity requirement under fullflow conditions, the *Manning* equation can be put in the form

$$V_{sc} = \frac{1}{n} R_h^{2/3} S_{ref3}^{1/2} = \frac{1}{n} \left(\frac{D}{4} \right)^{2/3} S_{ref3}^{1/2}$$

which can be put in a more convenient form

$$S_{ref3} = 6.35 \left(\frac{nV_{sc}}{D^{2/3}} \right)^2 = 6.35 \left(\frac{0.013 \times 1}{0.685^{2/3}} \right)^2 = 0.00178$$

Step 5: The minimum slope, S_0, for a pipe diameter of 685 mm is then given by

$$S_0 = \max(S_{ref1}, S_{ref2}, S_{ref3}) = \max(0.005, 0.0188, 0.00178) = 0.0188$$

Step 6: At a slope of 0.0188, the pipe flows full and the velocity is given by

$$V_{fs} = \frac{Q}{S} = \frac{1.2}{\frac{\pi}{4} 0.685^2} = 3.3 \text{ m/s}$$

Therefore, under design conditions, the velocity (3.3 m/s) is less than the maximum allowable velocity of 4.5 m/s.

These calculations show that using a pipe diameter of 685 mm and a slope of 0.0188 will meet the design requirements. The slope of 0.0188 is greater than the ground slope of 0.005 and so the minimum-cover requirement is exceeded at the downstream end of the pipe segment.

The required slope can be decreased by increasing the pipe diameter, and these options can be determined by repeating the above calculations for increased (commercial) pipe sizes. The results are as follows:

Table S4.4

D (mm)	S_0 (–)
685	0.0188
760	0.0108
840	0.00634
915	0.00402

Since the ground slope is 0.005, it is apparent that the pipe diameter can be increased up to 840 mm to reduce the pipe cover and still maintain the minimum cover. However, once the pipe diameter is increased it usually cannot be decreased further downstream. Hence, the final choice of pipe diameter and slope will be influenced by downstream conditions. ■

Example 4.5 A city plans to develop a new residential district on its outskirts. The area is not yet equipped for wastewater disposal. The total flows (wastewater + stormwater) are provided in Table E4.5. The layout of the network and the elevations of the associated manholes are given in

Figure E4.5. Using the *Bazin* charts, calculate the pipe diameters and hydraulic parameters. For each pipe, check that the minimum self-cleaning velocity is maintained. If not, propose a solution to meet this constraint.

Table E4.5

No. Segment	L (m)	Node Upstream	Node Downstream	Flow (m^3/s)
1	40	1	3	0.087
2	50	2	3	0.100
3	40	3	5	0.202
4	70	4	5	0.018
5	64	5	7	0.268
6	55	6	7	0.139
7	80	7	8	0.468

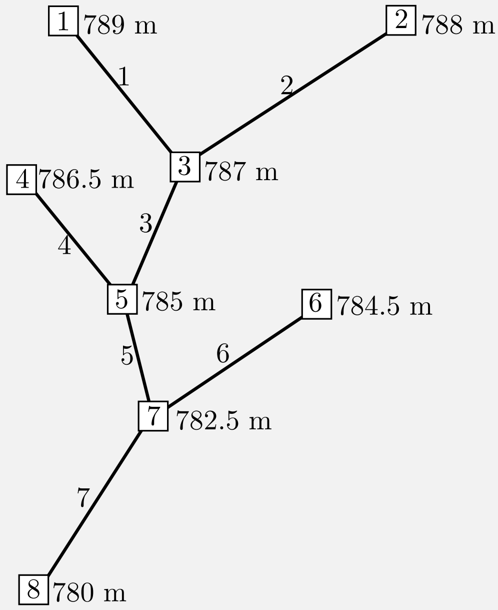


Figure E4.5

Solution 4.5

The calculations for the diameters and hydraulic parameters are shown in Table S4.5.
Columns 1 to 3 and 6 are data from Table E4.5.
Columns 4 and 5 are data from Figure E4.5.
Column 7 = (upstream elevation - downstream elevation)/L.
Columns 8 to 10 are obtained from Figure 4.3 using the slope.
Column 11 = Col. 3 / Col. 9: $R_Q = Q/Q_{fs}$.
Columns 12 and 13 are obtained from Figure 4.6 using the values from Col. 11.
Column 14 = Col. 8 $\times$ Col. 13: $y = R_y D$.
Column 15 = Col. 10 $\times$ Col. 12: $V = R_V V_{fs}$.
Column 16 = Col. 10 $\times$ 0.60: $V_{sc} = 0.54 V_{fs}$.
Examining the results in Col. 15 and Col. 16, we note that $V < 4$ m/s and $V_{sc} > 0.6$ for all segments.

4.7 Review Questions

1. What is the recommended minimum flow velocity in a sewer pipe?
2. Among the flow conditions to be met in sewer systems, there is the velocity condition $V_{\min} \leq V \leq V_{\max}$; explain the importance of this condition? In other words, why is this condition necessary?
3. What should be done if self-cleaning conditions are not achieved?
4. What does the term "self-cleaning" mean?
5. Why must a minimum velocity be ensured inside the pipes?
6. What are the main objectives of the hydraulic calculation for the drainage network?

Table S4.5: Solution to exercise 4.5

No.	Segment	Q	Ground Elevation		L	S_0	D	Full Section		Ratios			y	V	V_{sc}
	from...to...	(m^3/s)	upstream	downstream	(m)	(m/m)	(m)	Q_{fs}	V_{fs}	R_Q	R_V	R_y	(mm)	(m/s)	(m/s)
1	1-3	0.087	789	787	40	0.05	300	0.13	1.85	0.66	1.07	0.6	180	1.99	1.00
2	2-3	0.100	788	787	50	0.02	400	0.19	1.51	0.53	1.01	0.52	208	1.49	0.81
3	3-5	0.202	787	785	40	0.05	400	0.29	2.31	0.68	1.08	0.61	244	2.53	1.25
4	4-5	0.018	786.5	785	70	0.021	300	0.085	1.20	0.21	0.26	0.72	93	0.95	0.65
5	5-7	0.268	785	782.5	64	0.039	500	0.48	2.45	0.56	1.03	0.54	270	2.49	1.32
6	6-7	0.139	784.5	782.5	55	0.036	400	0.25	1.99	0.55	1.02	0.53	212	2.03	1.07
7	7-8	0.468	782.5	780	80	0.031	600	0.71	2.51	0.66	1.07	0.60	360	2.69	1.36

7. What data and parameters are necessary to perform an accurate hydraulic calculation of the drainage network?

8. Describe the different steps involved in the hydraulic calculation of a drainage network.

9. How are the physical characteristics of pipes and structures (diameter, slope, roughness) taken into account in the hydraulic calculation?

10. Describe the self-cleaning process and how it naturally occurs in pipes and sewers.

11. What are the main mechanisms that promote self-cleaning?

4.8 Objective Questions

Q 4.1 What is self-cleaning in sewer systems?

a) The process of manually cleaning pipes by workers.

b) The natural self-cleaning of sewer pipes by water flow.

c) The use of chemicals to remove blockages in pipes.

d) The installation of automatic cleaning devices in sewer networks.

■

Q 4.2 What are the benefits of self-cleaning in sewer systems?

a) Reduction in maintenance and cleaning costs.

b) Improvement in the hydraulic efficiency of the network.

c) Reduction in the risk of blockages and overflows of wastewater.

d) All of the above.

■

Q 4.3 What are the consequences if self-cleaning conditions are not met in a sewer network?

a) Accumulation of sediments and debris in the pipes.

b) Reduction in flow capacity in the pipes.

c) Risk of clogging and blocking of pipes.

d) All of the above.

■

Q 4.4 What factors can prevent the achievement of self-cleaning conditions?

a) Insufficient slope of the pipes.

b) Presence of grease deposits in the pipes.

c) Excessive pipe diameter relative to the flow.

d) All of the above.

■

Q 4.5 What measures can be taken to improve self-cleaning conditions in a sewer network?

a) Increase the slope of the pipes.

b) Use smoother lining materials to reduce roughness.

c) Implement regular maintenance programs for pipe cleaning.

d) All of the above.

■

Q 4.6 The minimum diameter in combined sewers is

a) 200 mm

b) 250 mm

c) 300 mm

d) 350 mm

■

Q 4.7 The minimum diameter in wastewater pipes of separate sewers is

a) 200 mm

b) 250 mm

c) 300 mm

d) 350 mm

■

Q 4.8 The minimum diameter in stormwater pipes of separate sewers is

a) 200 mm

b) 250 mm

c) 300 mm

d) 350 mm

■

Q 4.9 Which parameter is generally used to determine the pipe size in a sewer network?

a) The depth of the pipes.

b) The total length of the network.

c) The wastewater flow rate.

d) The type of pipe material.

■

Q 4.10 Two sewer pipes A and B with the same hydraulic radius and made of the same material are laid at slopes of 1/225 and 1/625, respectively. The ratio of their flow rates (Q_A/Q_B) is:

a) $\frac{5}{3}$

b) 1

c) $\frac{3}{5}$

d) $\frac{2}{5}$

■

Q 4.11 The self-cleaning conditions in combined sewers are met by achieving velocities

a) ≥ 1.0 m/s

b) ≥ 0.6 m/s

c) ≥ 0.3 m/s

d) < 4.0 m/s

■

Q 4.12 The sizing of sewer pipes involves the use of the following formula

a) *Manning-Strickler* formula

b) *Darcy-Weisbach* formula

c) *Bazin* formula

d) *Kutter* formula

■

Q 4.13 The curve of flow variations is used to find the hydraulic parameters corresponding to the following known element

a) Q/Q_{fs} (flow/full-section flow)

b) y/D (flow depth/pipe diameter)

c) V/V_{fs} (velocity/full-section velocity)

d) A/A_{fs} (wetted area/pipe area)

■

4.9 Problems

P 4.1 How much water is transported by an asbestos cement pipe with a diameter of 300 mm ($K_S = 98$) and a slope of 0.002? What is the velocity? What is the fill height if a water quantity of 33 L/s flows through this pipe, and what is the velocity? ■

P 4.2 In a circular sewer pipe with diameter D, the water height is $2D/3$ when the flow rate $Q_{2D/3}$ is 0.3 m^3/s. When it carries the minimum flow rate $Q_{\min}$ of 0.09 m^3/s, the flow velocity $V_{\min}$ is 0.6 m/s. Given that the *Manning* roughness coefficient is 0.015, calculate the diameter D and the slope of this pipe. ■

P 4.3 In a circular sewer pipe with diameter D, the water height is $0.75D$ when the flow rate $Q_{0.75D}$ is 0.14 m^3/s. When it carries the minimum flow rate $Q_{\min}$ of 0.03 m^3/s, the flow velocity $V_{\min}$ is 0.60 m/s. Given that the *Manning* roughness coefficient is 0.015, determine the diameter D, the slope S_0, and the capacity Q_{fs} of this pipe. ■

P 4.4 A concrete sewer pipe ($n = 0.013$) is to be laid parallel to the ground surface on a slope of 0.5% and is to be designed to carry 0.43 m^3/s of stormwater runoff. Estimate the required pipe diameter using (a) the *Manning* equation, and (b) the *Darcy–Weisbach* equation. ■

P 4.5 Determine the diameters for pipes 5.1, 5.2, 5.3, 6.1, and 7.1 shown in Fig. P4.5. The network characteristics are listed in Tables P4.5a and P4.5b.

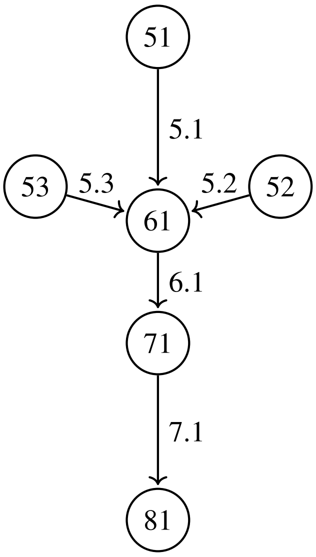


Figure P4.5

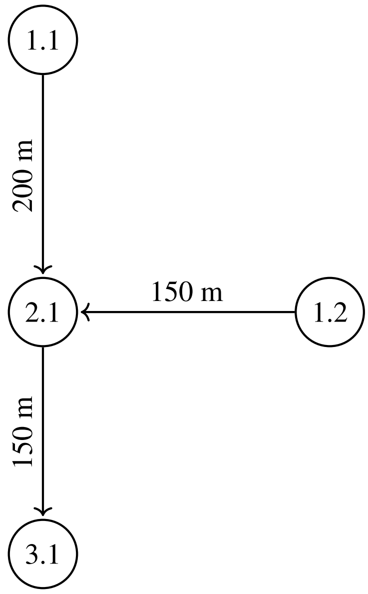


Figure P4.6

Table P4.5a: Sewers characteristics.

Sewer	L (m)	Q (m^3/s)
5.1	70	1.04
5.2	21	0.05
5.3	40	0.09
6.1	49	1.12
7.1	73	1.26

Table P4.5b: Manholes characteristics.

Manhole ID	Ground elevation (m)
51	219.49
52	219.83
53	219.54
61	219.30
71	218.05
81	217.96

P 4.6 Determine the pipe diameters for the storm sewer system in Figure P4.6. Assume that the design rainfall intensities can be determined from the equation $i = 977.265/(t_c + 10)^{0.786}$ (i in mm/h and t_c in minutes). Characteristics of the catchments are listed in Table E4.6. $n = 0.014$.

Table E4.6: Catchment Characteristics.

Catchment	Ground elev.(m)	A (km^2)	C	t_e (min)
1.1	300.0	0.010	0.60	25
1.2	298.0	0.008	0.75	20
2.1	296.0	0.005	0.80	15
3.1	294.5			

P 4.7 The layout of a sanitary sewer system is as shown in Figure P4.7. Data on area, length, and elevations are given in Table P4.7. The present population density, 100 persons/ha, is expected to increase to 250 persons/ha by conversation of the dwellings to apartments. The peak rate of sewage flow is 1600 L/d/person. Design the sewer system.

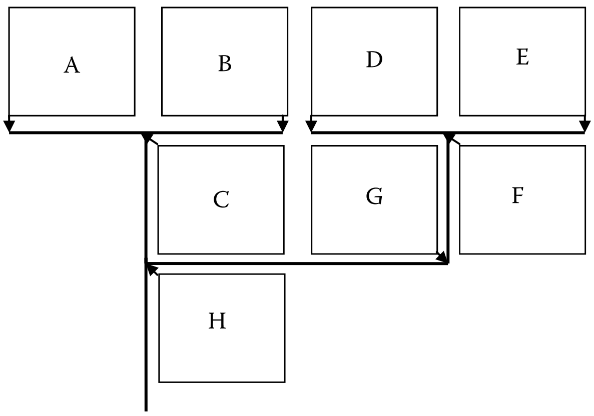


Figure P4.7

Table P4.7

Block	Area	Length	Elevation (m)	
			Up.	Dow.
	(m^2)	(m)	(m)	(m)
A	8000	118.87	30.94	29.62
B	10,000	106.68	30.68	29.62
C	6000	100.58	29.62	28.43
D	5200	70.1	30.08	29.73
E	4800	89.92	30.63	29.73
F	8400	91.44	29.73	28.74
G	22,800	198.12	28.74	28.43
H	14,000	167.64	28.43	26.34

P 4.8 A stormwater drainage system is to be designed for a new development, and the proposed pipeline segments and manholes are shown in Figure P4.8. Runoff contributions from individual subcatchments are added at the manhole locations, and the catchment and pipeline characteristics are presented in Tables P4.8a and P4.8b.

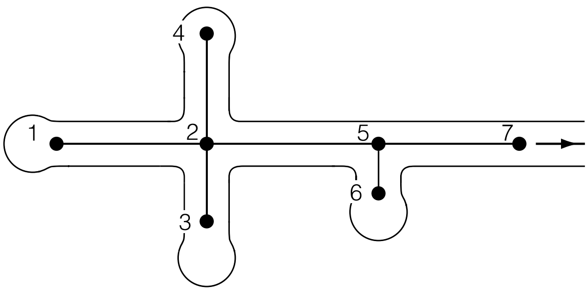


Figure P4.8

Table P4.8a

Location	Elev. (m)	A (ha)	t_e (min)	C (-)
1	30.064	2.00	8.0	0.70
2	29.263	1.50	7.4	0.85
3	29.655	0.71	6.5	0.55
4	29.705	0.86	6.8	0.62
5	28.328	1.20	7.0	0.9
6	28.558	0.30	5	0.75
7	27.653	—	—	—

Table P4.8b

Pipeline	Length (m)
1–2	100
3–2	55
4–2	73
2–5	110
6–5	30
5–7	90

Local regulations require that all pipelines be RCP with a design *Manning*'s n of 0.013, the minimum and maximum velocities under design conditions are 0.90 m/s and 4.5 m/s, respectively, the minimum pipe diameter is 455 mm, and the minimum cover is 1.0 m. The project is located in a region where the IDF curve is given by $i = \frac{2020}{(t+7.24)^0.73}$ where i is the average rainfall intensity [mm/h] and t is the duration [min]. Design the drainage system between Manholes 1 and 7.

P 4.9 Figure P4.9 schematically shows an urban watershed for which we want to calculate the dimensions of the various pipes. The characteristics of the pipes are listed in Table P4.9.

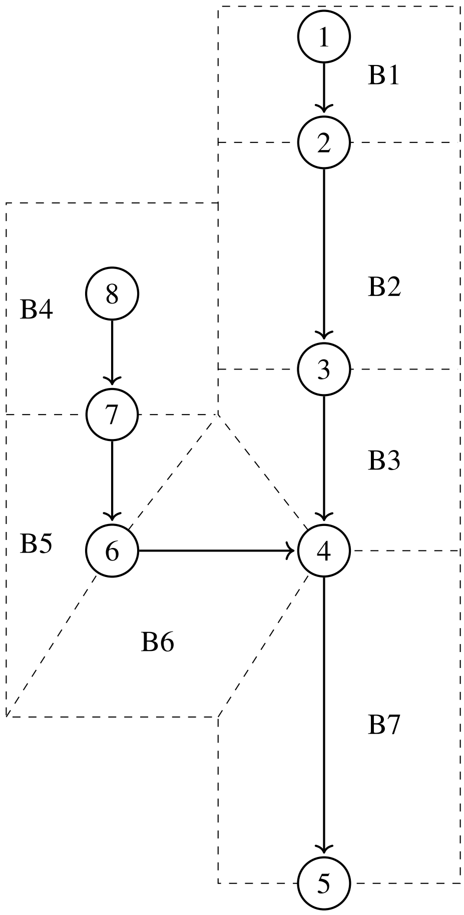


Figure P4.9

Table P4.9: Pipe Segment Characteristics

Segment	Ground Elevation Up.	Ground Elevation Dow.	L (m)	Q (L/s)
1-2	113.03	111.50	90	93.2
2-3	111.50	109.00	100	270.5
3-4	109.00	107.60	100	433.0
8-7	109.19	108.00	70	187.6
7-6	108.00	107.51	70	273.9
6-4	107.51	107.60	110	526.5
4-5	107.60	107.20	125	1279.0

■

P 4.10 The node-link system in Figure P4.10 represents a sewer system. The system has 8 sewers and 9 manholes. Manhole 16 is a confluence in the system. Manhole 99 is the system outlet. Tables P4.10a and P4.10b provides the network characteristics. Assuming that the Manning's $n = 0.013$, size the sewer pipes.

Table P4.10a: Sewers characteristics.

Sewer	L (m)	Q (m^3/s)
3512	137.2	0.258
1216	109.7	0.665
2316	140.2	0.416
1647	115.8	1.008
1547	89.9	0.379
4799	125.0	1.594
1747	61.0	0.059
1847	106.7	0.054

Table P4.10b: Manholes characteristics.

Manhole ID	Ground elevation (m)
35	33.8
12	33.2
23	33.5
16	30.9
15	31.7
47	30.2
99	29.7
17	30.4
18	30.4

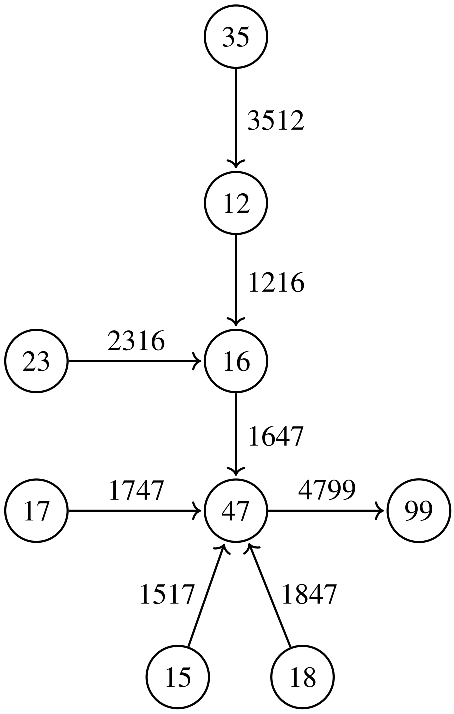


Figure P4.10

■

P 4.11 Use the following information to size the sewer pipes depicted in Figure P4.11. The design rainfall is given as: $i = 977.265/(t_c + 10)^{0.786}$ (i in mm/h and t_c in minutes).

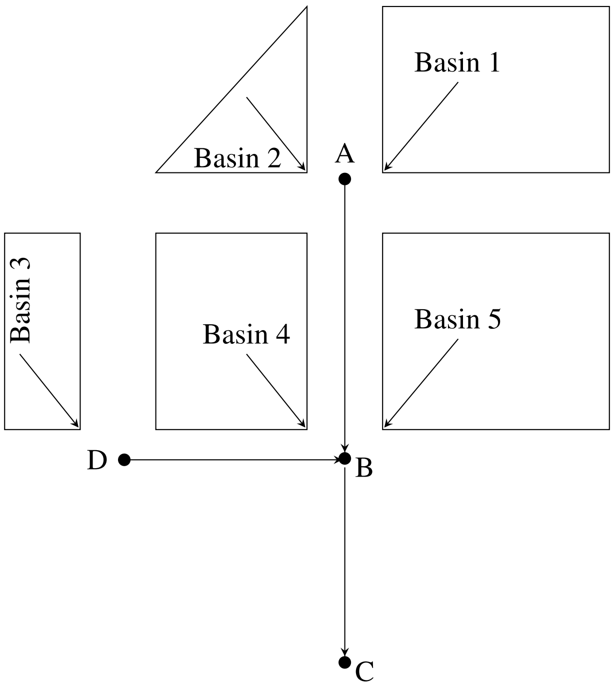


Figure P4.11

■

Table P4.11a: Information of sub-basins.

Basin Number	Area (ha)	Runoff Coeff.	Overland Slope (%)	Overland Length (m)	Channel Slope (%)	Channel Length (m)
1	1.62	0.8	1	53.3	I	167.6
2	3.24	0.6	2	76.2	2	198.1
3	1.21	0.75	1.5	91.4	1.5	106.7
4	3.44	0.8	1	83.8	1	198.1
5	2.83	0.85	1.5	76.2	I	182.9

Table P4.11b: Information of sewer pipes.

Sewer ID	L (m)	S_0 (%)	n
AB	182.9	1	0.02
DB	198.1	1	0.02
BC	198.1	1	0.02

P 4.12 Complete the Table P4.12

Table P4.12

Q (m^3/s)	S_0 (m/m)	D (mm)	Q_{fs} (m^3/s)	V_{fs} (m/s)	R_Q (-)	R_V (-)	R_y (-)	V (m/s)	y (mm)
	0.0025	800				1.02			
0.30	0.0020								
		500	0.45					2.29	
0.70		1000	1.00						
	0.0100	500							250
		300	0.08			0.70			
	0.0058		0.10		0.50				
	0.0020				0.50				400
	0.0350	500					0.26		
	0.0200	300						2.87	

5. Ancillary Structures of the Sewer Network

Ancillary structures are designed to facilitate the maintenance of the network, ensure the safety of inspection personnel, and increase the lifespan of the structures. They also serve to ensure the regular operation of the network by detecting any anomalies.

The consecutive elements of a sewer network are subdivided into:

1. Main structures: these are the structures responsible for the evacuation of water to their discharge point.

2. Ancillary structures: these are designed to facilitate the operation and maintenance of the network.

The objectives of this chapter are:

1. Understand the main structures of the sewer network;

2. Understand the ancillary structures of the sewer network.

5.1 Main Structures

The main structures include:

1. Circular sewers, which are distinguished by their internal diameters (nominal diameter in mm)

2. Ovoid sewers, which are distinguished by their internal heights (nominal height in cm)

3. Accessible structures with specific profiles, limited to large urban centers.

5.1.1 Circular Sewers

The use of circular conduits is reserved for small sections. The circular shape is simple to manufacture. It can be used for large sections with certain disadvantages:

- Significant width of the trench.

- Low flow velocity for small fill heights, leading to difficulties in cleaning and maintenance.

Table 5.1: Advantages and Disadvantages of Different Materials.

Type of Material	Advantages	Disadvantages
Compressed Concrete or Ordinary Vibrated Concrete (CC, OVC)	– Moderate cost	– Sensitive to H_2S – Poor performance in aggressive or poor-quality soils – Fragile – Short elements of 1 meter – Poor joints: mortar – High internal roughness – Short lifespan
Reinforced Vibrated Concrete (RVC)	– Good quality concrete (controllable) – Good breakage resistance – Elements of 2.5 meters – Moderate internal roughness that can be improved – 3 Strength Classes: 60 A, 90 A, and 135 A	– Sensitive to H_2S – Slightly higher costs due to the presence of steel – Steel coating needs monitoring (distance from the surface) – Limited diameter range – Sometimes needs appearance improvement – Heavy
Prestressed Concrete (PC)	– Steel savings – Good resistance – Fairly long elements: 4 to 6 meters	– Low resistance to mechanical attacks – Heavy
Ordinary Reinforced Centrifuged (ORC)	– Very good concrete quality – Good crush resistance – Impermeable – Waterproof joints – Elements: 3.50 meters	– Sensitive to H_2S – Low resistance to aggressive soils and waters – Moderate roughness – Heavy
Asbestos Cement (AC)	– Reduced weight per meter – Good corrosion resistance – Very impermeable – Elements: 5 meters	– Sensitive to H_2S – Moderate crush resistance – Poor resistance to aggressive waters – Diameter range does not exceed 1000 mm
PVC	– Very resistant to H_2S – Sufficient mechanical resistance – Very light – Very impermeable – Very easy to assemble – Waterproof joints – High resistance to common chemicals – Elements: 6 meters	– Expensive for diameters > 400 mm

5.1.2 Ovoid Sewers

It is preferable to use the ovoid shape when aiming to:

- Reduce the trench width

- Increase the flow velocity (or height) during low flows (improve self-cleaning)
- Facilitate network access

The standardized ovoid sections are: T100, T130, T150, T180, and T200. Other sections may be encountered and are defined by the dimensions: $D/H(H = 3/2D)$: 120/180, 160/240, 180/270, and 200/300.

5.1.3 Accessible Structures with Specific Profiles

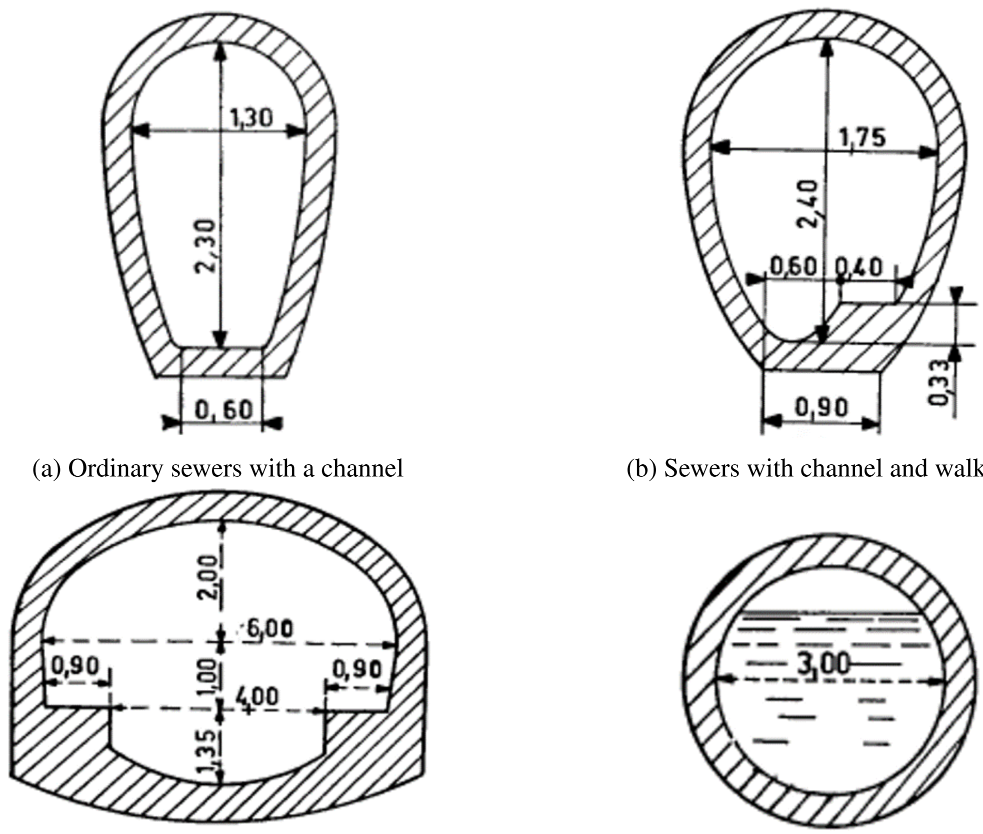
Special accessible structures built in situ in large urban centers can be classified into various categories (*Gomella et Guerrée*, 1986).

a) Ordinary sewers with a channel (Fig. 5.1a): Ordinary sewers with a channel ensure good water flow, existing in more or less ancient forms and in various dimensions.

b) Sewers with channel and walkway (Fig. 5.1b): The walkway allows for movement during dry periods for the maintenance of water pipes and telephone cables installed in the structure.

c) Collectors with channel and walkways (Fig. 5.1c): They include a central channel, up to 4 meters wide, and two lateral walkways for worker movement and guiding cleaning equipment.

d) Discharge outlets (Fig. 5.1d): The location of the WWTP at a considerable distance from the center necessitates the construction of special structures called discharge outlets. These structures, which receive no intermediate input, are not accessible during normal operation; to avoid sediment production, they are usually preceded by sand traps.



(a) Ordinary sewers with a channel

(b) Sewers with channel and walkway

(c) Collectors with channel and walkways

(d) Discharge outlets

Figure 5.1: Accessible structures with specific profiles (*Gomella et Guerrée*, 1986).

5.2 Auxiliary Structures

Auxiliary structures include:

- Manholes
- Gutters
- Pavement Drainage Inlets
- Special connections
- Blind manholes
- Storm overflows
- Retention basins

5.2.1 Manholes

Purpose: Manholes

- allow access to sewers for inspection, maintenance, cleaning and repair
- ensure network ventilation.

Components: They must be installed

- at the head of each run;
- at each change in direction;
- at each change in slope;
- at each change in pipe size;
- at the junctions of multiple collectors;
- in case of drops;
- every 40 to 50 meters for pipes with a diameter less than 1000 mm;
- every 80 to 120 meters for pipes with a diameter greater than 1000 mm.

Location: On non-accessible sewers, inspection manholes are placed in the axis of the pipeline. Inspection manholes include:

- a base;
- a vertical shaft;
- an upper slab;
- a cover system (a frame and a circular sealing cap of at least 600 mm in diameter);
- a ladder or descent steps.

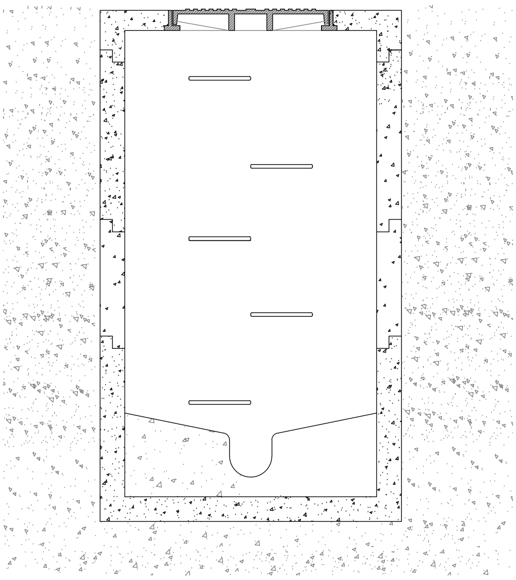
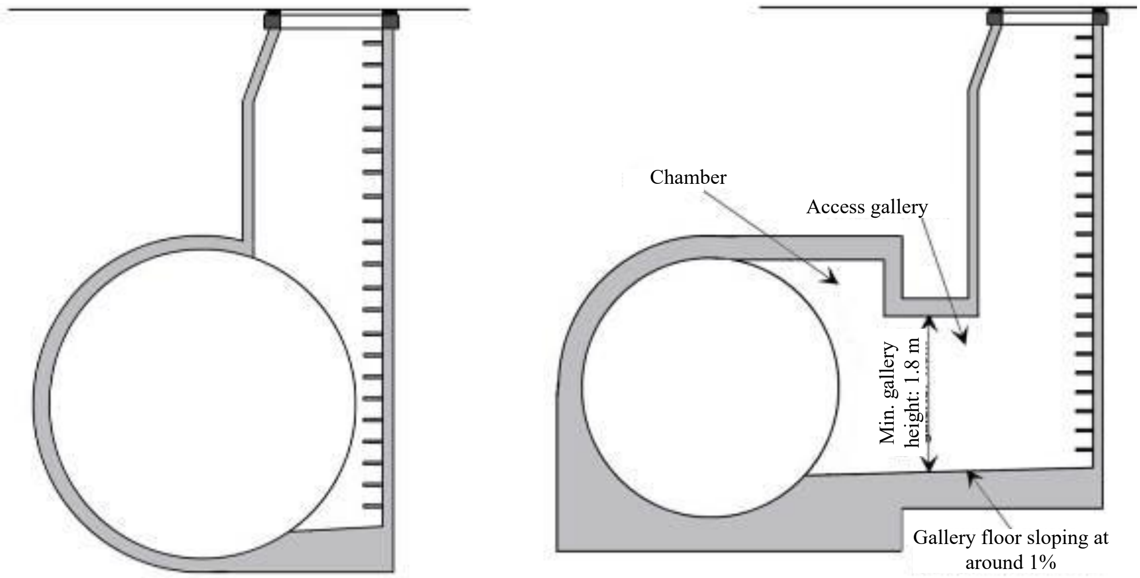


Figure 5.2: Manhole.

In accessible sewers, manholes can have side access to the collector (Fig. 5.3a) or include an access gallery with a base inclined towards the sewer (Fig. 5.3b).



(a) Manhole with side access to the collector (b) Manhole with access to the collector via gallery

Figure 5.3: Access structures on accessible networks.

5.2.2 Gutters

Gutters are designed to collect street water into storm drains. When there is a sidewalk, they consist of a curb and a paved surface or a prefabricated slab (*Gomella et Guerrée*, 1986). Various gutter sections are presented in Figure 5.4.

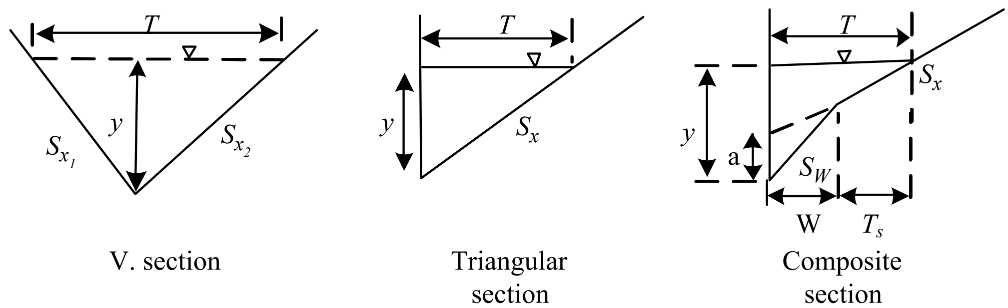


Figure 5.4: Various gutter sections.

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane, or pavement section. The following form of *Manning*'s equation can be used to evaluate gutter flow hydraulics:

$$Q = \frac{T^{8/3} S_x^{5/3} S_0^{1/2}}{2.64n} \tag{5.1}$$

where

- Q = gutter flow,
- T = top width,
- n = *Manning*'s roughness,
- S_x = cross slope, and

- S_0 = longitudinal bottom slope.

A nomograph for solving Equation (5.1) is presented in Figure 5.5. *Manning*'s n values for various pavement surfaces are given in Table 5.2.

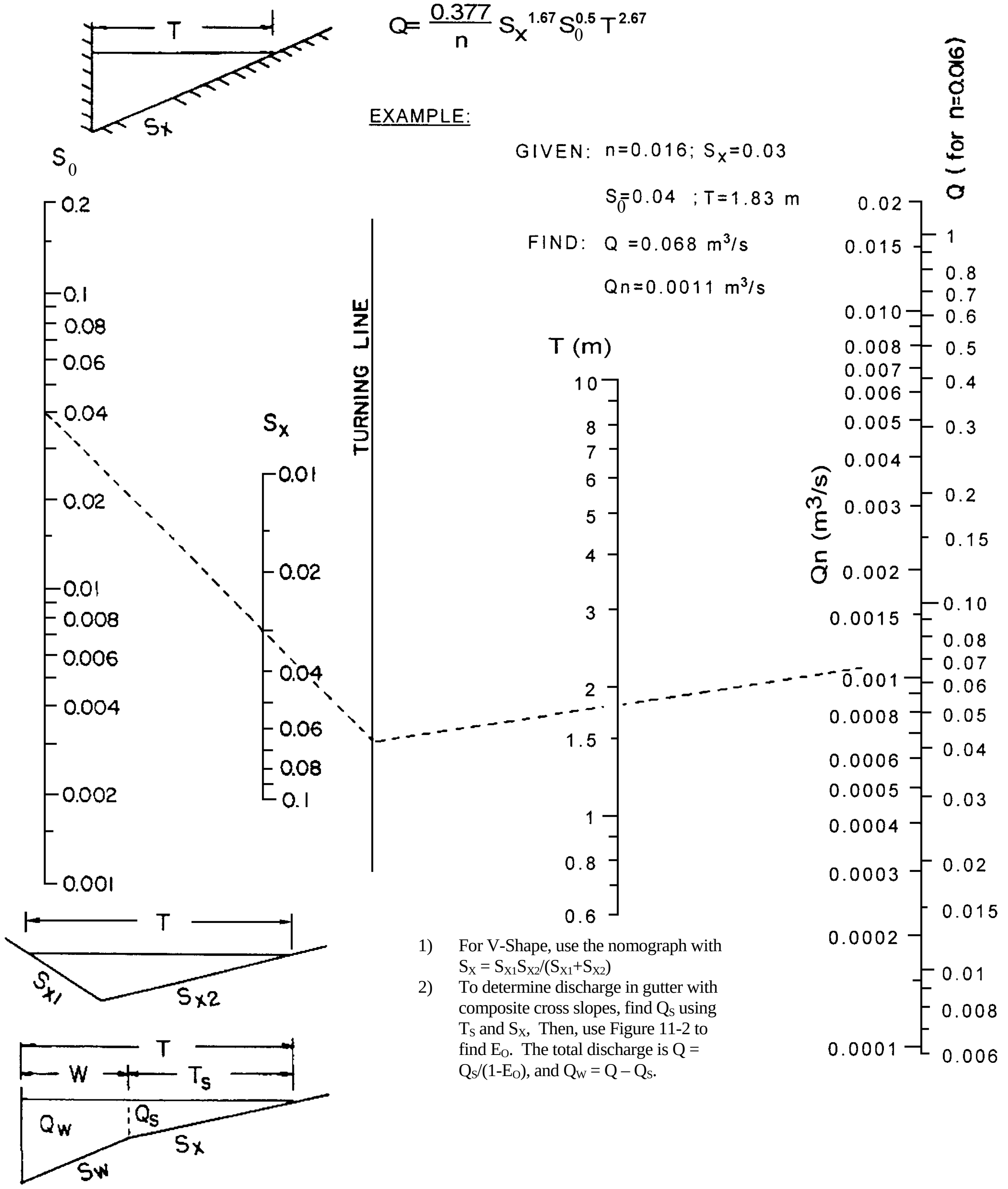


Figure 5.5: Flow in triangular gutter sections.

5.2.2.1 Triangular Gutters

For a triangular gutter shown in Figure 5.4, the top width T also represents the water spread. Noting that

$$y = S_x T \tag{5.2}$$

Table 5.2: *Manning*'s n Value for Street and Pavement Gutters

Type of Gutter or Pavement	Range of *Manning*'s n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

where y is the water depth, equation (5.1) can be written in terms of the flow depth y as

$$Q = \frac{y^{8/3} S_0^{1/2}}{2.64 n S_x} \tag{5.3}$$

Also, note that the flow area A is expressed as

$$A = \frac{1}{2} S_x T^2 \tag{5.4}$$

Example 5.1 A triangular gutter has a longitudinal slope of $S_0 = 0.01$, cross-slope of $S_x = 0.02$, and *Manning* roughness of $n = 0.016$. Determine the flow depth and spread at a discharge of 0.0566 m^3/s. ■

Solution 5.1

Using Equation (5.1), we find: $T = \left(\frac{2.64Qn}{S_x^{5/3} S_0^{1/2}}\right)^{3/8} = \left(\frac{2.64 \times 0.0566 \times 0.016}{0.025^{5/3} 0.01^{1/2}}\right)^{3/8}$, $\boxed{T = 2.48 \text{ m}}$.

Alternatively, we can obtain T by using Figure 5.5. With $S_0 = 0.01$, $S_x = 0.02$ and $Q = 0.0566$ m^3/s, the figure yields $\boxed{T = 2.5 \text{ m}}$.

Then by using Equation (5.2), we get: $y = S_x T = 0.02 \times 2.84$, $\boxed{y = 0.057 \text{ m}}$ ■

5.2.2.2 Composite Gutter Sections

For a composite gutter section such as shown in Figure 5.4:

$$Q = Q_w + Q_s \tag{5.5}$$

where

- Q_w = discharge in the depressed section, and
- Q_s = discharge in the section that is not depressed.

It can be shown that

$$Q = \frac{Q_s}{1 - E_0} \tag{5.6}$$

in which

$$E_0 = \cfrac{1}{1 + \cfrac{S_w/S_x}{\left[1 + \cfrac{S_w/S_x}{(T/W - 1)}\right]^{8/3} - 1}} \tag{5.7}$$

and

$$S_w = S_x + \frac{a}{W} \tag{5.8}$$

Also, note that from the geometry

$$y = a + TS_x \tag{5.9}$$

and

$$A = \frac{1}{2}S_xT^2 + \frac{1}{2}aW \tag{5.10}$$

where y is the flow depth at the curb, W is the width of the depressed gutter or inlet and A is the flow area.

Example 5.2 A composite gutter section has the dimensions $W = 0.5$ m, $S_0 = 0.008$, $S_x = 0.02$, and $a = 0.05$ m. The *Manning* roughness factor is $n = 0.016$. Determine the discharge in the gutter at a spread T of 2.0 m. ■

Solution 5.2
We first calculate the cross-slope of the depressed gutter S_w, by using Equation (5.8) as :
$S_w = S_x + \frac{a}{W} = 0.02 + \frac{0.05}{0.5} = 0.12$
Also, with reference to Figure 5.4, $T_s = T - W = 2.0 - 0.5 = 1.5$ m.
To find Q_s, Equation (5.1) can be rewritten for the triangular portion of the composite gutter having top a width T_s and evaluated as : $Q_s = \frac{T_s^{8/3}S_x^{5/3}S_0^{1/2}}{2.64n} = \frac{1.5^{8/3}0.02^{5/3}0.008^{1/2}}{2.64 \times 0.016} = 0.0091 \text{ m}^3/\text{s}$.
Now with $S_w/S_x = 0.12/0.02 = 6$, $T/W = 2.0/0.5 = 4$, and $(T/W - 1) = 4.0 - 1.0 = 3.0$, by using Equation (5.7), we find

$$E_0 = \cfrac{1}{1 + \cfrac{6}{\left[1 + \cfrac{6}{3}\right]^{8/3} - 1}} = 0.75$$

Finally, by using Equation (5.6) we get : $Q = \frac{Q_s}{1 - E_0} = \frac{0.0091}{1 - 0.75}$, $\boxed{Q = 0.036 \text{ m}^3/\text{s}}$.
Alternatively, we can obtain a solution to this problem by using Figure 5.5. With $T_s = 1.5$ m, $S_0 = 0.008$ and $S_x = 0.02$, the figure yields $Q_s = 0.009 \text{ m}^3/\text{s}$.
Using Figure 5.6 with $W/T = 0.5/2.0 = 0.25$ and $S_w/S_x = 0.12/0.02 = 6$ then $E_0 = 0.75$.
Finally, by using Equation (5.6) we get : $Q = \frac{Q_s}{1 - E_0} = \frac{0.009}{1 - 0.75}$, $\boxed{Q = 0.036 \text{ m}^3/\text{s}}$. ■

5.2.2.3 Swale Sections

V-shaped (Figure 5.4) and circular swale sections are used to convey runoff from pavements where curbs are not used. The flow within a V-section can be calculated using Equations (5.1) and (5.2) with

$$S_x = \frac{S_{x1}S_{x2}}{S_{x1} + S_{x2}} \tag{5.11}$$

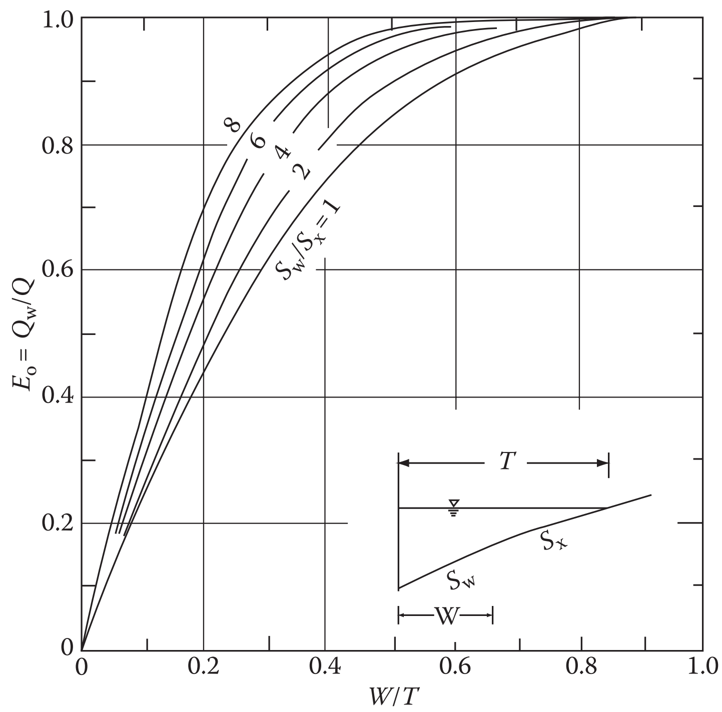


Figure 5.6: Ratio of frontal flow to total gutter flow.

The flow in a circular swale or gutter can be calculated using (*Akan et Houghtalen*, 2003):

$$\frac{y}{D} = 1.179\left(\frac{Qn}{D^{2.67}S_0^{0.5}}\right)^{0.488} \tag{5.12}$$

where

- y = flow depth (m),
- D = diameter of circular gutter (m), and
- S_0 = longitudinal slope

The top width of the flow within the circular section is expressed as:

$$T = 2\left[\frac{D^2}{4} - \left(\frac{D}{2} - y\right)^2\right]^{1/2} \tag{5.13}$$

Example 5.3 A V-section swale has $S_{x1} = 0.04$, $S_{x2} = 0.06$, $n = 0.016$, $S_0 = 0.01$, and $T = 2.5$ m. Determine the maximum discharge this swale can convey without water spreading over the pavement surface. Also determine the depth of flow. ■

Solution 5.3
For the flow not to spread over the pavement, the top width should not exceed 2.48 m. Using Equation (5.11), we find

$$S_x = \frac{S_{x1}S_{x2}}{S_{x1}+S_{x2}} = \frac{0.04 \times 0.06}{0.04+0.06} = 0.024$$

Now, by using Equation (5.1), we obtain:

$$Q = \frac{T^{8/3}S_x^{5/3}S_0^{1/2}}{2.64n} = \frac{2.44^{8/3}0.024^{5/3}0.01^{1/2}}{2.64 \times 0.016}, \boxed{Q = 0.054 \text{ m}^3/\text{s}}.$$

Also, by using Equation (5.3), we get:

$y = S_x T = 0.024 \times 2.48$, $\boxed{y = 0.06 \text{ m}}$ ■

5.2.3 Pavement Drainage Inlets

Role: Stormwater inlets serve to introduce rainwater and street washing water into a sewer (*Gomella et Guerrée*, 1986).

Variants: Stormwater inlets can be classified according to two criteria:

1. Method of water collection: Grate inlet, Curb opening inlet, or Combination inlet (Fig. 5.8-5.9).
2. Method of solid waste retention: regular inlets and selective inlets. Regular inlets have no barrier to the entry of solid materials, which can lead to clogging. Selective inlets have a selective device that prevents foreign objects from entering the sewer system. Again, there are three types: decantation inlets, bucket inlets, and siphon inlets (Fig. 5.10).

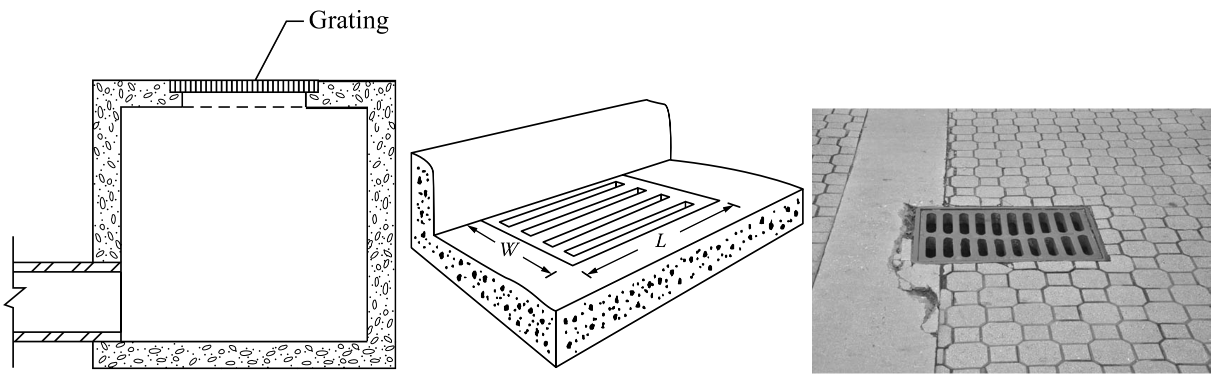


Figure 5.7: Grate inlet.

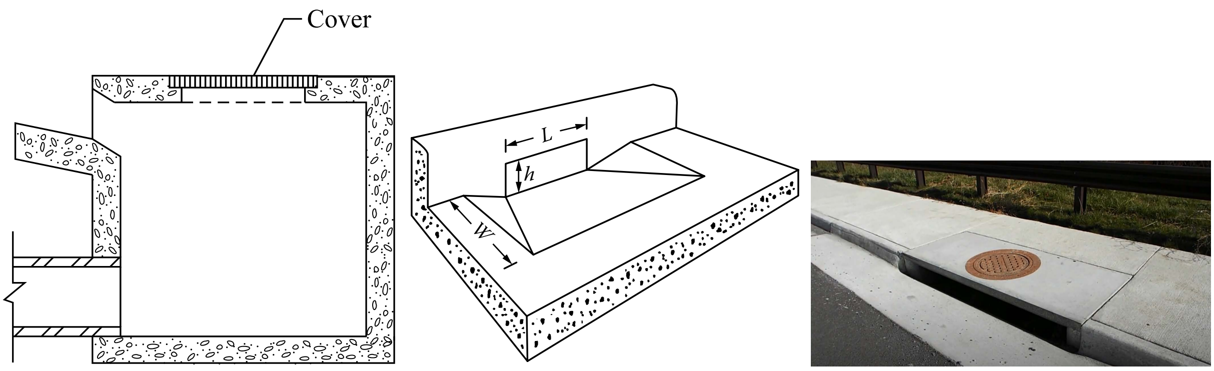


Figure 5.8: Curb opening inlet.

The interception efficiency of an inlet is defined as

$$E = \frac{Q_i}{Q} \tag{5.14}$$

where Q is the total gutter flow rate, and Q_i is the intercepted flow rate.

The flow that is not intercepted by an inlet is called carryover or bypass. By definition,

$$Q_b = Q - Q_i \tag{5.15}$$

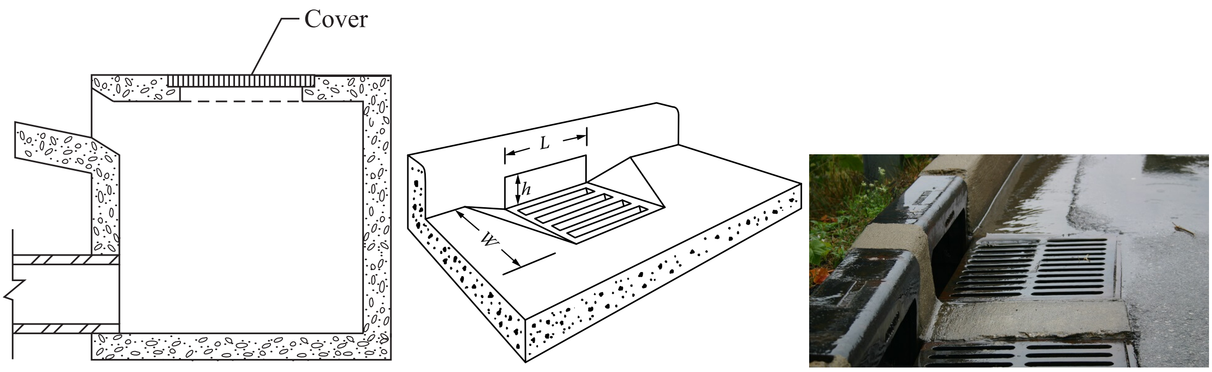


Figure 5.9: Combination inlet.

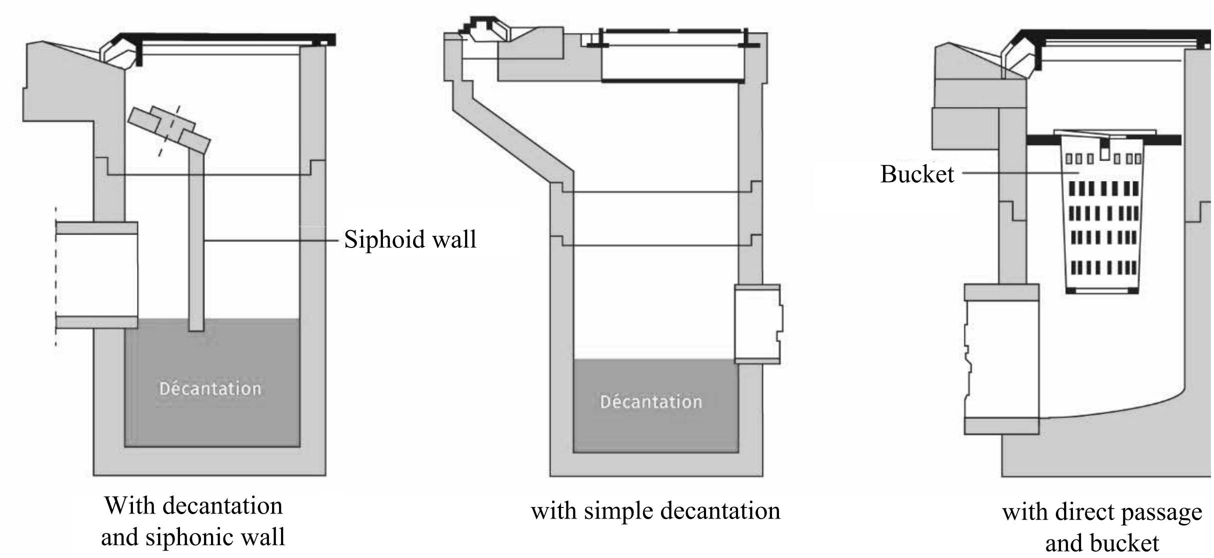


Figure 5.10: Examples of selective inlets

5.2.3.1 Grate Inlets

A grate inlet consists of an opening covered by a grate (Fig. 5.7). The main advantage of grate inlets is that they can be installed in the gutters of roadways where water is flowing, and their main disadvantage is the tendency for debris blockage. Grate inlets also provide an effective means of draining parking-lot pavements and roadways without curbs. Grates typically consist of longitudinal and/or transverse bars oriented parallel and perpendicular to the gutter flow, respectively, and design procedures have been developed for the grates listed in Table 5.3 (*Chin*, 2013a). In "P-xx" designations of grate inlets, "P" indicates parallel bar grates and xx is the approximate bar spacing in millimeters, while in "P-xx$\times$yy" designations yy indicates the approximate spacing of transverse bars or rods in millimeters. Typical P–50$\times$100 and reticuline grates are shown in Figure 5.11.

To determine the efficiency of a grate inlet, the total gutter flow is treated as having two parts: frontal flow and side flow. The frontal flow is the portion of the total gutter flow within the width of the inlet. It is expressed as (*Akan et Houghtalen*, 2003):

$$Q_w = Q\left[1-\left(1-\frac{W}{T}\right)^{2.67}\right] \tag{5.16}$$

where

- Q_w = frontal discharge,

Table 5.3: Grate inlets.

Name	Description
P–50	Parallel bar grate with bar spacing 48 mm on center.
P–50×100	Parallel bar grate with bar spacing 48 mm on center and 10-mm-diameter lateral rods spaced at 102 mm on center.
P–30	Parallel bar grate with 29 mm on center bar spacing.
Curved vane	Curved vane grate with 83-mm longitudinal bar and 108-mm transverse bar spacing on center.
45°–60 tilt bar	45°tilt-bar grate with 57-mm longitudinal bar and 102-mm transverse bar spacing on center.
45°–85 tilt bar	45°tilt-bar grate with 83-mm longitudinal bar and 102-mm transverse bar spacing on center.
30°–85 tilt bar	30°tilt-bar grate with 83-mm longitudinal bar and 102-mm transverse bar spacing on center.
Reticuline	"Honeycomb" pattern of lateral bars and longitudinal bearing bars.

- W = width of the depressed gutter or inlet, and
- T = total spread of water in the gutter.

Also,

$$Q_s = Q - Q_w \tag{5.17}$$

where Q_s = side discharge corresponding to the flow outside the width of the inlet $(T - W)$.

The ratio R_f of frontal intercepted flow to total frontal flow is expressed as:

$$R_f = \frac{Q_{wi}}{Q_w} = 1 - 0.295\,(V - V_0) \qquad \text{for } V > V_0 \tag{5.18a}$$

$$R_f = 1 \qquad \text{for } V \leq V_0 \tag{5.18b}$$

where

- Q_{wi} = frontal flow intercepted,
- V = velocity of flow in the gutter, and
- V_0 = splashover velocity.

Splashover velocity is the minimum velocity that will cause some water to shoot over the inlet. This velocity depends on the gutter length and type. Figure 5.12 displays the splashover velocities for several standard grates. The ratio R_s of intercepted side flow to total side flow is expressed as

$$R_s = \frac{Q_{si}}{Q_s} = \left[1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]^{-1} \tag{5.19}$$

where Q_{si} = side flow intercepted and L = length of grate.

The efficiency E of a grate inlet is evaluated by using

$$E = R_f \frac{Q_w}{Q} + R_s \frac{Q_s}{Q} \tag{5.20}$$

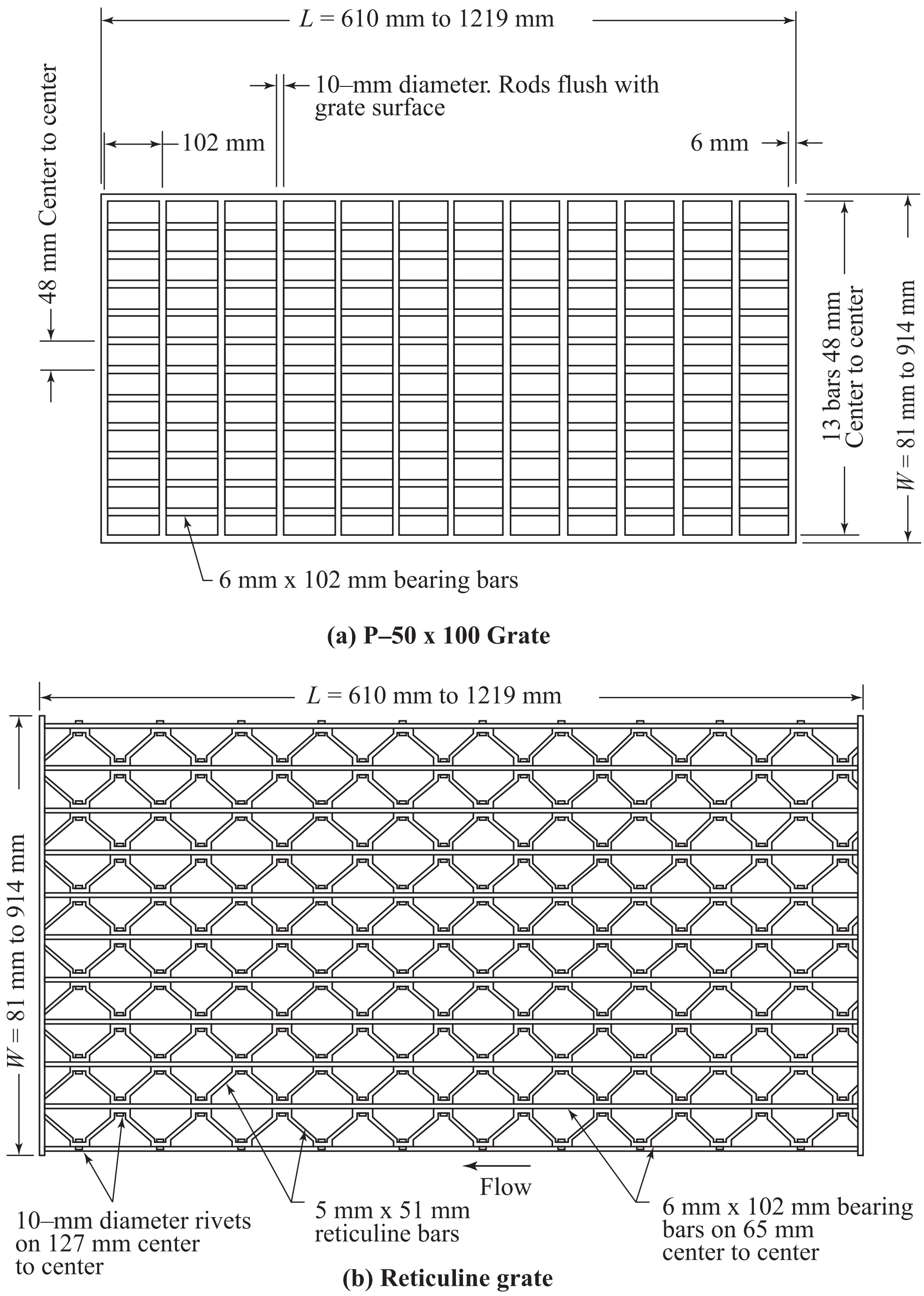


Figure 5.11: Types of grates

Example 5.4 A triangular gutter with $S_x = 0.02$, $S_0 = 0.01$, and $T = 2.4$ m carries $Q = 0.07$ m^3/s. A curved vane grate placed in this gutter has $W = 0.6$ m and $L = 0.6$ m. Determine the efficiency of this grate ■

Solution 5.4
We first determine the flow area by using Equation (5.4) as
$A = \frac{1}{2}S_xT^2 = \frac{1}{2}0.02 \times 2.4^2 = 0.0576$ m^2.
The average cross-sectional velocity becomes $V = Q/A = 0.07/0.0576 = 1.215$ m/s. Also, by using

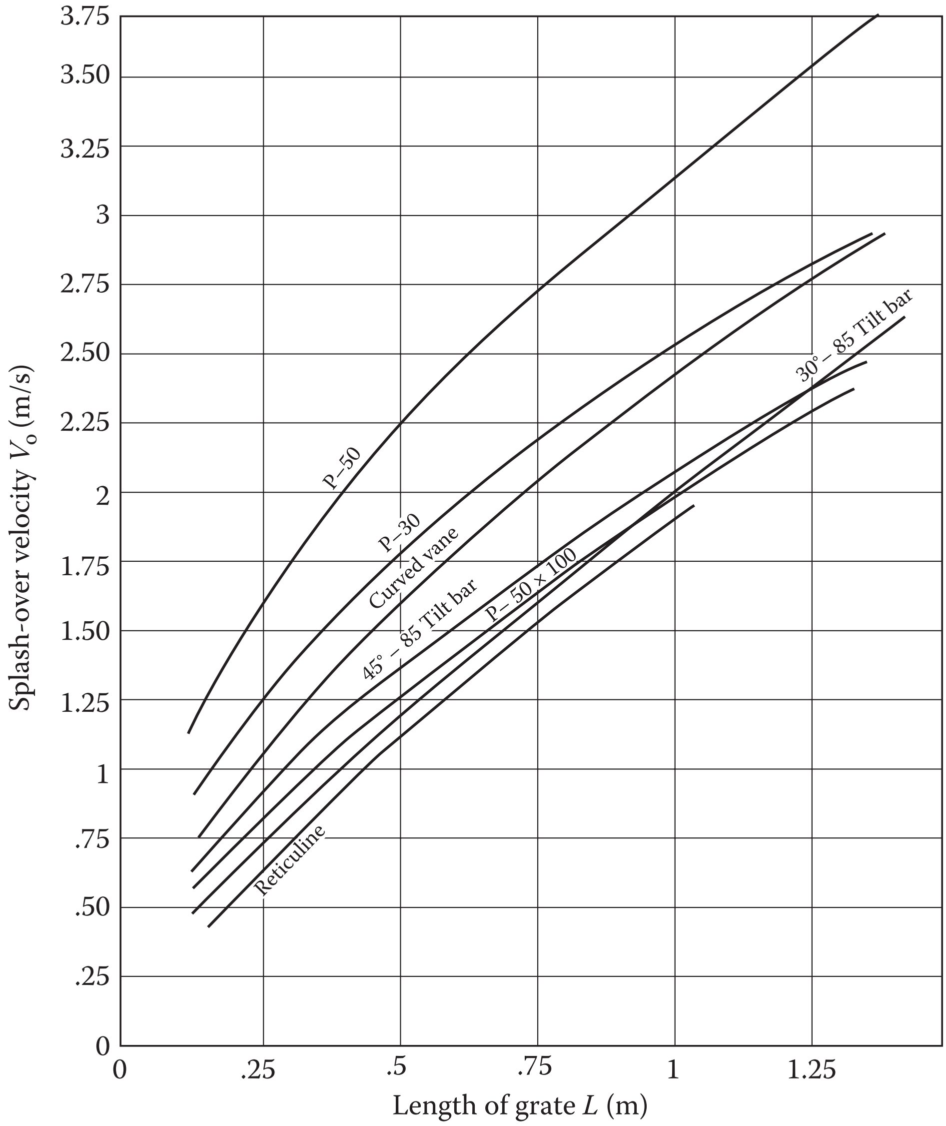


Figure 5.12: Splash-over velocity for various grates

Equation (5.16) we have

$$Q_w = Q\left[1-\left(1-\frac{W}{T}\right)^{2.67}\right] = 0.07\left[1-\left(1-\frac{0.6}{2.4}\right)^{2.67}\right] = 0.0375 \text{ m}^3/\text{s}.$$

and $Q_s = Q - Q_w = 0.07 - 0.0375 = 0.0325$ m^3/s. For the curved vane grate with $L = 0.6$ m, the splashover velocity is obtained from Figure 5.12 as being $V_0 = 1.8$ m. Because $V_0 > V$ in this case, by using Equation (5.18b), we obtain $R_f = 1.0$. Also by using Equation (5.19) we find

$$R_s = \frac{Q_{si}}{Q_s} = \left[1+\frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]^{-1} = \frac{Q_{si}}{Q_s} = \left[1+\frac{0.08281.215^{1.8}}{0.02\times 0.6^{2.3}}\right]^{-1} = 0.05$$

Finally, Equation (5.20) yields

$$E = R_f\frac{Q_w}{Q} + R_s\frac{Q_s}{Q} = 1\frac{0.0375}{0.07} + 0.05\frac{0.0325}{0.07}, \boxed{E = 0.56}.$$

Thus, the intercepted flow is (Eq. (5.14)): $Q_i = EQ = 0.56 \times 0.07 = 0.0391$ m^3/s, and the bypass is (Eq. (5.15)) $Q_b = Q - Q_i = 0.07 - 0.0391 = 0.0309$ m^3/s. ■

5.2.3.2 Curb-Opening Inlets

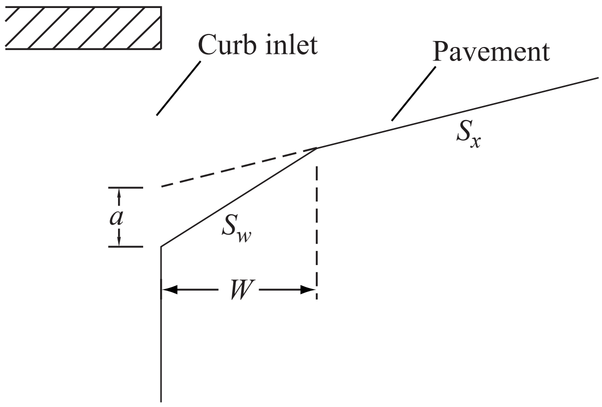


Figure 5.13: Combined System.

Curb inlets are vertical openings in the curb covered by a top slab, and an advantage of a curb inlet is that it does not interfere with traffic. Curb-opening heights vary in magnitude, with typical opening heights in the range of 10–15 cm (4–6 in.). Curb inlets are most effective on flatter slopes (<3%) and in sags. The efficiency of a curb-opening inlet is calculated as

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \qquad \text{for } L < L_T \tag{5.21a}$$

$$E = 1 \qquad \text{for } L \geq L_T \tag{5.21b}$$

where L = curb-opening length and L_T = curb-opening length required to capture 100% of gutter flow,

$$L_T = 0.817 Q^{0.42} S_0^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} \tag{5.22}$$

For a depressed curb-opening inlet as shown in Figure 5.13,

$$L_T = 0.817 Q^{0.42} S_0^{0.3} \left(\frac{1}{nS_e}\right)^{0.6} \tag{5.23}$$

where the equivalent cross slope, S_e, given by

$$S_e = S_x + \frac{a}{W} E_0 \tag{5.24}$$

Example 5.5 A curb-opening inlet is placed in a triangular gutter that has a longitudinal slope of $S_0 = 0.01$, cross-slope of $S_x = 0.02$, and *Manning* roughness factor of $n = 0.016$. The curb-opening inlet has a length of $L = 3$ m. Determine the flow intercepted by the curb-opening inlet when the gutter discharge is $Q = 0.05$ m^3/s. ■

Solution 5.5
Equations (5.22) and (5.21a), respectively, yield

$$L_T = 0.817 Q^{0.42} S_0^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} = 0.817 \times 0.05^{0.42} \times 0.01^{0.3} \left(\frac{1}{0.016 \times 0.02}\right)^{0.6} = 7.29$$

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{3}{7.29}\right)^{1.8} = 0.61$$

Therefore, the intercepted flow is (5.14)

$$Q_i = EQ = 0.61 \times 0.05, \ \boxed{Q_i = 0.03 \text{ m}^3/\text{s}}$$

and the bypass flow is

$$Q_b = Q - Q_i = 0.05 - 0.03, \ \boxed{Q_b = 0.02 \text{ m}^3/\text{s}}.$$ ■

Example 5.6 The composite gutter section considered in Example 5.2 has the dimensions $W = 0.5$ m, $S_0 = 0.008$, $a = 0.05$ m, and $S_x = 0.02$ and a *Manning* roughness factor of $n = 0.016$. It was determined in the example that at a spread of $T = 2.0$ m, the total gutter discharge was $Q = 0.036$ m^3/s and the frontal to total flow ratio was $E_0 = 0.75$. Determine the efficiency of a curb-opening inlet placed in the composite gutter if the length of the inlet is $L = 1.75$ m. ■

Solution 5.6
By using Equations (5.24) and (5.23), respectively, we find

$$S_e = S_x + \frac{a}{W}E_0 = 0.02 + \frac{0.05}{0.5}0.75 = 0.095$$

$$L_T = 0.817Q^{0.42}S_0^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} = 0.817 \times 0.036^{0.42} \times 0.008^{0.3}\left(\frac{1}{0.016 \times 0.0095}\right)^{0.6} = 2.3$$

Then, Equation (5.21a) gives

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{1.75}{2.3}\right)^{1.8} = 0.92$$

Therefore, the intercepted flow is (5.14)

$$Q_i = EQ = 0.92 \times 0.036, \ \boxed{Q_i = 0.033\ \mathrm{m^3/s}}$$

and the bypass flow is

$$Q_b = Q - Q_i = 0.036 - 0.033, \ \boxed{Q_b = 0.003\ \mathrm{m^3/s}}.$$ ■

5.2.3.3 Combination Inlets

Combination inlets usually consist of a curb opening and a grate. The flow interception capacity of a combination inlet is about the same as the grate alone if the curb opening and the grate are placed side by side with nearly equal lengths. If this is the case, the curb inlet is neglected in efficiency calculations. Often, a combination inlet is used with part of the curb opening placed upstream of the grate. That part of the curb opening placed upstream will intercept part of the gutter flow as well as the debris in such installations. In this case, the curb-opening length upstream of the grate is considered in efficiency calculations.

The interception capacity of a combination inlet is equal to the sum of the capacity of the curb opening upstream of the grate plus the grate capacity. The three-step procedure to calculate the capacity of a combination inlet is as follows:

1. Calculate the capacity of the curb inlet upstream of the grate inlet using the procedure described previously for curb inlets. Subtract the curb-inlet capacity from the flow rate in the gutter to obtain the flow rate approaching the grate inlet.

2. Calculate the capacity of the grate inlet. If the gutter cross section is depressed over a width, W, equal to the width of the grate. In this case, the ratio, E_0, is calculated using Eq. (5.7). The side-flow interception capacity, R_s, is calculated using Equation (5.19), the frontal-flow interception, R_f, is calculated using Equation (5.18), and the fraction of the flow rate in the gutter intercepted by the grate is given by Equation (5.20).

3. Add the capacity of the curb opening calculated in Step 1 to the capacity of the grate inlet calculated in Step 2 to obtain the capacity of the combination inlet.

Example 5.7 Determine the interception capacity of a combination curb opening grate inlet in a triangular gutter section that carries 0.15 $\mathrm{m^3/s}$ and has a cross slope of 0.03 m/m, longitudinal slope of 0.035 m/m, and *Manning*'s roughness of 0.016. The grate inlet is a 0.6 m × 0.6 m P-30 grate and the curb-opening length is 3.5 m with 2.9 m of the opening located upstream of the grate. ■

Solution 5.7
Data: $Q = 0.15\ \mathrm{m^3/s}$, $S_x = 0.03$ m/m, $S_0 = 0.035$ m/m, $n = 0.016$, $L = 3.5$ m, $W = 0.6$ m.

Step 1: Calculate the interception capacity of the curb opening upstream of the grate inlet.

The curb-opening length upstream of the grate is $L = 3.5 - 0.6 = 2.9$ m
The curb-opening length for 100% interception is calculated using Eq. (5.22):
$$L_T = 0.817 Q^{0.42} S_0^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} = 0.817 \times 0.15^{0.42} 0.035^{0.3} \left(\frac{1}{0.016 \times 0.03}\right)^{0.6} = 13.2 \text{ m}$$
The efficiency of a curb-opening inlet, E_c, is calculated using Eq. (5.21a):
$$E_c = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{2.9}{13.2}\right)^{1.8} = 0.36 = 36\%$$
The interception capacity of the curb opening, Q_{ic}, is computed using Eq. (5.14):
$Q_{ic} = E_c Q = 0.36 \times 0.15 = 0.054 \text{ m}^3/\text{s}$.

Step 2: Calculate the interception capacity of the grate inlet
The flow rate in the gutter immediately upstream of the grate (the remaining flow), Q_g, is given by $Q_g = Q - Q_{ic} = 0.15 - 0.054 = 0.096 \text{ m}^3/\text{s}$.
The spread corresponding to this discharge is calculated using Equation (5.1) as
$$T = \left(\frac{Q_g n}{0.376 S_x^{5/3} S_0^{1/2}}\right)^{3/8} = \left(\frac{0.096 \times 0.016}{0.376 \times 0.03^{5/3} \times 0.035^{12}}\right)^{3/8} = 2.13 \text{ m}$$
The frontal flow using Equation (5.16) is
$$Q_w = Q_g \left[1 - \left(1 - \frac{W}{T}\right)^{2.67}\right] = 0.096 \left[1 - \left(1 - \frac{0.6}{2.05}\right)^{2.67}\right] = 0.056 \text{ m}^3/\text{s}$$
and the side discharge using Equation (5.17) is
$Q_s = Q_g - Q_w = 0.096 - 0.056 = 0.04 \text{ m}^3/\text{s}$.
Also, from Equation (5.4), the flow area just upstream from the grate is
$$A = \frac{1}{2} S_x T^2 = \frac{1}{2} 0.03\ 2.13^2 = 0.068 \text{ m}^2$$
and hence the average velocity, V, in the gutter is given by $V = \dfrac{0.096}{0.068} = 1.41$ m/s.
For a 0.6-m-long P-30 grate, the splash-over velocity is approximately $V_0 = 2$ m (see Figure 5.12), and since $V < V_0$, the frontal-flow interception efficiency is 100% and hence $R_f = 1.0$ from Equation (5.18b).
Next, by using Equation (5.19) we get the side-flow interception efficiency,
$$R_s = \left[1 + \frac{0.0828 V^{1.8}}{S_x L^{2.3}}\right]^{-1} = \left[1 + \frac{0.0828 \times 1.41^{1.8}}{0.03 \times 0.6^{2.3}}\right]^{-1} = 0.057$$
Then, from Equation (5.20), the efficiency of the grate is
$$E_g = R_f \frac{Q_w}{Q_g} + R_s \frac{Q_s}{Q_g} = 1 \frac{0.056}{0.096} + 0.057 \frac{0.04}{0.096} = 0.61$$
The flow rate intercepted by the grate using Eq. (5.14) becomes $Q_{ig} = Q_g E_g = 0.61 \times 0.096 = 0.059 \text{m}^3/\text{s}$.

Step 3: Calculate the total interception capacity of the combination inlet.
The interception capacity of the combination inlet, Q_i, is the sum of the curb opening capacity, Q_{ic}, and the grate capacity, Q_{ig}, hence
$Q_i = Q_{ic} + Q_{ig} = 0.054 + 0.059$, $\boxed{Q_i = 0.113 \text{ m}^3/\text{s}}$.
Since the gutter flow rate upstream of the combination inlet is $Q = 0.15 \text{ m}^3/\text{s}$, the bypass flow is calculated using Equation (5.15)
$Q_b = Q - Q_i = 0.15 - 0.113$, $\boxed{Q_b = 0.037 \text{ m}^3/\text{s}}$. ■

5.2.3.4 Inlets Spacing

The inlets locations should be marked on the plans prior to computations regarding runoff, spread, and inlet capacity, and generally include (*Mays*, 2001):

- Sag locations on the gutter grade

- Upstream of median breaks, entrance/exit ramps, street intersections, and pedestrian crosswalks
- Upstream and downstream of bridges
- Upstream of cross slope reversals
- End of any channel in cut sections
- Behind curbs, shoulders or sidewalks to drain low areas

The method for evaluating inlet spacing is outlined as follows;

1. Select a trial inlet location and evaluate its contributing drainage area.
2. Compute the peak runoff from the selected area using the rational equation (Eq. (3.32)).
3. The gutter flowrate is set equal to the peak runoff plus additional bypass flow from upgrade inlets, and Eqs. (5.1) and (5.2) are solved for the resulting spread and depth at the curb.
4. If the depth at the curb is greater than the actual curb height or computed spread is greater than that allowable, return to step 1 with a reduced drainage area, and thus spacing. Likewise, if the computed spread is significantly less than the allowable, return to step 1 with an increased spacing. Otherwise, compute the interception capacity and bypass flow for the inlet.
5. Repeat steps 1 to 4 for subsequent inlets.

Uniform spacing can be established for continuous grades and in cases where the drainage area consists of pavement only or has uniform runoff properties and is rectangular in shape. In this case, the inherent assumption is that time of concentration is the same for all inlets. The location of the first inlet from the crest can be found directly by solving the rational equation for the length of pavement that will generate the design runoff, expressed as

$$L_1 = \frac{3.6 \times 10^6 Q}{CiW_p} \tag{5.25}$$

where

- L_1 = length from the crest to the first inlet in m,
- Q = gutter discharge in m^3/s computed from Eq. (5.1) using the design spread,
- W_p = lateral distance in m from the pavement crown to the curb
- C = dimensionless runoff coefficient
- i = rainfall intensity in mm/h.

Interception capacity and subsequent evaluation of bypass flow of this upstream inlet then determines the spread at the inlet. Downstream inlets should be spaced according to where the design spread is reached, or

$$L_i = \frac{3.6 \times 10^6 Q}{CiW_p} E \tag{5.26}$$

where

- L_i = spacing between subsequent inlets in m
- E = interception efficiency of the upstream inlet

Note that the last inlet, which will likely be placed at the low point in the roadway grade, should be designed for complete interception. In addition, more frequent spacing of smaller inlets, and consequently the allowance of larger bypass flows, can reduce capital costs in some cases.

Example 5.8 Determine the spacing required for a series of 0.6 m × 0.6 m reticuline grate inlets that drain a 7.5-m width of pavement at a design spread of 2.0 m. The gutter has a uniform cross slope of 0.02 m/m, longitudinal slope of 0.018 m/m, and *Manning*'s roughness of 0.015. The design rainfall intensity and runoff coefficient are estimated at 150 mm/h and 0.95, respectively. ■

Solution 5.8

Data: $T = 2$ m, $S_x = 0.02$ m/m, $S_0 = 0.018$ m/m, $n = 0.015$, $W_p = 7.5$ m $i = 150$ mm/h, $C = 0.95$, $W = 0.6$ m and $L = 0.6$ m.

Step 1. Compute the gutter discharge, Q, using Eq. (5.1).

$$Q = \frac{T^{8/3} S_x^{5/3} S_0^{1/2}}{2.64n} = \frac{2^{8/3} \times 0.02^{5/3} \times 0.018^{1/2}}{2.64 \times 0.015} = 0.032\ \mathrm{m^3/s}.$$

Step 2. Determine the location of the first inlet, L_1 , from Eq. (5.25).

$$L_1 = \frac{3.6 \times 10^6 Q}{CiW_p} = \frac{3.6 \times 10^6 0.32}{0.95 \times 150 \times 7.5}.\ \boxed{L_1 = 107\ \mathrm{m}}.$$

Step 3. Evaluate the efficiency of a 0.6 m × 0.6 m reticuline grate inlet

a. Calculate the flow velocity from gutter discharge, Q, and effective flow area, A.

Using Eq. (5.4) : $A = \frac{1}{2} S_x T^2 = \frac{1}{2} 0.02 \times 2^2 = 0.04\ \mathrm{m}^2$.

$$V = \frac{Q}{A} = \frac{0.032}{0.04} = 0.80\ \mathrm{m/s}.$$

b. Determine the frontal flow efficiency, R_f, and side flow efficiency, R_s.

For a 0.6 m × 0.6 m reticuline grate, the splash-over velocity is approximately $V_0 = 1.3$ m (see Figure 5.12), and since $V < V_0$, the frontal-flow interception efficiency is 100% and hence $R_f = 1.0$ from Equation (5.18b).

Next, by using Equation (5.19) we get the side-flow interception efficiency,

$$R_s = \left[1 + \frac{0.0828 V^{1.8}}{S_x L^{2.3}}\right]^{-1} = \left[1 + \frac{0.0828 \times 0.80^{1.8}}{0.02 \times 0.6^{2.3}}\right]^{-1} = 0.1.$$

c. Calculate the frontal and side discharges

The frontal flow using Equation (5.16) is

$$Q_w = Q\left[1 - \left(1 - \frac{W}{T}\right)^{2.67}\right] = 0.032\left[1 - \left(1 - \frac{0.6}{2}\right)^{2.67}\right] = 0.020\ \mathrm{m^3/s}$$

and the side discharge using Equation (5.17) is

$Q_s = Q - Q_w = 0.032 - 0.020 = 0.012\ \mathrm{m^3/s}$.

d. Evaluate the interception efficiency, E, from Eq. (5.20)

$$E = R_f \frac{Q_w}{Q_g} + R_s \frac{Q_s}{Q_g} = 1\frac{0.020}{0.032} + 0.1\frac{0.012}{0.032} = 0.65$$

Step 4. Determine the spacing of subsequent inlets, L_i , from Eq. (5.26)

$$L_i = \frac{3.6 \times 10^6 Q}{CiW_p} E = \frac{3.6 \times 10^6 \times 0.032}{0.95 \times 150 \times 7.5} 0.65,\ \boxed{L_i = 70\ \mathrm{m}}.$$

The first inlet should be placed 107 m from the crest and subsequent inlets should be spaced at 70-m increments. ■

Where the highway grade changes, computations proceed in a similar manner, but spacing will vary according to longitudinal slope. As the grade becomes flatter, both capacity and spacing will be reduced and additional inlets will be required. Conversely, as slope increases, fewer inlets will be needed due to the increased capacity of gutter sections.

5.2.4 Connections

Role: The purpose of individual connections is to discharge household wastewater and, where applicable, stormwater into the sewer system (Fig. 5.14).

Constituent Elements: A connection consists of

- A device for connecting to the main sewer (Manhole or inspection chamber);
- A connection pipeline;
- A junction box.

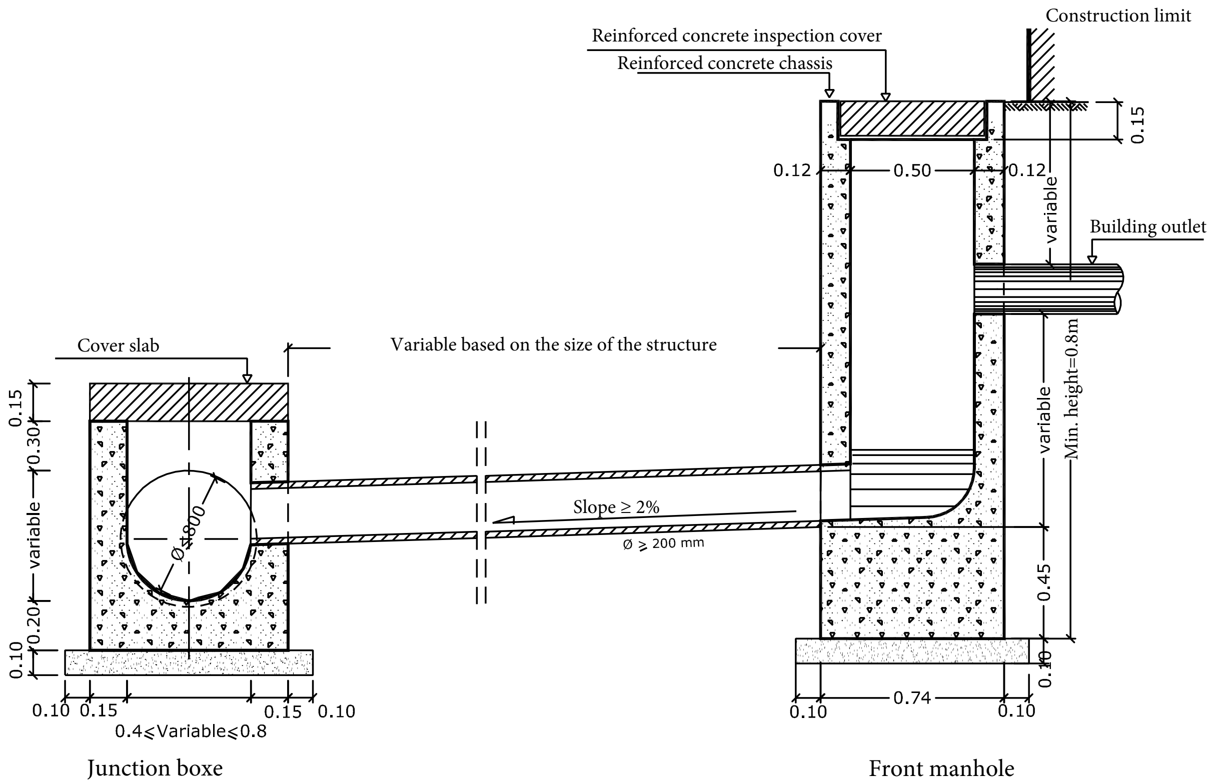


Figure 5.14: Simple individual connection

5.2.5 Inspection Chamber

Not accessible, it allows the connection between the junction box and the main sewer and/or between a street inlet and the sewer network (Fig. 5.14).

5.2.6 Storm Overflows

Role: Their primary function is to discharge exceptional stormwater peaks into the receiving environment.

Variants: The most common types in sanitation are:

- Front weir overflows: the weir is straight and perpendicular to the flow.
- Side weir overflows: the weir is straight and parallel to the flow.
- Double side weir overflows: a weir is placed on each side of the structure.
- Bed opening overflows: excess rainwater flow is discharged through the opening to continue along the upstream conduit alignment.

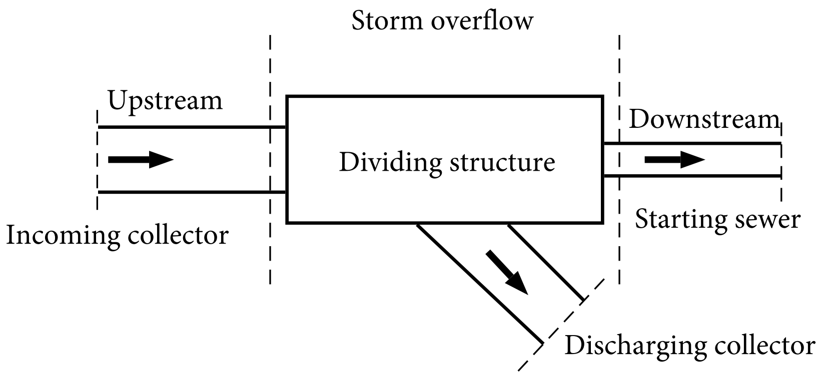


Figure 5.15: Schematic representation of a storm overflow.

Storm Overflow Calculation: The procedure for sizing storm overflows is:

- Evaluate the maximum flow retained downstream to the treatment plant, based on an acceptable dilution at the plant. This flow is typically 3 to 6 times the dry weather flow.
- Determine the operational weir height and the filling level of the inlet pipe, the latter determining the overflow level.
- Then, calculate the weir length.

The formula used is *Poléni*'s formula:

$$Q_{overflow} = \frac{2}{3} C_d \sqrt{2g} b_w h_e^{3/2} \tag{5.27}$$

where

- $Q_{overflow}$: flow rate of the overflow (m^3/s).
- C_d: discharge coefficient, which varies depending on the weir type.
- b_w: length of the weir crest (m).
- h_e: fictitious head height over the overflow (m).

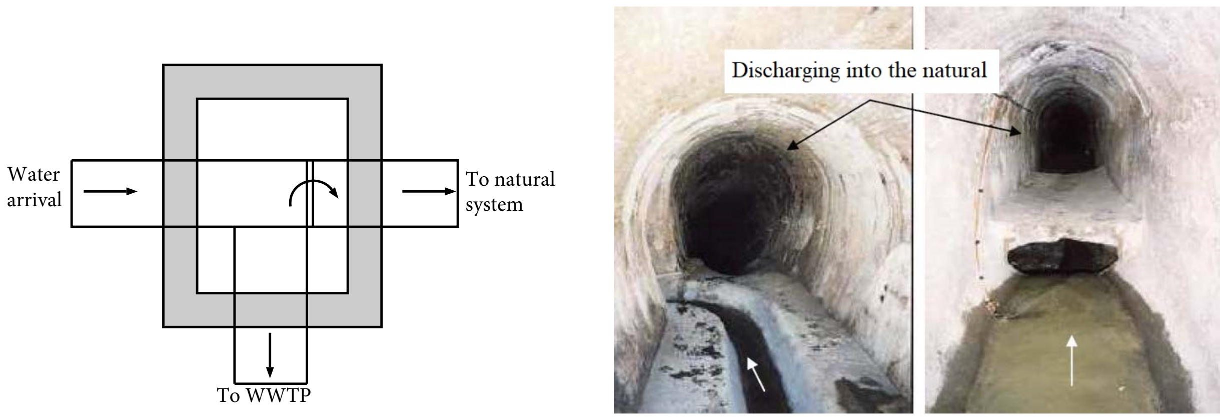


Figure 5.16: Front weir overflows.

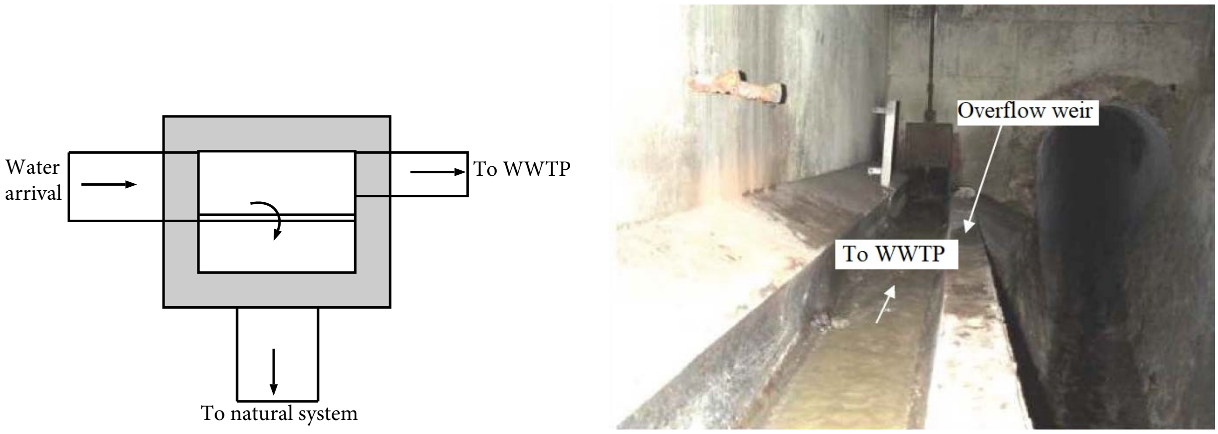


Figure 5.17: Side weir overflows

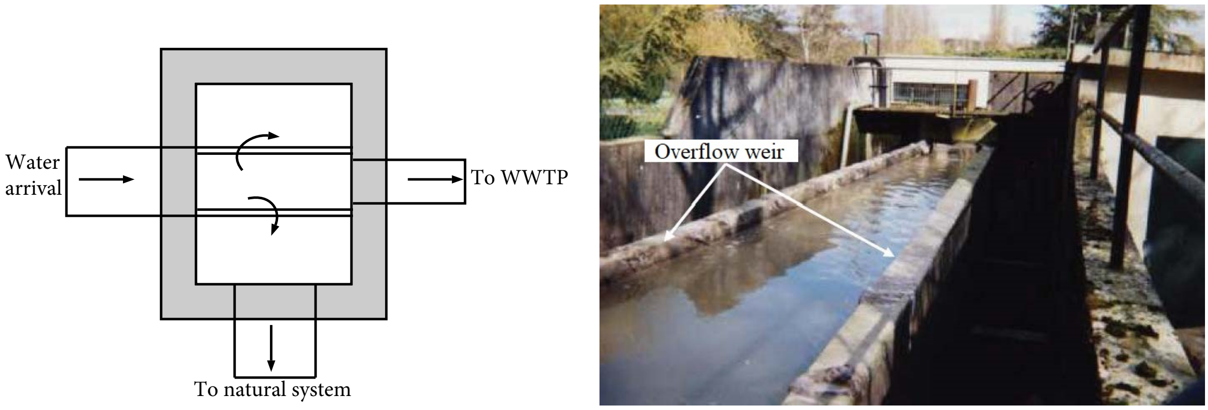


Figure 5.18: Double side weir overflows.

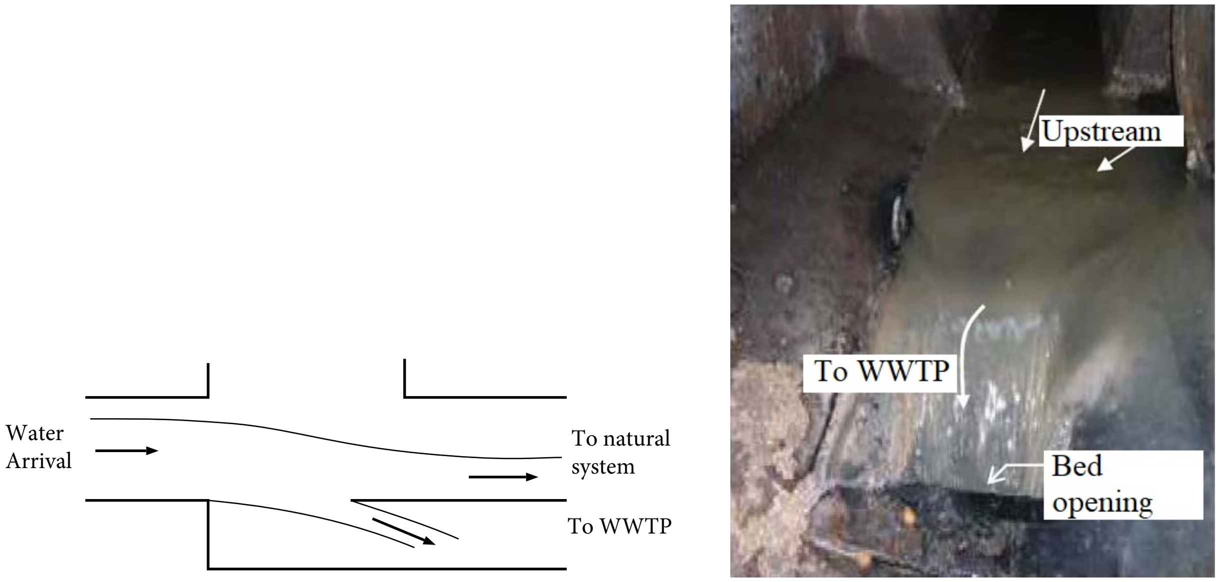


Figure 5.19: Bed opening overflows.

Example 5.9 At the outskirts of a city, the main concrete sewer is relieved by a storm overflow. When a dilution of 5 (1 part wastewater to 4 parts stormwater) is reached relative to the dry weather flow, calculate:

a) the length of the overflow weir

b) the diameter of the throttling pipe (wastewater) downstream of the overflow over a length of 40 m.

Given: the diameter of the main sewer upstream of the storm overflow is $D = 600$ mm, the slope is $S_0 = 1.6\%$, the flow rate during rainfall is 800 L/s, the dry weather flow rate is 60 L/s, $C_d = 0.6$, and $K_S = 90$. ■

Solution 5.9

The flow rate arriving at the treatment plant with a dilution of 5:
$Q_{WWTP} = (60 \times 4) + 60 = 300$ L/s
The flow directly discharged into the natural watercourse (discharge flow):
$Q_{overflow} = Q_T - Q_{WWTP} = 800 - 300 = 500$ L/s
Calculation of the water level y_1 in the inlet pipe
The full-section flow rate is calculated using the *Manning-Strickler* equation with $R_h = D/4$:

$$Q_{fs} = SV_{fs} = \left(\frac{\pi D^2}{4}\right)\left[K_S R_h^{2/3} S_0^{1/2}\right] = \left(\frac{\pi D^2}{4}\right)\left[K_S \left(\frac{D}{4}\right)^{2/3} S_0^{1/2}\right] = K_S \frac{\pi D^{8/3}}{4^{5/3}} S_0^{1/2}$$

$$Q_{fs} = 90 \frac{\pi 0.6^{8/3}}{4^{5/3}} 0.016^{1/2} = 0.909 \text{ m}^3/\text{s} = 909 \text{ L/s}$$

The ratio of flow rate during rainfall: $R_{Q1} = \dfrac{Q_T}{Q_{fs}} = \dfrac{800}{909} = 0.88$
This corresponds (using the chart provided in figure 4.7) to a height ratio of $R_{y1} = 0.73$
Therefore, $y_1 = R_{y1} D = 0.73 \times 600 = 438$ mm
The partial height y_2 when the flow rate equals 5 times the dry weather flow rate = 300 L/s
The dry weather flow rate ratio: $R_{Q2} = \dfrac{Q_1}{Q_{fs}} = \dfrac{300}{909} = 0.33$
This corresponds (using the chart provided in figure 4.7) to a height ratio of $R_{y2} = 0.40$
Therefore, $y_2 = R_{y2} D = 0.40 \times 600 = 240$ mm
The height of the overflow weir crest is 240 mm.

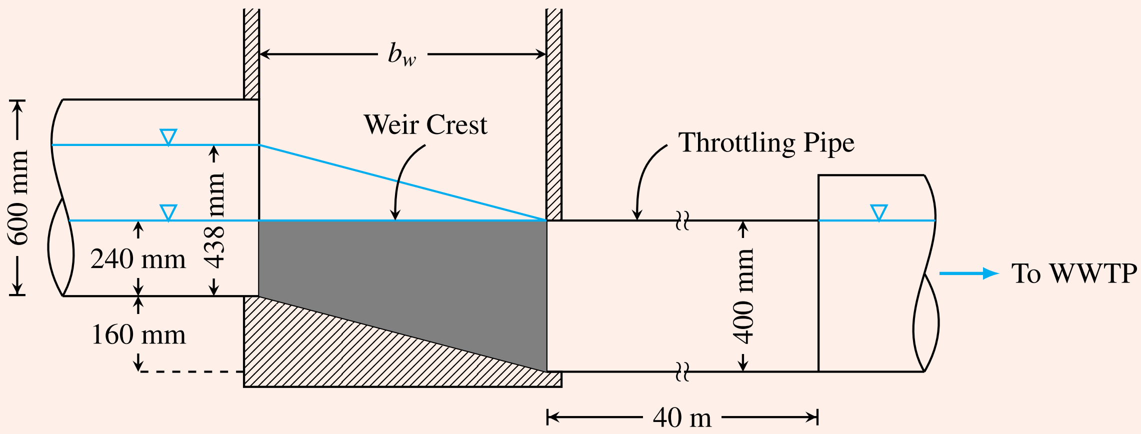


Figure P5.9a: Cup

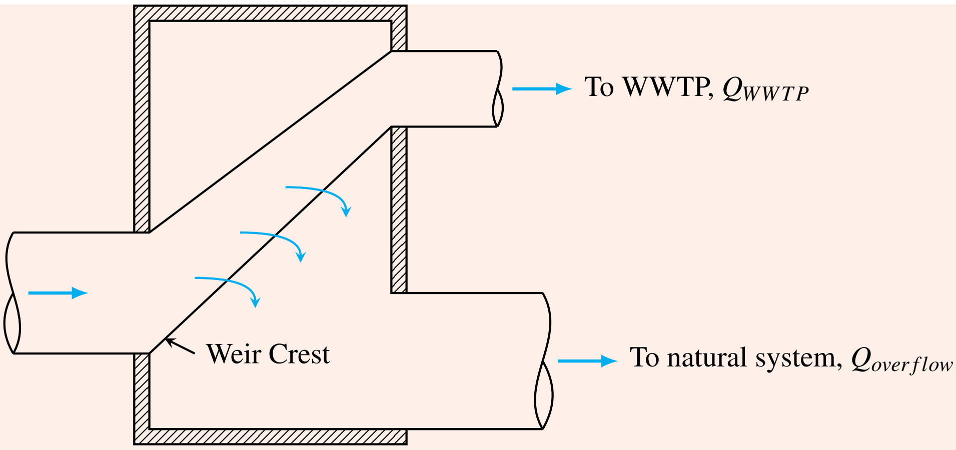


Figure P5.9b: Horizontal plane

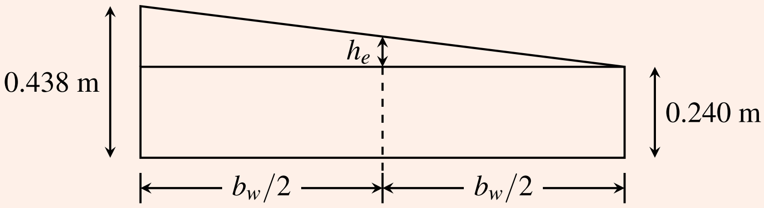


Figure P5.9c

a) Calculation of the Weir Crest Length

Using *Poleni*'s Formula (Eq.(5.27)): $Q_{overflow} = \frac{2}{3}C_d\sqrt{2g}bh_e^{3/2} \Rightarrow b_w = \frac{2}{3}\frac{Q_{overflow}}{C_d\sqrt{2g}h_d^{3/2}}$

For a side weir (Figs. P5.9a and P5.9b), the head at the entrance is 438 mm but must reduce to zero at the end of the weir length. For calculation purposes, we approximate the actual head on the weir as the average of the head at the entrance and the head at the end of the crest (see Fig. P5.9c), which gives:

$h_e = \frac{0.438-0.240}{2} = 0.1$ m, so

$b_w = \frac{2}{3}\frac{0.5}{0.6\sqrt{2\times 9.81}0.1^{3/2}} = 9$ m,

If a safety factor of 1.5 is applied, then $b_w = 9\times 1.5 = 13.5$ m.

For a double side weir overflow (Fig. 5.18), this gives $b_w = 13.5/2 = 6.75$ m.

In the case of a side weir (Fig 5.16), the head at the entrance is: $h_e = 0.438 - 0.240 = 0.198$ m, so

$b_w = \frac{2}{3}\frac{0.5}{0.6\sqrt{2\times 9.81}0.198^{3/2}} = 3.2$ m,

If a safety factor of 1.5 is applied, then $b_w = 3.2\times 1.5 = 4.8$ m.

b) Calculation of the Throttling Pipe Diameter

The pipe must be designed so that the flow does not exceed 300 L/s towards the treatment plant. The sizing is done by considering the full section of the pipe, i.e., the full cross-section.

Suppose this throttling is placed over a length of 40 m with a pipe diameter of $D_{\text{Throttling}} = 400$ mm.

$$Q_{WWTP} = K_S\frac{\pi D_{\text{Throttling}}^{8/3}}{4^{5/3}}S_0^{1/2} \Rightarrow S_0 = \left(\frac{Q_{WWTP}}{K_S\frac{\pi D_{\text{Throttling}}^{8/3}}{4^{5/3}}}\right)^2 = \left(\frac{0.3}{90\frac{\pi 0.4^{8/3}}{4^{5/3}}}\right)^2 = 0.015 \text{ m/m}$$

In conclusion, the side weir will have a total length of 13.5 m, and the throttling pipe with a diameter of 400 mm and a length of 40 m will be installed in a trench with a slope of 0.015 m/m (Fig. P5.9a). ■

5.2.7 Stormwater Retention Basin

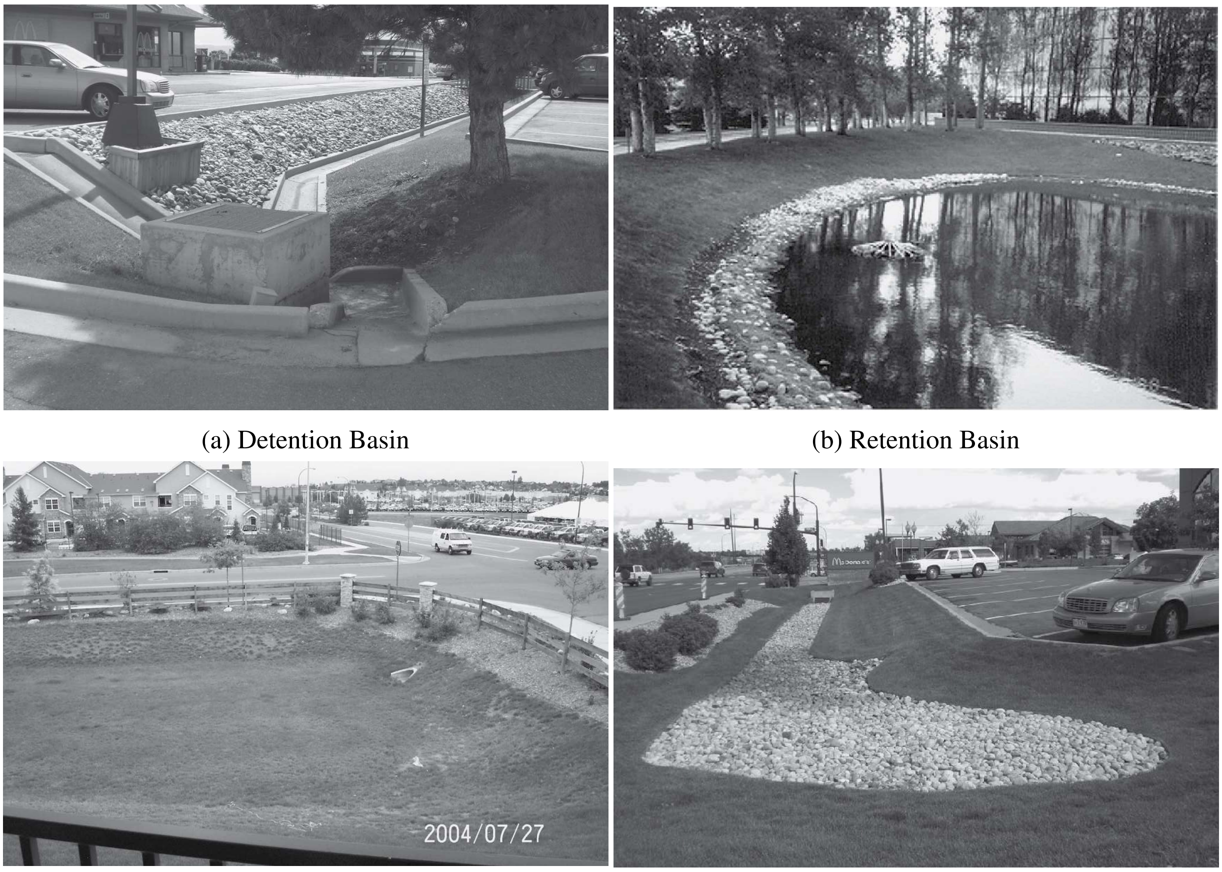
One of the methods for managing runoff and addressing the issues of insufficient outfalls and sudden discharge into receiving environments is to promote all forms of stormwater flow retention.

The main benefits offered by retention systems include:

- Flood control by delaying or diffusing the flood wave in runoff and flow;
- Reduced sizing of collectors due to the delaying and regulating effects on flow;
- Pollution retention: capturing floatables, hydrocarbons, and accidental pollutants upon entry, sedimentation of suspended solids, and the additional purifying capacity of large basins;
- Ecological aspects (aesthetic appeal and attraction as a landscape feature).

5.2.7.1 Types de bassins de retenue

Detention basins are classified into three major categories. (*Guo et al.*, 2023):



(a) Detention Basin

(b) Retention Basin

(c) Infiltration Basin

(d) Porous Pool in Detention Basin

Figure 5.20: Examples of storage basins.

Flood- control detention basin (dry basin)

A flood- control detention basin in Fig. 5.20a, is placed at the stormwater system outfall to temporarily store excess storm runoff and then to discharge the stored water volume at a rate no more than the allowable (Akan 1990). Between two sequential storm events, a flood control detention basin remains dry and can be accessible as an open space for the public.

Stormwater retention basin (wet basin)

A retention basin, shown in Fig. 5.20b, is installed at the low point and operated with a permanent pool. The basin is sized to capture stormwater for the purposes of groundwater recharge, water quality enhancement, stormwater reus, or/ and local runoff volume disposal. A retention basin is often mixed with wetland features to settle the solids and pollutants in stormwater.

Infiltrating basin and trench (porous basin)

Infiltrating basins and trenches are utilized as common low-impact-development devices to reduce the increased runoff volume. Infiltrating basins, shown in Fig. 5.20c, include rain gardens, infiltration beds, riprap trenches, and vegetation beds. An infiltrating basin consists of an on-surface storage basin, vegetal landscape, porous bottom, and overtopping weir. They are often located at the outlet of an industrial park, business district, or highway intersection as a pollutant-source control device (see Fig. 5.20d).

5.2.7.2 Classification based on location

Basin location is an important factor in determining the collection of stormwater. Based on the location, stormwater storage basins are classified into the following:

Upstream and downstream basins

Upstream basins include small, shallow water-quality porous basins and infiltration basins. As shown in Fig. 5.21, an upstream porous basin shall be placed to target solids removal, while a downstream basin shall be installed for the purpose of peak-flow reduction. For instance, urban trash carried in storm runoff from a sports field should be collected into an upstream basin before the street inlet. At the exit of a sewer trunk line, a downstream basin is needed to control the flow release into the downstream natural water body.

Local and regional basins

As shown in Fig. 5.21, a local detention basin serves a small, local residential subdivision for flow release control, and a regional detention basin is designed to mitigate flood flows in a major waterway.

On- stream and off- stream basins

Widening the floodplain width and constructing an embankment across the floodplain bottom create on-stream detention (see Fig. 5.22b). Diverting the excess flood flow from a waterway into adjacent open areas such as depressed parks and sports fields to control stormwater release is termed off- stream detention (Fig. 5.22a).

On- site and off- site basins

An on-site detention is implemented to dispose of the increased runoff volumes on the project site. However, if the easement is available, the flood flows may be transferred to downstream off-site detention basin to reduce increased peak flows. Generally, a porous basin is more suitable for on-site infiltration operation, while a detention basin is often recommended for off-site flood control operation.

5.2.7.3 Retention basin Location

In an urban area, parking lot, parks, sport fields, road embankments, and depressed areas provide stormwater storage volume.

5.2.7.4 Basic layout

The basic elements for a detention basin include

1. an inlet structure to collect runoff flows,
2. energy dissipation system for erosion control at the entrance,
3. trickle channel (Fig. 5.23a) to pass frequent nuisance flows (1.0 to 3.0% of the 100-year peak discharge),
4. fore bay (Fig. 5.23b) for sediment settlement,

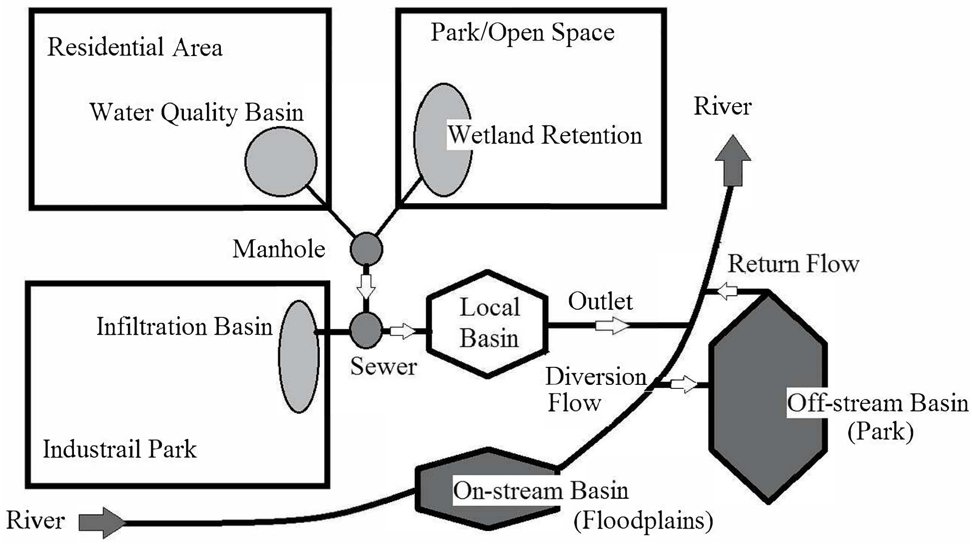


Figure 5.21: Types and locations of detention basins.

(a) Off-stream detention basin (sport field) (b) On-stream detention basin (floodplain)

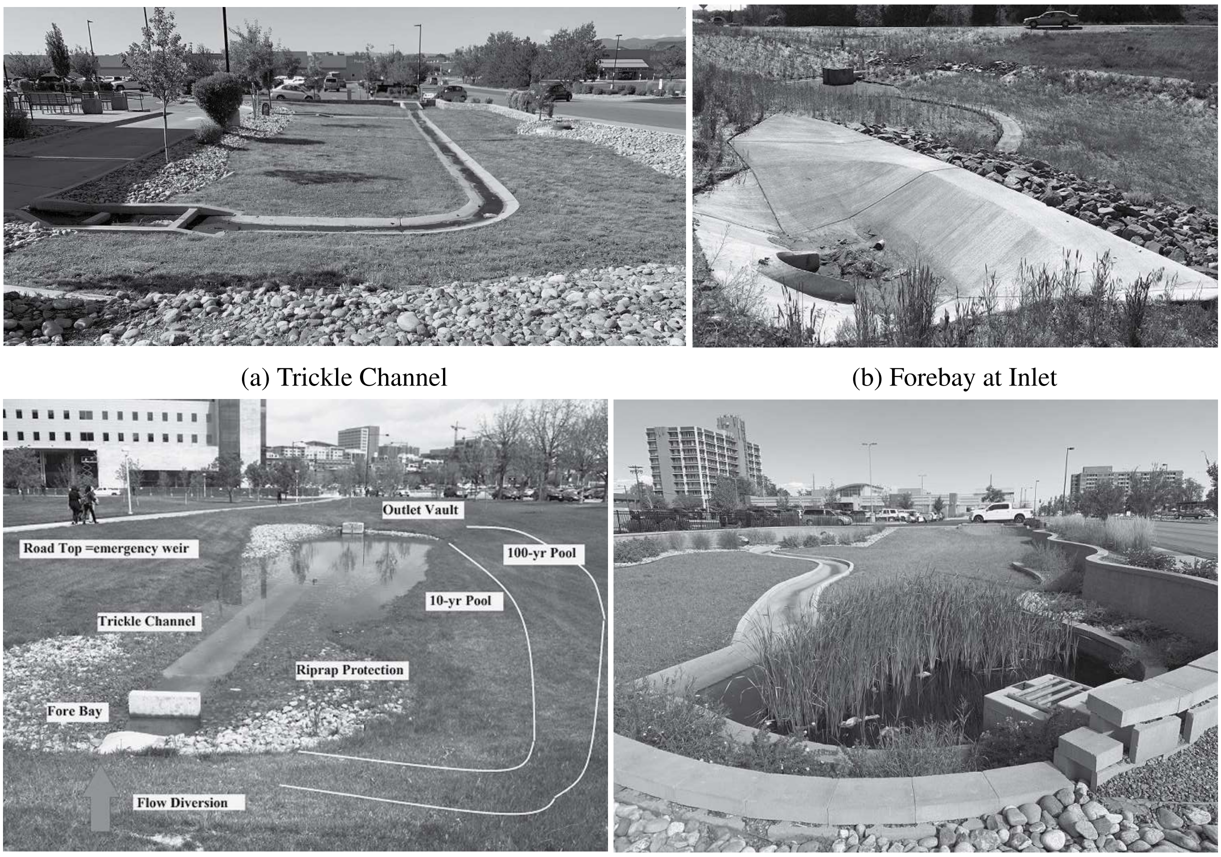
Figure 5.22: Off- stream and on- stream detention basins.

5. storage basin for the mitigation of design events (Fig. 5.23c), and

6. outlet structure (Fig. 5.23d) to control flow releases formed from a perforated plate, riser, orifices, and weirs to collect low to high flows into the concrete vault, and outfall pipes discharge the water flow from the concrete vault into the downstream receiving water body.

The basin width-to-length ratio must be greater than two so that the flood flows can be sufficiently expanded and diffused into the water body to enhance the sedimentation process. Slopes on embankments have to maintain the stability of the bank slope. As a rule of thumb, slopes on earthen embankments should not be steeper than 1V:4H and on riprap embankments should not be steeper than 1V:2H.

The cross- sectional geometry of the basin should be designed for multiple events. As shown in Fig. 5.24, the bottom storage volume in a basin is shaped for the waterquality capture volume. The mid layer is shaped to store the 10- year detention volume. From the 10-year water surface up to the weir crest should provide an additional storage volume to accommodate a 100-year event. From the weir crest up to the brimful of the basin is the height of the freeboard. To mimic pre-development watershed hydrologic conditions, it is preferred to drain a low storage volume over 6 to 24 hours, whereas a 100- yr storage volume should be emptied within 24 to 72 hours.



(a) Trickle Channel

(b) Forebay at Inlet

(c) Multiple Layer Detention Basin

(d) Wetland Pool and Outlet Box

Figure 5.23: basic elements for a detention basin.

5.2.7.5 Inlet and outlet works

Inlets and outlets of a detention basin should be protected from erosion and the deposition of sediment. The concentrated inflow from an inlet pipe shown in Fig. 5.25a has to be diffused into a forebay for energy dissipation and solid settlement. The forebay is a shallow pool that evenly spreads the flow overtopping a level spreader.

The outlet box in Fig. 5.25b is composed of a vertical perforated plate and a horizontal grate on top of the concrete box. The flow capacity through the outlet box is dictated by the operations of the orifices, weirs, and culverts attached on the concrete box.

The trash rack shown in Fig. 5.25c is necessary at the entrance of any outfall pipe larger than 450 mm in diameter. Often, a monitoring system, such as the one shown in Fig. 5.25d,is installed to measure the inflow and outflow using weirs and orifices.

5.2.7.6 Designing a Retention Basin

The principle for calculating the volume of a retention basin is based on determining the maximum storage volume, $\mathcal{V}_{\max}$, reached at time t_m when the inflow, $Q_p(t)$, becomes less than the outflow, $Q_r(t)$, of the attenuation system and is given by (*Bourrier et Claudon*, 1981):

$$\mathcal{V}_{\max} = \int_{t_0}^{t_m} (Q_p - Q_r)\, dt \tag{5.28}$$

The stormwater discharge rate, Q_r, is the minimum of the following (*Guo et al.*, 2023):

1. critical capacity of the downstream existing drainage facility,
2. allowable flow release published on the regional master drainage plan, and

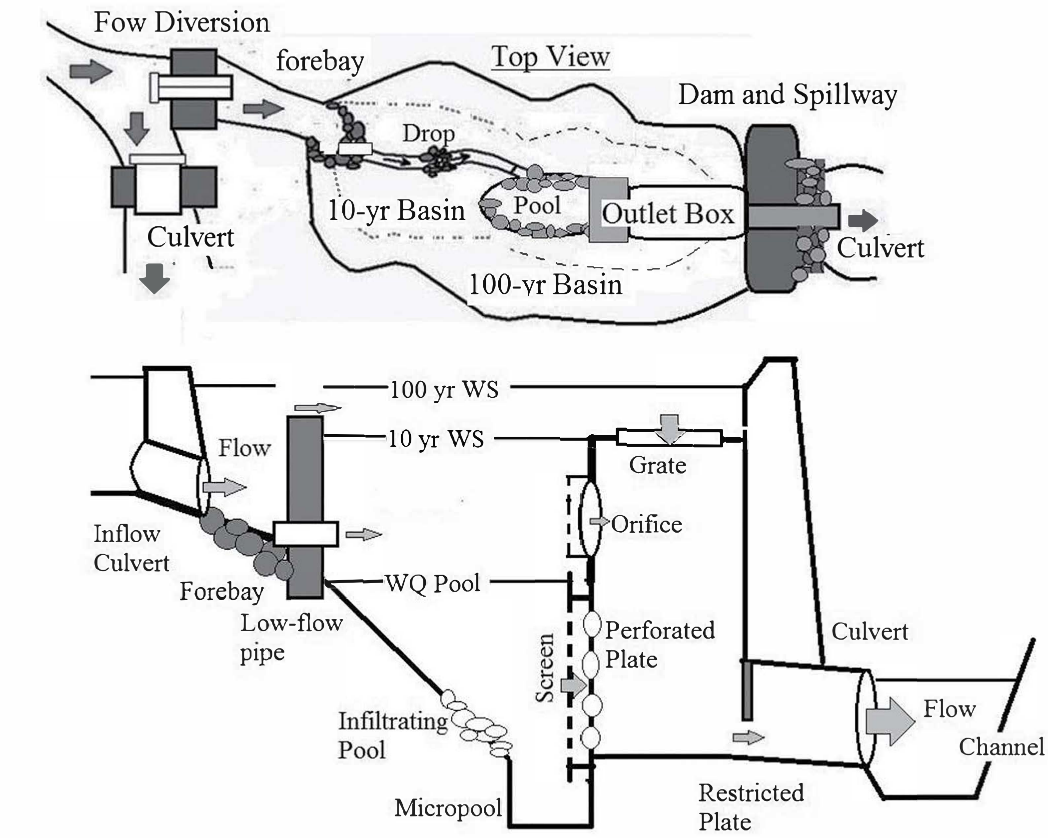


Figure 5.24: Layout of a detention basin for multiple storm events

3. recommended flow release using local design criteria.

There are three methods for calculating the required volume (to which a safety margin should be added, mainly to account for sedimentation between two cleanings) of a storage or retention basin:

- the Rainfall Method;
- the Volume Method;
- the Hydrograph Method.

The intensity of rainfall over a duration t follows an exponential pattern:

$$i = at^b \qquad (\text{mm}/\text{min}) \tag{(3.6)}$$

or a homographic pattern:

$$i = \frac{a}{t+b} \qquad (\text{mm}/\text{min}) \tag{(3.7)}$$

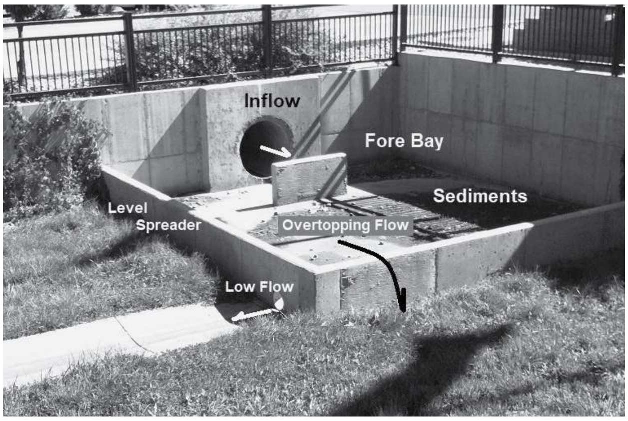
where a is in mm and t in minutes.

The cumulative rainfall at time t is then:

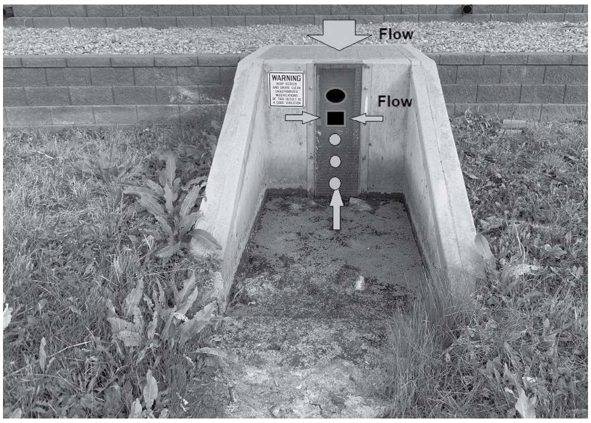
$$h(t) = i \cdot t \qquad (\text{mm}) \tag{(3.5)}$$

If C and A represent the equivalent runoff coefficient (see eq. (3.20)) and the total area respectively, the active area of the basin is given by:

$$A_a = C \cdot A \qquad (\text{ha}) \tag{5.29}$$



(a) Inlet and Energy Dissipator



(b) Outlet Box for Slow Flow Release

(c) Steel Trash Rack

(d) Notch Weir for Flow Measurement

Figure 5.25: Inlet and outlet works of retention basin.

Rainfall Method

The inflow volume into the storage facility, $\mathcal{V}_p(t)$, for a rainfall event lasting time t is given by:

$$\mathcal{V}_p(t) = 10h(t) \cdot A_a \qquad (\mathrm{m}^3) \tag{5.30}$$

where 10 accounts for unit consistency (millimeters–hectares), h is in mm, and A_a is in hectares.

If Q_v represents the total outflow discharge (drainage or release rate) to an external outlet, assumed constant in $(\mathrm{m}^3/\mathrm{min})$, the volume, $\mathcal{V}_r$, discharged after a duration t is:

$$\mathcal{V}_r(t) = t \cdot Q_v \qquad (\mathrm{m}^3) \tag{5.31}$$

The difference between the inflow volume and the discharged volume, which represents the storage volume $\mathcal{V}_s$ required for a rainfall event of this duration t, can then be calculated for each rainfall duration t (see Figure 5.26):

$$\mathcal{V}_s(t) = \mathcal{V}_p(t) - \mathcal{V}_r(t) \qquad (\mathrm{m}^3) \tag{5.32}$$

There exists a duration t_m for which this difference is at its maximum. Therefore, the necessary storage volume for the return period $\mathcal{T}$ should be calculated for this duration. If the IDF curve can be approximated by an exponential expression over 24 hours (Eq. (3.6)), then the duration t_m can be determined analytically:

$$t_m = \left(\frac{Q_v}{a\,(b+1)A_a} \right)^{1/b} \qquad (\mathrm{min}) \tag{5.33}$$

where a is in mm, Q_v in m^3/min, and A_a in m^2.

The volume then becomes:

$$\mathcal{V}_{max} = \left(\frac{-b}{b+1}\right) Q_v \cdot t_m \qquad (m^3) \tag{5.34}$$

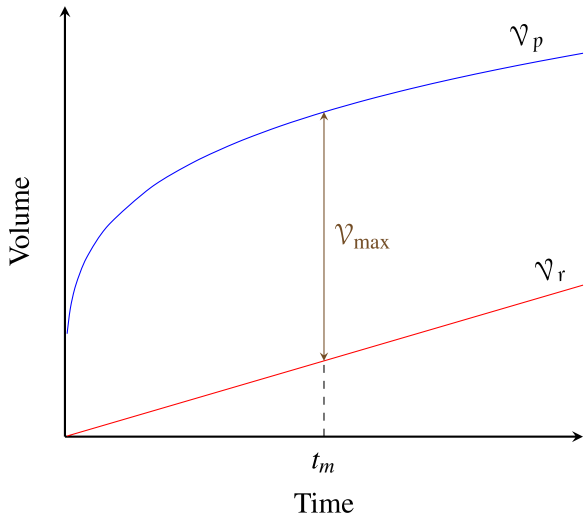


Figure 5.26: Concept of the rainfall method.

Example 5.10 A development area is comprised of three catchments with the characteristics provided in Table E5.10. The development can discharge a maximum of 0.018 m^3/s. Determine the required volume of the retention basin using the rainfall method. For a return period of 10 years, the parameters are given as: $a = 11.76$ mm and $b = -0.76$.

Table E5.10

BV	A (ha)	C
1	3.7	0.9
2	2.1	0.6
3	3.4	0.4

■

Solution 5.10

The total catchment area is: $A = 3.7 + 2.1 + 3.4 = 9.2$ ha.

The equivalent runoff coefficient is:

$$C = \frac{\sum C_j A_j}{\sum A_j} = \frac{0.9 \times 3.7 + 0.6 \times 2.1 + 0.4 \times 3.4}{9.2} = 0.65$$

The effective area of the catchment is:

$A_a = C \cdot A = 0.65 \times 9.2 = 5.95$ ha

The volume of the retention basin is calculated as shown in Table S5.10 and Figure S5.10.

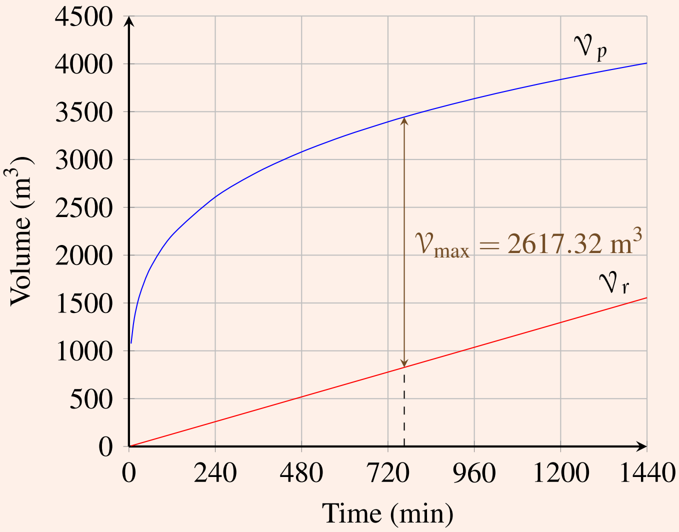


Figure S5.10

Table S5.10

t (min)	i (mm/min)	h (mm)	$\mathcal{V}_p$ (m^3)	Q_v (m^3/s)	$\mathcal{V}_r$ (m^3)	$\mathcal{V}_s$(m^3)
6	3.01	18.08	1075.67	0.018	6.48	1069.19
15	1.50	22.53	1340.25	0.018	16.20	1324.05
30	0.89	26.60	1582.83	0.018	32.40	1550.43
60	0.52	31.42	1869.31	0.018	64.80	1804.51
120	0.31	37.10	2207.64	0.018	129.60	2078.04
240	0.18	43.82	2607.20	0.018	259.20	2348.00
360	0.13	48.30	2873.67	0.018	388.80	2484.87
480	0.11	51.75	3079.09	0.018	518.40	2560.69
600	0.09	54.60	3248.48	0.018	648.00	2600.48
720	0.08	57.04	3393.78	0.018	777.60	2616.18
765.3	0.08	57.88	3443.85	0.018	826.52	2617.32
840	0.07	59.19	3521.69	0.018	907.20	2614.49
960	0.06	61.12	3636.38	0.018	1036.80	2599.58
1080	0.06	62.87	3740.64	0.018	1166.40	2574.24
1200	0.05	64.48	3836.43	0.018	1296.00	2540.43
1320	0.05	65.97	3925.20	0.018	1425.60	2499.60
1440	0.05	67.36	4008.03	0.018	1555.20	2452.83

Assuming the IDF curve can be approximated by an exponential expression, the duration t_m can be calculated using Equation (5.33):

$$t_m = \left(\frac{Q_v}{a\,(b+1)\,A_a}\right)^{1/b} = \left(\frac{0.018\times 60}{11.76\times 10^{-3}\,(1-0.76)\,5.95\times 10^4}\right)^{-1/0.76}, \quad \boxed{t_m = 765.3\text{ min}}$$

The volume can then be calculated using Equation (5.34):

$$\mathcal{V}_{\max} = \left(\frac{-b}{b+1}\right) Q_v \cdot t_m = \left(\frac{0.76}{1-0.76}\right)(0.018\times 60)\times 765.3$$

$$\boxed{\mathcal{V}_{\max} = 2617.32\text{ m}^3}$$

■

Volume Method

The fundamental equation for calculating the capacity of a detention or retention basin is given by:

$$\mathcal{V}_{\max} = h\cdot A_a - Q_v\cdot t_m \qquad (\text{m}^3) \tag{5.35}$$

where: h in m, A_a in m^2, Q_v in m^3/min, and t_m in min.

The applicable formulas for an exponential intensity type are as follows:

$$t_m = \left(\frac{a \cdot A_a(1+b)}{Q_v}\right)^{-1/b} \qquad (\text{min}) \tag{5.36}$$

$$\mathcal{V}_{\max} = a \cdot A_a \cdot t_m^{1+b} - Q_v \cdot t_m \qquad (\text{m}^3) \tag{5.37}$$

Alternatively, for a homographic intensity type:

$$t_m = \sqrt{\frac{a \cdot A_a \cdot b}{Q_v} - b} \qquad (\text{min}) \tag{5.38}$$

$$\mathcal{V}_{\max} = \frac{a \cdot A_a \cdot t_m}{t_m + b} - Q_v \cdot t_m \qquad (\text{m}^3) \tag{5.39}$$

where: a in mm, Q_v in m^3/min, and A_a in m^2.

Example 5.11 Consider a 10-year return period rainfall intensity over a watershed area of 150 ha, a runoff coefficient of 0.50, and a discharge rate of 0.8 m^3/s. Calculate the retention basin volume using:

1. The exponential intensity formula: $i = 6.3t^{-0.74}$ with t in min and i in mm/min
2. The homographic intensity formula: $i = \dfrac{68}{t+10}$ with t in min and i in mm/min

Solution 5.11

The effective area: $A_a = C \cdot A = 0.50 \times 150 \times 10^4 = 750000$ m^2.
The outflow rate: $Q_v = 0.8 \times 60 = 48$ m^3/min.

1) Using the exponential intensity formula

From Eq. (5.36): $t_m = \left(\dfrac{a \cdot A_a(1+b)}{Q_v}\right)^{-1/b} = \left(\dfrac{6.3 \times 10^{-3} \times 750000(1-0.74)}{48}\right)^{-1/0.74}$,

$\boxed{t_m = 80\ \text{min}}$.

The volume is calculated using Eq. (5.37):

$\mathcal{V}_{\max} = a \cdot A_a \cdot t_m^{1+b} - Q_v \cdot t_m = 6.3 \times 10^{-3} \cdot 750000 \times 80^{1-0.74} - 48 \times 80$,

$\boxed{\mathcal{V}_{\max} = 10924\ \text{m}^3}$.

2) Using the homographic intensity formula

From Eq. (5.38): $t_m = \sqrt{\dfrac{a \cdot A_a \cdot b}{Q_v} - b} = \sqrt{\dfrac{68 \times 10^{-3} \times 750000 \times 10}{48} - 10}$, $\boxed{t_m = 93\ \text{min}}$.

The volume is calculated using Eq. (5.39):

$\mathcal{V}_{\max} = \dfrac{a \cdot A_a \cdot t_m}{t_m + b} - Q_v \cdot t_m = \dfrac{68 \times 10^{-3} \times 750000 \times 93}{93 + 10} - 48 \times 93$, $\boxed{\mathcal{V}_{\max} = 41585\ \text{m}^3}$.

Hydrograph method

A detention basin is designed to release the allowable flow when the developed inflow hydrograph enters the basin. At the planning or preliminary stage, the information of the outlet system in the basin is not yet available. As a result, it is suggested that the rising limb of the outflow hydrograph be approximated by a linear line. As shown in Figure 5.27, the outflow rate, $Q(t)$, at time t on the linear rising limb 1s estimated as:

$$Q_r(t) = \frac{Q_v \cdot t}{t_p} \qquad \text{for } 0 \le t \le T_p \tag{5.40}$$

in which $Q(t)$ = outflow rate, Q_v = allowable release, T_p = time to peak on outflow hydrograph and t = elapsed time.

The total detention storage volume is the volume difference between the inflow and outflow hydrographs from the beginning of the event to the time when the allowable release occurs. As a result, the required storage volume is:

$$\mathcal{V}_{\max} = \sum_{t=0}^{T_p} [Q_p(t) - Q_r(t)]\,\Delta t \qquad \text{for } 0 \le t \le T_p \tag{5.41}$$

in which $\mathcal{V}_{\max}$ = detention storage volume, $Q_p(t)$ = inflow rate at time t, and Δt = time increment such as five minutes.

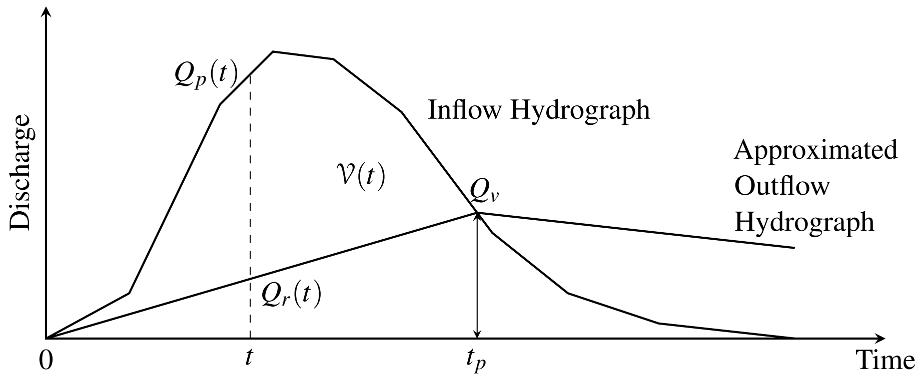


Figure 5.27: Volume de rétention par la méthode de l'hydrogramme

Example 5.12 As shown in Table E5.12, the inflow hydrograph has a peak flow of 28,32 m^3/s. The allowable peak release rate is set to be 15 m^3/s which will be reached after 45 min. Determine the volume of the retention basin using the hydrograph method. Draw the two hydrographs.

Table E5.12

Time (min)	0	5	10	15	20	25	30	35	40	45
Inflow (m^3/s)	0.00	2.83	14.16	21.24	26.90	28.32	24.07	19.82	16.99	15.50

■

Solution 5.12

Under the assumption of a linear rising outflow hydrograph, Eq (5.40) becomes

$Q_r(t) = \dfrac{15t}{45}$

With a time increment of five minutes, the accumulative storage, $\mathcal{V}(t)$, is computed using Eq. (5.41):

$$\mathcal{V}_{\max} = \sum_{t=0}^{45} [Q_p(t) - Q_r(t)]\,\Delta t \qquad \text{pour } 0 \le t \le 45 \text{ min}$$

Table S5.12 is the calculation of cumulative storage volume. The total detention storage volume,

$\boxed{V_{max} = 28448,04 \text{ m}^3}$.

Table S5.12

Time	Inflow	Outflow	Volume	
			Partial	cumulative
min	m^3/s	m^3/s	m^3	m^3
0	0.00	0.00	0.00	0.00
5	2.83	1.67	349.51	349.51
10	14.16	3.33	3247.53	3597.03
15	21.24	5.00	4871.29	8468.32
20	26.90	6.67	6070.30	14538.62
25	28.32	8.33	5995.05	20533.68
30	24.07	10.00	4220.80	24754.47
35	19.82	11.67	2446.54	27201.01
40	16.99	13.33	1097.03	28298.04
45	15.50	15.00	150.00	28448.04

Example 5.13 A watershed covering an area of 16 hectares will be developed to have a detention basin. The allowable discharge rates for 10-year and 100-year return periods are: $Q_{10} = 0.26 \text{ m}^3/\text{s}$ and $Q_{100} = 0.92 \text{ m}^3/\text{s}$, respectively. The inflow hydrographs for the 10-year and 100-year return periods are presented in Table E5.13. Calculate the retention volumes for the rainfall events with 10-year and 100-year return periods.

Table E5.13

Time (min)	10-yr hydrograph (m^3/s)	100-yr hydrograph (m^3/s)
0	0.00	0.00
10	0.00	0.00
20	0.00	0.57
30	0.64	1.84
40	2.14	4.39
50	1.01	2.69
60	0.66	1.84
70	0.52	1.37
80	0.48	0.92
90	0.44	0.61
100	0.34	0.47
110	0.29	0.39
120	0.27	0.34
130	0.20	0.31
140	0.14	0.17
150	0.11	0.08

Solution 5.13

For a 10-year return period

Assuming $T_p = 120$ minutes, Equation (5.40) becomes:

$$Q_r(t) = \frac{0.26t}{120}$$

With a time increment of 10 minutes, the cumulative storage, $\mathcal{V}(t)$, is calculated using Eq. (5.41):

$$\mathcal{V}_{\max} = \sum_{t=0}^{120} [Q_p(t) - Q_r(t)]\,\Delta t \qquad \text{for } 0 \le t \le 120 \text{ min}$$

Table S5.13a presents the calculation of the cumulative storage volume. The total retention storage volume is $\boxed{V_{\max} = 3099 \text{ m}^3}$.

Table S5.13a: Computation of retention volume for a return period of 10 years with $T_p = 120$ min

Time	Inflow	Outflow	Volume	
			Partial	cumulative
min	m^3/s	m^3/s	m^3	m^3
0	0.00	0.00	0	0
10	0.00	0.00	0	0
20	0.00	0.00	0	0
30	0.64	0.07	345	345
40	2.14	0.09	1232	1577
50	1.01	0.11	541	2118
60	0.66	0.13	318	2436
70	0.52	0.15	221	2657
80	0.48	0.17	184	2841
90	0.44	0.20	147	2988
100	0.34	0.22	74	3062
110	0.29	0.24	31	3093
120	0.27	0.26	6	3099
130	0.20	0.00	0	3099
140	0.14	0.00	0	3099
150	0.11	0.00	0	3099

For a 100-year return period

Assuming $T_p = 80$ minutes, Equation (5.40) becomes:

$$Q_r(t) = \frac{0.92t}{80}$$

With a time increment of 10 minutes, the cumulative storage, $\mathcal{V}(t)$, is calculated using Eq. (5.41):

$$\mathcal{V}_{\max} = \sum_{t=0}^{80} [Q_p(t) - Q_r(t)]\,\Delta t \qquad \text{for } 0 \le t \le 80 \text{ min}$$

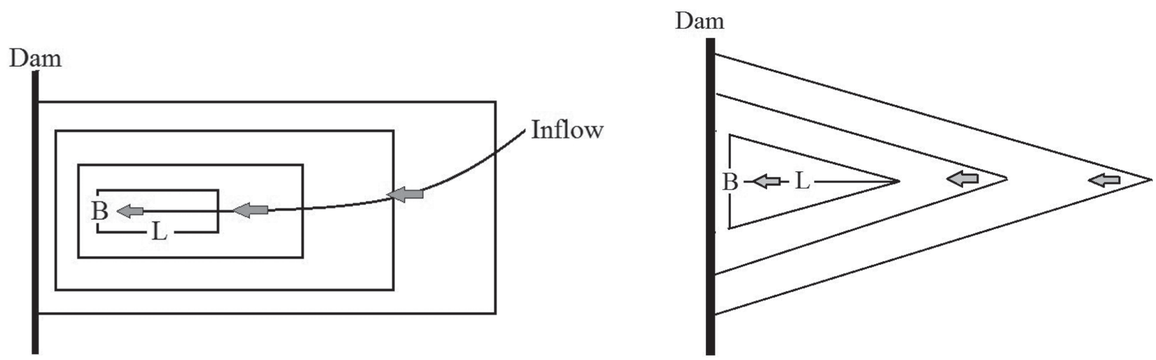
Table S5.13b presents the calculation of the cumulative storage volume. The total retention storage volume is $\boxed{V_{\max} = 5757 \text{ m}^3}$.

Table S5.13b: Computation of retention volume for a return period of 100 years with $T_p = 80$ min

Time	Inflow	Outflow	Volume	
			Partial	cumulative
min	m^3/s	m^3/s	m^3	m^3
0	0.00	0.00	0	0
10	0.00	0.00	0	0
20	0.57	0.23	204	204
30	1.84	0.35	897	1101
40	4.39	0.46	2358	3459
50	2.69	0.58	1269	4728
60	1.84	0.69	690	5418
70	1.37	0.81	339	5757
80	0.92	0.92	0	5757
90	0.61	0.00	0	5757
100	0.47	0.00	0	5757
110	0.39	0.00	0	5757
120	0.34	0.00	0	5757
130	0.31	0.00	0	5757
140	0.17	0.00	0	5757
150	0.08	0.00	0	$\boxed{5757}$

5.2.7.7 Preliminary Shaping

Hydraulic structures are designed to process flows generated from small to extreme events. As a result, a detention basin is built with multiple layers of storage volume, starting from the bottom layer for a 2-yr storage volume, the mid layer to store up to a 10-yr detention volume, and the additional top layer to accommodate a 100-yr storage volume. Having known the detention volumes, the cross-sections of the basin can be approximated by a truncated cone with a circular, triangular, or rectangular base (*Guo et al.*, 2023).



(a) Rectangular basin. (b) Triangular basin.

Figure 5.28: Shapes of retention basin.

Bassin rectangulaire

The basic geometric parameters are the width and length of the cross-sectional area for each layer in a rectangular basin, as shown in Fig. 5.28a. The cumulative storage volume between two cross- sectional areas is calculated as:

$$L_2 = L_1 + 2mH \tag{5.42}$$

$$B_2 = B_1 + 2mH \tag{5.43}$$

$$A_1 = L_1 B_1 \tag{5.44}$$

$$A_2 = L_2 B_2 \tag{5.45}$$

$$\mathcal{V} = 0,5\,(A_1 + A_2)\,H \tag{5.46}$$

where L length in m, B width in m, m average side slope, H vertical distance in m, and $\mathcal{V}$ storage volume in m^3. The subscript 1 represents the variables for the bottom layer, and 2 represents the variables for the top layer.

Usually, a basin is divided into multiple layers, and the slope of the basin's embankment varies with respect to water depth, starting from as steep as 1V:2H at the bottom to as flat as 1V:10H at the top.

Triangular basin

Repeating the calculations in the previous section, the cross- sectional area in a triangular basin as shown in Fig. 5.28b is calculated by the base width, B_2, and height, L_2, as:

$$A_2 = 0,5 B_2 L_2 \tag{5.47}$$

where L_2 and B_2 are determined, respectively, using equations (5.42) and (5.43). The volume between two triangular layers is estimated by Eq. (5.46).

Applying Eqs (5.42) through (5.47) to 2-, 10-, and 100-year storage volumes, the basin is shaped with various stages. Between two adjacent layers, the required side slope can be incorporated into the storage volume calculation. The stage-storage curve and stage-contour area curve can then be established. During the final design, the detention basin can be further refined to fit the site grading plan.

Example 5.14 The 10- and 100-yr detention volumes for the basin are 3250 m^3 and 5950 m^3. Distribute this volume using a triangular basin. The bottom triangle has a width of 200 feet and height of 60 m. The side slope starts with 1V:4H for water depths less than 0.9 m, and then decreases to 1V:10H for recreational bank areas. ■

Solution 5.14

The pond geometry is summarized in Table S5.14.

Table S5.14 shows that for this case, the 10-yr water surface elevation is 1.5 m, while the 100-yr water depth is 1,5 m. The 0.3 m freeboard provides a brim-full capacity of 7642.93 m^3.

Table S5.14

m	Water Depth	*b*	*L*	*A*	Storage Pond Volume Partial	Cumulative	Remark
	(m)	(m)	(m)	(m^2)	(m^3)	(m^3)	
4	0.00	60.0	110.0	3300.00	0.00	0.00	Input
4	0.15	61.2	111.2	3402.72	502.70	502.70	
4	0.30	62.4	112.4	3506.88	518.22	1020.92	
4	0.45	63.6	113.6	3612.48	533.95	1554.88	
4	0.60	64.8	114.8	3719.52	549.90	2104.78	
4	0.75	66.0	116.0	3828.00	566.06	2670.84	
4	0.90	67.2	117.2	3937.92	582.44	3253.28	10-yr WS
10	1.05	70.2	120.2	4219.02	611.77	3865.05	
10	1.20	73.2	123.2	4509.12	654.61	4519.67	
10	1.35	76.2	126.2	4808.22	698.80	5218.47	
10	1.50	79.2	129.2	5116.32	744.34	5962.81	100-yr WS
15	1.65	83.7	133.7	5595.35	803.37	6766.18	Freeboar
15	1.80	88.2	138.2	6094.62	876.75	7642.93	Freeboar

5.3 Review Questions

1. What are the main roles of auxiliary structures in sewer networks?
2. What are the primary types of structures used in sewer networks?
3. Where are manholes typically located on a sewer main?
4. List three advantages of using PVC pipes.
5. What types of cross-sections are generally used in stormwater gutters?
6. The shape of sewer pipes is generally circular; list three cases where it is preferable to use an egg-shaped (ovoid) design (provide three reasons).
7. Explain the role of the following auxiliary structures: Manhole, Storm Drain, Gutter, and Overflow Weir.
8. List three types of materials commonly used in sewer pipes.
9. Name two methods for calculating the volume of a stormwater retention basin.
10. What are the main criteria to consider when placing a manhole?
11. What auxiliary structures are commonly found in a sewer network?
12. How do the auxiliary structures in a sewer network contribute to the overall system's proper functioning?
13. What are the different types of accessible structures with special profiles commonly found in sewer networks?
14. How are weirs used to control stormwater flow rates in sewer networks?

5.4 Objective Questions

Q 5.1 What is the main disadvantage of using PVC pipes in wastewater management?

a) Susceptible to H_2S

b) Weak resistance to aggressive soils and water

c) Expensive for diameter $>$ 400 mm

d) High internal roughness

■

Q 5.2 What are the common applications of PVC pipes in wastewater management?

a) Domestic wastewater disposal.

b) Stormwater collection.

c) Industrial effluent transportation.

d) All of the above.

■

Q 5.3 What are the advantages of reinforced concrete pipes in wastewater management?

a) High resistance to mechanical stresses.

b) Easy and lightweight installation.

c) Low initial cost.

d) Adaptability to all types of terrains.

■

Q 5.4 What is prestressing in prestressed reinforced concrete pipes in wastewater management?

a) A concrete coating applied to the inner surface of the pipes.

b) A thermal treatment to enhance concrete strength.

c) The use of tensioned cables or bars to preload the structure and reduce stress.

d) The injection of chemicals to increase concrete durability.

■

Q 5.5 What are the advantages of prestressed reinforced concrete pipes in wastewater management?

a) Steel savings

b) Good strength

c) Relatively long element lengths: 4 to 6 m

d) All of the above.

■

Q 5.6 What is the primary function of a gutter in a wastewater network?

a) Collect rainwater and direct it to sewer inlets.

b) Temporarily store runoff water during heavy rainfall.

c) Separate wastewater from stormwater.

d) Treat wastewater before its discharge into the environment.

■

Q 5.7 What is a manhole in a wastewater network?

a) A flow regulation device.

b) An access point for inspection and maintenance.

c) A wastewater treatment system.

d) A stormwater pumping station.

■

Q 5.8 In wastewater networks, drop manholes are provided when there is [Rat:2022/2023]:

a) a transition from a gravity system to a pressurized system

b) a change in pipe section

c) a steep slope

d) a change in direction

■

Q 5.9 What is the main purpose of sewer inlets? [Rat:2022/2023]

a) Prevent water infiltration into sewer pipes

b) Provide access for maintenance and repair of sewer pipes

c) Prevent unpleasant odors from sewers

d) Allow the introduction of rainwater and street cleaning water into a sewer system

■

Q 5.10 What is the primary function of a stormwater overflow in a wastewater network?

a) Temporarily store runoff water during intense rain events.

b) Remove solid waste and sediments from wastewater.

c) Regulate flow rates in pipelines.

d) Discharge exceptional peak flows to the receiving environment.

■

5.5 problems

P 5.1 A composite gutter section as seen in Figure 5.4 has a design spread of $T = 2.45$ m; a cross-slope $S_x = 0.03$; $W = 0.60$ m with a gutter depression $a = 5$ cm; the gutter slope $S_0 = 0.005$; and the *Manning*'s n is 0.016. If this gutter section leads to a P-50×100 Grate of length 0.6 m and width 0.6 m. Calculate the following:

1. the side discharge Q_s
2. the frontal discharge Q_w
3. the Interception capacity Q_i

■

P 5.2 For the gutter with a depressed curb-opening inlet, the longitudinal slope is 0.01, the manning roughness coefficient is $n = 0.016$, the length of the curb-opening inlet is 2.5 m. Other parameters are: $T = 2.75$ m, $W = 0.7$ m, $S_x = 0.02$, and $a = 50$ mm. Find: the side flow rate (Q_s), frontal flow rate a (Q_w), total gutter flow rate and the intercepted flow rate. ■

P 5.3 Calculate the maximum frontal flow and side flow through a composite gutter section when maximum gutter spread is 5 m, and gutter width is 1 m. Gutter section has roadside slope of $S_x 0,015$, gutter slope of $S_w = 0,0325$, and longitudinal road slope of $S_0 = 0,015$. Assume *Manning*'s $n = 0.013$. ■

P 5.4 Calculate the flow spread in a 2 m wide asphalt gutter for a flow of 0.08 m^3/s. The gutter is at 2% longitudinal and 4% cross slopes, respectively. ■

P 5.5 Calculate the spread in a 1.8 m wide asphalt gutter for a flow of 0.07 m^3/s. The gutter is at 1.5% longitudinal slope and 4% cross slope. ■

P 5.6 Determine the flow in a composite gutter with $W = 0.5$ m, $S_0 = 0.015$, $S_x = 0.03$, and $a = 0.08$ m. Assume that T is equal to 2 m and the *Manning* coefficient is 0.01. ■

P 5.7 Determine the flow depth and spread of a triangular gutter with $S_0 = 0.02$, $S_x = 0.03$, and $n = 0.02$ where the peak flow is estimated as 0.1 m^3/s. ■

P 5.8 Curb-opening inlets are placed along a 20-m wide street. The length of the inlets is 1 m that are placed on a continuous grade and the runoff coefficient is 0.9. The inlets drain the stormwater into a triangular gutter with $S_x = 0.03$, $S_0 = 0.02$, and $n = 0.03$. Determine the maximum allowable distance between inlets considering that the rainfall intensity is equal to 15 mm/h and the allowable spread is 3 m. ■

P 5.9 The design discharge for a triangular gutter with $S_0 = 0.015$, $S_x = 0.025$, and $n = 0.015$ is estimated as 0.073 m^3/s. Determine the flow depth and spread for this gutter. ■

P 5.10 Determine the flow in a composite gutter with the following geometry when $T = 1.75$ m. $W = 0.4$ m, $S_0 = 0.01$, $S_x = 0.022$, and $a = 0.07$ m. Assume that the *Manning* roughness factor is 0.015. ■

P 5.11 A composite gutter section has $S_x = 0.02$, $S_0 = 0.01$, $a = 0.05$, $n = 0.016$, and $W = 0.6$ m. Determine the spread T at $Q = 0.028$ m^3/s. ■

P 5.12 A V-section swale will be used in an 2.5 m shoulder to convey 0.028 m^3/s. The longitudinal slope is $S_0 = 0.008$ and the *Manning* roughness factor is $n = 0.016$. Determine the cross-slopes and the depth of the swale. ■

P 5.13 A circular swale with a diameter of $D = 1.5$ m is to carry $Q = 0.042$ m^3/s. The *Manning* roughness factor is $n = 0.016$, and the longitudinal slope is $S_0 = 0.01$. Determine the required depth and the top width of the swale. ■

P 5.14 A composite gutter section has $T = 2.5$ m, $n = 0.015$, $S_x = 0.025$ and $S_0 = 0.01$. Find:

- Flow in gutter at design spread
- Flow in width ($W = 0.6$ m) adjacent to the curb

■

P 5.15 Determine the interception capacity and bypass flow for a 0.6 m × 0.6 m curved vane grate inlet in a composite gutter section having a cross slope of 0.022 m/m, *Manning*'s roughness of 0.015, and longitudinal slope of 0.014 m/m. The gutter depression is 50.0-mm deep and 0.60-m wide, and the gutter discharge and spread at the inlet are 0.12 m^3/s and 2.9 m, respectively. ■

P 5.16 Given: $T = 1.8$ m, $S_x = 0.04$, $S_0 = 0.03$, $n = 0.016$, calculate interception capacity for the the following grates:

1. 0.6 m × 0.6 m P-50 grate
2. 0.6 m × 1.2 m (long) P-50 grate
3. 0.6 m × 0.6 m reticuline grate

■

P 5.17 Given: $W = 0.6$ m, $T = 2.4$ m, $S_x = 0.025$, $S_0 = 0.01$, $E_0 = 0.69$, $Q = 0.085$ m^3/s, $V = 0.94$ m/s and Gutter depression =5 cm. Find the interception capacity of:

1. A curved vane grate
2. A reticuline grate 0.6-m long and 0.6-m wide

■

P 5.18 A smooth-asphalt ($n = 0.013$.) roadway has a cross slope of 3%, a longitudinal slope of 2%, a curb height of 15 cm, and a 90-cm-wide concrete gutter. If the flow rate in the gutter is 0.08 m^3/s, determine the length of a 12-cm-high curb inlet that is required to remove all the water from the gutter. Consider the cases where: (a) there is an inlet depression of 25 mm; and (b) there is no inlet depression. What inlet length would reduce the gutter flow rate by 80%? ■

P 5.19 Compute the interception capacity of a 3.5-m long curb-opening inlet in a triangular gutter section having a cross slope of 0.025 m/m, longitudinal slope of 0.03 m/m, and *Manning*'s roughness of 0.015. The gutter carries a design discharge of 0.08 m^3/s. ■

P 5.20 A rectangular parking lot measuring 150 by 300 m makes a watershed for which the inlet time, t_e, is 15 min (the sewer inlet is located in the geometric centre of the watershed) and the time of flow, t_f, is 5 min. Figure P5.20 shows the watershed with isochrones of 5, 10, 15, and 20 min. If the runoff coefficient is constant and uniform and equal to 0,85, calculate the maximum outflow at the sewer inlet following a precipitation of 50 mm/h falling for 5 min over the entire surface of the watershed.

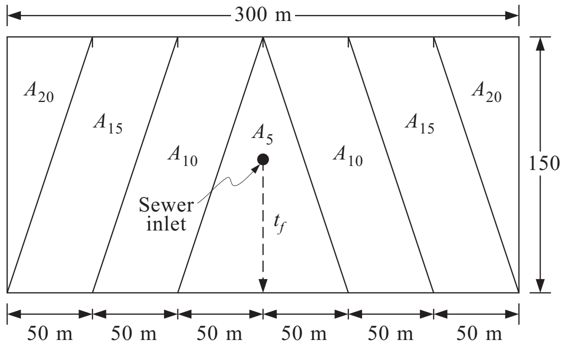


Figure P5.20

■

P 5.21 A concrete rectangular parking lot ($n = 0.013$) 30 m by 50 m drains into a single water inlet with a capacity of 30 L/s in one of its lower corners. The two corners of the highest small side of the parking lot are at an elevation of 100 m and the two others are at 99,5 m. Precipitation intensity (mm/h) is represented by $i = 2184.4/(t_c + 12)$; t_c is in minutes.

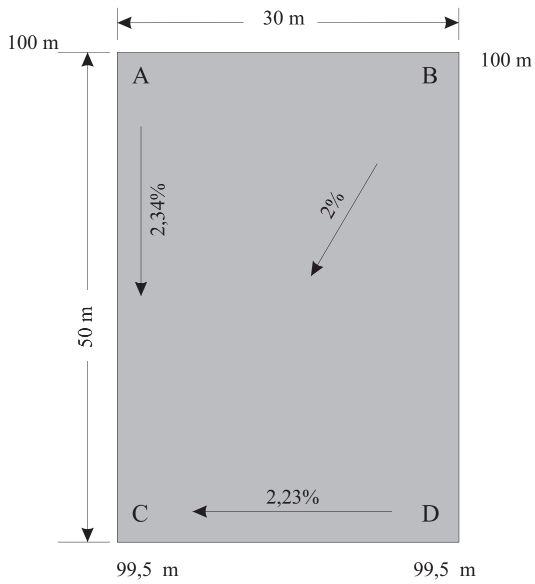


Figure P5.21

1. At what level should the water inlet be installed knowing that the surface of the parking lot must be shaped like a cone with a slope toward the inlet as small as possible but not less than 2%?
2. What is the value of the inlet time?

3. What is the maximum runoff flow at the water inlet? Precipitations with a five year return period must be chosen and the runoff coefficient of the parking lot is 0.85. Use the Kerby equation.

4. Knowing that if maximum flow at a water inlet must not exceed 30 L/s, will the unique proposed water inlet be sufficient? If not, what would you suggest? When repetitive computations are required, do not make more than three, judiciously chosen ones.

■

P 5.22 A watershed has a drainage area of 20 ha and runoff coefficient of 0.65. The 10-year and 100-year allowable release rates are 0.708 and 1.416 m^3/s.

1. Estimate the required 10-year and 100-yr storm water detention volumes by the volume method.

2. Use a triangular basin to establish the stage-storage curve. The bottom triangular area has a base width of 60 m and a length of 120 m. The average side slope of the basin vanes from 1V:2H for water depth < 0,6 m, 1V:5H for water depth from 0.6 to 1.5 m, and 1V:10H for water depth > 1.5 m.

The rainfall intensity formula for the homework problems is

$$i(\mathrm{mm/h}) = \frac{28.5 P_{1-h}}{(10+t)^{0.789}}$$

in which $P_{1-h} = 40$ mm for the 10-year event, and 66 mm for the 100-yar event, and t = rainfall duration in minutes. ■

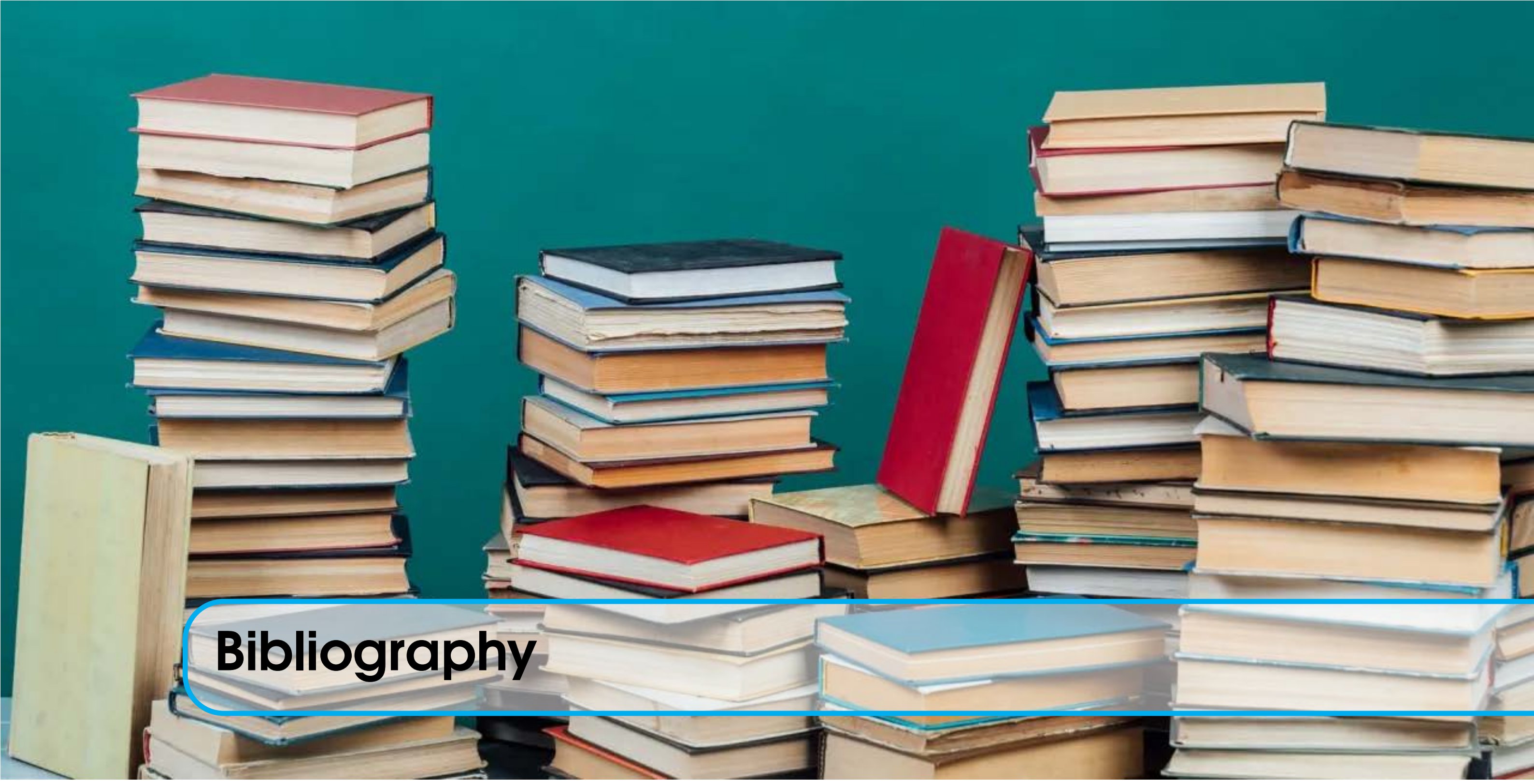
P 5.23 To design a detention basin the allowable flow release rates are: 0.016 $m^3/s/ha$ for a 10-yr event and 0.06 $m^3/s/ha$ for a 100-yr event. The watershed area is 16 ha. The 10- and 100-yr post-development hydrographs are given in Table P5.23. The 10-year and 100-year allowable release rates are 0.260 and 0,963 m^3/s. Apply the hydrograph method to determine the 10- and 100- yr detention volumes.

Table P5.23

Time (min)	10-yr hydrograph (m^3/s)	100-yr hydrograph (m^3/s)
0	0.000	0.000
10	0.538	0.481
20	1.303	1.812
30	1.444	3.058
40	1.133	2.917
50	0.821	2.209
60	0.623	1.586
70	0.481	1.048
80	0.368	0.623
90	0.283	0.368
100	0.255	0.283
110	0.198	0.198
120	0.113	0.113

■

P 5.24 Estimate using the rainfall method the storage volume of a retention basin downstream of the catchment area for a 10-year return period rainfall event, assuming a constant outflow rate of 450 L/s. The given data are: $A = 558.9$ ha, $C = 22.6\%$, *Montana* coefficients for a 10-year return period: $a = 7.40$ and $b = -0.72$. ■



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Appendix A. Mini-Project

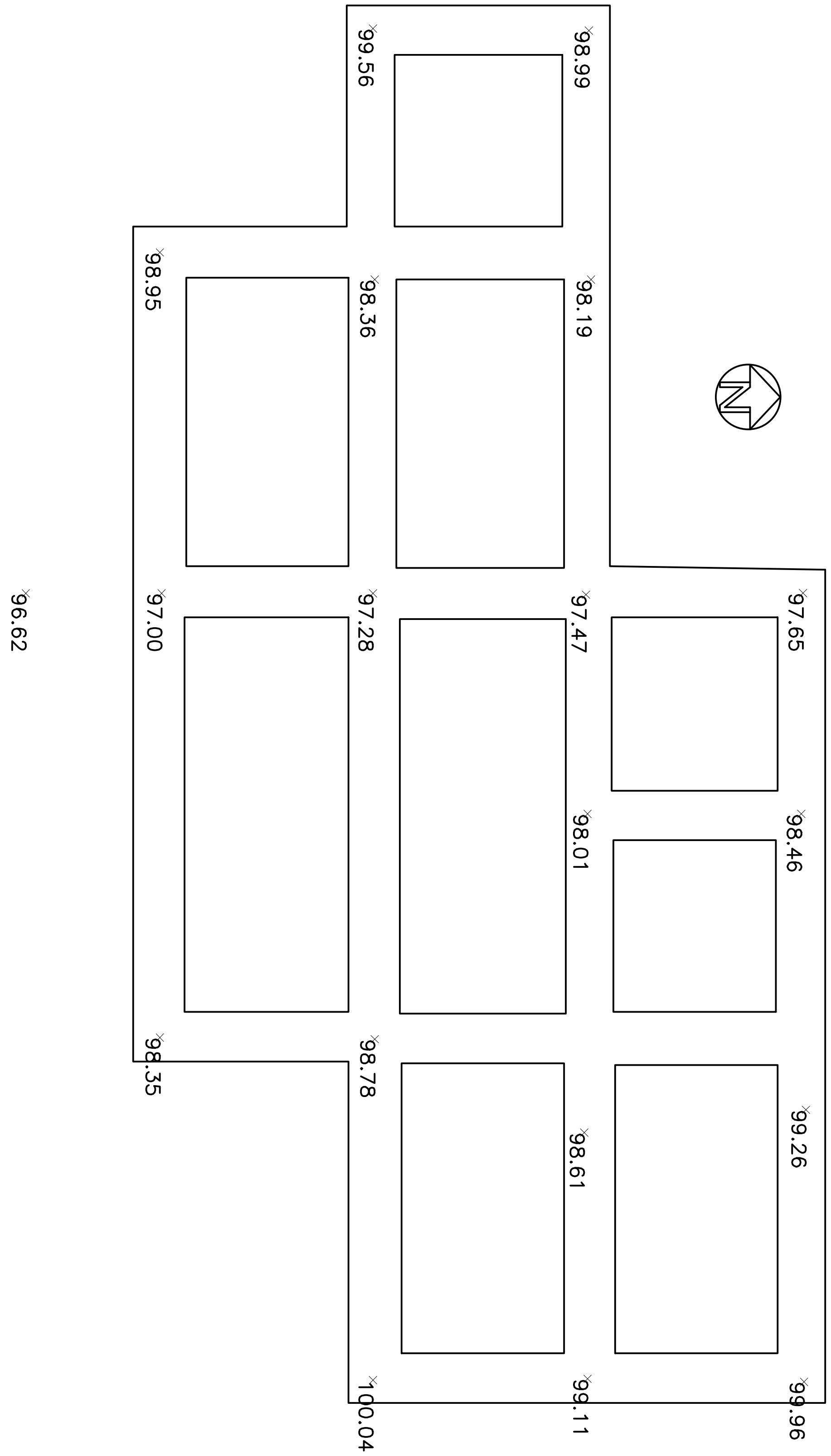
The city where you reside plans to develop a new residential district on its outskirts (see site plan). The area, designed for *N* residents (see Table A1), is not yet equipped for stormwater and wastewater disposal. The engineering firm you work for has been tasked with designing the sewer network (combined system). Wastewater flow rates will be estimated based on the potable water consumption of the residents. Stormwater flow rates will be estimated using the rational method, with the *Montana* coefficients (a and b) for a 10-year return period, as shown in Table A1.

Table A1: Project data for each student.

N°	Full name	a	b	N
1		5.9	-0.71	2000
2		6.7	-0.66	1970
3		6.37	-0.64	1940
4		6	-0.62	1910
5		7	-0.57	1880
6		6.2	-0.57	1850
7		5.5	-0.67	1820
8		5.4	-0.66	1790
9		4	-0.58	1760
10		7.3	-0.57	1730
11		3.3	-0.54	1700
12		5.6	-0.48	1670
13		7.7	-0.68	1640
14		5.1	-0.69	1610
15		5.2	-0.6	1580
16		5.8	-0.6	1550
17		5.39	-0.68	1520
18		5.1	-0.64	1490
19		4.8	-0.59	1460
20		4.2	-0.64	1430

21	8.7	-0.74	1400
22	4.6	-0.57	1370
23	4.2	-0.56	1340
24	3.8	-0.54	1310
25	5.2	-0.61	1280
26	3.4	-0.47	1250
27	4.7	-0.55	1220
28	4	-0.6	1190
29	5.9	-0.64	1160

1. On the site plan:
   - Draw the sewer network layout;
   - Number the pipes and manholes;
   - Determine the lengths of the sections and the natural ground elevations at the manholes;
   - Delineate the catchment areas and calculate their areas in hectares;
2. Use the previous results to calculate the peak stormwater flow rate if the runoff coefficient for all catchment areas is $C = 0.7$ and the time of concentration is $t_e = 5$ min (Table A2);
3. Determine the average and peak discharge at the outlet of each sub-catchment if the population growth rate is 1.5%, the number of years is 30 years, the discharge coefficient is 80%, and the water supply allocation is 150 l/d/cap (Table A2);
4. Calculate the total discharge rates (Table A2).
5. Design the sewer network (Table A3). The *Bazin* formula will be used for calculating nominal flow rates and velocities in the pipes.
6. Determine the hydraulic parameters of each section (Table A4).
7. For each pipe, verify that the minimum self-cleaning velocity is maintained. If not, propose a solution to meet this requirement;
8. Draw the longitudinal profile of the main sewer line.



Urban Sanitation Project

Full names	
Scale : 1 / 1000	
Date :	

Table A2: Flow rate calculations.

Manhole	A	C	CA	$\sum CA$	Path	t_e	Length From Path	t_f	t_c	i	Q_{Storm}	Q_{Waste} partial	Q_{Waste} Cumul.	Q_T
	(ha)		(ha)	(ha)		(min)	(m)	(min)	(min)	(mm/min)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
R1														
R5														
R9														
R8														
R11														
R15														
R14														
R17														
R20														
R19														

Table A3: Sewer network design.

Collect.	Manholes		Up. Grades			Dow. Grades			Depths		Drops		Lengths	Slopes	Q_T	Diameters
	Up.	Dow.	Elv.	WL In	WL Out	Elv.	WL IN	WL Out	Up.	Dow.	Up.	Dow.				
			(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m/m)	(m^3/s)	(mm)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
C1	M1	M2														
	M2	M3														
	M3	M4														
	M4	M8														
	M8	M14														
	M14	M19														
	M19	Mrelease														
C1-1	M5	M6														
	M6	M7														
	M7	M8														
C1-2	M9	M10														
	M10	M8														
C1-3	M11	M12														
	M12	M13														
	M13	M14														
C1-4	M15	M16														
	M16	M14														
C1-5	M17	M18														
	M18	M19														
C1-6	M20	M19														

Table A4: Determination of hydraulic parameters.

Collect.	Manholes		Q_T	Slopes	Diameter	Full section		Ratios			y	V	V_{sc}	Remarks
	Up	Dow.				V_{fs}	Q_{fs}	R_Q	R_V	R_y				
			(m^3/s)	(m/m)	(mm)	(m/s)	(m^3/s)				(mm)	(m/s)	(m/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
C1	M1	M2												
	M2	M3												
	M3	M4												
	M4	M8												
	M8	M14												
	M14	M19												
	M19	Mrelease												
C1-1	M5	M6												
	M6	M7												
	M7	M8												
C1-2	M9	M10												
	M10	M8												
C1-3	M11	M12												
	M12	M13												
	M13	M14												
C1-4	M15	M16												
	M16	M14												
C1-5	M17	M18												
	M18	M19												
C1-6	M20	M19												