Manual on drainage in urbanized areas

Volume I
Planning and design of drainage systems

A contribution to
the International Hydrological
Programme

Prepared by
the Working Group,
Project A 2.9

Edited by:
W. F. Geiger
J. Marsalek
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F. C. Zuidema, Chairman

UNESCO

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Preface

Although the total amount of water on earth is generally assumed to have remained virtually constant, the rapid growth of population, together with the extension of irrigated agriculture and industrial development, are stressing the quantity and quality aspects of the natural system. Because of the increasing problems, man has begun to realize that he can no longer follow a "use and discard" philosophy – either with water resources or any other natural resources. As a result, the need for a consistent policy of rational management of water resources has become evident.

Rational water management, however, should be founded upon a thorough understanding of water availability and movement. Thus, as a contribution to the solution of the world's water problems, Unesco, in 1965, began the first world-wide programme of studies of the hydrological cycle – the International Hydrological Decade (IHD). The research programme was complemented by a major effort in the field of hydrological education and training. The activities undertaken during the Decade proved to be of great interest and value to Member States. By the end of that period, a majority of Unesco's Member States had formed IHD National Committees to carry out relevant national activities and to participate in regional and international co-operation within the IHD programme. The knowledge of the world's water resources had substantially improved. Hydrology became widely recognized as an independent professional option and facilities for the training of hydrologists had been developed.

Conscious of the need to expand upon the efforts initiated during the International Hydrological Decade and, following the recommendations of Member States, Unesco, in 1975, launched a new long-term intergovernmental programme, the International Hydrological Programme (IHP), to follow the Decade.

Although the IHP is basically a scientific and educational programme, Unesco has been aware from the beginning of a need to direct its activities toward the practical solutions of the world's very real water resources problems. Accordingly, and in line with the recommendations of the 1977 United Nations Water Conference, the objectives of the International Hydrological Programme have been gradually expanded in order to cover not only hydrological processes considered in interrelationship with the environment and human activities, but also the scientific aspects of multi-purpose utilization and conservation of water resources to meet the needs of economic and social development. Thus, while maintaining IHP's scientific concept, the objectives have shifted perceptibly towards a multidisciplinary approach to the assessment, planning, and rational management of water resources.

As part of Unesco's contribution to the objectives of the IHP, two publication series are issued: "Studies and Reports in Hydrology" and "Technical Papers in Hydrology." In addition to these publications, and in order to expedite exchange of information in the areas in which it is most needed, works of a preliminary nature are issued in the form of Technical Documents.

The purpose of the continuing series "Studies and Reports in Hydrology" to which this volume belongs, is to present data collected and the main results of hydrological studies, as well as to provide information on hydrological research techniques. The proceedings of symposia are also sometimes included. It is hoped that these volumes will furnish material of both practical and theoretical interest to water resources scientists and also to those involved in water resources assessments and the planning for rational water resources management.

Contents

Page

Page

Foreword

Large concentration of people in relatively small areas is recognized as an inevitable historic process of urbanization. This process is continuing and predictions indicate that by the end of this century, about one half of the total world population will live in urban areas. High concentrations of population change the landscape and hydrological cycle of the affected area and increase demands on such services as water supply, flood protection and drainage, wastewater disposal, and water-based recreation. Even though the importance of individual services greatly varies, they are often interactive and all need to be considered in successful management of water resources. From perilous events, such as flooding, health hazards, and loss of human life, and the concomitant economic losses, we have learned that urban development and its water systems have to be planned in an orderly manner.

The planning and design of drainage systems in urban areas is strongly affected by climatic, physiographic and socio-economic conditions, and institutional arrangements. Consequently, the planning of drainage systems in urbanized areas varies considerably among countries, or even regions. Nevertheless, a common technically-sound and cost-effective approach to drainage planning and design can be identified together with many appropriate existing design methods. Besides the general need for drainage planning, there is an increasing need to base the technical drainage design and the related political decisions on good supporting data, their careful analysis and interpretation, and on analysis of design alternatives. It is expected that new technology for data collection and analysis will bring about further progress in this area.

Since 1971, Unesco has paid special attention to urban hydrology under the International Hydrological Decade and the International Hydrological Programme (IHP). In particular, Unesco has sponsored many activities in this field including symposia and workshops on the effects of urbanization and industrialization on water resources and their management. Recognizing the importance of drainage for the well-being of urban population, particularly in less economically developed countries, and at the same time recognizing the availability of expertise on drainage planning and design, Unesco initiated Project A.2.9 on urban hydrology under the IHP-II. To conduct this project, Unesco established a Working Group and charged it with responsibility to develop a manual for planning and design of drainage in urban areas. This Working Group comprised eight internationally recognized experts on urban drainage and was further assisted by several observers and the Unesco staff. The members of the Working Group prepared written materials for the manual in their respective areas of expertise. Such materials were reviewed and expanded by the Editorial Board and eventually integrated into a unified comprehensive document. The members of the Working Group and the Editorial Board are listed below in the acknowledgement.

Although there are numerous national urban drainage manuals available, the Unesco manual is unique by being the most comprehensive document in this category, by reflecting the most advanced approaches from many countries, and by paying special attention to the conditions in economically less developed countries. The manual's comprehensiveness consists in addressing not only the problems of conventional drainage, but also modern methods of stormwater management and the collection and analysis of supporting data. The manual is divided into two volumes which are fairly independent. While the first volume deals with the planning and design of drainage systems, the second one deals with the collection and analysis of data required in drainage design. The manual consequently combines design and data collection aspects by many cross-references between the various chapters of both volumes.

The main objectives of the manual are to advance the understanding of complex interactions between urban drainage and other facets of urban water resources, to increase the awareness of various planning alternatives, to aid in the selection of appropriate calculation procedures, to demonstrate the importance of input and supporting data, to guide the decision-makers and designers in implementation of urban drainage projects, and to increase awareness of pitfalls of drainage planning. Although the manual is meant primarily for designers, engineers and planners, other professions, such as researchers and decision-makers, should find it also of interest.

The first volume describes the urban drainage system, various approaches to urban drainage, structural elements of drainage systems, design parameters and computational procedures for runoff calculations, hydraulic design of drainage systems and their components, network design methods, and organization and administration of urban drainage projects.

The second volume describes the methods of collection and analysis of meteorological, hydrological, ancillary and water quality data used in drainage design, the handling of such data, and organization of data collection programmes. Although both volumes are more or less independent documents, in order to obtain full benefits of this manual, it is desirable to work jointly with both volumes.

Further activities related to the Unesco urban drainage manual are expected under IHP-III and IHP-IV. Such activities will concentrate on technology transfer in the form of meetings and training courses.

Acknowledgement

The Unesco drainage manual is a result of an intensive international effort involving many individuals and institutions. For brevity, only the key contributors to this effort can be acknowledged here. In this connection, the members of the Unesco IHP-II, Project A.2.9 Working Group on Urban Hydrology are listed below in the alphabetical order. Footnotes identify the members of the Editorial Board and the Chairman of the Group.

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[1] Member of the Editorial Board

[2] Chairman of the Working Group and the Editorial Board

1 Introduction

1.1 URBANIZATION AND THE HYDROLOGICAL REGIME

Evaluation of the effects of urban and industrial development on the hydrological cycle and water quality is one of the most important yet least understood problems of scientific and applied hydrology. This follows from the fact that solutions of such problems as water supplies for municipal and industrial needs, drainage, and protection against floodings and rise of groundwater levels were considered as purely engineering problems to be solved according to availability or special features of local water resources. In the course of time, however, it has become evident that the hydrological cycle and its individual components are subject to great changes in urbanized areas and these changes then demonstrate themselves according to physiographic features and water use.

At present, the development and implementation of systems for the monitoring of urban water resources and implementation of environmental protection measures are impossible without a careful consideration of changes of the hydrological cycle resulting from urbanization. The importance of problems of urban water resources is steadily increasing, because urbanization is an inevitable continuing process leading to dramatic changes in land use, and water regime and resources.

1.1.1 Urbanization and population growth

The increasing population and the movement of people from rural to urban areas on all continents lead to changes in land use. The data on population trends in different parts of the world are useful for examining the relationship between social and economical factors and urban water characteristics in the development planning. The data in Table 1.1 elucidate the trends in the expansion and concentration of population.

While in 1800 only one per cent of the world population lived in cities, it appears from Table 1.1 that, in 1970, 37 per cent of the entire world population were urban and by the year 2000, the urban population is expected to reach 51 per cent of the total. Moreover, it was predicted that the combined urban population of the world would increase almost two and a half times during the period from 1970 to 2000. It is important to note that the urban population grows differently in various parts of the world. If we look at the projected trends in more developed countries, we find that the urban population may grow from 717 million in 1970 to 1174 million in the year 2000. This corresponds to an increase of 457 million or 54 percent. In contrast, the urban population in less developed areas may increase from 635 million in 1970 to 2155 million in the year 2000. Table 1.1 also shows that, during the same period, the percentage of total population in urban areas may rise from 66 to 81 per cent in the more developed regions whereas there may be an increase from 25 to 43 per cent in the less developed countries. Comparisons of projected population of some areas for the year 2000 and the urban populations in 1970, shown as ratios in Table 1.2, demonstrate such increases.

In 1971, for instance, the urban population in India was 13 per cent of the total poulation, a smaller percentage than in more developed countries. The number of towns with a population of more than 20000, however, has increased from 536 in 1951 to 957 in 1971. Most migration from rural areas was, however, into towns with a population of 50000 and more (Rameshan and Sarma, 1978). Considering urban areas of more than 1 million inhabitants such a trend becomes even more obvious. In 1820, London was the only city in the world with 1 million people. In 1910 there were 11 cities with more than 1 million of inhabitants, six of which were in Europe. Only 50 years later this number increased to 75 cities, 24 of which were located in less developed countries. For 1985, 270 urban areas of such size were predicted, 140 of which would be located in less developed countries.

Table 1.1 Urban and Rural Populations, 1965 to 2000 (Unesco, 1979)

	Year				
	1965	1970	1980	1990	2000
Urban Population (millions)					
World total	1158	1352	1854	2517	3329
More developed regions	651	717	864	1021	1174
Less developed regions	507	635	990	1496	2155
Rural Population (millions)					
World total	2131	2284	2614	2939	3186
More developed regions	386	374	347	316	280
Less developed regions	1745	1910	2267	2623	2906
Percentage of Urban Population					
World total	35,2	37,2	41,5	56,1	51,1
More developed regions	62,8	65,7	71,4	76,4	80,2
Less developed regions	22,5	25,0	30,4	36,3	42,6

Table 1.2 Ratio of Urban Population in the Year 2000 to that in 1970 (Unesco, 1979)

Europe	1,5	East Asia	2,7
North America	1,7	Latin America	3,1
U.S.S.R.	1,8	South Asia	3,3
Oceanic	1,9	Africa	4,2

1.1.2 Effects of urbanization on catchment hydrology

The combined effects of population growth, urbanization and industrialization change adversely the hydrological response of the affected area and cause various environmental impacts. The natural environment is changed to a man-made one, with concomitant quantitative and qualitative changes of the water cycle. Urban drainage is part of this cycle and is related to the urban hydrological system in a very complex way. Therefore, the planning of urban drainage systems should not only evaluate the linkages to other urban planning efforts, but it should also be integrated with the planning of other aspects of urban water resources, such as, water supply, wastewater treatment and use of the receiving waters.

When urbanization and industrialization reach a certain degree of development, changes of the environment become inevitable. Because water forms a substantial basis for life, it is necessary to deal with the mechanism of the hydrological cycle and the consequences of its changes. The changes imposed by urbanization may affect the hydrological cycle well beyond the boundaries of the urban area. Although such effects often are indirect, their consideration is pertinent. The implications of urbanization for urban water resources are also discussed in Chapter 1 of Volume II of this manual (Unesco, 1987).

Water needed in urban areas for domestic and industrial purposes vastly exceeds the amount needed in rural areas of comparable size. Intensive groundwater withdrawals for human needs may lead to land subsidence and saltwater intrusion in coastal areas. Natural water resources of urban areas are quickly depleted and it becomes necessary to import water from both adjacent and remote areas, thus affecting their water budgets. Consequently, large quantities of wastewater are discharged into rivers, lakes or coastal waters, thus transporting pollution to both adjacent and remote areas and endangering their ecosystems.

Urban land use itself has a great impact on water quantity and quality aspects of the hydrological regime. Basic aspects of this impact are discussed below.

Increase in the incidence of floods

Fast concentration of runoff on impervious surfaces and hydraulic improvements in the form of gutters, storm sewers and drains result in quickly-peaking high runoff rates. This tendency is supported by the straightening, deepening and lining of natural river beds and channels within urban areas. Rao et al. (1972) found that an increase in the area imperviousness from 0 to 40 per cent would approximately halve the time to peak discharge and increase its magnitude by 90 per cent. The increase in surface runoff from urban areas may cause local flooding as well as flooding in downstream rural or urban areas, thus causing threats to human life and wildlife, disrupting ecosystems and human activities, and damaging houses and other properties.

Reduction of base flows and groundwater recharge

The impregnation of the catchment surface by impervious elements such as rooftops, streets, sidewalks and parking lots, and intensive use of soils and their consolidation in pervious areas greatly reduce rainfall abstractions in urban areas. Such abstractions include interception by vegetation, depression storage and infiltration. This then results in an immediate surface runoff and reduction in groundwater recharge which in turn contributes to the lowering of groundwater tables. Furthermore, the water which does infiltrate is polluted and contributes to increased groundwater pollution. The changes of runoff processes caused by urbanization not only affect the water budget of the urban area, but they also affect water budgets of downstream areas. Progressing urbanization of river basins leads to diminishing stream base flows and thereby aggravates background pollution. Calculations for a catchment in California indicated that complete urbanization increased runoff volumes 2,3 times compared to the predevelopment state, while the stream base flow fell to 0,7 of its natural magnitude (James, 1965).

Increase in downstream pollution

All construction activities increase soil erosion and discharge of suspended solids into receiving waters. Keller (1962) observed that erosion of lands undergoing a transition from rural to urban use in the vicinity of Washington, D.C. increased the suspended sediment loads discharged into local streams by about 6,6 tons/km^2/year. Dallaire (1976) estimated erosion rates for disturbed urban areas as 28 tons/ha/year and 0,2 tons/ha/year for well-established urban areas. Furthermore, concentrated human activities lead to deposition of dust, dirt, and various pollutants on the catchment surface. Such materials are eventually washed off by storm runoff and contribute to the pollution of receiving waters. Sartor et al. (1974) found about 400 kg of total solids, 27 kg of COD, 0,3 kg of phosphate, and 0,6 kg of Kjeldahl nitrogen per one curb-kilometre of street surface. Other substances present in urban dust and dirt include heavy metals, pesticides and bacteria. The above accumulation rates are dependent on the degree of urbanization and local climatological conditions, especially precipitation. Increased flow velocities during storm runoff enhance the transport of suspended solids and pollutants from the catchment surface to receiving waters and lead to increased erosion and scouring of river and channel beds. Although many of the transported materials are of natural origin and relatively harmless when deposited on land, they become water pollutants in receiving waters and contribute to the degradation of water quality.

Large urban areas affect the local climate through increases in air temperatures, reduced humidity, higher incidence of fog, and increased precipitation. Air temperatures in urban areas are commonly higher than in their environs. This is caused by a much smaller latent heat flux in cities; different physical properties and structures of the urban or industrial area; the energy generated by house heating and air conditioning, factories and automobiles; and the pall of dust and carbon dioxide that reduces the amount of solar radiation and the net outgoing longwave radiation. The urban heat islands vary in size and temperature differential depending on their geographical location, size and other specific characteristics. The wind velocity is smaller in urban areas than in rural areas because of the obstacles caused by houses and buildings which change the natural flow and turbulence of the air. The humidity of the air is smaller in urban areas than in rural areas because the rainwater is quickly removed from impervious areas and drainage systems. The frequency of fog is higher in a polluted urban area than in the environs of the city, because of an increased amount of condensation nuclei. Finally, increased precipitation amounts have been observed on the downwind sides of large metropolitan areas (Unesco, 1980).

The above listed impacts of urbanization do not cover all aspects of the complex problems caused by urbanization. The intent was simply to provide some background information before proceeding with further discussion. Additional information on changes in the hydrological cycle caused by urbanization is given in Chapter 1 of Volume II (Unesco, 1987).

The conventional hydrology addressed hydrological processes in rural areas. Because many problems and solutions in urban areas are quite different, a new branch of hydrology, referred to as urban hydrology, has evolved. Urban hydrology deals with hydrological behaviour of urban catchments. It covers both natural processes, such as formation of runoff, and processes controlled by man's activities, such as water supply and wastewater disposal. One of the important problems addressed by urban hydrology is the effect of urbanization on catchment hydrological response and water quality. Many such adverse effects can be abated by properly planned, designed and operated urban drainage facilities as discussed in the following sections.

1.2 URBAN DRAINAGE

1.2.1 Need for urban drainage systems

In the context of this manual, urban areas are considered as large densely populated areas with a characteristic infrastructure. In some urban areas, the population density may be as low as 30 inhabitants per ha, while in other areas, as many as 300 or more people live on one hectare. Usually the population density in an urban area is not uniform, but may greatly vary. Besides public and office buildings, commercial and industrial zones, urban areas may also include playgrounds, parks and cemeteries.

Drainage serves for removal of excess water from an area by surface or subsurface means. Excess water in urban areas may be domestic and industrial wastewater or storm runoff. The need for urban drainage systems seems to be obvious considering the number of people living in urban areas and the effects of wastewaters on health or the threat of stormwater flooding. Appropriate disposal of wastewater and storm runoff contributes to human well-being and to the proper functioning of urban communities.

In some less developed countries, the methods of excreta disposal which the majority of people use are deposition on surface of ground in surrounding bush, the bucket latrine and the pit latrine. Only a very small percentage of people use flush toilets connected to a septic tank or a sewer system. Health hazards of depositing excreta on the surface of the ground in bush and open fields around houses are grave. The spread of diseases like typhoid, cholera, shigella and amoebic dysentries is enhanced, as shown in many publications (Moore et al., 1965; Schliessmann, 1959; Kourany and Vasquez, 1969; and Cvjetanovic, 1971). These diseases are still causing epidemics in many developing countries. Other worm diseases like ascariasis, hookworm, trichuris and schistosomiasis are also very common in communities where feces are disposed in an insanitary manner (Sanders and Watford, 1974; Schliessman et al., 1958).

Sanitary conditions in many developing countries may be visualized by considering the situation in Nigeria, a very urbanized country with a population of about 70 million. In 1975 there was not a single town or city with a central sewage system though there were many types of modern sewage treatment plants serving institutions, housing estates and army barracks. Even in main cities and towns, the majority of people did not have a water carriage toilet system. In smaller towns and rural areas where more than 80 per cent of the entire population lived, the flush toilet was almost totally absent (Oluwande, 1979).

Some countries also set their priorities in such an order that prestigeous projects which are "eye catching" and "vote winning" are undertaken in place of simple schemes which would benefit the majority of population. The old popular saying that "there are no votes in sewage" is still very appropriate in some developing countries, with the result that up to 90 per cent of the diseases which doctors see in hospitals and clinics are caused by poor, inadequate or nonexisting sanitary conditions (Oluwande, 1979). European cities such as London and Munich faced similar problems in the 19th century, when they were plagued by epidemics of cholera and typhoid. Obviously this had been caused by the lack of sewerage systems at that time.

In urban areas numerous cases of flooding caused by urbanization and resulting in loss of lives and immense damages were reported in almost all countries. Such problems emphasize the need for appropriate urban drainage which would provide for disposal of wastewater and storm runoff without increasing runoff peaks to a critical level at which downstream flooding would occur and without creating local flooding and health hazards within urban areas.

As a current example of drainage problems aggravated by flooding, drainage problems in a South Pacific island town are mentioned. Nuku'alofa, a town of 22000 inhabitants (in 1978), is located on the north coast of Tongatapu Island in the Kingdom of Tonga. During heavy rainfalls, heavy runoff is generated throughout the town and runoff drains into a valley in the centre of the town. Consequently, during and following rainy periods, parts of the town are flooded and the accumulated water often remains on the surface for days or weeks. There is no sewage collection network and properties rely on individual wastewater disposal facilities. Sanitation consists of septic tanks and pit latrines, and many properties have no latrine at all. When storm runoff passes through the town, it becomes polluted with overflow from septic tanks and pit latrines. As a result of such overflows, filariasis is endemic and there are also some occurrences of elephantiasis. Although filariasis can be cured, it is a serious disease which in this case is spread by surface water (Evans, 1979). Thus, improvements in surface water drainage would lead directly to improvements in health conditions.

Since urban runoff is a significant source of pollution, the design of urban drainage systems cannot be done any longer on a quantitative basis only, but must consider water quality aspects as well. Comprehensive planning integrated with planning for wastewater treatment is of utmost importance.

1.2.2 Drainage concepts and practices

Drainage problems in many areas of developing countries are similar to those illustrated by the above examples. On the other hand, there are relatively problem-free areas with flush toilets and appropriate storm drainage. In this context attention is drawn to equally effective low-cost sanitation measures, further explained in Chapter 2, which do not require water for transport of wastes. Such alternatives are especially valuable when water is scarce.

Traditionally, waste management was restricted to the technical, organizational, and economic aspects of waste handling. Thus, most of the activities and financial investments concentrated on the construction of treatment facilities. This so-called end-of-pipe approach was successful insofar as the environmental situation would have deteriorated even further without providing such treatment. However, evaluations of cost/benefit ratios were not carried out for alternative approaches to waste management (Wasmer, 1979). This has been especially true for urban drainage planning in relation to environmental design. Until now, the planning of urban drainage systems has been conducted by means of less sophisticated methods than the planning at the watershed level. For most drainage projects, the rational method has been applied, although its application to larger areas may be inadequate and may contribute to flooding of downstream areas (see Chapter 5).

The planning of urban drainage systems which traditionally focussed on individual aspects cannot be generally beneficial for the system on the whole and it may even result in damages exceeding the originally perceived benefits. For example, the lack of solid waste disposal facilities may lead to the conditions shown in Figures 1.1 and 1.2. Especially when open channels are used for wastewater drainage, inhabitants are inclined to dispose solid wastes in such channels. This contributes to health hazards, particularly during flooding, and further deteriorates the urban environmental conditions. With storm runoff, wastes are flushed into the receiving waters, thus deteriorating their quality and limiting their use. Obviously, a successful operation of an open drainage system also depends on providing an effective solid waste disposal system.

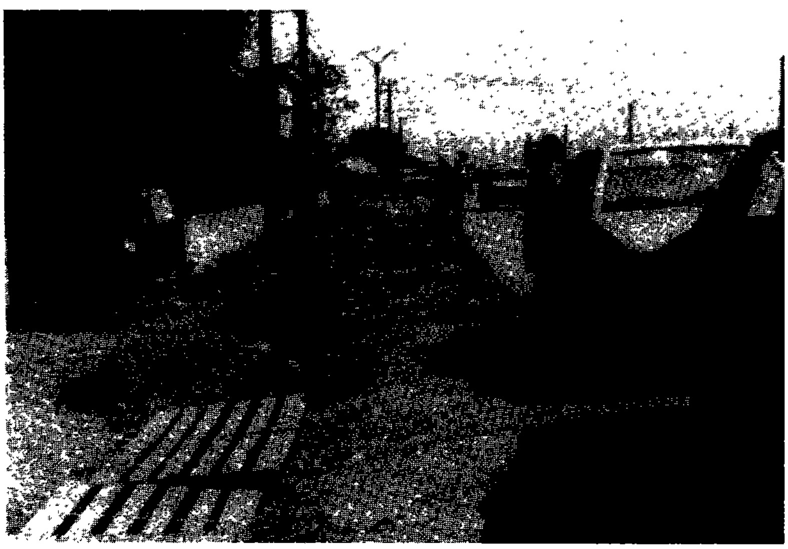


Figure 1.1 Road-side ditch in Douala (Cameroon)

Furthermore, the proper maintenance and cleanliness of the entire urban area can have a significant impact on the quantity of pollutants washed off by stormwater. Cleanliness of an urban area starts with control of litter, debris, de-icing agents, and agricultural chemicals, such as pesticides and fertilizers. Regular street repair can further minimize the quantity of pollutants picked up by stormwater runoff. Street sweeping, incidentally, removes only the coarse material and its effectiveness depends largely on the methods applied. The proper use and maintenance of both catch basins and the collection system can maximize control of pollutants by directing them to treatment or disposal facilities.

There are two basic types of urban drainage system:

- Separate systems conveying separately domestic/industrial wastewaters and stormwater, thus allowing for good handling of domestic wastewaters with low-cost sanitation schemes, and
- combined systems conveying domestic and industrial wastewaters and stormwater together in a single system of pipes and channels.

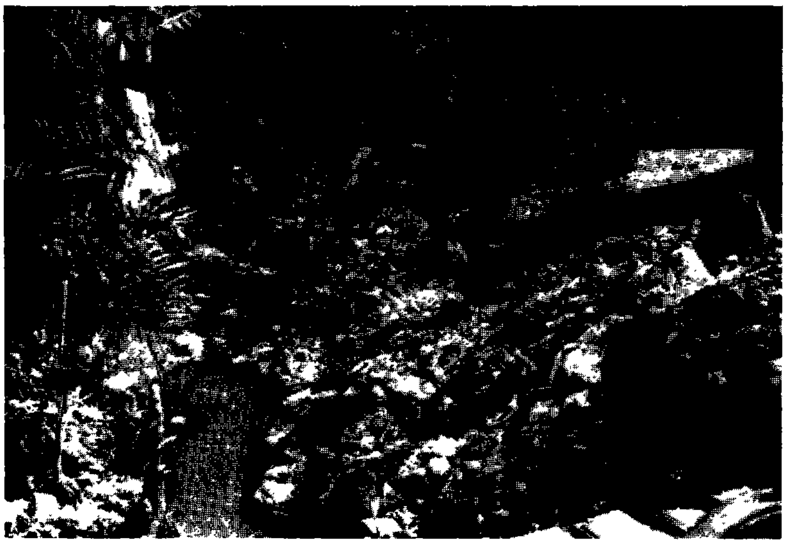


Figure 1.2 Illicit solid waste disposal in an urban drainage channel in Douala (Cameroon)

Advantages, disadvantages and modifications of these two basic types are discussed in Chapter 2.

In Europe and North America, urban drainage practice is in a transition stage. Although traditional methods continue to be applied on a large scale, there are many attempts to apply new concepts such as various types of runoff storage, runoff control and standard analysis of sewage systems on the whole, including treatment operations and effluent criteria. New simulation methods are also increasingly applied instead of the rational method (see Chapters 5 and 8).

In the United Kingdom, for instance, it has been estimated that in 1979/80 prices some 100 million U.S. dollars are spent every year by public sector authorities on the removal of stormwater by urban drainage systems. This amount excludes any considerations of the removal and treatment of sanitary sewage, and also excludes a large expenditure in the private sector on the drainage of small estates and other private developments (Colyer, 1982). The magnitude of such expenditures emphasizes the need for efficient basin-wide planning to prevent piecemeal drainage construction which at a later date may be found incompatible.

1.2.3 Urban drainage within the urban system

The previous sections showed that urban drainage systems are not self-contained, but are influenced by and do affect other aspects of urban planning. There are indeed strong interactions between urban and industrial development and water resources planning. Urban water resources planning deals with preparation and weighting of alternatives on the basis of technical requirements as well as socio-economic and environmental constraints. The former includes urban drainage, flood control, water reuse, and water supply, and the latter comprises living conditions, industrial development and quality of life. Figure 1.3 shows a simplified diagram of the urban planning process showing the interrelation of water resources investigations and urban planning.

The socio-economic considerations of urban drainage systems have been addressed by Unesco (Lindh, 1979). The basic conclusions of this report are summarized below.

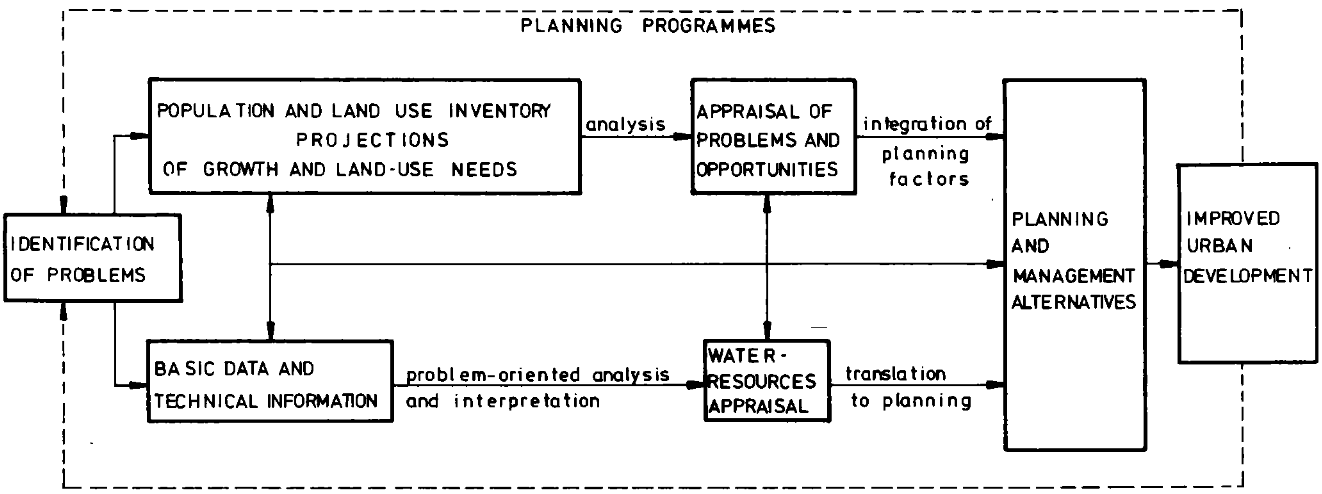


Figure 1.3 Simplified flow diagram of the urban planning process showing the interrelation of water resources investigations and urban planning (after Schneider et al., 1973).

The main benefits from sewerage schemes, such as the alleviation of health hazards, reduced inconvenience and improved aesthetic conditions, are mostly intangible. Fundamentally, it is the service itself that is desirable. This service does have a value, but the value cannot be determined by using the traditional cost-benefit analysis and is usually set arbitrarily by, or on behalf of, the community in the form of normative criteria.

Wastewater in sewers consists of water which has been supplied and passed through a polluting process. Thus, where a cost-recovery basis is adopted for financing the works, the pricing structure for a sewer service often imitates that for water service. On the other hand, in many countries sewer services are not separately levied. Moreover, the number of water users without access to public sewer systems is very large in certain regions. In order to reduce potential health hazards, it is necessary that the provision, or extension, of sewer services to high-density, low-income urban areas should have a substantially high social priority.

There is a two-directional linkage between urban fringe growth and availability of sewers. For example, in some North American cities, wastewater sewers (and water mains) are not approved for construction when they do not satisfy growth policy. On the other hand, availability of such services is recognized as a stimulus to growth.

Furthermore, various aspects of resource conservation, pollution control and environment cannot be considered out of context. The complex interactions within the materials cycle, the direct correlation between energy and minerals production, the effects on industry and the consumer, and the interrelationships of national and international policies all need to be taken into account for the formulation of objectives (Wasmer, 1979).

1.3 DESIGN OF URBAN DRAINAGE SYSTEMS: OBJECTIVES AND TOOLS

1.3.1 Planning objectives

Planning is the process of deciding what resources to allocate, over time and space, to achieve a set of specified objectives (McPherson, 1978). Planning and design of drainage systems for urbanized areas may be done for a number of objectives, such as

- health and hygienic reasons,
- prevention of flooding,
- improvement of living conditions and human well-being,
- rehabilitation of urban areas,
- protection of environment, and
- aesthetic reasons.

However, it has to be checked if urban drainage is the right and best tool to achieve these objectives. Sometimes this might require a combined effort linking water supply, urban drainage and solid waste disposal schemes.

Sizing a drainage system for domestic and industrial wastewaters involves estimates of population growth, commercial or industrial development, and water consumption. Designing a storm or combined sewer system requires reliable data on rainfall characteristics, surface/soil characteristics, and adequate methods for runoff computations.

Computerization of calculation methods outlined in Chapter 8 makes it possible to investigate various planning alternatives. To take full advantage of this possibility, various levels of planning have been suggested in order to limit the planning costs (U.S.-EPA, 1974; U.S.-EPA, 1977). Such planning levels are briefly characterized below.

- Providing a first estimate of the magnitude of urban runoff quantity and quality problems, prior to investment of time into more detailed planning.
- Providing an overall assessment of urban runoff problems and estimates of the effectiveness and costs of different abatement procedures. Trade-off among various control options has to be demonstrated on a macroscale prior to the decision for the design of a specific sewer system layout.
- Designing the overall system and sizing the individual components of the chosen layout of the drainage system.
- Providing the final layout of the individual components of the drainage system and providing the necessary drawings for construction.
- Planning of operational measures and controls for the implemented system.

1.3.2 Comprehensive planning

Until recently, urban storm drainage design served only to devise measures to protect urban development from stormwater. It usually consisted of evaluating the peak runoff rate and designing a network of sewers and ditches to collect and convey the stormwater downstream, just away from the urbanized area. Stream flood plains and sewered drainage systems were considered separately, and urban runoff was regarded as merely an adjunct consideration in land use planning. The urban development and industrialization of recent years yielded new public demands and policies which require the use of true comprehensive planning including integrated land and water management. Greater emphasis has to be placed upon the total impact of projects and the overall optimal solution as opposed to optimizing individual devices for presumed sets of criteria.

A review of a number of local government planning projects in the United States has revealed a commonality of six discrete steps, which might be regarded as the framework of the standard planning process (McPherson, 1978):

- Define problems, establish goals and identify constraints,
- assemble data and refine definitions of problems,
- formulate alternatives,
- analyze alternatives,
- identify more promising alternatives, and
- evaluate the trade-offs among selected alternatives.

Embracing the above six parts of the basic planning process, Figure 1.4 is a block diagram depicting the urban runoff control planning concept, which attempts to accommodate different situations of land already developed and projected for development, considering also the impact of urban drainage on receiving waters.

Comprehensive planning is done for integrated management including flood protection, pollution abatement, erosion control, and water conservation and is usually conducted by metropolitan planning agencies. So-called master drainage plans are comprehensive in scope, but locally confined and prepared by local agencies. Chapter 9 elaborates on these matters and provides examples.

The lack of hydrological data and institutional shortcomings are found to be the major handicap to promotion, successful planning and efficient operation of all water development projects; in most cases failure is caused by the lack of proper water development authorities, conflicts between multiple agencies having divided authority and working under conflicting policies, and the absence of up-to-date water legislation. The institutional background for study, development and implementation of urban drainage is discussed in Chapter 9.

1.3.3 Use of models in planning

All but a small fraction of storm sewers in the world have been sized by means of fully empirical methods. Given the lack of evidence of superior methods, these overly simplistic procedures proved adequate when the primary purpose of storm sewers was to drain the land and transport quickly surface runoff to receiving waters. Out of sight, out of mind. Once restraint or containment of flows and their pollutant burdens become added primary objectives, traditional procedures of analysis are no longer adequate because of added system complexities for which conventional tools are unsuited (McPherson, 1978).

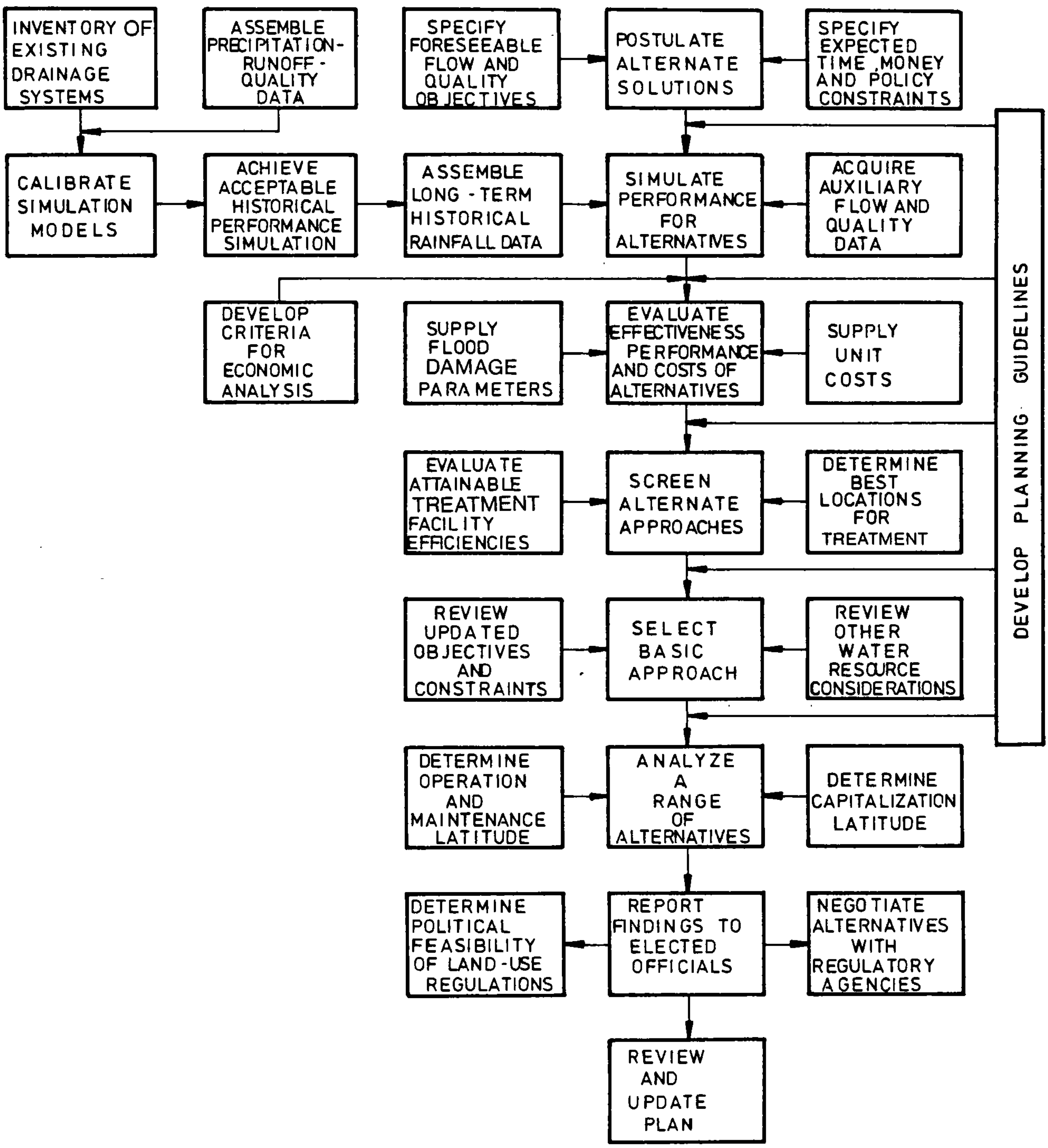


Figure 1.4 Block diagram of urban runoff control master planning concept (McPherson, 1978)

Along with the growing understanding of the complexity of optimal planning of urban drainage systems, many nations have reported an obvious recent change in design of urban drainage systems by an increased use of mathematical runoff models and computerized methods (Unesco, 1977; Unesco, 1978; McPherson, 1979). A runoff model is defined here as a computational procedure which mathematically simulates runoff processes in time and space. To choose the right calculation procedure for a given design problem has become quite difficult. For larger and more complex systems, application of a simulation model is a necessity, especially because the climbing costs of drainage construction and the increasing awareness of the environmental impacts of runoff require a broader and more careful approach for drainage facility development. While mathematical simulation is undoubtedly an effective means for analyzing alternative urban runoff control strategies, its most important use can be in the assessment of expected systems performance (McPherson, 1978).

Larger cities usually have sufficient resources to operate their own planning groups and eventually to develop their own methodologies for the planning of their urban drainage systems. Medium size communities usually can afford only a limited amount of planning effort and do not have the resources to develop their own expertise and methodologies. To assist these numerous medium size communities, it is desirable to have a number of models for the planning and design of drainage systems that are not specific to a particular location but are "transferrable". This class of models comprises urban runoff models which are generally applicable to a large number of communities in a broad geographical area. Examples of such models are given in Chapter 8.

Emphasized is that simulation techniques adopted should not exceed the level of mastery of such tools by the user and that tools should be selected on the basis of their suitability for solving defined problems (McPherson, 1978). Models do not make decisions but provide qualified quantitative information to serve as one input into the decision-making process. To ensure the validity of planning models, their verification against local field data or data from catchments with similar geographical, climatological and hydrological characteristics is recommended. Some urban catchments which served for the collection of field data in numerous countries are listed in Appendix A.

1.4 STRUCTURE OF THE MANUAL

The manual consists of two volumes. Volume I that follows deals with the planning and design of urban drainage systems and Volume II (Unesco, 1987) deals with data collection and analysis for drainage design. Each chapter of both manuals not only provides general information on the subject matter, but it also discusses practical applications. For example, for calculation formulae, the sources of input data are indicated together with the practical range of values of various parameters, or their default values. As far as practical, all basic information was included in the manual. Whenever necessary, references to publications with additional detailed information are made. An international list of urban test catchments is provided in the Appendix. This list includes basic characteristics of the catchments documented and the sources of collected data. As detailed below, Volume I is divided into nine chapters and three appendices.

Chapter 1, Introduction, gives some background information on the effects of urbanization on the hydrological cycle, explains the need for drainage systems in urbanized areas, shows the role of drainage in catchment hydrology, and discusses the need for comprehensive planning.

Chapter 2, Basic approaches to urban drainage, provides the information necessary for the selection of a suitable drainage concept. In this context, various drainage concepts, their construction and operational costs, and their effects on the human well-being and the environment are discussed.

Chapter 3, Elements of drainage systems, briefly describes common structural elements of drainage systems. It also provides examples of typical layouts of various drainage structures.

Chapter 4, Design parameters, discusses various design parameters for urban drainage. Design procedures incorporating step-by-step planning, regular updating, and considerations of possible changes in the planning horizon are discussed. For individual design parameters, data sources and links to data collection described in Volume II are given.

Chapter 5, Quantity of stormwater, explains the basic principles of various methods for calculation of storm runoff, including the input data requirements. It also describes the calculation procedures necessary for drainage network design.

Chapter 6, Hydraulics of conduits and open channels, presents the basic principles for hydraulic design of conduits and open channels.

Chapter 7, Design of components of drainage systems, describes the basic hydraulics of special drainage structures and various procedures of their sizing.

Chapter 8, Network design methods, describes the methods of network design, which are largely based on the information given in Chapters 4 to 7. Simple common methods are discussed in detail. For comprehensive models, frequent references to the literature are made. Limitations of various network design methods are also discussed.

Chapter 9, Organization and administration of urban drainge projects, presents economic, financial, institutional and managerial considerations and their interdependencies in design of drainage systems in urbanized areas.

Volume I further contains three appendices titled Summary of urban hydrological test catchments, Rational method calculations for the example given in Figure 7.1, and Rational method calculations for the example given in Figure 7.2.

Volume II is divided into eight chapters and three appendices.

Chapter 1, Needs and objectives of urban drainage, contains general information on the development of urbanization in the world and its effects on the hydrological regime and water quality. Special attention is given to data collection needs for urban drainage design.

Chapter 2, Meteorological data, provides the information necessary for collecting meteorological data. The principles of organizing a network of observation stations, measuring equipment, and data processing and verification are described.

Chapter 3, Hydrological data with emphasis on runoff, identifies the hydrological parameters to be measured in urban drainage studies. Recommendations for the selection of test catchments, data processing, and data quality control are given.

Chapter 4, Ancillary data, lists the ancillary data also required in the design of urban drainage systems and the means of obtaining such data. The data discussed include physiographic characteristics of the area, land use, and demographic data.

Chapter 5, Special problems of water quality analysis, describes the problems encountered in the determination of water quality. Recommendations are given for the selection of water quality parameters to be studied, and the types of laboratory analyses and equipment to be used.

Chapter 6, Data handling, recommends procedures for data handling, storage, and quality control.

Chapter 7, Statistical analysis of data, presents statistical analyses of data which are used in hydrological applications.

Chapter 8, Organization and administration of data collection programs, discusses the importance of a good organization of urban data collection programs. Various aspects of financing and catchment instrumentation are also discussed.

Volume II further contains three appendices entitled Operation and maintenance of a standard non-recording rain gauge, Operation and maintenance of a pluviograph with a 0,04 m^2 collector, and Operation and maintenance of a ground-level rain gauge.

2 Basic approaches to urban drainage

2.1 URBAN WASTES AND STORM RUNOFF

2.1.1 Sanitation of urban areas

Sanitation of urban areas comprises disposal of solid waste, excreta and sullage, and storm drainage flows. While the "production" of waste depends mainly on the standard of living, population density, traditional ways of life, and the level of water-supply services, storm runoff depends mostly on climatological, meteorological, and geological conditions. Consequently, sanitation problems cannot be solved by a general standard solution. Instead, individual approaches, taking into account local conditions, have to be developed for sanitation of specific urban areas.

In industrialized countries, a frequent solution for sanitary disposal of human excreta is the waterborne sewerage. In this context, users and operators of such systems often consider the flush toilet as the absolutely essential part of any adequate solution to the problem of excreta disposal. This approach, however, has been developed to maximize users' convenience rather than health benefits. While such an objective may be important in developed countries, it has much lower priority in developing countries (Kalbermatten et al., 1982). Waterborne sewerage has evolved over decades and centuries from the pit latrine to the flush toilet and the present standard of convenience has been achieved at substantial economic and environmental costs.

In developing countries high expectations often are contradicated by limited resources. Decision makers in these countries are asked to achieve the standards of convenience seen in the highly industrialized countries. Given the backlog in providing various services, the massive size of required sewerage investments, and the demands on financial resources by other sectors, they do not have the funds to realize this goal. Sewerage could be provided for relatively few,but at the expense of the vast majority of the population. As a consequence, many developing countries have taken no steps at all towards improving sanitation. The very magnitude of the task has effectively discouraged any action (Kalbermatten et al., 1982.).

At present, the first priority of excreta disposal programmes for urban areas in developing countries should be the protection of human health by accomplishing a significant reduction in the transmission of excreta-related diseases. This health objective can be achieved by sanitation technologies that are much less costly than waterborne sewerage. Some flooding and health hazards, however, may be caused by inadequate solid waste disposal. Figures 1.1 and 1.2 in Chapter 1 show situations where storm runoff may inundate urban areas and spread diseases as a result of solid waste deposits. Thus, the general goal of water management in urban areas should be to provide as many people as possible with safe potable water and sanitation using technologies which can achieve these objectives with the resources available.

2.1.2 Development and transport of storm runoff

To control storm runoff and to provide adequate storm drainage, it is necessary to recognize and understand various facets of the runoff process. Storm runoff is caused by rainfall and, therefore, occurs randomly. When the same amount of rain falls on two different areas, the resulting runoff may be different because of different hydrologic and meteorologic conditions, surface covers, and layouts of the drainage system. Temperature, humidity and wind determine the amount of evaporation from wetted surfaces. Voids and depressions in the surface are filled and emptied according to the permeability of the top layer of the ground. Only after the infiltration capacities have been exceeded and the depressions filled, runoff starts with water following the surface slopes. Water may flow in sheets (layers) or in small rivulets. It then collects in ditches and channels draining to creeks, rivers or impoundments. If the capacities of the collecting system are exceeded or the pathways are blocked (i.e., by solid wastes), smaller or larger areas may become flooded. Runoff flow paths, discharges and composition on the catchment surface continuously change, partly as a result of ongoing deposition and wash-off of matters during the runoff process. In the case of snow-covered catchments, the runoff process is delayed until snowmelt takes place. If the catchment is drained by a sewer system, runoff may be accelerated with concomitant increases in peak flows.

The physical mechanisms of the runoff processes on the catchment and surface transport in ditches, channels or sewer systems are linked, especially when backwater conditions occur. Although runoff is a continuous unsteady process involving and linking many parts of an urban area, for calculation of flows this process often is subdivided into the phases of runoff generation, flow concentration, and runoff transport in the system of channels or sewers.

There are two basic types of sewerage systems. In the combined sewer system, all wastewaters and storm runoff are collected and transported together in common combined sewers. In the separate system, two separate sewer pipe subsystems are employed. Sanitary sewers transport wastewaters and storm sewers transport storm runoff. More details on both sewerage systems and their variants are presented in Section 2.2.

While most of the existing urban drainage systems comprise overland flow systems and a transport network of channels and sewers, which rapidly convey the flow to the receiving waters, future urban drainage systems will increasingly utilize such concepts as enhanced infiltration of stormwater into the ground, surface and subsurface flow detention using the sewer network storage capacity, the prolongation of the time of concentration by increasing the travelling time of the water to the inlets of the collecting system, provision and definition of flooding areas, and storage and use of stormwater for industrial or irrigation purposes. Reuse of urban storm runoff as drinking water is rarely feasible because of its polluted character and need for costly treatment. Some of the above listed new runoff management concepts are further described in Chapter 3.

2.1.3 Runoff quality

Numerous studies of stormwater runoff composition demonstrated that runoff may carry relatively high loads of such pollutants as solids, substances exerting oxygen demand, toxic substances, bacteria, and nutrients (Torno et al., 1986). In spite of this fact, it has been rather difficult to evaluate urban runoff as a source of pollution, because such an evaluation inherently depends on the type of pollutant, loadings carried by runoff in relation to those from other sources, and the receiving waters. With regard to the receiving waters, both their self-purification capacity and water use are important. If the level of pollutant loadings discharged into the receiving waters exceeds their self-purification potential, the receiving water ecosystem is endangered.

Stormwater quality may be described by such conventional parameters as total suspended solids, settleable solids, biochemical oxygen demand (BOD), chemical oxygen demand (COD), total organic carbon (TOC), total nitrogen, total phosphorus, and indicator bacteria. The significance of these parameters is briefly explained below. For details of analyses of such parameters, reference is made to Chapter 5 of Volume II of this manual (Unesco, 1987).

Stormwater carries large quantities of both inorganic and organic solids in either particulate or colloidal form. From the environmental point of view, the suspended solids fraction is particuarly important because of induced turbidity, pollutant adsorption, substrate smothering, and benthal accumulation. To estimate the fraction of the suspended load consisting of organic matter, the volatile fraction of total suspended solids is determined. Where gravity separation of solids is of interest, settleable solids are also determined (Alley, 1977).

Primary organic indicators include BOD, COD, and TOC. The aforementioned tests are not substitutes, but rather independent measurements. BOD is a measure of dissolved oxygen depletion by biological and chemical reactions over a standard time period, such as five days. In the case of stormwater, BOD values may be erratic because of inhibitory effects of commonly present toxic metals. Chemical oxygen demand is a measure of inorganic and organic oxidizable materials, including materials which cannot be utilized by microorganisms. Total, dissolved, and/or suspended carbon measurements are sometimes used in lieu of BOD and COD. Such tests are well applicable to small concentrations of organic matter and, like COD, indicate more than just the easily-degradable organic load (Alley, 1977).

Both nitrogen (N) and phosphorus (P) are measured in stormwater, in order to evaluate its impact on the eutrophication of lakes and impoundments and on the productivity of rivers and estuaries. For such purposes, it is desirable to measure various forms of N and P as described in Chapter 5 of Volume II (Unesco, 1987).

Because pathogens in stormwater and overflows are of numerous types, few in number per type, and difficult to isolate, indicator organisms are used instead (Alley, 1977). Typically, the total coliform, or its portion associated with the faeces of warm-blooded animals, fecal coliform, are studied.

Besides the general parameters, storm runoff quality is also described by concentrations of various toxic substances. Heavy metals, such as lead, zinc, càdmium, mercury, copper, nickel and chromium are of primary interest. Recent studies indicate that many organic priority pollutants are also present in storm runoff in significant quantities (Marsalek, 1986).

The sources of pollutants in storm runoff are quite numerous. Pollutants enter into runoff through the scavenging of the atmosphere by rain and through wash-off of materials deposited on the catchment surface. The scavenging process may be particularly important in industrial areas with high air pollution. The wash-off of materials by runoff is strongly affected by the amount of accumulated material. Such accumulations are in turn affected by the cleanliness of the area and methods of solid waste disposal. Water running off the urban catchment surface washes off the accumulated dust and dirt, deposits from automobile exhausts, oil and other materials arising from operation of automobiles, and all other materials of anthropogenic origin. For example, waste paper, cigarette butts, and fruit and vegetable wastes contribute to organic matter loads in stormwater. Animal and human excreta add to the organic loads and also contribute to the bacteriological pollution of runoff. Traffic by-products and industrial operations are the main sources of toxic substances. Such substances may be also produced by decomposition of other materials. Finally, soil erosion on bare soil surfaces is a primary source of solids in stormwater.

Depending on runoff flow discharges and velocities, the materials deposited on the catchment surfaces may be picked up and transported in suspended or dissolved form. In this process, some particles are transported continuously through the drainage system, others are transported over a limited distance. Therefore, the transport of pollutants through urban drainage systems is a highly variable process which depends on many factors. It has to be recognized, however, that the pollutional loads in stormwater are influenced by urban population. This is further illustrated by examples given in Figures 1.1 and 1.2 in Chapter 1.

As a result of random occurrence of storms and widely varying accumulation of polluting materials on the catchment surface and in the transport network, storm runoff pollution is highly variable and strongly dependent on local conditions. Table 2.1 gives concentrations of various pollution parameters measured in both separate and combined sewer systems in some industrialized countries. The techniques, methodologies, and goals varied from study to study, but the tabled results present a good indication of concentrations of pollutants that can be expected in urban runoff. These data were collected in various catchments with diverse land use, during different annual seasons, and during various types of rainfall events. The ranges of pollutant concentrations shown in Table 2.1 indicate wide variations in concentrations. The data presented were derived mainly from European and North American studies and may be even exceeded under some conditions. It should be emphasized that stormwater sometimes creates pollutional problems only if it collects pollutants during transport or if it is mixed with domestic, commercial or industrial sewage.

Table 2.1 Ranges of Pollutant Concentrations in Combined Sewer Overflows (CSO's) and Stormwater Runoff

Land Use	Residential		Commercial		Industrial	
Source Units	CSO's mg/L	Stormwater mg/L	CSO's mg/L	Stormwater mg/L	CSO's mg/L	Stormwater mg/L
BOD_5	2 - 600 (23 - 114)	1 - 145 (8 - 55)	4 - 600 (46 - 95)	0,5 - 173 (17 - 38)	82 - 685 (86 - 153)	0,5 - 88 (9 - 28)
COD	33 - 1762 (138 - 209)	4 - 1740 (28 - 213)	41 - 626 (138 - 145)	3 - 610 (46 - 170)		0,7 - 605 (86 - 343)
Settleable Solids	0,1 - 656 (165 - 238)	0,1 - 4500 (50 - 435)		0,1 - 440 (76 - 160)		0,1 - 1270 (151 - 374)
Total Suspended Solids	24 - 1260 (177 - 271)	1 - 12000 (28 - 736)	20 - 1800 (90 - 391)	1 - 4803 (56 - 275)	124 - 1000 274 - 637)	1 - 11900 (114 - 1220)

First line = minimum and maximum values of data from all studies.
Second line (in parentheses) = range of average concentrations found in different studies.

While the domestic sewage containing excreta and sullage is polluted mostly by organic matter, commercial and industrial wastewaters may convey high inorganic pollutant loads depending on their origin. The composition of domestic, industrial and commercial wastewaters can be determined relatively easily by sampling during the periods of dry weather. The pollutant loads carried by such wastewaters contribute to loads in combined sewer overflows as further discussed in Chapter 4. Attention is drawn to Appendix A which lists urban test catchments where runoff quality has been monitored.

2.2 APPROACHES TO URBAN DRAINAGE

2.2.1 Options for excreta and sullage disposal

Excreta disposal in European and North American countries went through many stages of development before it reached the current level of waterborne sewerage. Improvements were implemented over a long period of time, whenever the old system was no longer satisfactory and funds for improvements became available. Sewerage was not a grand design implemented in one giant step, but rather the end result of a long series of measures with progressively increasing technological sophistication. For example, the collection of night soil from bucket latrines in London in the eighteenth century was a step toward reducing the overall urban pollution. This was followed by piped water supplies, the development of combined sewerage, then separate sanitary sewerage, and eventually sewage treatment prior to discharge into the river. This particular series of improvements spanned over 100 years, a time frame necessitated by historical constraints in science, technology, and resources. The present level of knowledge of sanitation techniques makes it possible for planners to select from a wide range of sanitation options and to design a sequence of incremental sanitation improvements (Kalbermatten et al., 1982).

The population pressures and the requirements for improved health in developing countries will require a much more rapid and extensive implementation of effective waste disposal than that which took place in European and North American countries in the past. It is unlikely that the systems that were appropriate for rather small populations of the relatively wealthy countries of Northern Europe in the 19th century will be successfull in solving the formidable problems of less affluent and more populous developing countries (Rybaczynski et al., 1982). A World Health Organization survey (WHO, 1976) indicates that while in 1970, 27% of the urban population in developing countries had sewerage connections, by 1975 this fraction had actually declined to 25%. Even more serious is the fact that another 25% of urban population had no access to sanitary facilities at all.

It is recognized that the problems of excreta and sullage disposal and the storm drainage problems should not be solved by a single common approach applied in various climatic regions. For disposal of excreta and sullage, Kalbermatten et al. (1982) and Mara (1982) offer a variety of approaches shown in Table 2.2 The selection of the most appropriate approach is affected, among other factors, by the water supply service and water consumption.

Table 2.2 Water Supply Service Levels and Associated Options for Excreta and Sullage Disposal in Urban Areas (Mara, 1982)

Water Supply Service Level	Typical Water Consumption L/cap./day	Options for Extreta Disposal[1]	Options for Sullage Disposal[1]
Stand-pipes	20 - 40[2]	Pit latrines Pour-flush toilets[3] Vault toilets	Soakage pits
Yard taps	50 - 100	Pit latrines Pour-flush toilets[3] Vault toilets Sewered pour-flush toilets Septic tanks	Soakage pits Stormwater drains Sewered pour-flush toilets Septic tanks
Multiple tap in-house connections	>100	Sewered pour-flush toilets Septic tanks Conventional sewerage	Sewered pour-flush toilets Septic tanks Conventional sewerage

[1] The options are not listed in any order of preference.
[2] Consumption depends on stand-pipe density.
[3] Feasible only if sufficient water carried home for flushing.

2.2.2 Storm drainage by separate and combined systems

Although the primary objectives in developing countries are water supply and domestic sanitation, these aspects are closely linked to stormwater drainage in urban areas as indicated in Chapter 1. Short-circuiting between water supplies and urban drainage discharges has to be avoided.

There are several different concepts for urban drainage:

- Storm drainage by ditches and channels discharging to the nearest water course and on-site and off-site sanitation.
- Separate sewer systems, conveying separately domestic and industrial wastewaters and stormwater; and,
- Combined sewer systems which convey domestic and industrial wastewater and stormwater in the same pipes or channels.

The discharge of stormwater into the nearest water course and on-site or off-site sanitation, if applied adequately, may represent an acceptable method of runoff and wastewater disposal in some urban areas with lower population densitites.

Another approach to the drainage of urban areas is through the use of the separate sewer system which comprises storm and sanitary sewers. Storm sewers convey stormwater discharges directly into receiving waters and sanitary sewers carry sanitary sewage to the sewage treatment plant. Disadvantages of this system include the costs of providing and maintaining two parallel sewer systems and pollution of receiving waters by untreated storm runoff.

A sewer system which drains together domestic wastewater, industrial wastewater and storm runoff is called a combined sewer system. Thus, combined sewage is a mixture of dry weather flow (i.e., domestic and industrial wastewaters) and storm runoff. All dry weather flows or, during wet weather, combined flows, should be transported to a sewage treatment plant for treatment. During heavy rain storms, however, the combined sewage inflow into the system exceeds its capacity and that of the treatment facility. Therefore, the excess flow has to be allowed to escape the collection system and be discharged to the receiving waters as combined sewer overflows. Such overflows are polluted and the overflow pollutant loadings to the receiving waters comprise pollutants from domestic sewage, industrial wastewaters, surface runoff and materials scoured from depositions in sewers. When such pollutants are discharged directly into the receiving water body, its water quality may be severely downgraded. Direct discharges of surface runoff into the receiving waters also reduce the recharge of groundwater aquifers. On the other hand, blockages in combined sewer systems may cause flooding and, because of the polluted character of such flows, they represent a real health hazard to the population. Figures 2.1 and 2.2 show basic layouts of separate and combined sewer systems. The separate system with two underground pipe systems requires much more resources for construction and operation. Therefore, it may be desirable to place only the sanitary sewers underground and operate the storm drainage system as a system of open channels (see Section 2.2.3). Such an arrangement may be particularly acceptable in urban areas with low housing densities.

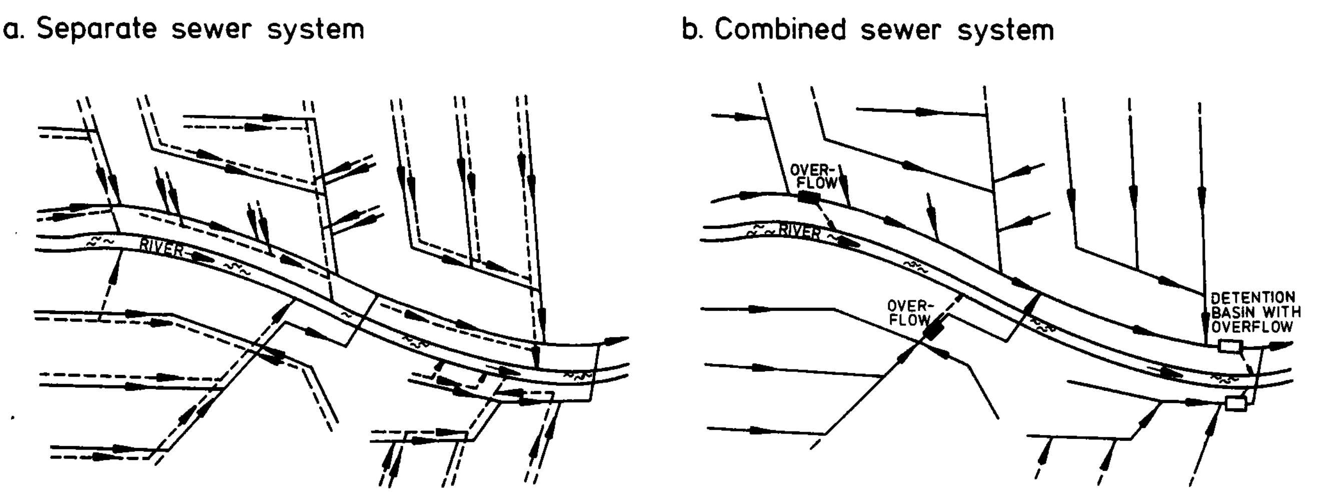
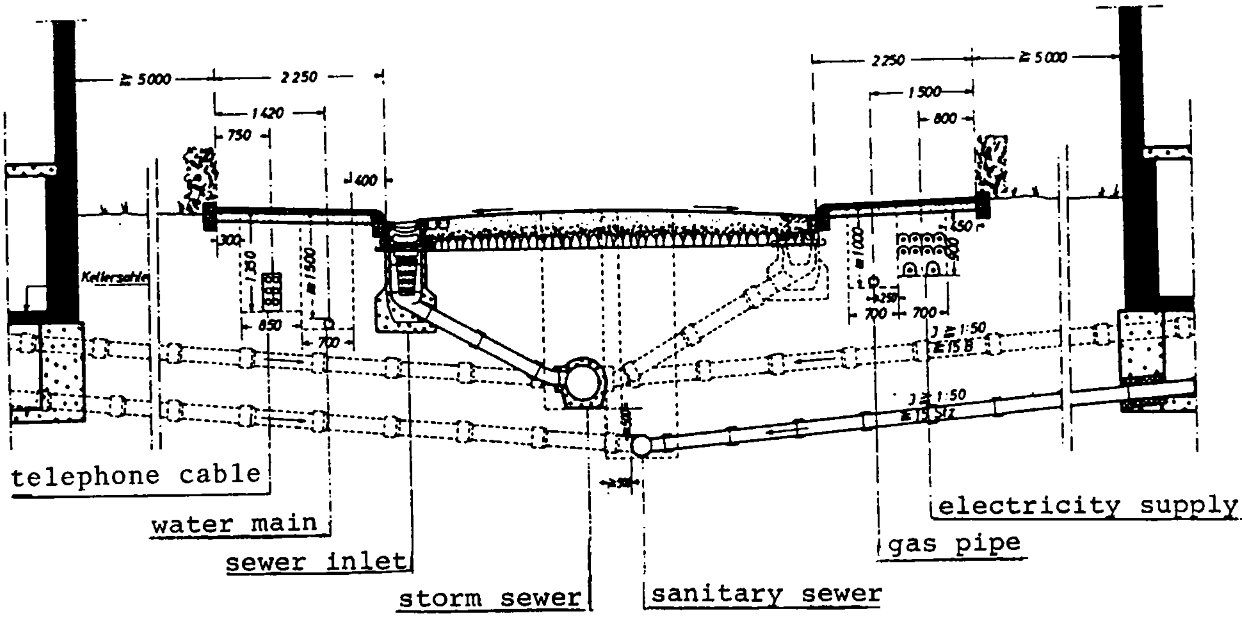


Fig. 2.1 Plan view of separate and combined sewer systems

a. Separate sewer system with underground storm sewers



b. Combined sewer system

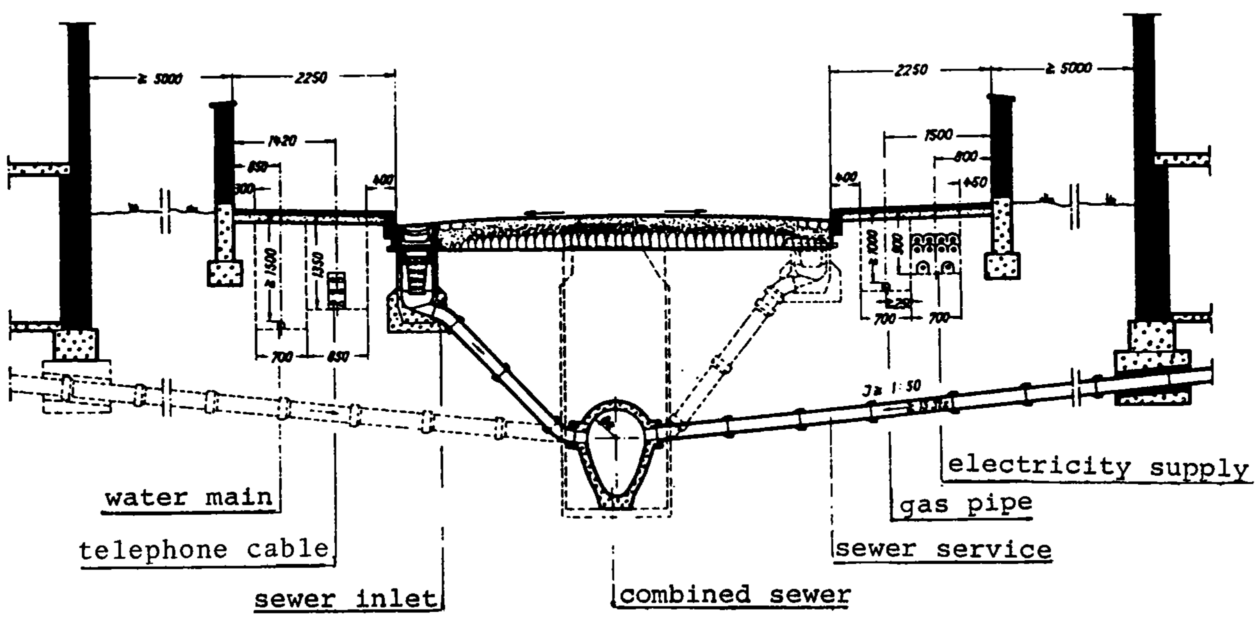


Fig. 2.2 Cross-sections of separate and combined sewer connections (after Bischofsberger, 1982)

There are, of course, many other variations of the fully separate or combined systems, some of which are further discussed in Section 2.2.4. In principal, any sewer system comprises interconnected components which serve for:

- drainage of buildings, properties and public grounds utilizing detention measures, such as surface or roof-top ponding,
- collection and transport of stormwater in sewers or channels utilizing in-line and off-line storage,

- treatment in sewage treatment plants and settling tanks, and
- discharge of effluents to receiving waters.

The first two components represent the urban drainage systems aiming to remove wastewater and storm runoff from built-up areas in such a way that flooding and pollution of receiving waters can be prevented.

Advantages and disadvantages of combined and separate systems are still under discussion. Although combined systems dominated the sewerage practice in the past, the separate systems became fairly popular during the last thirty years. In spite of this popularity, the effectiveness of separate systems without stormwater treatment for pollution control is questioned. In many countries, the distinction between combined and separate systems is rather academic because the existing networks, combined or separate, are generally used for disposal of all kinds of urban wastes, particularly in the case of open channels. Furthermore, climatic conditions are often unfavourable for successful operation of sewer networks. Very long dry weather periods lead to consolidation of depositions which prevents self-cleansing of sewers during wet-weather periods. Under such circumstances, even separate storm sewers may cause similar pollution problems as combined systems do.

2.2.3 Open channels and closed conduits

Drainage channels are natural or man-made channels used for transport of stormwater. Such transport may be done in the form of an open-channel flow with a free water surface. Other means for stormwater transport are closed conduits which can operate either with pressure or free flow. The choice between open channels and closed conduits depends strongly on local conditions. Open channels are used more frequently in less developed countries or in smaller communities. Closed conduits are preferable for hygienic and aesthetic reasons. They provide better safety, reduce the possibility of illicit discharges of wastes and, therefore, provide a better operation of the sewerage system. The use of closed conduits in drainage design is limited by the local availability of the appropriate pipe materials. Because the transport of solids is more important in open channels and the weed growth in such channels often cannot be avoided, the maintenance of open channels must be much more frequent than that of closed conduits. However, control and cleaning of open channels is much easier and does not require special equipment or trained staff as is the case of maintenance of closed conduits. On the other hand, self-cleansing of closed conduits which is largely given by their proper shape and slope is much better than in open channels. Because of construction costs and maintenance reasons, open channels are frequently favoured in spite of the benefits of closed conduits. Open channels and ditches, however, require additional land and, consequently, are not always feasible in built-up areas.

A compromise solution is to cover open channels and gutters with concrete slabs, thus reducing the risk of accidents and illicit disposal of solid wastes, yet still allowing for maintenance by less skilled labour. The use of concrete slabs has some disadvantages because they break easily and must be often replaced. Also, the handling of large slabs may be difficult when conducting channel maintenance.

The frequent misuse of open drainage channels for solid waste disposal shows that the choice between open channels and closed conduits is very important. In many urban areas it would be sufficient if only a few town districts were drained by closed conduit networks, while for cost reasons residential areas were drained by open channels and gutters. Such drainage systems, however, can be operated successfully only, if at the same time, sufficient services for solid waste removal are provided. Furthermore, public information and education programmes explaining advantages of these systems and disadvantages of their misuse should lead to the acceptance of these new approaches and guarantee their feasibility. Participation of local residents in the control and maintenance work is considered very helpful. Where domestic and industrial wastewaters are collected by sewers, closed conduit systems should be adopted. Pressurized flows require polyethylene or steel pipes, which are very costly. Furthermore, pressurized systems generally require expensive pumping stations. Therefore, pressurized sewer systems with high capital and maintenance costs should be avoided whenever possible.

2.2.4 Advanced drainage schemes

When the drawbacks of combined and separate sewer systems became evident, water scientists and engineers, particularly in Europe and North America, began to search for better solutions. What emerged was a new drainage concept advocating a minimum interference of stormwater control measures with the existing hydrological regime. A central feature of this concept is that, if feasible, the runoff should be retained in the urban area, rather than being diverted out of the catchment. In order to interfere minimally with the hydrological regime, storm runoff should be infiltrated into urban soils wherever possible. It must be kept in mind, however, that the infiltration of polluted storm runoff in a groundwater recharge area is not without hazards. Once an aquifer has been contaminated, the removal of pollutants from the aquifer is physically and economically unfeasible. Thus, treatment of stormwater, in special treatment facilities or sewage treatment plants, prior to infiltration or discharge into receiving waters, is often advisable.

Other recently proposed runoff control techniques include temporary runoff storage on the catchment surface. The areas used for this purpose may include such public areas as parking lots, streets, playgrounds, public gardens, parks and special flood plains whose surface cover and installations can withstand flooding. Such measures, however, require additional construction costs in the design of the surface structures. Open storage facilities in urban areas should be designed so that they flood only under severe storm conditions. The water retained is released slowly after the storm, or infiltrated into the ground. In addition, direct infiltration of runoff from impervious areas such as roofs, paved streets, or sidewalks may be enhanced, especially in residential areas. Even the use of porous road and parking lot pavements has been considered. If the runoff from streets and parking lots is expected to be heavily polluted, its infiltration into the ground should be avoided.

Another technique for runoff detention or retention consists in increasing the runoff time of travel to the sewer inlets. This can be achieved by limiting the density of gutters and inlets, or by designing the gutters and connecting sewers with minimal slopes.

Storage is a common runoff control measure frequently used in existing densely built-up areas. Storage may be provided within the sewer system itself, or off-line, in the form of storage basins or storage channels. Advanced control measures include real-time flow regulation within a sewer network as demonstrated for example in Hamburg, FRG, Seine Saint Dennis County, France, and Seattle, U.S.A. Under appropriate climatological conditions, runoff storage and reuse of stormwater for irrigation or other subpotable water supply should be also considered among drainage alternatives. All of the aforementioned measures reduce the runoff peak significantly and, thereby, avoid possible downstream flooding. Although the idea of advanced drainage schemes is attractive, sanitary problems are best solved by standard on-site or off-site measures or, if unavoidable, by waterborne sewerage using simple network design. It is very important to realize that alternative drainage concepts can be designed most effectively prior to the area development. To incorporate such alternatives into existing urban areas is often difficult and costly.

It should be also realized that the above listed advanced drainage schemes, which have been developed for European and North American cities, may warrant consideration only in densely populated areas of less developed countries. In smaller towns and villages, stormwater may be still discharged into local receiving waters. Groundwater and surface waters from such areas cannot be used for water supply without treatment.

2.2.5 Wastewater and stormwater reuse

Increasing water demands may exceed the total available water supply in certain areas. Under such circumstances, it is necessary to examine the means of augmenting water supply by such measures as wastewater and stormwater reclamation. The information available on this subject indicates that the reuse of municipal wastewater effluent and stormwater for such purposes as industrial cooling, non-potable domestic water supply, and irrigation is technically and economically feasible. The reclamation of stormwater is particularly attractive because it also yields secondary benefits in the form of reduced pollutant discharges, drainage control, groundwater recharge, and improvement and preservation of ecology in the urban area. There is a documented case of reuse of treated stormwater from ponds in a residential area for potable water supply in Singapore (Tay and Chan, 1984). It can be expected that the technology for wastewater and stormwater reclamation will further evolve as water demands will exceed supplies in the future.

2.3 MASTER DRAINAGE PLANS

A master plan of an urban drainage system is a general technical layout plan of the sanitation and storm drainage system for the entire urban area. Such a plan indicates the existing conditions as well as different future development and implementation stages up to the planning horizon.

2.3.1 The planning horizon

The planning of drainage projects requires several types of information on the physical system, development trends, jurisdiction divisions, and financial arrangements. All these factors affect the planning process and the selection of system components. Various design parameters employed in planning are discussed in Chapter 4.

The planning itself is undertaken at two levels - the short-term and long-term planning. In the short-term plan, drainage facilities are planned on the basis of the proposed development plans for a period from five to ten years. Such short-term plans must be compatible with the long-term plan. If needed, provisions are made for future expansion and extention of proposed drainage facilites. The long-term planning reflects the ultimate development of the watershed and is based on a longer planning period which may be as long as 25 to 50 years. To reduce uncertainties in the long-term plan, it is regularly updated, about every five years, to account for the actual development taking place and to reflect changes in technology and engineering practices.

While the planning horizon reflects the envisaged land use projected for a certain number of years, the design period of drainage structures indicates the expected level of protection against such phenomena as flooding, health hazards, or deterioration of quality in the receiving waters. Besides the desired level of protection, the design period depends on construction costs and damage losses resulting from system failures. The planning horizon may be linked to the longest design life of the structures under design. While there is only one planning horizon for the drainage area, the design periods may differ for the individual system components. Both planning horizon and design period are further discussed in Chapter 4 of this volume.

2.3.2 Major and minor drainage systems

Urban drainage planning is obviously linked to the capacity of the receiving water system. Therefore, a master drainage plan should be part of a watershed plan and preferably be prepared at an early stage of the watershed development. Consideration of overland flow, flood plain management and flood control in conjunction with urban drainage is probably one of the greatest achievements in urban hydrology. Severe flooding often is caused by the lack of overland routes, the lack of capacity of creeks or main channels, and backwater effects in the sewer system.

A master drainage plan incorporates the whole urban drainage system including the connections and interrelation between the so-called minor and major system components. The minor drainage system comprises swales, street gutters, catch basins, storm sewers, and surface and subsurface detention facilities. It conveys runoff from frequent storms with return periods up to the design period, namely from one to ten years. The minor system primarily reduces the frequency of inconvenience, caused by stormwater ponding, to both pedestrians and motorists. The consequences of the failure of the minor drainage system, except for possible health hazards, are often insignificant, provided that there is a properly functioning major drainage system. Therefore, the design periods for the minor system are relatively short. The shorter periods apply to residential areas, the longer periods apply to commercial and industrial areas and transport facilities where drainage failures could cause large damage. The minor drainage system has to be properly interfaced with the major system to avoid overloading of minor drainage elements.

The major drainage system comprises natural streams and valleys as well as main man-made drainage elements such as swales, channels, and ponds. The system should accommodate runoff from infrequent storms with long return periods, up to 100 years, or other critical regional events. A properly designed, constructed, and maintained major system greatly reduces the risk of loss of life and property damage caused by flooding of urban areas. It should be emphasized that the major drainage system exists in the nature regardless of whether it is identified as such and preserved during the urban development. It is the route which is followed by water during severe storms. Consequences of the major drainage system failure are quite severe and may include appreciable flood damages and even the loss of life. Therefore, major drainage systems are designed at least for a 50-year storm and often for a 100-year storm or a regional critical storm defined by the local drainage authority.

A master drainage plan of an urban area comprises the minor drainage system components, the links to the major drainage system, and sometimes parts of the major drainage system within the urban area. It should be emphasized that an identical design period does not have to be maintained throughout the drainage system. Some parts of the system, whose failure would be particularly costly, may be designed for longer design return periods than others. Furthermore, the choice of the return period for the minor system may have some implications for the design of the major system. By reducing the minor system design period and capacity, inflows to the major system will be reduced, thus reducing the costs of the major system.

2.3.3 Development and components of master drainage plans

Logically, the sanitation and storm drainage master plans assemble all actions to be taken in an urban area under the existing and future conditions. The master plan must define the technical requirements and layout of the system considering the existing or proposed regulations and institutional concepts. The plan must present technical measures to be taken in a chronological order defining realistic emergency programmes as the first level to be implemented within a short-term period.

The preparation of master drainage plans starts with identification of drainage related problems and definition of study objectives. These objectives generally include the abatement of local flooding and inconvenience, abatement of downstream flood damages and threat to human life, protection of water supplies, and protection of water quality in receiving waters which depends on various water uses. Study objectives are assigned various priorities and need to be accomplished under a given set of constraints wich includes natural, policy, regulation and cost constraints. The definition of objectives requires the assessment of the existing conditions and a forecast of the future development. Obviously, this forecast forms a vital basis for the design of drainage system components.

In the development stage of the planning horizon, the necessary system components must be defined including all quantities and magnitudes necessary for design which are also called design parameters (see Chapter 4). Examples of such input quantities are design rainfall data, water consumption and unit costs. Obviously, the various options available in urban drainage, as discussed in Section 2.2, yield a number of non-structural and structural alternatives for any specific condition. Non-structural measures include land use policies or prohibition of flood plain occupancy. Structural measures include various surface pavements, drainage conduit and channel configurations, storage and diversion structures, channelization and dikes. For each configuration, the overall system outputs are provided in the form of flood hydrographs and costs of various alternatives. These outputs are produced by means of the computational procedures which are discussed in Chapter 5. All drainage alternatives are screened, compared and the best alternative is selected on the basis of decision and evaluation criteria. It is one of the major concerns of the master plan to explore possible alternatives and substantiate the selection of the proposed system. When developing and evaluating various alternatives, financial constraints for the implementation of the system, the assignment of responsibilities to one or more authorities, and reliability and financing of maintenance operations must be considered. Regarding the design parameters and the calculation methods employed, their applicability to local conditions must be checked carefully.

Once the most suitable system layout is selected, all possible proprietary rights or constraints, which may hinder plan implementation, must be checked. The next step is to define the sequence of implementation stages in such a way that the monies invested for each step contribute maximum benefits towards reaching the selected objectives. The final design of individual system components follows and it must accommodate various implementation stages. For example, a particular stage facility within the drainage system may be designed in such a way that it can be enlarged at some later time according to the ongoing development. Such a procedure considers the dynamics of urban development and changes in drainage technology. Therefore, the master drainage plan should be regularly updated and modified, about once every five years. At the same time, the efficiency of already constructed system components may be checked allowing for incorporation of improvements in planning updates. The finalization of a master drainage plan, which depends on the availability of data and the size of the drainage area, may require from one to five years. During the same period, maintenance and operational crews must be trained and, if necessary, the responsible authorities organized in such a way that effective operation and control of the system will be guaranteed. Furthermore, public information programmes should be undertaken.

2.3.4 Example of master drainage plan contents

The following captions may guide the development of a master drainage plan and the preparation of a study report:

- Purpose and background of the study
  - need
  - legal background
  - regulatory background (health, flooding and pollution control criteria)
  - possible updates and regulations
  - institutional background
- Identification of drainage related problems
  - description of the drainage area (topographic and soil conditions, existing drainage facilities, land use)

• extent of the study area
• assessment of existing conditions
• planning horizon
• *forecast of future development* (population, land use, industrialization)
• expected drainage problems for the planning horizon and intermediate development stages

- Definition of study objectives
  • human needs
  • health requirements
  • environmental concerns
  • socio-economic aspects
  • natural constraints
  • policy constraints
  • financial constraints
  • the list of attainable objectives
  • priorities of objectives
  • scope of work
- Data base for planning
  • programme for collection of field data
  • preliminary basic system layout
  • design periods for conduits and individual structures
  • rainfall data as a design basis
  • dry-weather flow quantity and quality
  • stormwater or combined sewage quality
  • infiltration and inflow
  • unit costs
- Methods for planning and design
  • choice of analysis procedures
  • procedures to be used
  • check for local applicability
  • possible comparison with field data for existing conditions
  • availability of input data
- Identification and investigation of drainage alternatives
  • performance of existing systems
  • optimization of existing facilities
  • non-structural measures (land use policies, zoning, prohibition of flood plain occupancy, maintenance of streets, solid waste disposal)
  • minimal structural alternative (open channels, surface ponding)
  • structurally-intensive alternatives (infiltration ponds, diversion structures, surface and subsurface detention facilites, pumping stations, channels)
  • operational measures (maintenance, flow control)
  • performance of alternatives
  • implementation and operational costs
  • screening for alternatives yielding an optimum solution with respect to technical feasibility, environmental impact, political acceptance, costs and financial reality
  • selection of the final drainage scheme
- Impact of the future drainage system
  • proof that objectives are met
  • environmental impact
  • acceptance by the public
  • control and operational requirements of the selected scheme
  • check of proprietary rights
  • other constraints
  • overall financing
- Final design of individual structures
  • local availability of construction materials
  • availability of skilled labour
  • layout and design of all components
  • specifications for future operation and maintenance
- Implementation
  • stages of implementation
  • time frame
  • financing of the individual stages
  • indication that objectives are met at the individual stages of implementation

* interim benefits for the urban population, and
* schedules and requirements for planning updates.

Naturally, not all of the subjects listed apply to every study. On the other hand, special problems encountered may require some additional considerations not listed. It is important that maps and construction drawings are prepared and incorporated in the report. Major calculations must be attached as appendices. Besides the detailed report with all technical and financial aspects, executive summaries should be prepared for politicians and the public. Information of the public and preparation and training for control, operation, and maintenance of the planned system are vital for the success of the overall effort.

2.4 CONDITIONS AFFECTING URBAN DRAINAGE DESIGN

2.4.1 Physical conditions

The major physical conditions affecting urban drainage design include the climate, topography, geology, geohydrology, land use, area size, receiving waters and existing drainage systems.

The major input to any drainage design is the rainfall intensity and its variation over an area. Other climatic factors of interest are air temperature, wind velocity and solar radiation. These factors influence evaporation, snowfall and snowmelt. While in the equator zone rainfall occurs throughout the year in conjunction with high temperatures and evaporation potentials smaller than the rainfall depth, north and south of the equator zone the rainfall occurrence is dependent on the movement of the sun. Rainy seasons are followed by dry spells. During the rainy seasons, the evaporation potential is smaller than the rainfall depth and during the dry seasons the evaporation potential exceeds the rainfall. Thus, humid conditions are followed by arid conditions in a typical seasonal cycle. Arid and desert zones follow to the north and south of the tropical zone. These zones are characterized by the evaporation potential greater than the rainfall depth. Further on towards the poles, there are zones with a mediterranean climate and these are followed by the temperate climate zones. Precipitation in temperate zones is affected by the distance from the sea, continental locations and mountains.

Drainage systems are designed for rainfalls of short duration. Intensities of tropical rainfalls are extremely high. The intensity of tropical rainfalls with a duration of 15 minutes and a frequency of once per year is 2.5 to 4 times higher than that of rainfalls of similar duration and frequency in Western Europe. For a 60-minute rainfall, the above factor is even higher, from 3 to 6. Consequently, combined sewers collecting domestic and industrial wastewaters, and stormwater are impractical in tropical areas, because such sewers would have to be very large and be laid with relatively steep slopes to maintain self-cleansing velocities during the periods of dry weather. Thus in tropical climates as well as in arid or semi-arid climates with few rain storms per year, separate sewer systems preventing petrifaction of the sewer sediment during dry weather should be used. In other climatological zones, both separate and combined sewer systems may be equally well applicable.

Concerning the topography, the surface slope and cover, physical characteristics of soils in the top and underlying layers, vegetation cover and distribution of paved areas affect the choice of a sewer system. In areas with abundant water courses and channels, stormwater may be discharged directly to the nearest water body. This favours the use of separate systems, if domestic and industrial wastewaters have to be also collected. In areas without suitable receiving waters and in hillside areas, long sewer lines are required and, under such conditions, the combined system may be preferred. In any case, pumping of stormwater from flat or low-lying areas should be avoided whenever possible. This recommendation may also favour the use of separate storm sewers which can be laid at smaller depth and milder slopes than the combined or sanitary sewers. The remaining factors, including soil characteristics, vegetation cover and paved areas strongly affect storm runoff and stormwater infiltration. The rates of runoff may then influence the choice of the sewerage system, because relatively low runoff flows may favour the use of a combined system.

Erosion is a very important geological factor affecting drainage design. Particularly in mountainous areas, large quantities of various types of sediments enter the drainage system. Such sediments are then deposited in sewer sections with low velocities, particularly during the periods of low flows. Sediment deposits may then restrict the sewer flow and thereby cause flooding. To avoid such flooding, sewers susceptible to sediment deposition have to be regularly maintained at fairly high costs. The problems of flooding and costly maintenance can be avoided by installing sewers on slopes equal to or greater than the minimum slope required to achieve the self-cleansing flow velocity which allows a continuous transport of sediments. Experience indicates that to establish the self-cleansing velocity, the sediment size needs to be considered only for particle diameters greater than 0,5 mm. Further information on self-cleansing sewer velocities is given in Chapter 6. It should be emphasized that the best way to solve the problems of sediment deposition in sewers is by source controls - minimizing soil erosion through the use of common soil conservation practices.

Urban drainage operation is also interconnected with hydrogeological conditions of the area drained. Limited water supplies and increasing water demands may lead to over-exploitation of groundwater and land subsidence in the affected areas (Lindh, 1985). The lowering of the groundwater table is further aggravated by reduced groundwater recharge because of provision of urban drainage comprising numerous impervious elements.

Urban areas with high or medium density of development have to be often serviced by a conventional waterborne sewage system, if there are no opportunities for low-cost sanitation systems and direct disposal of stormwater. Flooding of these areas may create serious health hazards caused by wash-out of the content of pit latrines. Furthermore, the implementation of a fully adequate drainage system may be hindered by limited access and space, because of the proprietary rights, or existing or planned installations of streets, water mains, electricity and gas supply lines, and drains.

The hydraulic and self-purification capacities of receiving waters and water level variations also affect the sewer design. In some developing countries, the hygienic conditions of surface waters are considered to be the most important water quality criterion. Surface water from rivers and drains is often used for household purposes because of the lack of public water supply or water wells. Tropical surface waters have certain general characteristics which make them different from most surface waters in temperate zones in respect to their capacity for handling waste discharges. Owing to the nature of tropical rainfall, streams usually have very high ratios of the maximum to minimum discharge and low flows occur for extended periods. Because dilution is of major importance in waste assimilation, long periods of low stream flows reduce the stream ability to accept continuously heavy waste loads without damage. Low streamflows, particularly in flat delta areas, where many major cities are located, produce little turbulence, which in turn reduces oxygen diffusion at the surface. Surface diffusion normally provides the greatest source of oxygen for biological breakdown of organic matters discharged to surface waters. In addition, high temperatures not only reduce the amount of oxygen which can dissolve in water, thereby minimizing the oxygen supply available to microorganisms, but also the rate at which oxygen is utilized by microorganisms (Pescod, 1982).

2.4.2 Socio-economic aspects

Socio-cultural aspects have a significant influence on the acceptance and, consequently, the maintenance of any sanitation system. Aside from the educational level, the cultural behaviour and customs have a strong influence on the demand for and maintenance of a drainage system. For the planning engineer, who may be a foreign expert unfamiliar with the traditions and customs of the population, the actual willingness to change customs and traditions is extremely difficult to judge. As a helpful indicator of the readiness to accept the drainage system, the payment of project fees is regarded. If there are delays in payments of appropriate fees, the drainage project under consideration is likely to fail.

For a minimum standard of living at least the basic human needs in respect to water consumption should be satisfied. Sanitation systems must satisfy a certain level of safety at which health hazards are prevented and minimum aesthetics is provided. Desired service standards should be carefully reviewed, because of the failure of past practices which attempted to transfer conventional views of water use and drainage from developed countries to developing countries with little modification. There were "Water and Sewerage Boards" established in countries or regions with no sewers. In several cities, money was spent on master sewerage plans, later superseded by other master plans, without building any sewers. And where sewers were constructed, they often benefitted only a more fortunate minority of the population. Such approaches are completely inappropriate. The planning of drainage works should consider the city on the whole and should start in areas where most people will benefit from such systems.

In the past, economic development was regarded to be closely dependent on the infrastructure of water supply and wastewater disposal. This is not necessarily true. It is now recognized that investments in the field of water supply and wastewater disposal are relatively small in comparison with investments for the industrial development and energy supply. Nevertheless, aside from providing employment, the improvement of drainage system still has a positive effect on the economic development. Furthermore, the availability of construction materials for installations and maintenance is often a guiding factor for the construction of the drainage works. Note also that the affordability of house or property connections to an urban drainage network depends on the income of the population.

2.4.3 Financial and institutional aspects

In developing countries, urban drainage design usually deals with the planning of new systems. Therefore, there is a lack of local experience and proper institutions. In addition to the design task, it may be necessary to provide funds for training of construction and maintenance crews as well as funds for a public information programme. In the long run, this will help to establish public services. Chapter 9 further elaborates on institutional aspects.

The amount of money spent on urban drainage in the past was small in comparison to other sectors. As an example, the resources required to provide adequate drainage and solid waste disposal in five Asian countries are listed in Table 2.3. The countries selected are neither extremely poor nor particularly representative for their region. They were selected more or less because of the availability of homogeneous and nearly-complete data. Table 2.3 separates the resources required for urban and rural areas. In rural areas, the measures to be taken include the installation of dry toilets, pit latrines and proper storm drainage. In urban areas, the measures may reach up to the installation of separate or even combined sewer systems. The necessary per capita expenditures reach in some cases the magnitude of the gross national product (GNP). This means that the financing of such systems by these countries alone is simply impossible. It also implies that low-cost sanitation systems have to be used in countries with limited financial resources, although the high density of their urban developments would justify a waterborne sewerage system. The drainage water has to be collected by a system of open channels and ditches. Obviously, there will be a little interest in the use of low-cost sanitation systems in those urbanized areas where public funds are available for the provision of sewerage systems. On the other hand, many developing countries have to cope with serious flooding problems in urbanized areas and, consequently, the priority is given to flood control measures. Because of limited financial resources, the implementation of new construction programmes and the improvement of existing facilities can only be carried out step by step.

Table 2.3 Estimates of Resources Needed for Wastewater and Solid Waste Disposal (Hahn, 1982)

		Urban Areas		Rural Areas	
Country	GNP in U.S. $ (1978)	Per Capita Range (in U.S. $)	Deficit %	Per Capita Range (in U.S. $)	Deficit %
Bangladesh	90	18,6 - 155	62	19,8 - 247,5	99
India	180	15,9 - 132,5	53	19,6 - 245	98
Malaysia	1090	7,2 - 60	24	7,4 - 92,5	37
Philippines	510	20,4 - 170	68	14,6 - 182,5	73
Thailand	490	6,6 - 55	22	15 - 187,5	75
Average GNP in industrialized countries	8000				

For acquisition of project funds, the maintenance costs and any other follow-up costs have to be included in the budget. A new financial management system may need to be developed which would include these expenses and also secure some financial return on reserved funds.

In many countries, water laws are relatively few and simple, and water is generally perceived as a gift rather than a resource to be managed. Private water uses are often uncontrolled or controlled by ambiguous legislation which allows widespread overuse and misuse of water resources. Water pollution control legislation is only now being introduced in many countries. The existing legislation has not been effectively enforced because of the priority assigned to industrial development as a major contributor to the national economic growth. Water quality is yet to receive significant attention in the development planning, although water resources development projects are generally given high priority in national plans.

There is a proliferation of government agencies responsible for different aspects of water use and pollution control but little co-ordination among them (Pescod, 1982).

The lack of public investment in urban sanitation contributed to the rise in private sector management of domestic wastes in on-site disposal systems. Poorly designed and constructed installations and inadequate regulations governing waste disposal have often caused contamination of groundwater and surface waters by polluted runoff. Official ignorance of the widespread pollution of urban surface waters by domestic sources has made industry adopt an irresponsible attitude towards the pollution impacts of industrial effluent discharges. Poor enforcement of inadequate pollution control legislation has caused the industry to be generally unco-operative in regulating its wastewater releases. Mistrust on both sides leads to wrong decisions in formulating water quality objectives and in planning to meet them (Pescod, 1982).

There is an obvious need for greater co-operation between the government and industry. Management of water resources, however, must be for the public good rather than for private interests, although the two might be compatible if a flexible official policy is adopted. In view of the increasing attention being paid to urban drainage in developing countries, it is likely that investments in urban sanitation and industrial wastewater treatment will expand rapidly in the future. It is essential that sensible water quality objectives are established and rational management approaches adopted to minimize the required investments in both the public and private sectors and to achieve least-cost solutions to local problems (Pescod, 1982).

To implement an urban drainage programme, it is essential to have an institutional structure which allows the adoption of a practicable approach. Although one regional water authority with responsibility for water management in an entire river basin would be an efficient institutional arrangement, in reality, a large number of authorities are typically concerned with water in developing countries. Nevertheless, governments must attempt to co-ordinate the activities of different agencies to achieve desirable water quality management objectives and to work towards a consolidation of authority in fewer agencies over time (Pescod, 1982).

A regional water authority, properly funded and staffed with qualified engineers and scientists, will be in the best position to take advantage of the wide range of alternatives available in urban drainage design. The division of responsibility among several agencies belonging to different government ministries will inhibit the application of a holistic approach. However, even a regional water authority must be empowered by the law to carry out its functions. Regulations can be formulated in such a manner that various strategies can be applied to suit the needs of different situations. Controlling authorities should be prepared to enforce regulations and must be equipped with sufficient staff and resources to monitor the situation. Government policy should be made public and regular discussions should be held between water agencies and water users in a specific area. The priorities have to be set at the national, regional and local levels (Pescod, 1982).

2.4.4 Operation and maintenance

Many developing countries suffer from the lack of skilled labour for the maintenance of waterborne sewerage systems. For maintenance of drainage systems, it is not only necessary to train personnel on the job, but also to ensure that the trained personnel does not leave to accept other positions. This problem is further aggravated by the low image of wastewater disposal in many countries. When choosing the appropriate technology, these aspects also have to be considered.

Simple techniques for the cleaning of sewers and drains and the operation of pumping stations will have to be applied. Sewer cleaning may be carried out manually, which requires short distances between sewer manholes. Sewers are subject to infiltration, particularly if they are laid at great depths below the groundwater table. In general, it will be difficult to construct watertight conduits under such conditions. The lack of suitable materials may further aggravate this problem and lead to maintenance problems, because continuously flooded sewers and drains cannot be properly maintained.

Another aspect hampering sewer maintenance is the location of drains on private property, particularly where underground drains have been constructed under buildings. As a minimum level of service, open storm drains are recommended for urbanized areas with unpaved roads, to facilitate easy maintenance of the system. However, measures have to be taken to prevent the misuse of the open channels for solid waste disposal. Siphons and sewers without self-cleansing characteristics should be avoided and the number of pumping stations has to be limited as much as possible. In general, it is necessary that the user of the drainage system understands its purpose. The construction of channels or conduits should be done in such a way that the citizens can repair and maintain the system, in the worst case, themselves. Therefore, local materials should be used and inexpensive and easy maintenance and operation should be ensured. Despite using simple methods and materials, the operational safety must be ensured.

As explained above, the measures taken in the planning and construction of drainage must consider the local ecological, sociological, and financial conditions. It follows that in every case appropriate technologies have to be applied for successful planning. In most cases, the appropriate technologies are not mere transfers of the technologies of highly industrialized countries, but modified technologies which depend on the available labour skills, and commercial and industrial services.

3 Elements of drainage systems

3.1 DIVERSITY OF DRAINAGE SYSTEM ELEMENTS

Implementation and detailed design of drainage systems have to account not only for hydrologic but also for structural, geotechnical, operational, maintenance, economic and other considerations. Although urban hydrologists contribute significantly to the analysis of design alternatives, the ultimate responsibility for the selection of the best alternative rests with municipal engineers.

Successful application of new urban hydrologic concepts requires a good co-operation between hydrologists and municipal engineers. The former should get familiar with the existing drainage systems, which have been implemented over a long period of time and have, therefore, components based on different design principles. Although implementation of new methods is obviously easier in new communities, such methods may also apply to some of the older drainage systems.

The chapter that follows presents, therefore, descriptions of both recent and older facilities and refers mainly to those aspects which are of interest to the urban hydrologist. Structural aspects are not included in the discussion.

Detailed design aspects are frequently presented in drainage criteria developed by municipalities or other agencies. Common designs for such elements as conduits, appurtenances and culverts vary not only from one country to another, but quite often they are not the same even for neighbouring municipalities. The same diversity applies to various runoff control measures which were briefly discussed in Section 2.2.4. Most of these structures and measures can be incorporated in both separate and combined sewer systems which were described in Section 2.2.2 and whose layouts were shown in Figures 2.1 and 2.2.

3.2 CONVEYANCE ELEMENTS

Drainage flows are transported by conveyance elements which comprise underground conduits and/or open-channel drains. Advantages and disadvantages of both types of elements are discussed in Section 2.2.3 and summarized in Table 3.1 below.

3.2.1 Open-channel drains

In spite of some disadvantages listed in Table 3.1, open-channel drains are often used for economic reasons and they are preferentially used in areas subject to soil settling or land subsidence (ATV, 1982a). Basic considerations in practical design of such channels include the channel shape, bottom slope, susceptibility to erosion, and maintenance of flow velocities required to prevent the settling of solids and concomitant odour problems. Regarding the channel shape, common shapes include triangular, trapezoidal, rectangular, semi-circular and composite-shape channels. The composite channels represent various shapes with cunettes. The selection of the channel shape depends on soil properties, minimization of excavation, the need for lining and hydraulic considerations. Among the linings, in-situ poured concrete or prefabricated concrete slabs are most popular. The basic hydraulic design of open channels is given in Chapter 6 and practical design aspects are explained in Chapter 7.

Natural channels in drained areas may be used as elements of the major drainage system. This may require an improvement of their conveyance and considerations of such aspects as extent of flood plains, provisions of adequate depth for collection of runoff, evaluations of potential erosion and loss of embankment stability, effects of road crossings, and proper design of culverts or bridge openings of adequate flow capacity.

Table 3.1 Advantages and Disadvantages of Underground Conduit and Open-Channel Drains

Drainage Type	Advantages	Disadvantages
Underground conduit drains	Convenience Safety Aesthetics No land required Good control of inflows Easy maintenance, if self-cleansing	Higher construction costs. If poorly designed and/or constructed, high maintenance costs requiring special equipment and skilled labour. Requirements on construction materials.
Open-channel drains	Low construction costs Easy maintenance with unskilled labour	Poor control of inflows and waste disposal. Reduced safety and hygiene. Land requirements. More frequent maintenance.

Traditional drainage practice used channelization extensively by deepening natural channels or replacing them with lined canals. Such practice results in reduction of time of concentration, potential increase of downstream flooding and is not acceptable for recreational or aesthetic purposes. In some new developments in North America, regulations require preservation of natural creeks and do not allow development in the area within flood lines corresponding to a selected storm return period. In a less restrictive approach referred to as the "two-zone concept", some restricted development is allowed in flood fringes while floodways have to be maintained without any encroachment. Another solution may be provided by using aesthetically pleasant channels.

3.2.2 Underground conduit drains

Most drainage systems in European and North American countries are of the underground type. This preference applies to both combined and separate sewer systems. The design and construction of underground systems requires more considerations than in the case of open-channel drains. In particular, it involves the use of numerous special appurtenances and, therefore, this chapter deals mainly with underground systems.

The interface between the drainage within the private property and the public drainage system is shown in Figure 3.1. It is seen that where buildings have basements, flooding problems may occur at this interface.

In countries with potential freezing, a minimum depth of cover of about 1,5 to 2,0 m is required. In warmer climates, smaller minimum depths of cover, i.e. 0,8 m, are acceptable. The profile of the conduits is defined by using the minimum cover at the upper end and adjusting the profile slopes according to the criteria mentioned above. If velocities increase above an acceptable limit of about 6 m/s, or if there are sudden changes in the ground slope, drop structures may be required. The maximum and minimum flow velocities are further discussed in Section 6.5.

Depth of storm sewers is also related to the practice regarding foundation drains and use of basements (Figure 3.1). If there are no basements, surcharge of storm sewers does not cause a problem. In some cases, basements are used as dwelling space and foundation drains are connected to storm sewers. Consequently, storm sewers may have to be located under the basement level and special measures may be required to limit surcharge occurrences. Connections of foundation drains to sanitary or combined sewers should be avoided because the resulting treatment of increased diluted volumes of sewage is costly. Therefore, the problems of basement flooding should also be considered when choosing between combined and separate sewer systems.

In the past, many different conduit cross-sections have been used (e.g., various egg shapes), whose purpose was to achieve sufficiently high velocities at low flows. In modern practice, prefabricated circular conduits are used as much as possible for economic reasons. Common sizes of prefabricated circular pipes vary from a minimum of about 250 mm to about 2000 mm. However, even larger pipes are sometimes prefabricated. For exceptionally large conduits, rectangular or similar cross-sections with cunettes and an in-situ monolithic construction are used.

The choice of pipe material is affected by the local availability of various materials, aggressivity of conveyed flows and structural considerations. Typical materials include clay, concrete, asbestos cement, corrugated steel, cast iron and plastics (WPCF, 1970a).

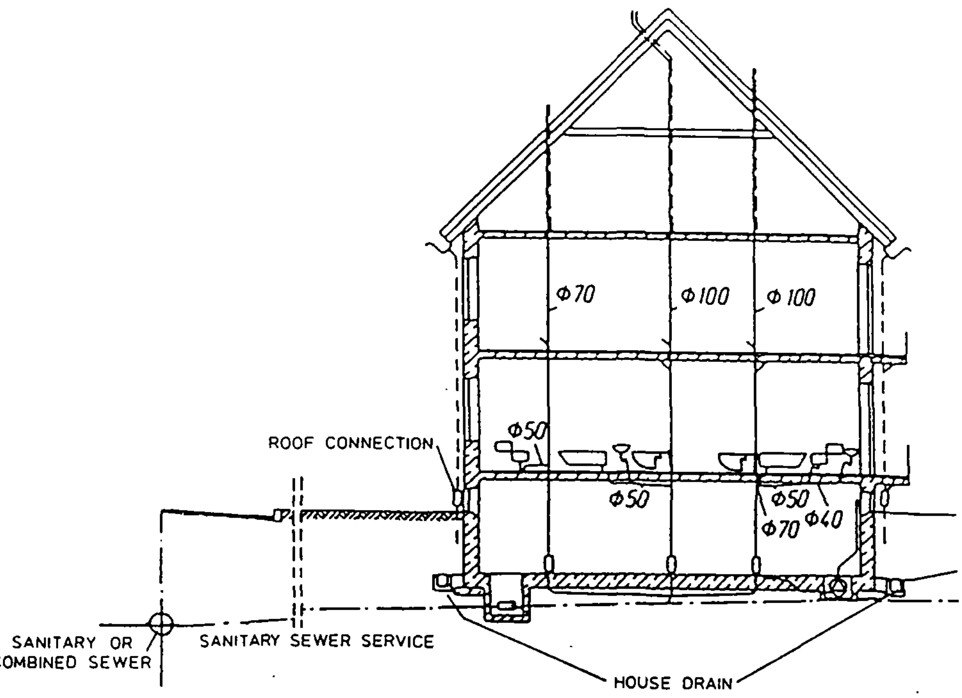


Fig. 3.1 Schematic of house connections

Details of network layout, final design and construction of sewers can be found in standard handbooks (WPCF, 1970a; ATV, 1982a; ATV, 1982b; Lautrich, 1980).

3.3 APPURTENANCES

The appurtenances discussed in this section include manholes, junctions, inlets, catch basins, drop structures and siphons. Such structures can be built using in-situ poured concrete, or by partial or full prefabrication using various materials.

3.3.1 Manholes and junction structures

Manholes are designed to provide convenient access to sewers for operation and maintenance. They should cause minimum interference with the sewer flow. Figure 3.2 shows some typical manhole layouts. Other manhole designs can be found in common handbooks (WPCF, 1970a; ATV, 1978).

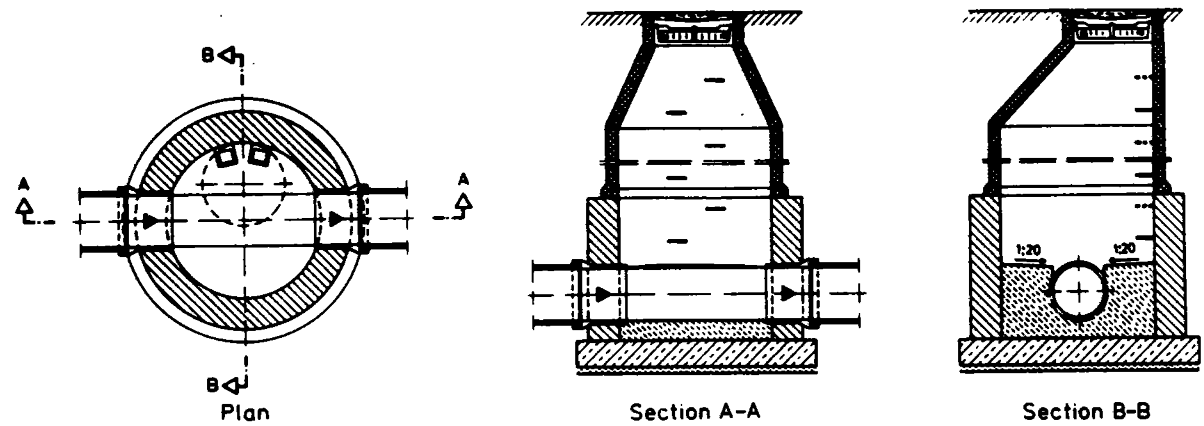
The spacing of manholes in a sewer network is often defined by local regulations. For ventilation and maintenance, the recommended manhole spacings range from 50 m to 80 m. In the case of storm sewers large enough to permit the passage of a man, even larger spacings may be used.

Most manholes are circular with inside dimensions sufficient to perform inspection and cleaning operations without difficulty. Manhole covers must take into account adequate strength to support superimposed loads, good fit between the cover and frame to avoid rattling in traffic, and provision for opening and reasonably tight closure. Manholes are also located at sewer junctions, locations where sewer grade or alignment change (except in curved sewers), and at street intersections. An example of a sewer junction manhole layout is shown in Figure 3.3. Other similar examples can be found in common handbooks (WPCF, 1970a; ATV, 1978).

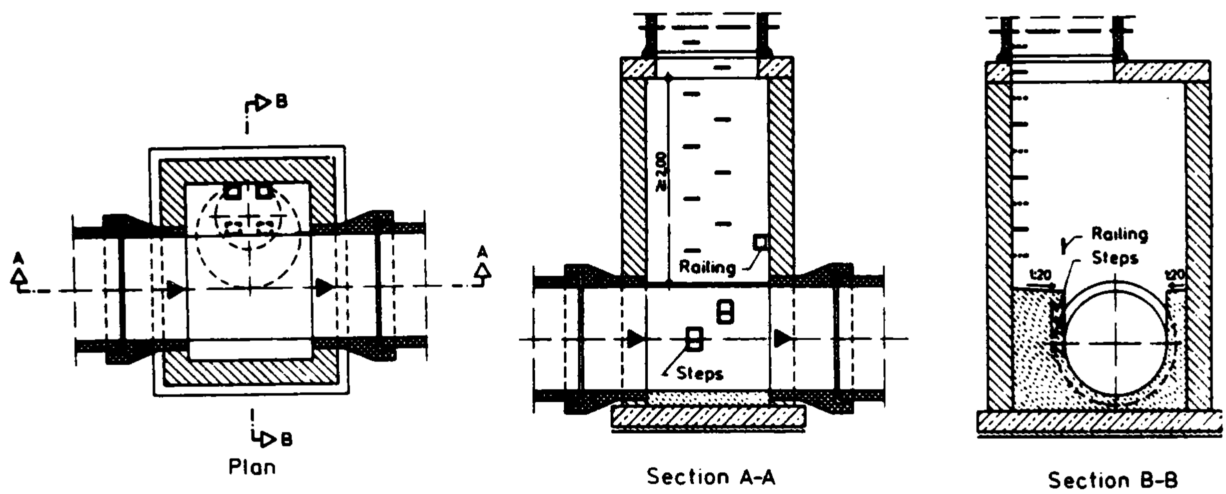
3.3.2 Inlets

Inlet design has received increased attention in recent years, because inlets control the quantity of flow entering the sewer system. The capacity and spacing of inlets determine the division of the flow between the underground sewers and street gutters. If the captured flow is greater than the sewer capacity, the sewer will surcharge and this may lead to flooding of basements. Hence, the inlet design has to meet the following two criteria:

a. Manhole preferably used up to 0.5 m in diameter



b. Manhole preferably used for manhole diameters larger than 0.5 m



c. Manhole with side entry

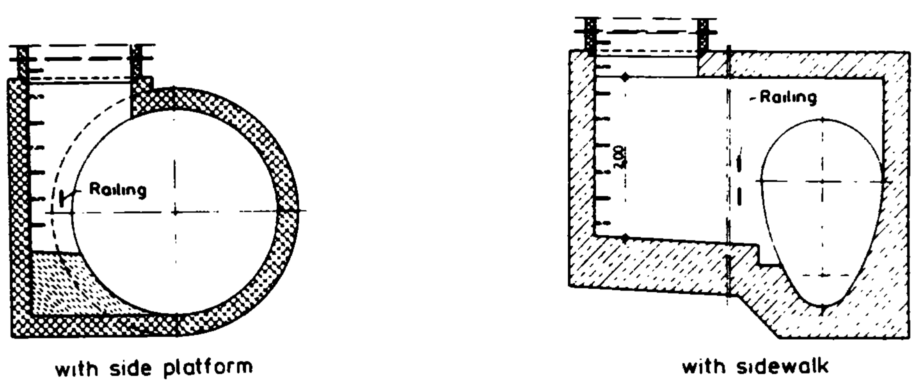
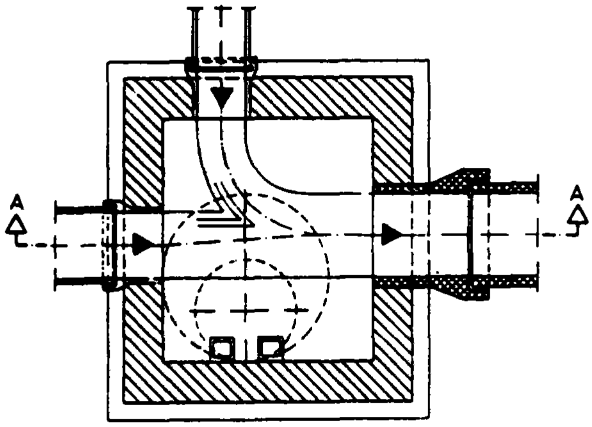
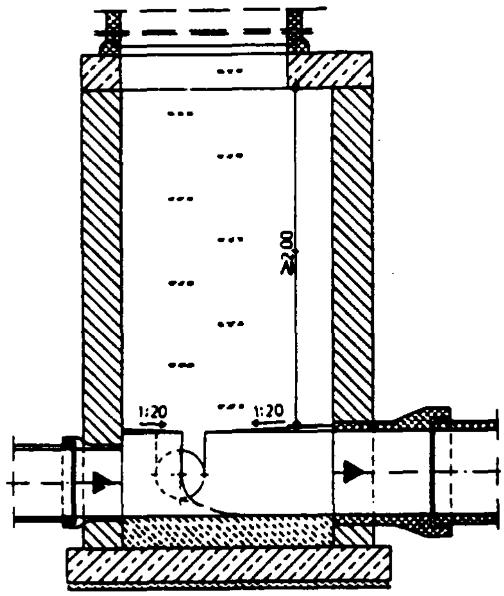


Fig. 3.2 Examples of manholes (ATV, 1978)

a. Junction manhole up to 0.5 m in diameter with perpendicular lateral sewer (other lateral angles may be used)

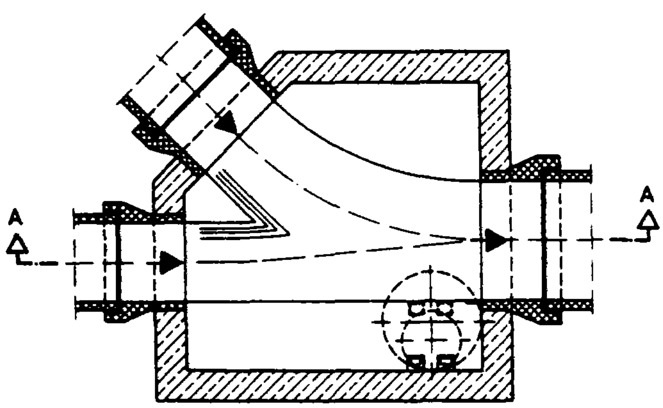


Plan

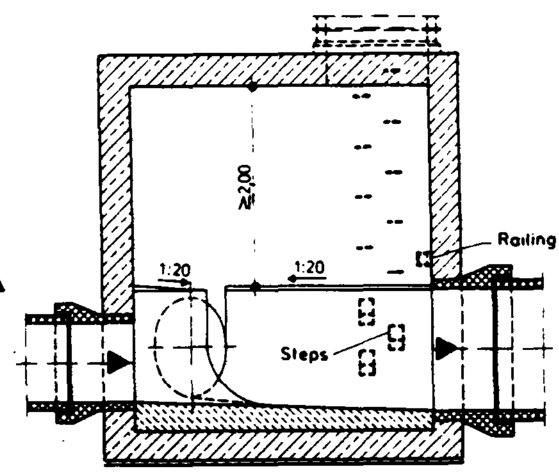


Section A-A

b. Junction manhole for manhole diameters larger than 0.5 m with 45° lateral sewer (other lateral angles may be used)



Plan



Section A-A

Fig. 3.3 Sewer junction manhole layout (ATV, 1978)

(a) For storms generating peak flows equal to or less than the flow capacity of the designed sewer, inlets should be sized to capture as much of the overland street flow as possible.

(b) For storms generating flows greater than the capacity of the sewers, inlet capacity should be controlled. Excess flow should be conveyed to the surrounding major system to avoid surcharge of the sewer network. Spacing of inlets should hence be determined by a rational analysis of these governing factors.

The choice of the inlet and grate types should not be based on the capacity alone. Inlet clogging by debris and bicycle safety of grate inlets have to be also considered. Some of the options of inlet design comprise the choice of curb inlets, grate inlets and combination inlets which are a composite of the two preceding ones (see Figure 3.4). Curb opening inlets are very effective in sags and for flows carrying floating debris. As the gutter grade steepens, their interception capacity decreases. The capacity of grate inlets is less affected by the gutter grade, but their principal disadvantage is their clogging by debris. Combination inlets provide both a curb opening and a grate. They are high-capacity inlets which combine many advantages of both kinds of openings. Sometimes, the curb opening is placed upstream of the grate inlet (the so-called sweeper) (APWA, 1981).

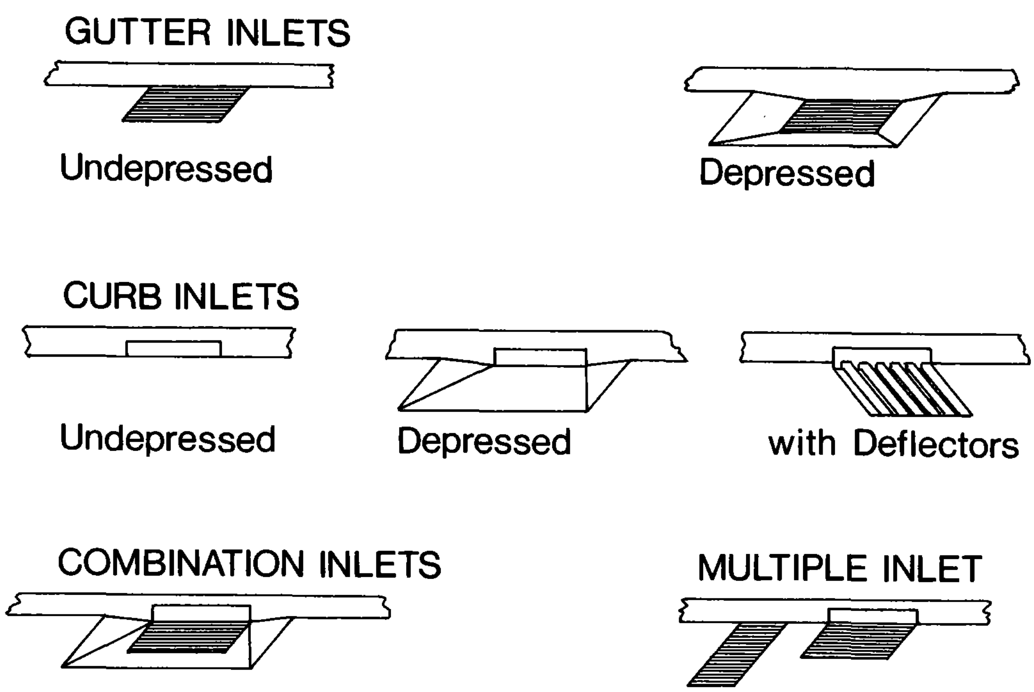
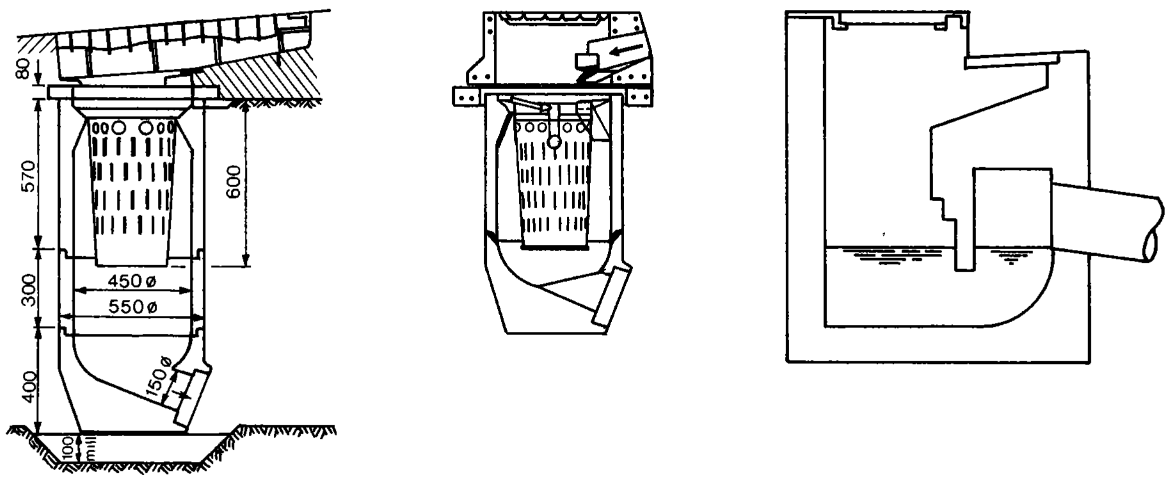


Fig. 3.4 Examples of inlets

3.3.3 Catch basins

Catch basins are chambers or wells, usually built under inlet gratings or openings, which are located either under the gutter or just back of the curb. A major function of catch basins is to prevent solid matter from the street from entering the sewers. The retention of solids is achieved by providing a sump or settling basin in which the heavy solids settle to the bottom while the light matter floats on top. Sometimes, catch basins are provided with a bucket strain for separation of various solid materials. Such buckets manufactured by various companies provide easy and rapid cleaning by maintenance crews (Figure 3.5). Removable bags made of special permeable materials have also been proposed to capture solids and reduce pollutant loading. Catch basins with sumps are used mainly for extremely flat pipe gradients. In this case, flow velocities lower than self-cleansing velocities may result in accumulation of debris in pipes and reduction of the pipe capacity.



(a) Inlet with Bucket Strain

(b) Inlet with Side Inflow and bucket Strain

(c) Inlet with Catch Basin (Atlanta Design)

Fig. 3.5 Examples of catch basins (Figures (a) and (b) are after ATV, 1982b)

Because the primary purpose of a catch basin is to trap solids that would otherwise enter the sewer and form deposits causing stoppages or impeding the flow, it is obvious that the trapped material has to be regularly removed.

In the case of combined sewers, catch basins have to be designed to trap sewer gases using various types of water seals.

3.3.4 Drop structures

Drop structures are used only when it is economically unfeasible to steepen the incoming sewer. In general, the use of these structures for drops of less than 0,6 m should be avoided. Figure 3.6 illustrates the common types of drop structures. For the type shown in Figure 3.6b, flow velocities are sometimes reduced by means of a series of steps. For higher drops, drop shafts of various types are used. Details of such designs may be found in design handbooks (ATV, 1982b; WPCF, 1970a).

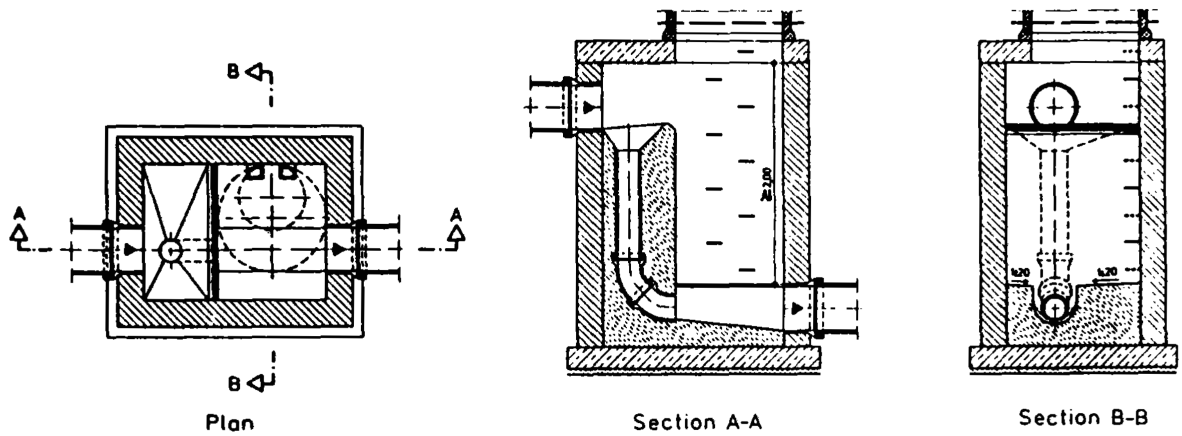
3.3.5 Siphons

Whenever it is required to carry flow under an obstruction, such as a stream or a depressed highway, and regain as much head as possible downstream of the obstruction, siphons are used. A common practice is to use multiple barrel siphons, at least when dealing with larger sewers (Figure 3.7). This helps to facilitate self-cleaning of individual barrels by allowing a flow velocity greater than 0,5 m/s at least once per day. In any case, siphons require more frequent cleaning than gravity sewers. In general, it is recommended that siphon layouts should have only smooth curves with an adequately large radius. The rising leg should not be too steep in order to facilitate cleaning. Changes in the siphon pipe diameter should be avoided since this would make cleaning operations more difficult. Additional information on siphon design can be found in common handbooks (WPCF, 1970a; ATV, 1982b).

3.4 OVERFLOW STRUCTURES

Overflow structures are provided in combined and partially-separate sewerage systems to restrict the through-flow to the treatment plant to a specified maximum and to divert the surplus flow directly to the nearest water course. Older overflow structures were designed to overflow when the flow exceeded a specified discharge without any considerations of the composition of the retained and escaping flows. Recent designs consider not only the hydraulic performance, but also the quantity of pollutants reaching the receiving water body. Overflow structures may be classified into two categories - overflows with static and dynamic controls.

a. Drop structure with underflow



b. Drop structure with chute

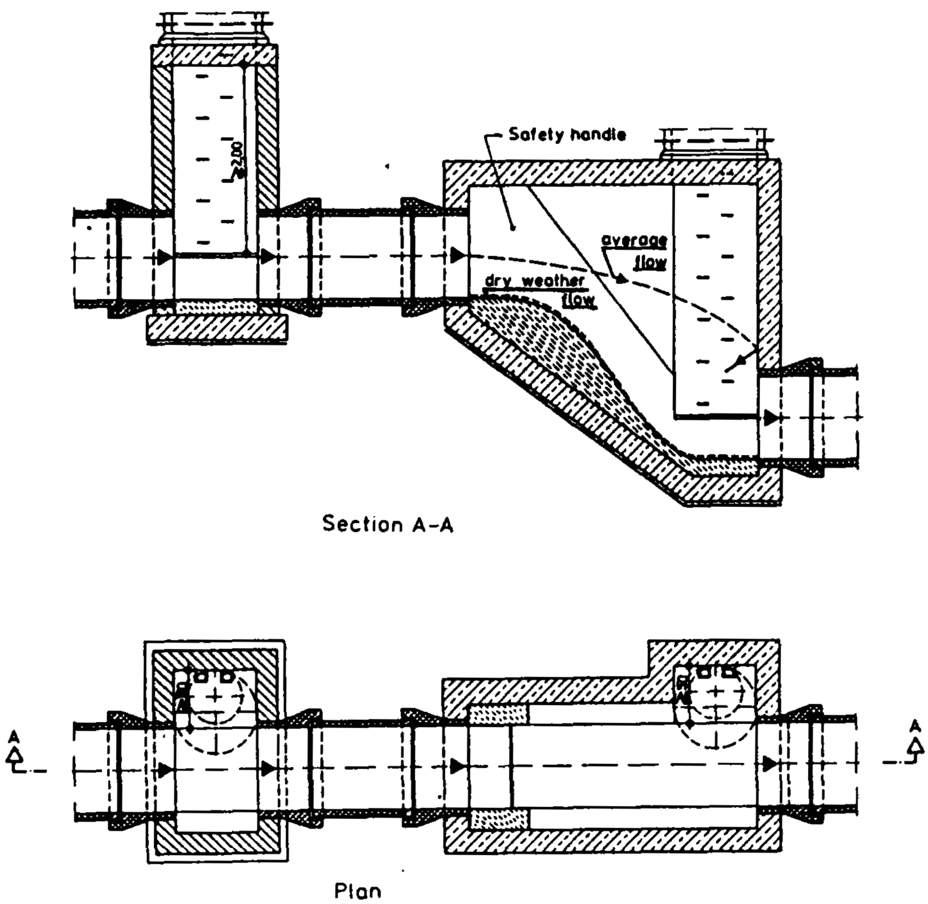


Fig. 3.6 Examples of drop structures (ATV, 1982b)

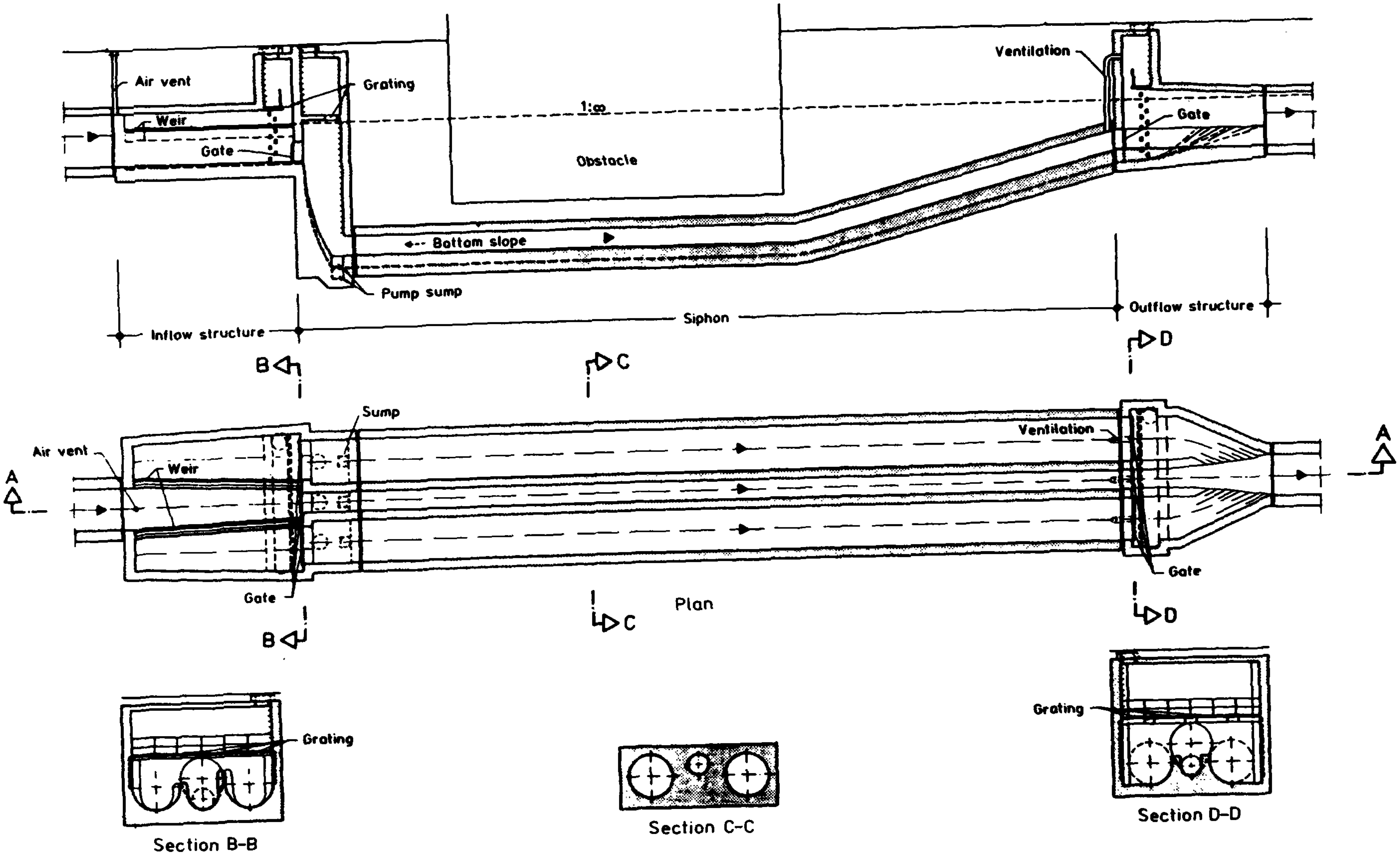


Fig. 3.7 Triple-barrel siphon (ATV, 1978)

3.4.1 Overflows with static controls

There are many types of overflows with static regulators, such as weirs, throttle pipes, and fixed-setting gates.

The simplest type of storm overflow is an outfall pipe, built into the side wall of a manhole, which comes into operation when the water level in the manhole exceeds a selected threshold. This alternative gives a poor hydraulic control of the flow passing forward for treatment because the water level in chamber rises with increasing runoff. Furthermore, this design does not control the quality of effluent being discharged through the overflow pipe. The above design can be improved by using a low side weir as an overflow regulator (Ackers, 1957). The overflow may be allowed along one or two sides of the channel, as shown in Figure 3.8a. Although this type of overflow regulator retains grit and coarse solids only to some extent, it may also retain floating debris, if scum boards are provided. Further improvements can be achieved by raising the weir crest almost to the crown level of the incoming pipe (high side weir regulator). Other similar designs with curved overflow crowns are also used (Ackers et al., 1967).

Another simple overflow design is the so-called leaping weir which provides an opening in the sewer invert. The dimensions of this opening control the intercepted flow which falls through the opening. The overflow leaps or passes over the opening and continues to the outlet (see Figure 3.8b).

Reductions in pollutant loadings in overflows can be achieved in specially designed overflow chambers. The first of such designs discussed here is the vortex overflow which comprises a circular overflow weir, a circular chamber with a spiral baffle and a scum baffle. The underflow is directed towards an outlet pipe of a selected capacity (Smisson, 1967). Another design in this category is the overflow with a stilling pond whose outflow is controlled by a throttle pipe. The pond has to be designed so that floating debris remains in quiescent zones at the surface near the upstream corners of the chamber. Retained flowing debris may be forwarded for treatment with other suspended material after the peak runoff passed (Sharpe and Kirkbride, 1959). Another method to improve hydraulic control of this design is to replace the side weir overflow with air-regulated siphons (Frederick and Markland, 1967).

3.4.2 Overflows with dynamic control

Operation of major overflow structures can be significantly improved by using dynamically-operated regulators. Such regulators are generally activated by sewage levels in the trunk or intercepting sewers. In the former case, they limit the intercepted quantity to a selected maximum amount and, in the latter case, the amount diverted increases until the intercepted sewer flow reaches a desired depth and is then throttled.

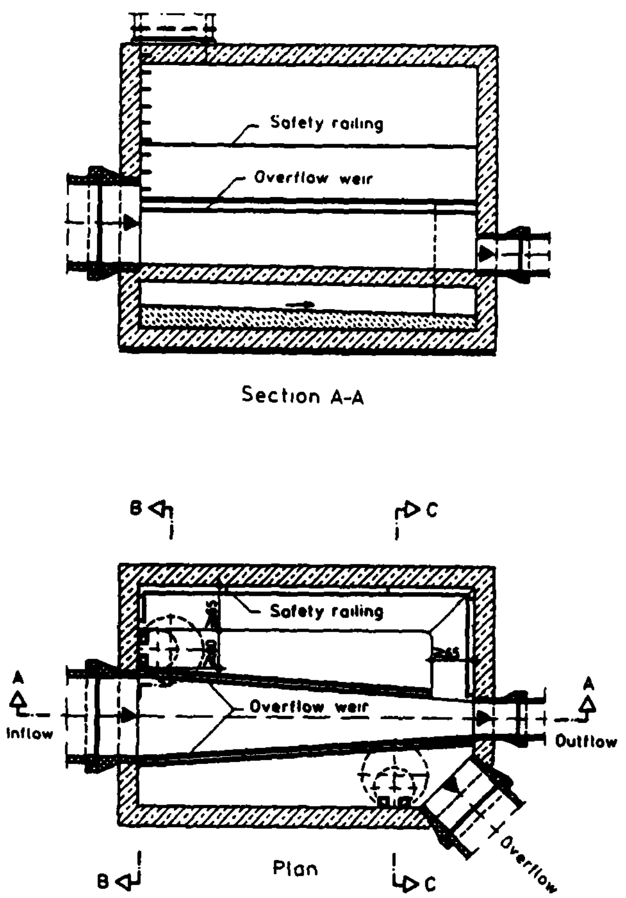
Review of operation of older static regulators indicated that their performance is quite often unsatisfactory. In comparison to static regulators, properly designed dynamic regulators may further reduce the frequency and volume of overflows. Studies have been conducted in many large cities to find the best solution for modernization of combined sewer systems. It was found that the number of overflow structures can be reduced by replacing the old static regulators with fewer dynamic regulators. This may involve the use of remote control and optimal operation based on real-time forecast. Such measures attempt to minimize the total pollution load discharged into the receiving waters. An example of a remotely controlled overflow regulator is shown in Figure 3.9. In recent studies, the analysis of overflows and the selection of control alternatives have been based on simulations of pollution loadings and their effects on the receiving water body. Different interception capacities and storage options can be analyzed using such models as the Stormwater Management Model of the U.S. Environmental Protection Agency (see Chapter 8).

In the cases of tidal outfalls, or large variations of the water level in the receiving water body, overflows should prevent backflow by means of a tidal gate controlled by a motor, a mechanical device, or both (see Figure 3.10).

3.5 RUNOFF CONTROL

Runoff controls comprise measures taken to reduce the volume of surface runoff entering the sewer system and measures involving control by storage and treatment of the collected flow. The former group of controls is referred to as source controls, and the latter group represents collection system or off-line controls. Both groups are discussed below.

a. Side weir with overflows on both sides



b. Leaping weir with straight overflow pipes

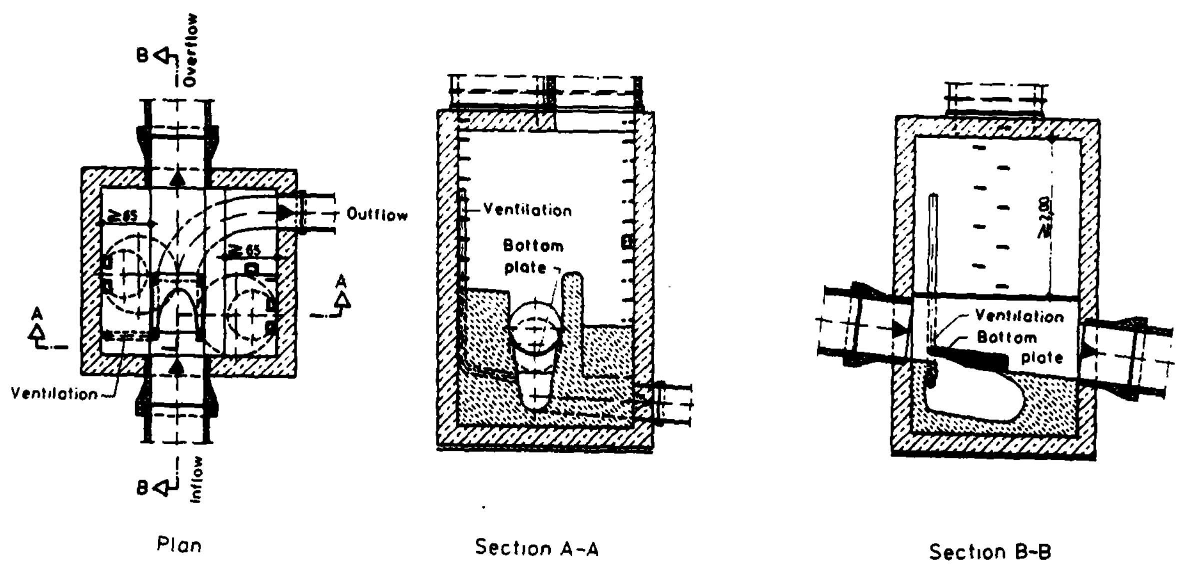


Fig. 3.8 Overflow structures (ATV, 1978)

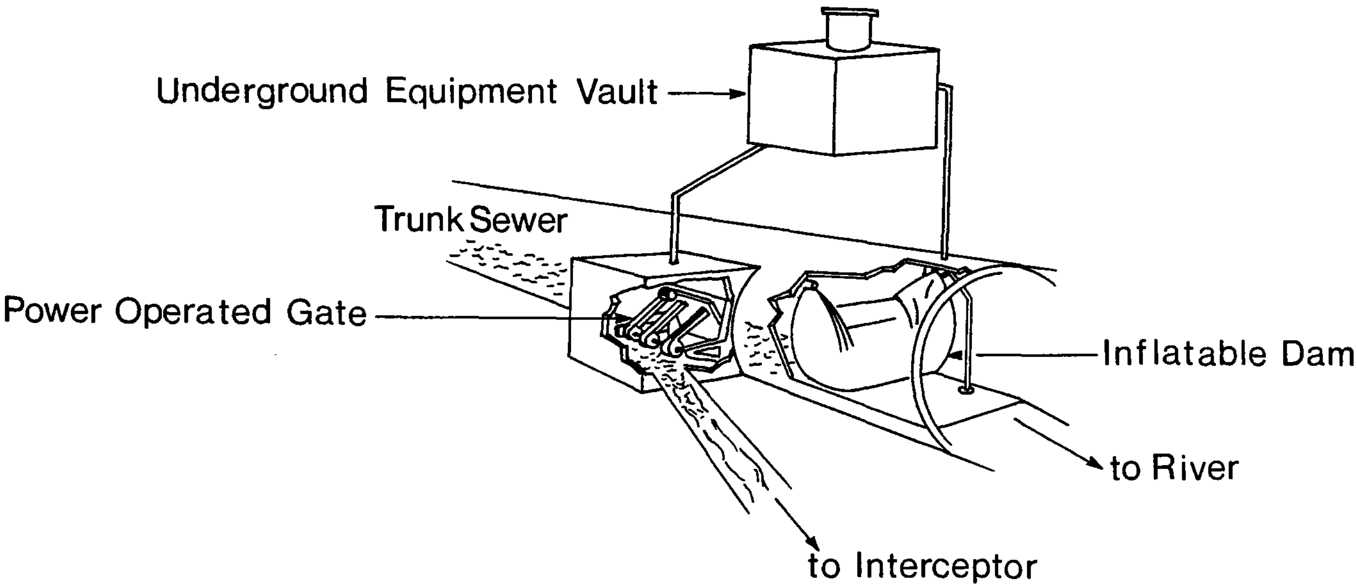


Fig. 3.9 Overflow regulator with remote control

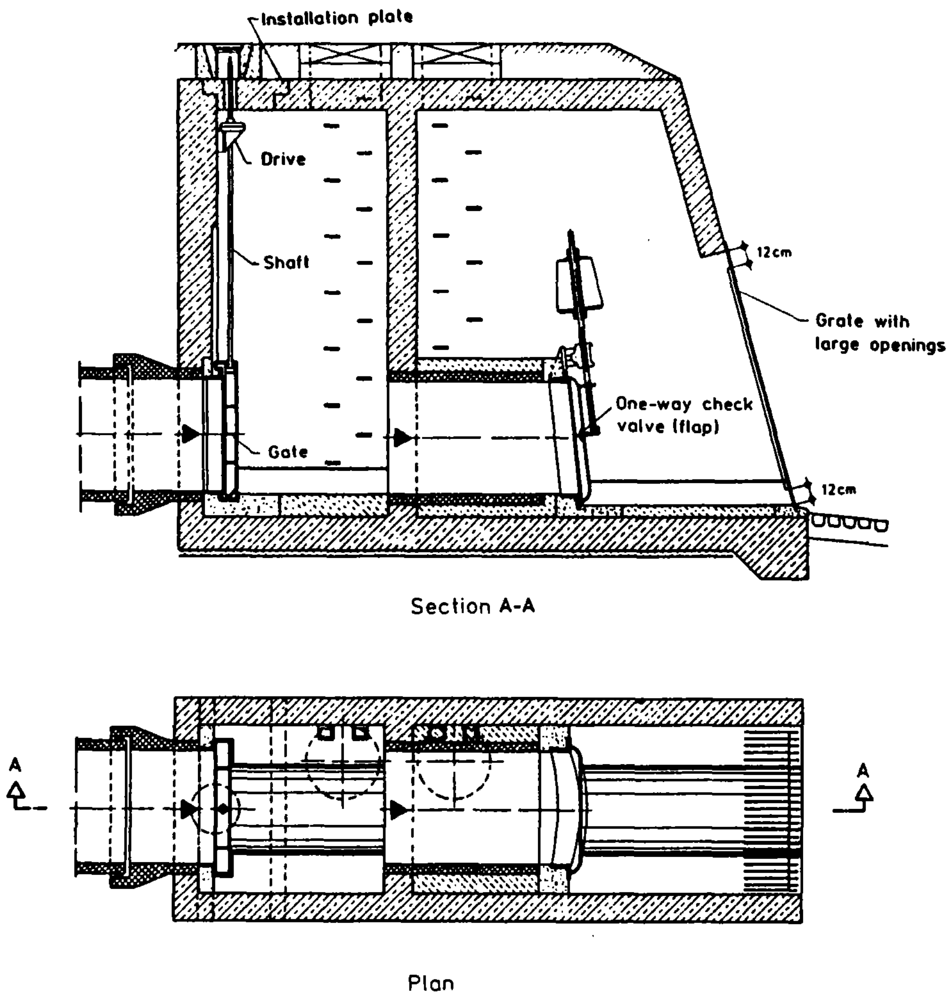


Fig. 3.10 Tidal gate structure with gate and flap (ATV, 1978)

3.5.1 Source controls

The most cost-effective methods for reduction of runoff in urban areas are source controls (Ministerie van verkeer en waterstaat, 1985). Some simple source control measures are shown in Figure 3.11. Explanatory comments on such measures follow.

Figures 3.11a and 3.11b depict discharges from rainwater leaders, fitted with splash pads or extension pipes, onto grassed areas instead of directly into storm sewers. Such an arrangement increases rainwater infiltration and hence reduces runoff. Where native soils are fairly pervious, the rainwater leaders may discharge into soak-away pits located some distance from the basement wall (see Figure 3.11d). Another possibility is to discharge flows from rainwater leaders into a cistern which may be used to store water for garden watering, particularly by root-feeding pipes (Figure 3.11c). The last source control measure, shown in Figure 3.11e, is a seepage trench. It is a stone-filled trench used to detain runoff temporarily. A controlled outlet, usually a perforated pipe, drains to the storm sewer and assures eventual emptying of water to prevent frost heave.

Another source control measure is porous pavements. Such pavements provide storage and enhance runoff infiltration into the ground. Porous asphalt-concrete pavements can be underlain by a gravel base with large storage capacity. Experience indicates that porous pavements can reduce runoff by up to 83%, their structural integrity is not impaired by heavy loads, and their clogging can be relieved by flushing (Field, 1986).

Porous pavements, soak-away pits, seepage trenches and grassed swales are basically groundwater recharge facilities which are dispersed throughout the catchment and, therefore, difficult to control and maintain. Although source controls are very effective in reducing runoff volumes and peaks, the protection of groundwater quality may restrict their application to less polluted stormwater originating on roof-tops, backyards and residential streets.

Centralized groundwater recharge facilities allow good operational control and maintenance. Such recharge basins generally have sufficient capacity to contain all the runoff from the area they drain during a storm of specified frequency. Sometimes, detention ponds or sedimentation basins are used in conjunction with infiltration basins so that suspended solids will settle out before the water is released into the basin (see Figure 3.12). Design of recharge storage facilities should account for the fact that lateral drainage through the basin wall is generally several times more rapid than vertical percolation. To prevent some water from staying at low points in the basin, a control drain should be installed to infiltrate water into the soil by means of trenches or vertical wells. The use of infiltration basins is particularly popular in areas where groundwater recharge is required. Such areas include coastal areas where urbanization may lead to saltwater intrusion and other areas where groundwater depletion would cause problems.

Without proper maintenance, storage facilities accumulate too much debris and become a community nuisance. On the other hand, well landscaped and maintained basins can be used as a park or recreation area. Such a project requires plants, trees and facilities capable of withstanding temporary inundation. Basin sides should be gently sloped to give a park-like appearance.

3.5.2 Storage facilities

Storage is perhaps the most frequently employed method of runoff control (APWA, 1981). Storage controls are implemented in the form of retention or detention facilities which can be designed as in-line or off-line reservoirs and underground tanks. Underground tanks which obviously require more resources for their design and construction are used in densely built-up areas.

Many of the above storage schemes can be implemented only in areas with adequate open space. This applies particularly to open reservoirs which may require large space. Several examples of detention facilities are shown in Figure 3.13 which illustrates schematically off-line detention (Figure 3.13a), on-line detention (Figure 3.13b), and on-line detention in a natural water course (Figure 3.13c). By using a system of storage facilities, it is possible to reduce drastically peak flows and use older existing systems to convey runoff from new developments.

In densely developed areas, and because of safety reasons and space limitations, stormwater storage facilities are often built underground. Such facilities include on-line and off-line storage tanks with regulated outlets. Large detention and retention facilities are often added to the existing sewer systems, in order to reduce pollutant loads discharged to the receiving waters. The design of such facilities depends largely on local conditions and sewer network conditions. Figures 3.14 and 3.15 show examples of an underground storage facility and of a facility which is a combination of underground storage with open reservoir.

It is a good practice to subdivide larger storage facilities into a number of cells. The entire facility is used only for the largest storms and runoff from more frequent storms is

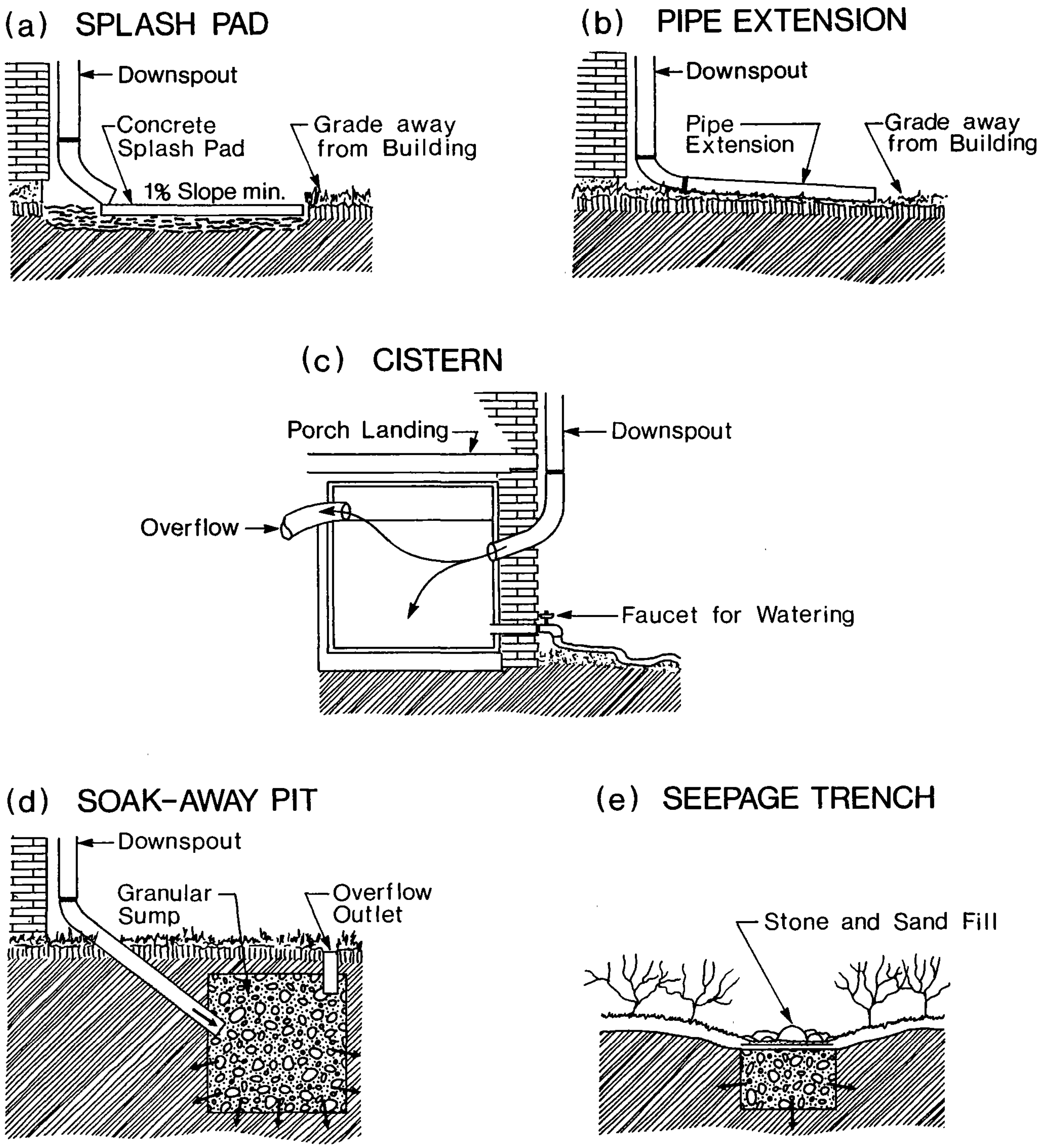


Fig. 3.11 Source control methods

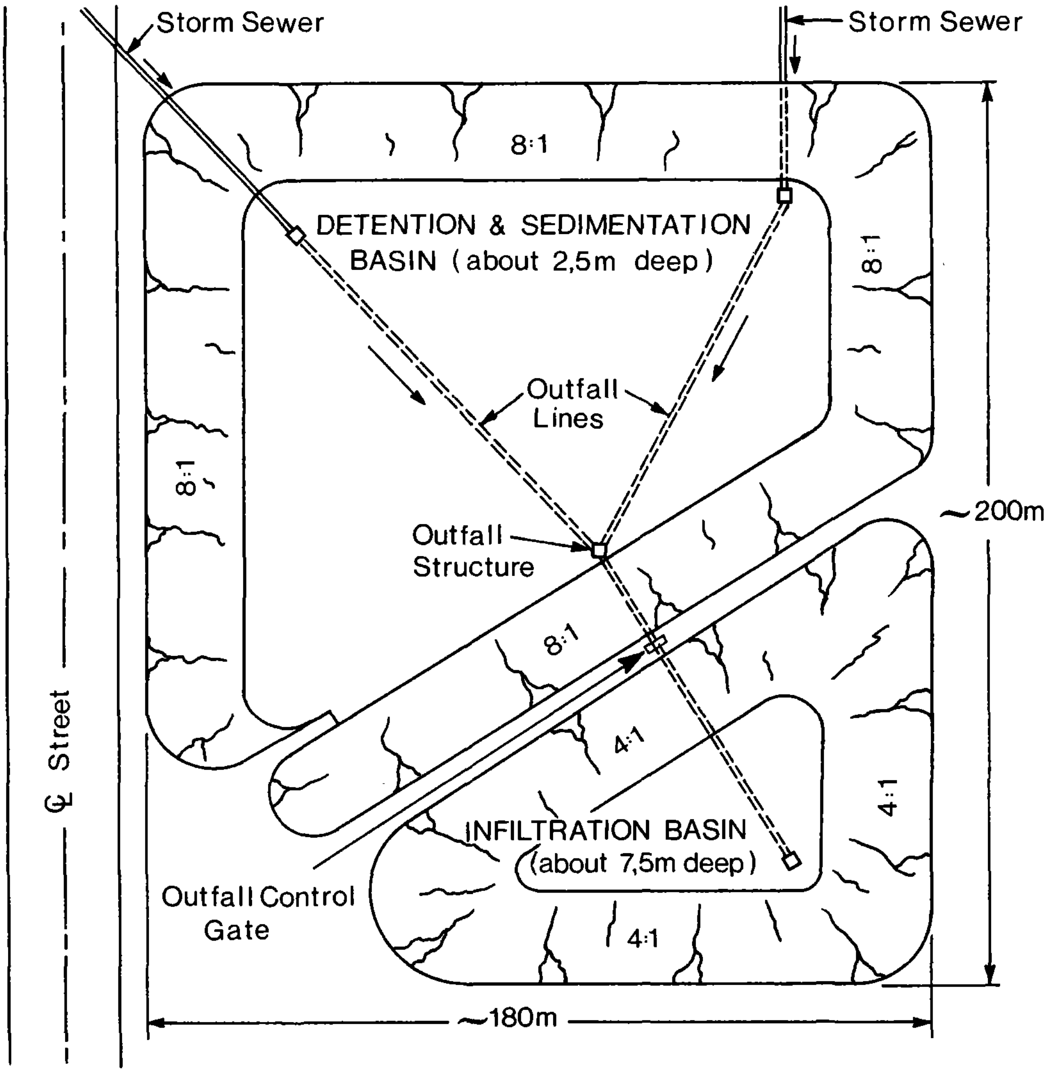


Fig. 3.12 Recharge basin

stored in a limited number of cells to reduce maintenance. For example, the storage facility in Figure 3.15 is designed so that runoff from more frequent storms is captured in the underground part and additional volumes from less frequent storms are retained in the open reservoir. More information on such facilities can be found in the literature (ATV, 1982a; ATV, 1982b; Lautrich, 1980).

On-line storage can be provided in the form of large-diameter sewer sections or by controls maximizing the utilization of storage in the existing sewer system. The latter measure has been adopted in a number of European and North American cities (Schilling, 1986).

In combined sewer systems, the problem of the first flush need to be considred in the design of layout of storage basins. Such layouts should attempt to capture the maximum quantity of pollutants which are often carried with the first flush (ATV, 1983b; Geiger, 1984).

3.5.3 Outlets for storage facilities

Outlet structures of surface detention ponds are designed according to the function of the storage facility. If the storage objective is "peak shaving" for rare storms (i.e., the return period of 100 years), a single weir or bottom outlet is usually satisfactory. For large ponds, this outlet is combined with an emergency spillway. This solution provides only little or no reduction of runoff peaks from more frequent, smaller storms. If the detention pond has as objective to reduce peak flows for a range of storms, multiple outlets are required. As an example, Figure 3.16 shows an outlet design whose lower orifice controls flows with return periods of 10 years, in order to avoid erosion problems in downstream creeks. The weir is

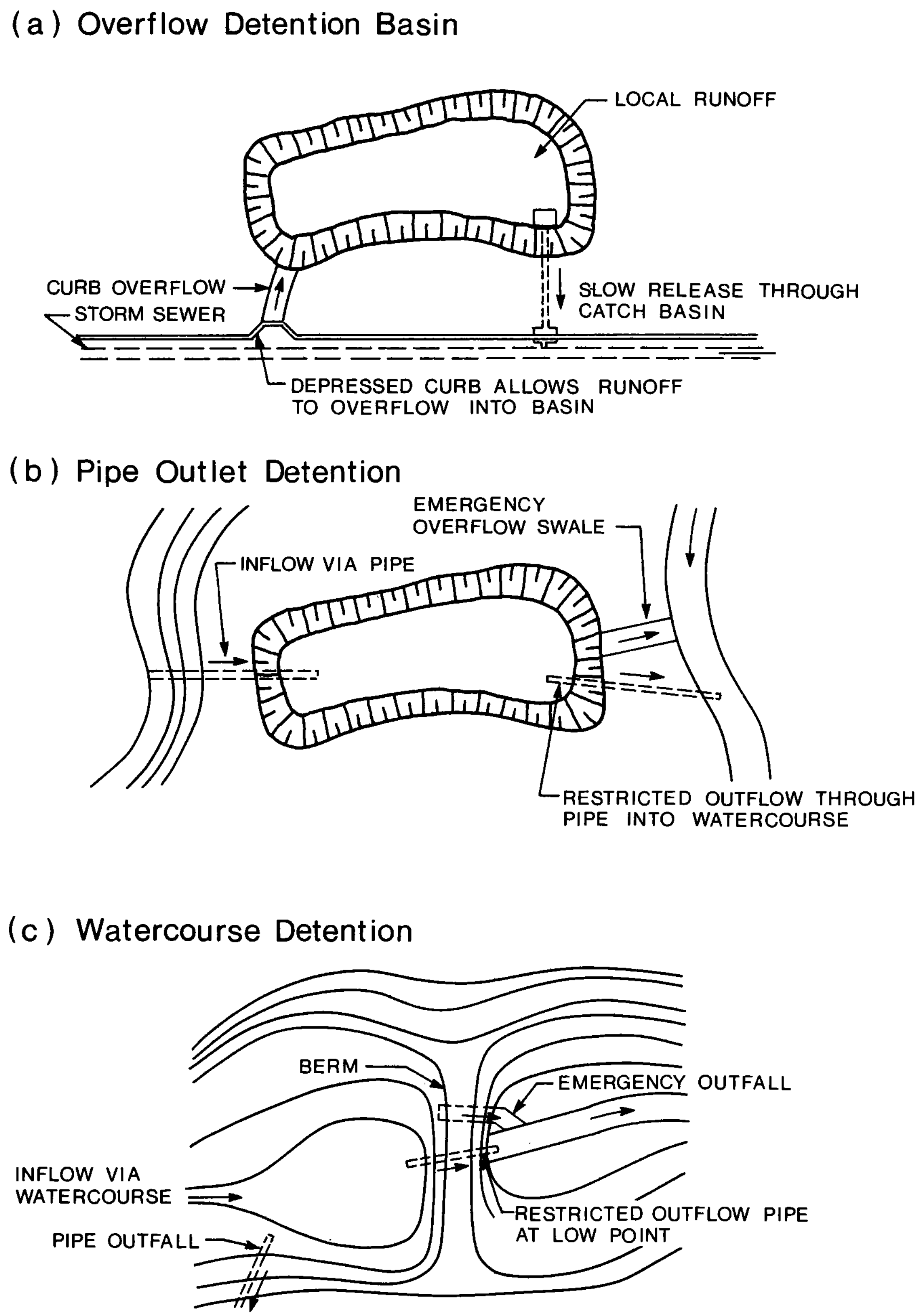


Fig. 3.13 Examples of detention basins

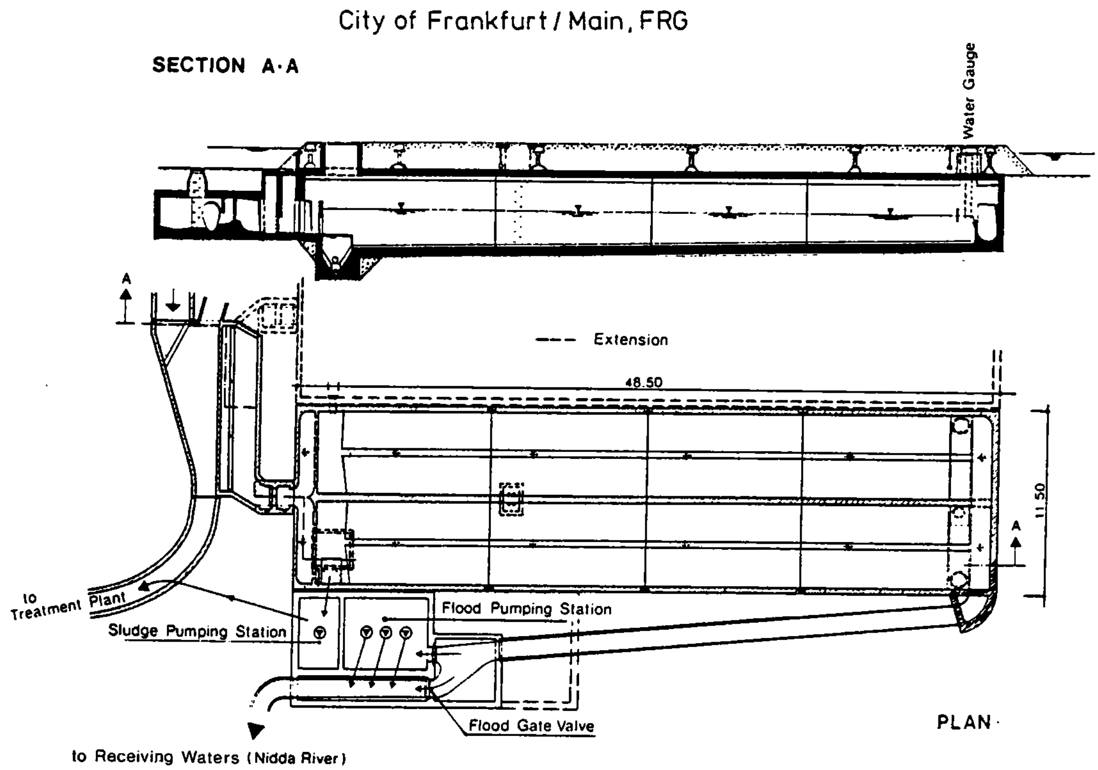


Fig. 3.14 Underground retention basin (reproduced by permission of Dorsch Consult, Munich, FRG)

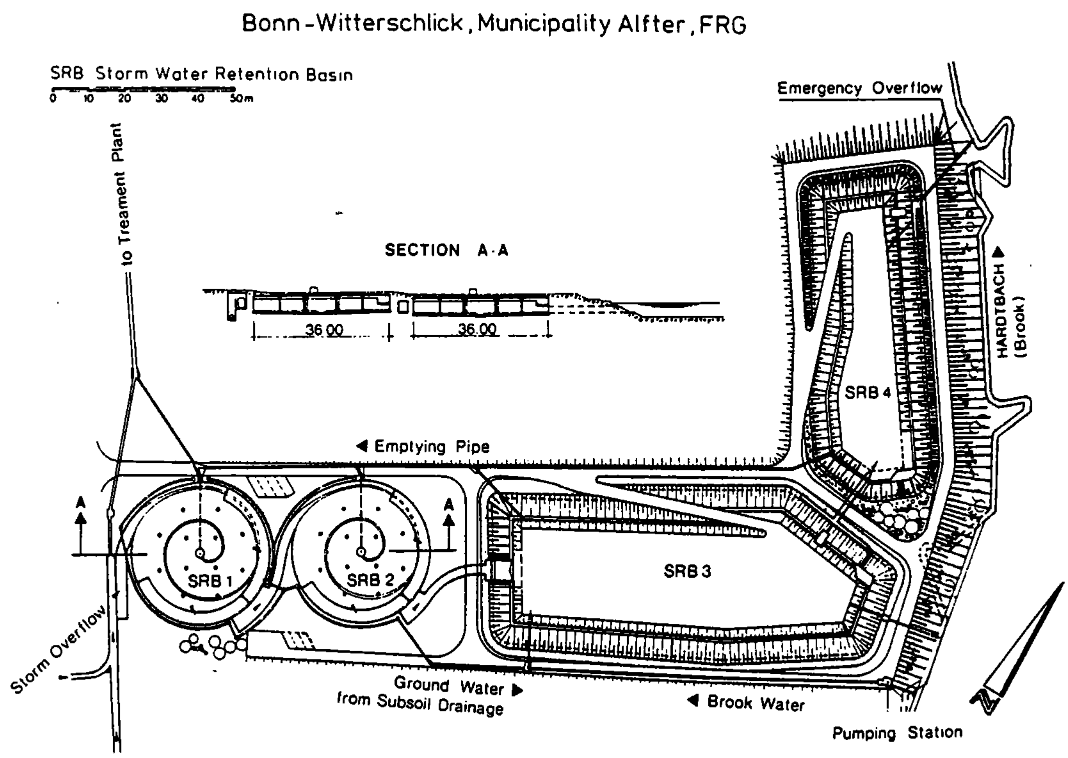


Fig. 3.15 Combination of underground and open reservoir detention basins (reproduced by permission of Dorsch Consult, Munich, FRG)

designed for reduction of the 100-year flood. For the maximum possible flood, an emergency spillway must be provided, usually over the embankment. A certain minimum size is required for the lower orifice in order to avoid clogging.

Outlets are also provided with trash racks with a large cross-section. For example, a New Jersey (USA) regulation recommends that retention outlets should be protected by removable devices with a surface area of at least 1 m^2. Openings in the trash racks should not exceed one-half the area of the retention outlet, if mesh screens are used, or one-third of the diameter of the outlet where bar screens are used. A concrete pad surrounding the retention outlet is also recommended to ensure that all water is evacuated.

Some multiple outlets are designed to meet a pollution control objective. The lower retention outlet provides a detention period by maintaining a small pond for settling of sediments. In this case, the smaller size of retention outlets makes them more susceptible to clogging by debris. A design frequently used in the U.S.A. consists of a perforated conduit surrounded by a gravel filter. The emergency spillway is formed by raisers.

Special orifice outlets are designed for very small runoff releases, usually required for underground storage or inlet controls. Several inlet control alternatives are considered to avoid or minimize clogging:

(a) Use of orifices of specific shapes. For example, diamond orifices in vertical walls were found less susceptible to clogging than circular orifices. Flow restrictions based on this finding are used in several Canadian municipalities (Townsend et al., 1981). Such restriction is achieved by installing special plastic inserts at the inlet of the conduit connecting the manhole with the main sewer.

(b) Use of flow restrictors based on the vortex principle. Flow passing through these devices will create a vortex with an open central air core.

Roof-top and parking lot storage is particularly useful in commercial and industrial areas, where buildings occupy a relatively large proportion of the developed area and building architecture provides for flat roof-tops. In this case, storage can be obtained by installing a control ring around the roof drain hopper as detailed in Figure 3.17. The roof storage depth has to be compatible with structural design of the building. In general, water depths greater than 80 mm should be avoided. Also, special considerations in waterproofing techniques are required. Parking lot storage requires provision of slopes and special restricted catch basins. Since these storage measures are associated with private property, the continued use of the roof-tops for storage and prevention of removal of control devices by owners are almost impossible to ensure.

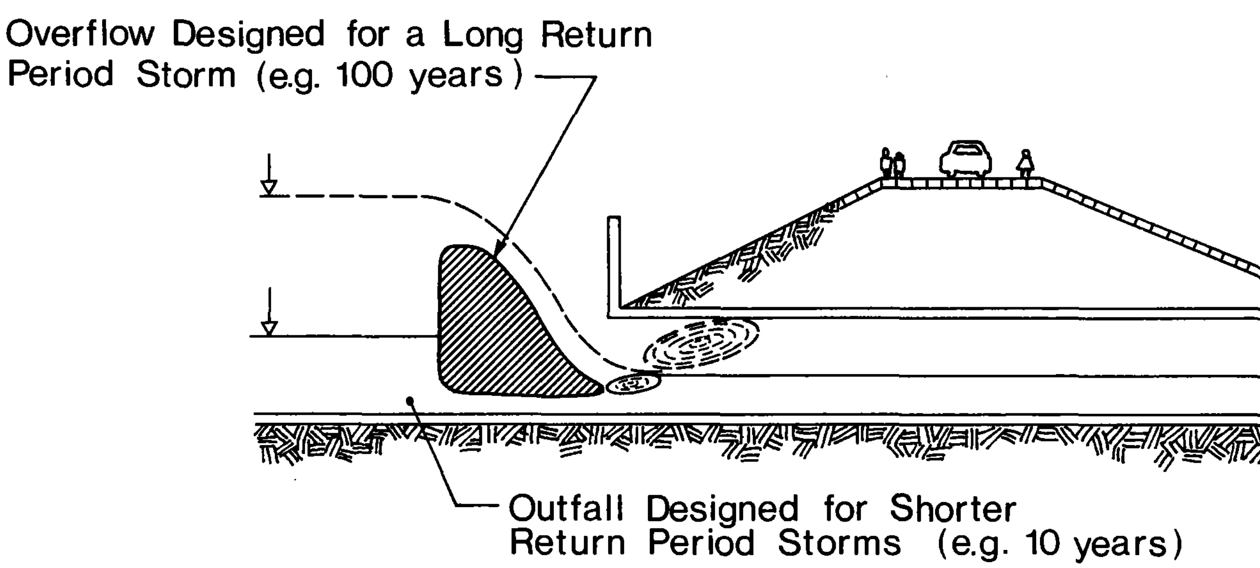


Fig. 3.16 Control outlet designed for two storm return periods

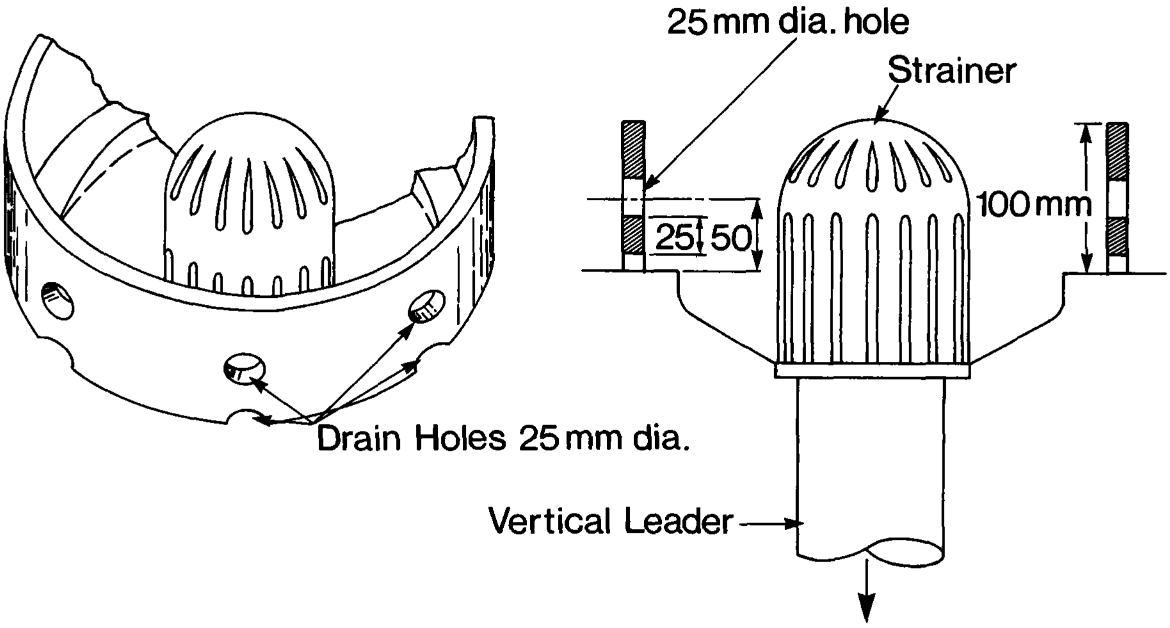


Fig. 3.17 Outlet control for roof storage

3.6 PUMPING STATIONS

Pumping stations are used in separate sewer systems to drain low-lying areas, in areas where tidal conditions at the outlet prevent gravity flow, or as an alternative to very deep sewers (see Figure 3.18). They are also frequently used in combined sewers or interceptors located below the treatment plant level.

There is a wide variety of pumping station designs. Station layouts depend on the type of pumps used. Commonly used pumps include pumps with horizontal or vertical shafts, and pumps located in wet wells (submerged) or in dry wells. Suction pipes of dry well pumps extend into wet wells, or into enlarged sections of incoming sewers. Centrifugal pumps driven by electric motors are particularly popular. In larger stations, it is a common practice to install an emergency diesel engine drive on at least one pump for protection during possible electric power failures. Additional information on pumps can be found elsewhere (ATV, 1982b).

Since stormwater inflow may vary significantly, many pumping stations include some storage volume in a suction sump or wet well. An automatic float switch is used to start or stop various pump units as water level rises or falls in the well. For pumping stations with flows of less than 100 l/s, this storage should be about equal to the volume of inflow over a period of about 2 or 3 minutes. In large pumping stations, such storage volume is reduced to an equivalent of 1,5 minutes of inflow.

Two typical layouts of pumping stations are shown in Figure 3.19. Figure 3.19a shows a station with the engine above the ground surface. If this is not possible, the whole pumping station can be underground as shown in Figure 3.19b.

It is also necessary to provide protection of pumps against large solids by means of trash racks at the end of the inlet conduit to the pump. The flow velocity through the trash racks is limited to a maximum of 1 m/s. It is also recommended to install mechanical cleaning systems at the inlet and hence the design has to account for head losses through the screens. Pumping stations are also provided with various types of check valves with controlled closure speed to avoid water hammer problems. In some installations pumps are located in large collecting channels.

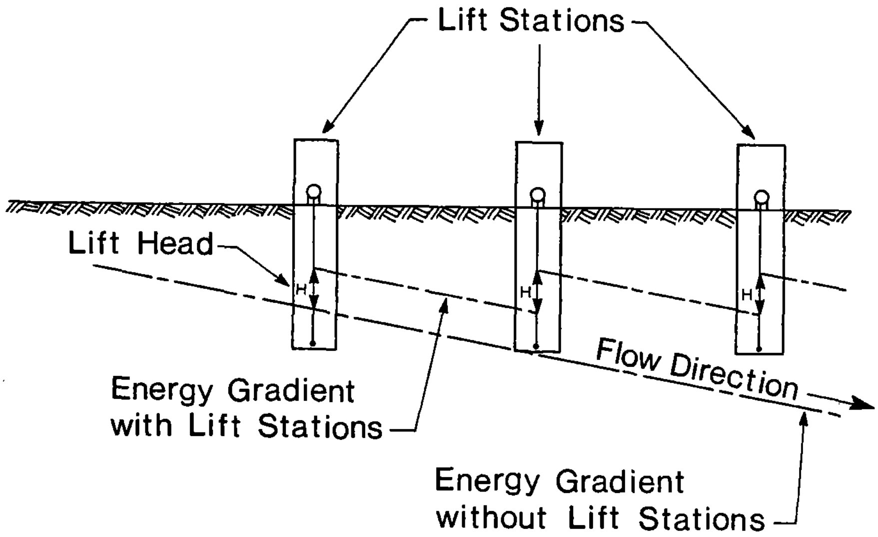
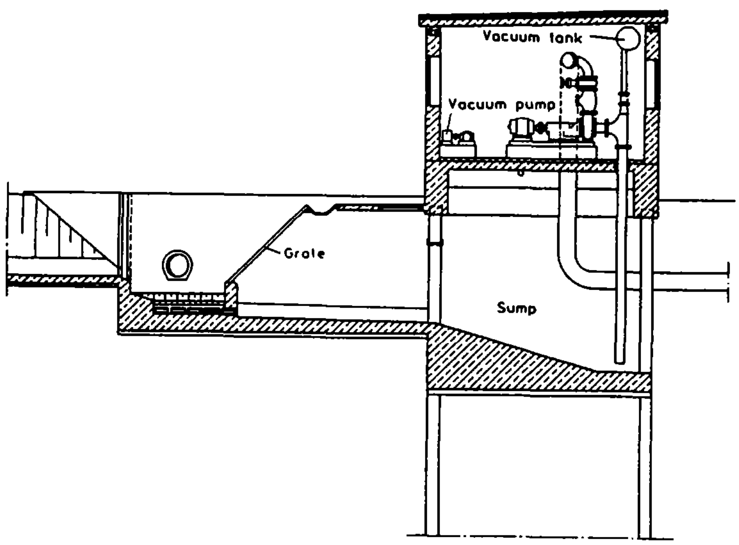


Fig. 3.18 Use of lift stations to replace deep sewers

a. Pumping station with a horizontal centrifugal pump and priming equipment



b. One-stage Archimedes screw pump

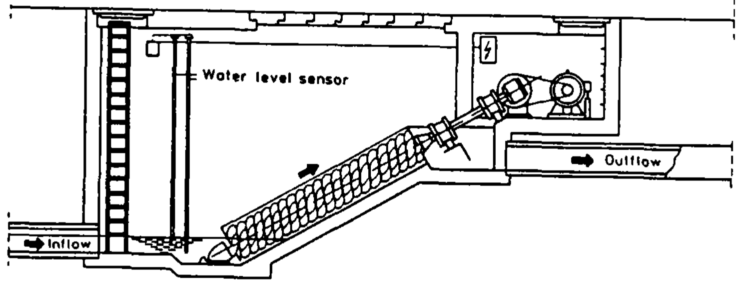


Fig. 3.19 Basic sewer pumping station layouts (ATV, 1982b)

4 Design parameters

4.1 BACKGROUND

Applications of general concepts and methodologies of drainage design discussed in Chapter 2 require the establishment of design parameters of the system under design. Such parameters specify the system performance with regard to various inputs. While the design concepts and methodologies are fairly general, the design parameters are site specific. Depending on the design objectives and the availability of supporting data, different design parameters may apply to each region, location, or project. Quite often, the basic drainage design parameters are established by a drainage authority (it can be a municipal, or a higher-level government) which issues drainage criteria and policies as further discussed in Chapter 9. In the absence of drainage criteria, the design parameters are established by the designer. For this purpose, the designer may have to examine various sources of supporting data, process such data, or even to undertake data collection programmes which are discussed in Volume II of this manual (Unesco, 1987).

The major topics discussed in this chapter include the planning horizon, the design period, catchment physical parameters, process parameters, rainfall data, water quality parameters, and infiltration and inflow of extraneous flows into sewers.

4.2 PLANNING HORIZON

The concept of the planning horizon, which was introduced earlier in Chapter 2, reflects the need to plan urban drainage facilities for the future development of the urban area. Such planning requires several types of information concerning not only the physical parameters of the system, but also the development trends, jurisdictional divisions, and financial arrangements. In drainage studies, the planning is best done on the watershed basis. As part of the watershed plan, a master drainage plan is prepared for the entire watershed, preferably at an early stage of the watershed development.

Considering the relatively long design life of drainage structures, it is necessary to consider development factors in the watershed being analyzed. Such factors include the population trends, comprehensive land use and zoning plans, and locations of future roads, airports and industrial areas which may affect the drainage design. Because of large uncertainties in projections of development trends and the resulting uncertainties in the need and utilization of physical facilities, the planning process is more an art than science.

The planning is undertaken at two levels - the short-term and long-term planning. In the short-term plan, drainage facilities are planned on the basis of the proposed development plans for the period from 5 to 10 years. Such short-term plans have to be compatible with the long-term plan. If needed, provisions are made for future expansion and extension of proposed drainage facilities. The long-term planning reflects the ultimate development of the watershed and is based on a longer planning period which may be as long as 25 to 50 years. To reduce uncertainties in the long-term plan, it is regularly updated, say every five years, to account for the actual development taking place and to reflect changes in technology and engineering practices.

4.3 DESIGN PERIOD FOR DRAINAGE PROJECTS

As discussed earlier in Chapter 2, the design period of drainage systems indicates the expected level of protection obtained under the proposed drainage systems. Depending on design objectives, this protection may apply to both flooding as well as water quality.

Sizing of storm sewers and drainage ditches is almost universally based on the concept of the design storm of a selected return period. In general, the selection of the design event return period is affected by the design life of structures involved, construction costs, and damage costs resulting from the system failure. Drainage structures are typically characterized by long design life and then the ideal design period should be the one for which the total annual drainage costs, defined as the sum of annual construction and drainage failure costs, are minimal. The procedure which would be used to select such a least-cost design period is schematically shown in Figure 4.1.

Most drainage projects do not warrant detailed analysis of the construction and failure costs and the selection of the design period is largely based on experience. Even under those circumstances, the designer should consider the probability of the system failure and the consequences of the failure in terms of damage costs. The probability P of a design event with the return period T occuring at least once in N years may be expressed as

$$P = 1 - (1 - \frac{1}{T})^N \tag{4.1}$$

Theoretical probabilities of failure for a given project life and design return period are shown in Figure 4.2. Considering the design return period equal to the project life (T = N), the resulting probabilities of failure vary from 0,75 to 0,63, depending on the selected design period. Depending on the consequences of the drainage failure, such levels of risk may or may not be acceptable. To evaluate the acceptability of such risks, one has to differentiate between the minor and major drainage systems which were briefly introducted in Chapter 2. Further comments on both systems follow.

The minor drainage system primarily reduces the incidence of inconvenience, caused by stormwater ponding, to both pedestrians and motorists. The consequences of its failure are not significant, provided that there is a properly functioning major drainage system. Therefore, the design periods of the minor system are relatively short, typically from 1 to 10 years.

The major drainage system is designed to convey runoff from infrequent storms with long return periods (up to 100 years), or other critical regional events. A properly designed, constructed, and maintained major drainage system greatly reduces the risk of loss of life and property damage caused by flooding of urban areas. Consequences of the major drainage system failure are quite severe and may include appreciable flood damages and even the loss of life. Consequently, major drainage systems are designed at least for a 50 year storm, or more often, for a 100 year storm or a regional storm defined by a local drainage authority.

Two additional points should be made with regard to the design periods. Firstly, it should be emphasized that an identical design period does not have to be maintained throughout the drainage system. Some parts of the system, whose failure would be particularly costly, may be designed for longer design return periods than others. Secondly, the choice of the return period for the minor system may have some implications for the capacity and costs of the major system.

As an example, a summary of typical design return periods which are used in the Canadian drainage practice is given in Table 4.1 (Ministry of the Environment and Environment Canada, 1980). Table 4.1 provides some guidance for the selection of design return periods. There are, however, some additional considerations to be made. In particular, longer design return periods are used for design of combined sewers to reduce the incidence of basement flooding, design of special structures (such as pumping stations), and for design of those parts of the drainage system which are not amenable to future relief.

The return periods given in Table 4.1 for storm sewers vary over a fairly wide range (2 to 10 years). It should be recognized, however, that the costs of storm sewer systems are little sensitive to the design return period. Several case studies indicated (WPCF, 1970a) that by doubling the design return period from 5 to 10 years, the costs increased by only 5 to 11%. Thus, it may be possible to provide a higher level of protection with a marginal increase in costs.

Finally, it should be mentioned that, as an alternative to the design event approach, a risk-based design approach has been proposed for urban drainage. Although such an approach is commonly used for design of large hydraulic structures, it has not gained much acceptance in drainage practice. The description of the risk-based design of storm sewers is given elsewhere (Tang et al., 1975; Yen and Tang, 1976).

4.4 CATCHMENT PHYSICAL PARAMETERS

Physical characteristics of the catchment are required to establish drainage patterns in the catchment, linkages of various conveyance elements, and numerical values of process parameters. Because the computations often involve comparisons of pre-development and post-development flows, physical characteristics are needed for both states of the catchment

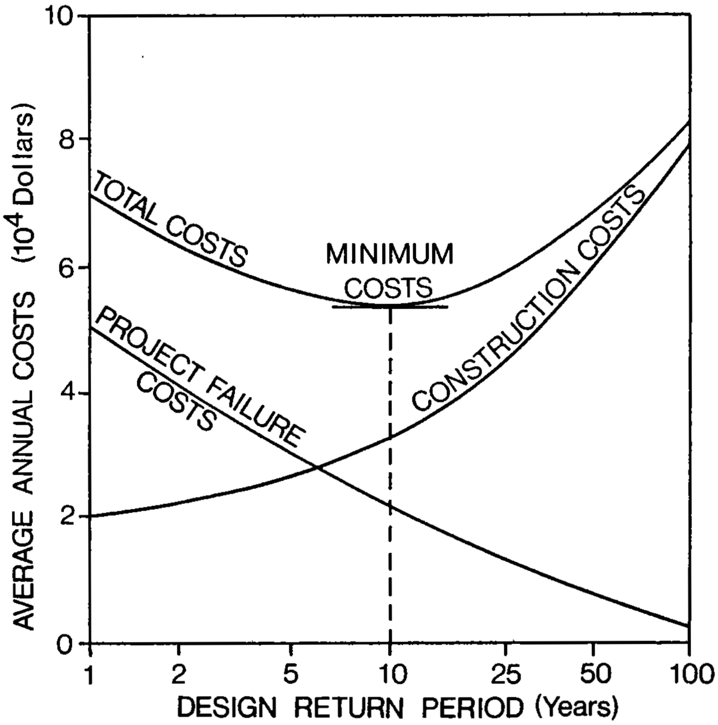


Fig. 4.1 Hypothetical average annual costs for various design periods

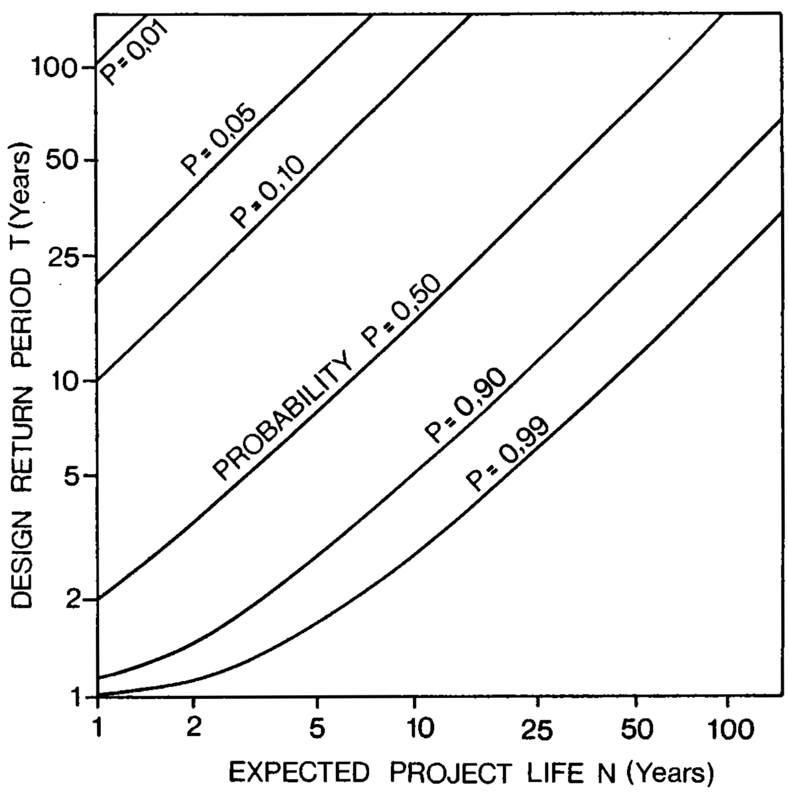


Fig. 4.2 Probability of failure for specified project life and design return period

development. The types of information required include catchment topography, drainage plans, and soil maps, as further discussed below and also in Chapter 4 of Volume II (Unesco, 1987).

Table 4.1 Typical Design Return Periods Used in the Canadian Drainage Practice (Ministry of the Environment and Environment Canada, 1980)

Structure	Drainage System	Design Return Period (Years)
Culverts - Driveways		2* - 5
- Local roads (span <6 m)		10
Storm sewers (no surcharging)	Minor	
- Residential areas		2* - 5
- High-value districts, commercial areas		5 - 10
Elements of the major drainage system (swales, streets, channels, etc.)	Major	100
- Culverts and bridges		100

* Shorter return periods (e.g., one year) can be used under special circumstances.

Site maps are required showing the entire watershed and the area under consideration; details of topography including contours, watercourses, wooded areas, rock outcrops, and marshes; details of existing and proposed land use; and, the existing and proposed major drainage channels. Using such maps, it is possible to establish the study area drainage boundaries, general drainage patterns in the area, surface slopes, and the total catchment area. The total catchment area is further subdivided into impervious and pervious parts. In the impervious part, an impermeable surface cover does not allow infiltration of water into the ground. Typical examples of impervious elements include paved streets, sidewalks and yards; roofs; and, rock outcrops. The catchment imperviousness is usually expressed as the ratio of the impervious area to the total catchment area.

The impervious area can be further divided into the effective and noneffective areas. The former one refers to the impervious elements which drain directly into transport elements (sewers). The latter areas represent impervious elements which drain onto pervious elements, or have no drainage outlet.

The pervious area is sometimes divided into the contributing and noncontributing parts. The contributing areas drain into the drainage transport system; the noncontributing part has no drainage outlets.

All the above listed areas need to be delineated and characterized in terms of the area, plan geometry, and surface slopes. For various reasons, the catchment studied is usually subdivided into a number of subcatchments and then the above information needs to be determined for each of these subcatchments.

All the existing drainage channels and conduits in the catchment need to be characterized in terms of the cross-sections, slopes, lengths, linkages, and installations.

4.5 PROCESS PARAMETERS

In deterministic hydrological calculations, it is assumed that the relationships between many interactive factors affecting the water balance can be defined analytically and the numeric values used to quantify the movement and storage of water are called parameters. Although such parameters can be quite numerous, only a few of these are used in typical urban runoff computations. Such parameters include infiltration rates, depression storage, flow roughness coefficients, and runoff coefficients. Numeric values of these parameters are obtained by field measurements, calibrations, or most frequently, by transposition from other similar catchments. Examples of numeric values of various process parameters, as used in hydrological computations, are listed in Chapter 5 and further discussed in Chapter 4 of Volume II (Unesco, 1987). The discussion that follows concentrates on general aspects of the use of such data in drainage design.

4.5.1 Infiltration rates

A proper representation of infiltration in a catchment is a complex task which often extends beyond the scope of hydrologic concepts and methods employed in urban drainage design. Most often, infiltration rates are evaluated from soil physical properties which are obtained from soil maps. For this purpose, soils are classified according to their drainage properties and the corresponding infiltration rates are selected from the literature. In urban applications in North America, the U.S. Soil Conservation Service (SCS) classification of soils into four hydrologic groups A-D is fairly common (U.S. Department of Agriculture, 1972). The knowledge of the hydrologic soil group and initial soil moisture conditions is sufficient to estimate the infiltration rates based on the SCS method and to select the appropriate parameter values for Horton's equation, or to select a runoff coefficient for pervious areas.

In physically based approaches, such as those described by the Holtan and Green-Ampt equations, more information on soils is required. In particular, the division of the soil profile into various horizons needs to be known, together with the soil porosity and various types of water storage (see Chapter 5).

Simplified approaches to infiltration may be acceptable for urban catchments in which the generation of runoff is mainly controlled by impervious elements. In other cases, the most comprehensive approach which can be supported by the available data should be used. Such approaches are generally physically based and involve continuous simulation of water storage in soils. Detailed listings of infiltration rates are given in Section 5.5.2.4.

4.5.2 Depression storage

Depression storage accounts for rainwater trapped on the catchment surface in minute depressions that does not run off or infiltrate into the soil. Generally, it represents a combination of several hydrologic abstractions. For drainage design, the depression storage is of secondary importance. Typical values are 1 mm for impervious areas and 5 mm for pervious areas. For a complete listing of values, see Section 5.5.2.3.

4.5.3 Roughness of transport elements

In flow routing calculations, the roughness of individual elements needs to be determined. In urban applications, this is estimated most often by means of the Manning roughness coefficient, n. For concrete conduits, concrete-lined channels, and impervious overland flow planes, the value of n = 0,013 is widely used. Manning's n values for overland flow on grassed areas vary from 0,2 to 0,35. Extensive listings of n-values for various conveyance elements can be found in Chapter 6.

4.6 RAINFALL DATA FOR DRAINAGE DESIGN

Rainfall data represent the most important input to runoff computations. Consequently, much attention in drainage design focuses on the selection and processing of rainfall data. Various procedures used for this purpose are discussed in this section. The main topics discussed include the sources of rainfall data, point vs. areal rainfall, frequency analysis of point rainfall, and design storms. The methods of collection of rainfall data are discussed in Chapter 2 of Volume II (Unesco, 1987).

4.6.1 Sources of rainfall data

Rainfall data are typically collected by national hydrometeorological agencies (or similar agencies), which have been established in most countries, in a network of stations distributed throughout the country. Where the density of such a network and the locations of individual stations do not meet the needs of drainage design, the local drainage authority may have to establish its own monitoring network using the information given in Chapter 2 of Volume II (Unesco, 1987). In fact, it is quite common in some countries that large metropolitan municipalities operate their own network of rain gauges.

Rainfall data are distributed in various forms, most often as hourly rainfall depths, daily rainfall records, or depth-duration-frequency curves. For drainage design, hourly rainfall data may be adequate for the planning stage. In detailed design, however, short-interval (5 to 15 minutes) data are needed, usually in the form of rainfall intensities (Geiger, 1984). Such intensities are obtained by analysis of rainfall records using the shorter intervals for small drainage areas and the longer intervals for larger areas. Some agencies distribute processed rainfall data in the form of depth-duration-frequency curves, intensity-duration-frequency curves, or design storm distributions. For example, the Comite Interafricain d'Etudes Hydrauliques (Puech and Gonni, 1984) produced an atlas of rainfall depth-duration-frequency curves for 87 urban locations in 15 countries of West and Central Africa. An example of such curves is shown in Figure 4.3.

In the absence of local rainfall data, desing data may be obtained through transposition or interpolation from other areas. Although individual rain gauge records may not be

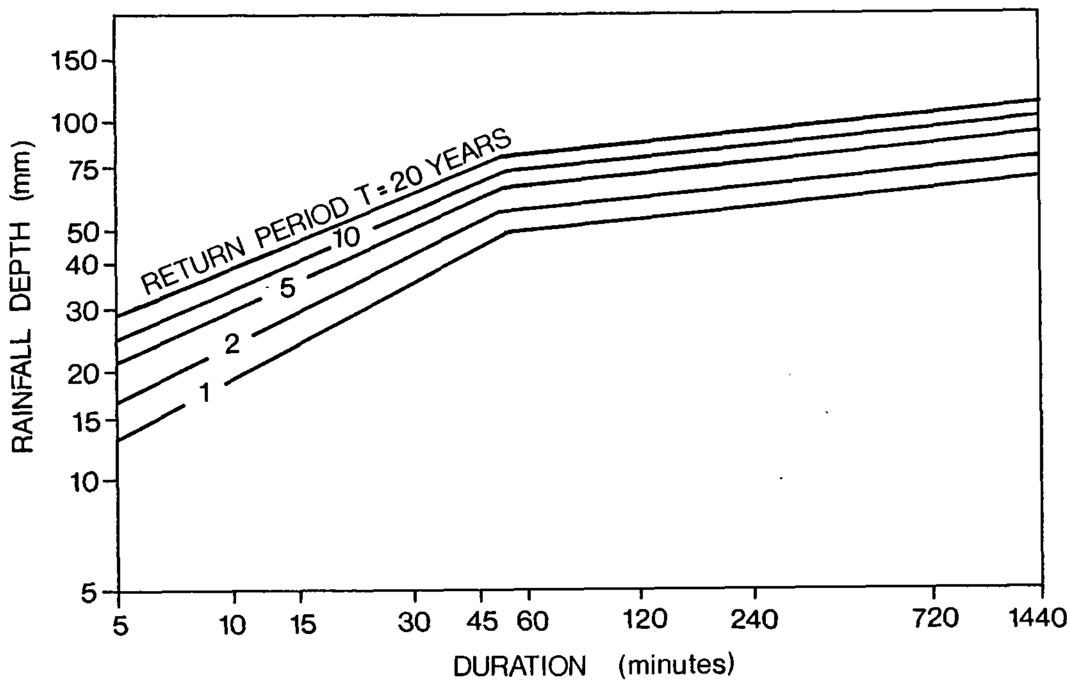


Fig. 4.3 Depth-duration-frequency for Yaounde, Cameroon (after Puech and Gonni, 1984)

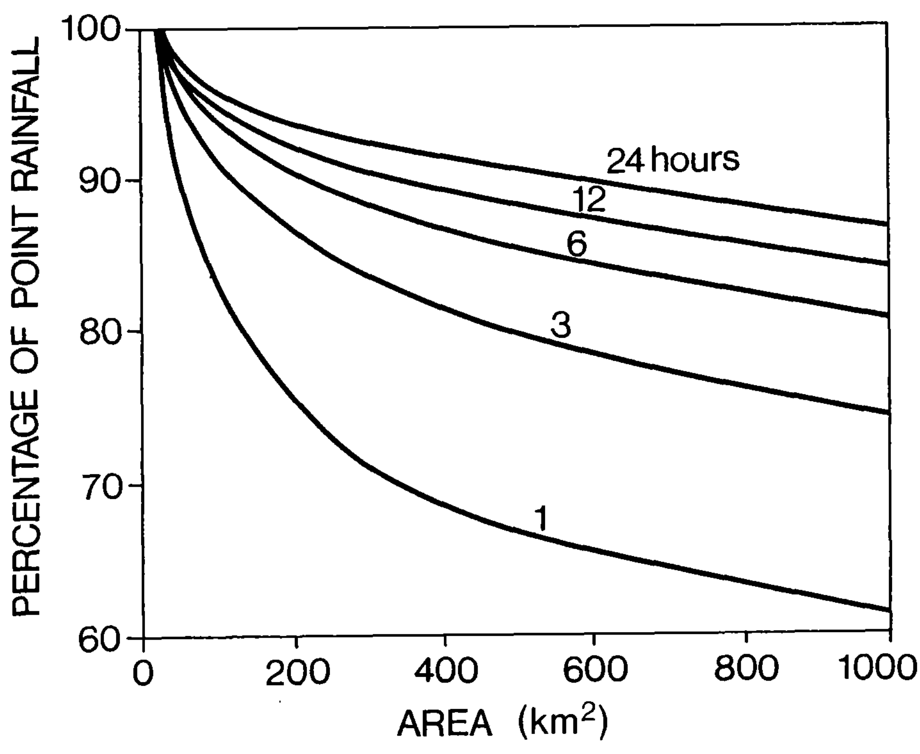


Fig. 4.4 Depth-area rainfall reduction curves (after WMO, 1974)

representative of rainfall over large areas, the general characteristics of rainfall are quite conservative in space and vary smoothly in non-mountainous areas (Pugsley, 1981). Thus such parameters as frequency of rainfall above a given value may be transposed or interpolated to nearby areas without data. Ideally, this task is carried out by a national hydrometeorological service which distributes standardized rainfall data for various climatic regions of the country.

Another important consideration in the evaluation of existing rainfall records is the record length. Excessive extrapolations of limited data, leading to large uncertainties, should be avoided. For example, the Canadian Atmospheric Environment Service (AES) undertakes annual maximum rainfall analysis only for records longer than seven years and the data are extrapolated for return periods up to twice as long as the record length (Pugsley, 1981).

4.6.2 Point vs. areal rainfall

In small-scale drainage projects, one may assume the validity of the point rainfall for the entire drainage area. It has been suggested that a single gauge be taken as representative of the rainfall over small basins up to 25 km^2 in area (WMO, 1974). In urban areas which are generally characterized by fast response, a more stringent criterion may be desirable. For areas larger than 25 km^2, the areal rainfall will be smaller than the point rainfall because of decrease in rainfall intensity with distance from the storm centre. To account for this decrease, reduction factors which depend on the basin area are applied to the point rainfall to obtain the average areal rainfall. Reduction factors recommended by WMO (1974) are shown in Figure 4.4. These factors depend on local climatological conditions and, therefore, whenever possible, should be derived from local data.

4.6.3 Frequency analysis of point rainfall

Frequency analysis is used to derive rainfall depth-duration-frequency data from which the Intensity-Duration-Frequency (IDF) curves are produced. The IDF curves indicate the probability of exceedance of rainfall of certain duration and intensity. In drainage design, the IDF curves represent the most important form of rainfall data. These curves are used in computations of runoff peaks by the rational method, or they can be used to develop certain types of design storms for runoff simulations. Because of the importance of the IDF curves, a detailed example of the development of such curves is given below. Similar procedures are discussed in Chapter 7 of Volume II (UNESCO, 1987) and in other handbooks (ATV, 1983b; WPCF, 1970a).

The preparation of IDF curves consists of two interrelated tasks, abstraction of data from the rainfall record and the frequency analysis of the abstracted data.

For frequency analysis, two types of rainfall data series may be used. Preferably, an annual series consisting of annual rainfall maxima for various durations, typically from five minutes to 24 hours, is used. Such an analysis is expedient (the data series is rather limited) and there is a theoretical basis for extrapolation of the annual series beyond the record length.

The other type of data series is referred to as the partial duration series and contains all large rainfall amounts above certain thresholds selected for individual durations. Thus for individual years, not only the annual maximum but possibly also other values are included in the analysis. The partial duration analysis appears to be preferable for short records, or where return periods shorter than two years are of interest.

In the frequency analysis, the abstracted large rainfall depths are ranked and assigned probabilities using various plotting position formulas (Chow, 1964). For the annual series, the Weibull's formula is generally recommended in the following form:

$$P = \frac{m}{N+1} \tag{4.2}$$

where P is the probability, m is the rank (the largest value corresponds to m = 1), and N is the record length in years.

For a partial duration series, a different position formula is sometimes used in the following form (Chow, 1964):

$$P = \frac{m}{N} \tag{4.3}$$

where N is the number of years, and only the top N items are considered. It should be emphasized that both equations 4.2 and 4.3 give almost identical results in the middle of the distributions but differ near the tails of the distributions.

The results of the annual series analysis can be modified to make them compatible to those obtained from the partial duration series by using the coefficients given in Table 4.2.

Table 4.2 Relationship Between Annual and Partial Series (Pugsley, 1981)

	Return Period (years)			
	2	5	10	>10
(Partial series value)/(Annual series value)	1,13	1,04	1,01	$\backsim$1,00

Having established the plotting positions, the abstracted rainfall amounts can be plotted either on the Gumbel probability paper (the annual series), or on a semi-logarithmic paper (ATV, 1983a). In the latter case, the probabilities (or the return periods) are plotted on the x-axis in the logarithmic scale and the rainfall amounts (or intensities) on the y-axis in the linear scale. Finally, the experimental data are approximated by a straight line which is used for further extrapolations or interpolations. The line fitting is accomplished either graphically or mathematically. One of the mathematical fitting procedures is demonstrated in the example given below.

4.6.4 Example of intensity-duration-frequency analysis

The following example of the IDF analysis has been adopted from a report of the (Canadian) Atmospheric Environment Service (Pugsley, 1981). Other agencies may recommend different procedures of equal merit.

Many attempts have been made to derive theoretical or empirical relationships which would describe well the probability distributions of hydrological variables. Such procedures are described in various standard texts on hydrology (Sokolov et al., 1976). The discussion below is limited to the asymptotic extremal distribution, also known as the Fisher-Tippet Type I distribution, which can be applied using the procedure developed by Gumbel (1958).

It can be shown that the hydrological frequency function for extreme rainfall can be described as

$$X_T = \bar{X} + KS_X \tag{4.4}$$

where $\bar{X}$ is the mean and S_X is the standard deviation of the sample of extreme rainfall, and X_T is the rainfall amount which is equalled or exceeded on an average once in T years. K is the frequency factor which can be expressed in terms of the return period T and the number of years of record, N. The values of K can be obtained from standard tables, such as the one reproduced in Table 4.3 (Pugsley, 1981).

Equation 4.4 is used to calculate the rainfall amount X_T of a return period T. For that purpose, the mean and standard deviation of the annual series are calculated first and the value of K, for the return period T and the record length N, is read from Table 4.3. If one needs to determine the return period of a rainfall value X_T, Equation 4.4 is used to calculate K, and the corresponding T is read from Table 4.3, i.e., for given $\bar{X}$, S_X and N. The return period T can also be determined from frequency plots on extreme (Gumbel) probability paper. If the plotting positions are calculated from Equation 4.2 then Equation 4.4 plots as a straight line on the Gumbel probability paper.

An example of the computation of the IDF curves is presented in Table 4.4 (Pugsley, 1981). Further explanations follow.

In the upper part of Table 4.4, annual maximum rainfall amounts are listed for nine durations from five minutes to 24 hours, and the record length of 12 years. For each duration, the mean and standard deviation were calculated. Subsequently, the rainfall amounts for various durations and return periods were calculated from Equation 4.4 by substituting for $\bar{X}$, S_X and K. The values of K were determined from Table 4.3 for N = 12 and return periods varying from 2 to 100 years.

In the bottom part of the table, rainfall amounts were converted into intensities ($i = h/\Delta t$, where i is the rainfall intensity, h is the rainfall amount, and Δt is the corresponding duration in hours). The calculated rainfall intensities were plotted in Figure 4.5 and approximated by smooth curves. For simplicity, the curves for T = 2 and 5 were not converted to partial series results (see the coefficients in Table 4.2) and, although the record length was adequate only for return periods up to 25 years, two IDF curves for T = 50 and 100 were also included.

Table 4.3 K Factors for the Gumbel Distribution (After Pugsley, 1981)

N (yr)	K Factors: Return Period T (years) 2	5	10	25	50	100
5	-0,116	1,313	2,260	3,456	4,343	5,224
6	-0,122	1,229	2,124	3,254	4,093	4,925
7	-0,127	1,169	2,026	3,110	3,914	4,712
8	-0,130	1,123	1,953	3,001	3,779	4,551
9	-0,133	1,087	1,895	2,916	3,673	4,425
10	-0,136	1,058	1,848	2,847	3,587	4,323
11	-0,138	1,034	1,809	2,789	3,516	4,238
12	-0,139	1,013	1,777	2,741	3,456	4,166
13	-0,141	0.996	1,748	2,699	3,405	4,105
14	-0,142	0,981	1,724	2,663	3,360	4,052
15	-0,143	0,967	1,703	2,632	3,321	4,005
16	-0,144	0,955	1,683	2,603	3,286	3,963
17	-0,145	0,945	1,666	2,578	3,255	3,926
18	-0,146	0,935	1,651	2,556	3,227	3,893
19	-0,147	0,927	1,637	2,535	3,202	3,863
20	-0,148	0,919	1,625	2,517	3,179	3,836
21	-0,148	0,911	1,613	2,500	3,158	3,811
22	-0,149	0,905	1,603	2,484	3,138	3,788
23	-0,150	0,899	1,593	2,470	3,120	3,766
24	-0,150	0,893	1,584	2,457	3,104	3,747
25	-0,151	0.888	1,575	2,444	3,089	3,728
26	-0,151	0,883	1,568	2,433	3,074	3,711
27	-0,151	0,878	1,560	2,422	3,061	3,695
28	-0,152	0,874	1,553	2,412	3,048	3,681
29	-0,152	0,870	1,547	2,402	3,037	3,667
30	-0,153	0,866	1,541	2,393	3,026	3,653
31	-0,153	0,863	1,535	2,385	3,015	3,641
32	-0,153	0,859	1,530	2,377	3,005	3,629
33	-0,153	0,856	1,525	2,369	2,996	3,618
34	-0,154	0,853	1,520	2,362	2,987	3,608
35	-0,154	0,850	1,515	2,356	2,979	3,598
36	-0,154	0,848	1,511	2,349	2,971	3,588
37	-0,155	0,845	1,507	2,343	2,963	3,579
38	-0,155	0,843	1,503	2,337	2,956	3,570
39	-0,155	0,840	1,499	2,332	2,949	3,562
40	-0,155	0,838	1,495	2,326	2,943	3,554
41	-0,155	0,836	1,492	2,321	2,936	3,547
42	-0,156	0,834	1,489	2,316	2,930	3,539
43	-0,156	0,832	1,485	2,311	2,924	3,533
44	-0,156	0,830	1,482	2,307	2,919	3,526
45	-0,156	0,828	1,479	2,303	2,913	3,519
46	-0,156	0,826	1,477	2,298	2,908	3,513
47	-0,156	0,824	1,474	2,294	2,903	3,507
48	-0,157	0,823	1,471	2,291	2,898	3,502
49	-0,157	0,821	1,469	2,287	2,894	3,496
50	-0,157	0,820	1,466	2,283	2,889	3,491

If no short-duration rainfall data are available and data transposition or interpolation is unfeasible, it is sometimes possible to estimate the short-duration rainfall statistics from other available data. Such data include the mean annual precipitation, the mean annual number of days with rain, and the mean annual number of thunderstorm days. WMO (1974) gives a summary

Table 4.4 Example of Calculation of Intensity-Duration-Frequency Curves

Year	Durations								
	Rainfall Amounts (mm)								
	5 min	10 min	15 min	30 min	1 hr	2 hr	6 hr	12 hr	24 h
1968	5,6	7,6	7,9	7,9	9,9	16,5	30,5	36,1	36
1969	5,1	9,1	13,7	15,5	15,5	15,5	34,3	41,1	41
1970	3,8	5,6	6,6	8,4	10,9	19,6	22,6	24,1	31
1971	2,5	3,6	4,8	7,6	11,7	18,8	20,8	21,1	23
1972	7,6	7,9	8,4	9,7	12,2	17,0	25,7	31,0	39
1973	6,9	8,9	10,4	11,7	16,8	23,6	27,9	30,0	36
1974	2,3	3,3	4,3	5,3	6,3	9,1	23,4	29,5	41
1975	6,3	9,7	10,2	10,9	15,5	16,8	28,4	28,4	33
1976	2,8	3,8	5,6	8,4	10,2	13,5	19,3	20,3	20
1977	6,3	6,3	6,3	9,9	14,0	15,5	18,3	33,3	33
1978	14,5	15,6	17,8	24,8	27,3	27,3	27,3	36,3	50
1979	3,9	4,8	5,4	7,5	8,1	13,4	15,8	22,2	26
Mean Extreme (N=12)	5,6	7,2	8,4	10,6	13,2	17,2	24,5	29,4	34
Standard Deviation	3,2	3,3	3,9	4,9	5,2	4,6	5,2	6,3	8
Return Period (yr)	Rainfall Amounts (mm)								
2	5,2	6,7	7,9	9,9	12,5	16,6	23,8	28,6	33
5	8,8	10,6	12,4	15,6	18,5	21,9	29,8	35,9	42
10	11,3	13,1	15,3	19,4	22,4	25,4	33,8	40,7	49
25	14,3	16,3	19,0	24,2	27,5	29,8	38,9	46,8	57
50*	16,6	18,7	21,8	27,7	31,2	33,0	42,6	51,3	63
100*	18,8	21,0	24,5	31,2	34,9	36,3	46,3	55,8	68
Return Period	Rainfall Intensities (mm/hr)								
2	62,4	40,2	31,6	19,8	12,5	8,3	4,0	2,4	1
5	105,6	63,6	49,6	31,2	18,5	11,0	5,0	3,0	1
10	135,6	78,6	61,2	38,8	22,4	12,7	5,6	3,4	2
23	171,6	97,8	76,0	48,4	27,5	14,9	6,5	3,9	2
50*	199,2	112,2	87,2	55,4	31,2	16,5	7,1	4,3	2
100*	225,6	126,0	98,0	62,4	34,9	18,2	7,7	4,7	2

* Extrapolations to these long return periods are inappropriate but were included for demonstrative purposes.

of such techniques. When applying these techniques, one should be aware of possible large uncertainaties in the results.

IDF curves are commonly determined for all stations in the network and mapped. Such maps are then used to interpolate the IDF statistics for locations without data.

IDF curves are used extensively in urban drainage design. Their primary use is to provide a rainfall input for the rational method or similar empirical formulae (see Chapter 8). Such an input is typically a constant-intensity rainfall of a certain duration and return period. For example, if the time of concentration is one hour (= duration), and the design return period is ten years, one obtains the design rainfall from Figure 4.5 as i = 22,7 mm/hr. This design value is valid only for the location for which the IDF curves in Figure 4.5 were produced.

4.6.5 Design storms

The design storm concept was introduced into urban drainage design about 25 years ago to replace constant-intensity design rainfalls which were adequate only for steady-state computations of runoff peak flows. As runoff computations progressed from empirical formulae for the peak flow to the hydrologic synthesis based on time-varying losses, it became necessary to

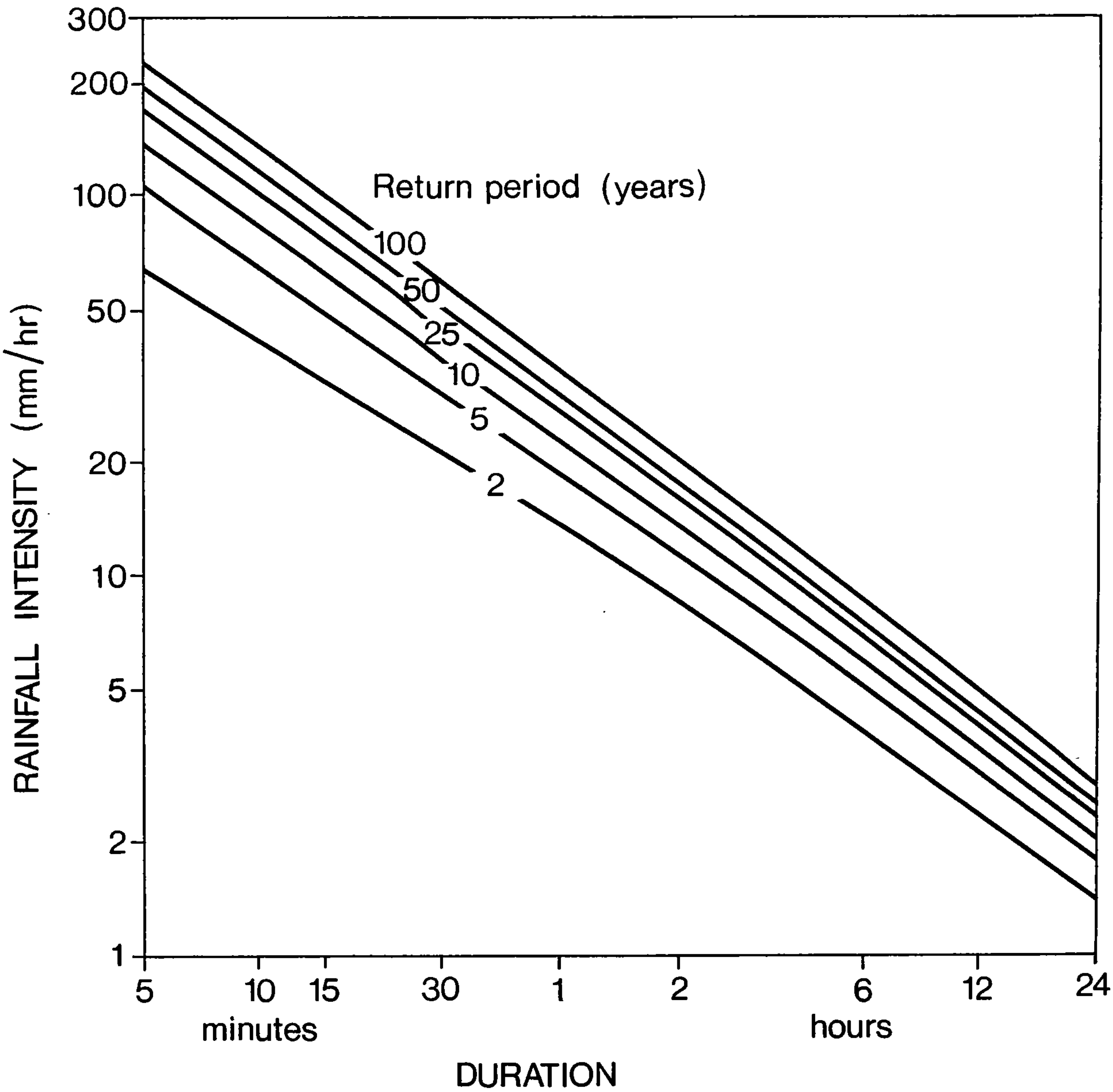


Fig. 4.5 Intensity-duration-frequency curves

consider time-varying rainfall inputs referred to as design storms. Such storms are typically derived by synthesizing general characteristics of a large number of actual events.

In the design storm concept, an assumption is made that the return period of the calculated runoff event is identical to that of the design storm. Although this assumption and some other aspects of design storms have been criticized by many researchers (McPherson, 1975; Marsalek, 1978; Geiger, 1984), the design storm concept is widely used in the engineering practice and can produce good results within its domain of applicability. Design storms can be developed expediently from local rainfall data and used economically in conjunction with event runoff models. Common design storms which are discussed below have been found adequate (Arnell, 1982) for sizing of storm sewers and drainage ditches. Their application, however, should not be extended to more complex design aspects, such as storage design, comparisons of pre-development and post-development flows, water quality considerations, and real-time operation of combined sewer systems. For these complex problems, it may be required to use actual rainfall data in conjunction with continuous simulation models, or to develop special rainfall inputs.

Design storms and their ability to produce design runoff peaks are difficult to evaluate. Attempts have been made to accomplish such evaluations through comparisons of runoff peak frequency curves produced from simulations with actual and design storms (Marsalek, 1978; Arnell, 1982; Geiger, 1984). The results of such studies are not fully conclusive and depend on the simulation model as well as on detailed simulation procedures. It appears, however, that in many situations both synthetic and actual design storms produce comparable results which are adequate for sizing of sewers (Arnell, 1982). These storms are sufficiently characterized by the total rainfall amount, temporal rainfall distribution and some indication of the antecedent conditions.

4.6.5.1 Design storms for sewer sizing

A number of design storms for sewer sizing have been proposed by various investigators and reviewed in the literature (Marsalek et al., 1983; Arnell et al., 1984). In the absence of systematic evaluation of various design storms, no particular storm can be recommended as the best one. Consequently, the following discussion focuses on basic parameters of design storms and on approaches to estimating such parameters. For a selected return period the design storms discussed in this section are fully characterized by the duration, the total rainfall amount, the maximum rainfall intensity of a certain short duration, the timing of peak intensity, and the temporal storm rainfall distribution. A discussion of individual storm parameters follows. Whenever appropriate, reference is made to the existing design storms.

The storm duration can be taken as a fixed-time period (Marsalek and Watt, 1984), or it can be derived from the statistical analysis of actual storms (Desbordes et al., 1981), or from the time of concentration of the area under design (Keifer and Chu, 1957). In the last case, the time of concentration is defined as the travel time from the most hydraulically remote point. The recommended storm durations vary from one (Keifer and Chu, 1957) to three times (Natural Environment Research Council, 1975) the time of concentration. It would appear that the best estimates of the storm duration should consider actual storm durations as well as the catchment concentration time. The selection of the storm duration is particularly important for those synthetic storms whose peak intensity is affected by the storm duration (Terstriep and Stall, 1974).

The total storm rainfall depth is determined from IDF curves for a selected storm duration and a return period. For example, considering a 6-hour 50-year storm, the total rainfall is read from Figure 4.5 as $H = 6 \times 7{,}5 = 45$ mm. This rainfall depth is then distributed over the total storm duration.

The peak intensity of a particular storm hyetograph is determined for a certain short duration which usually coincides with the computational time step used in runoff computations. Typically, this duration is equal to the shortest duration used in IDF curves, five minutes. The peak intensity is determined either by applying the temporal distribution to the storm rainfall (Terstriep and Stall, 1974), or from the IDF curves (Keifer and Chu, 1957). The latter approach is used in the Chicago storm which is based on an assumption that, for a selected return period, all maximum intensities for durations shorter than the storm duration are contained in the design storm. It is then possible to read the 5-minute peak intensity directly from IDF curves. For the 50-year storm considered above, the peak intensity is read from Figure 4.5 as $i_{5\text{-min, }50\text{-year}} = 190$ mm/hr. With other types of temporal distributions, different peak intensities would be obtained (most often lower). This would mean that the 5-minute peak intensity associated with a 6-hour 50-year storm has a different return period than 50 years.

The timing of the peak intensity during the storm is important for runoff computations. Recognizing that loss functions considered in hydrologic synthesis (e.g., depression storage, infiltration) attain their maximum values at the beginning of the storm, advanced storm profiles will produce lower runoff peaks because the peak intensity coincides with maximum losses. On the other hand, delayed storm profiles produce higher runoff peaks, because the peak intensity occurs in the later part of the storm when losses are low. Various storm patterns are shown in Figure 4.6. The timing t_r of the peak intensity is usually expressed as

$$t_r = t_p/T \tag{4.5}$$

where t_p is the time to the peak and T is the total storm duration. Thus for a fully advanced storm profile $t_r = 0$, and for a fully delayed profile $t_r = 1$. Although the timing of the peak intensity is a probabilistic factor, it is typically considered as a deterministic factor with a constant value. Various approaches to determining t_r have been recommended. In some design storms, the storm profile is assumed to be symmetrical with a centered peak intensity, $t_r = 0{,}5$ (Natural Environment Research Council, 1975). In other cases, the timing t_r is determined by analysis of actual local heavy storms. The timing is then taken as the

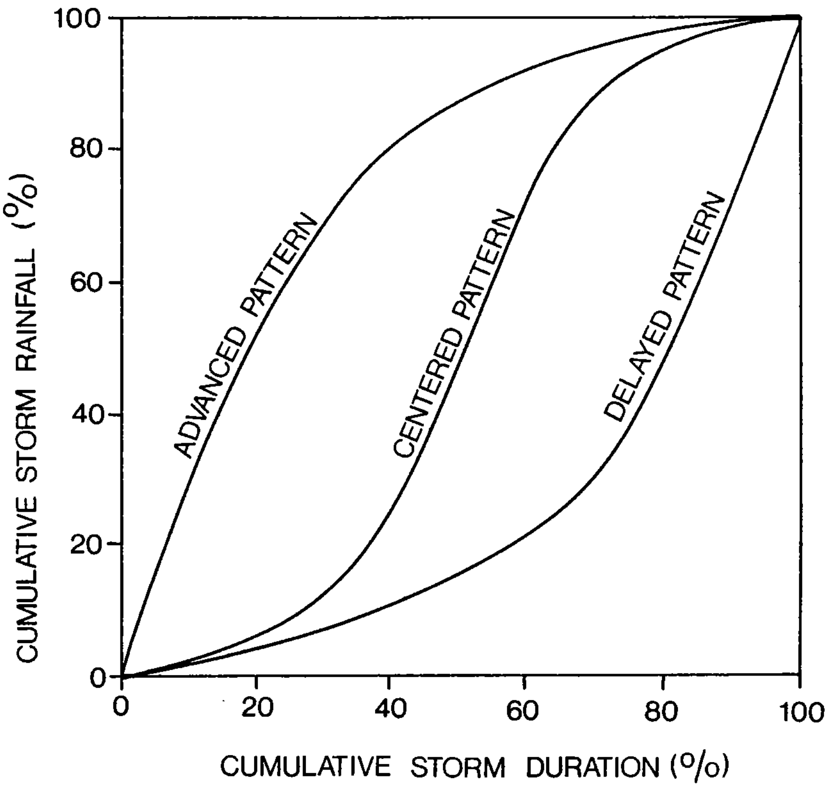


Fig. 4.6 Cumulative graphs of various storm patterns

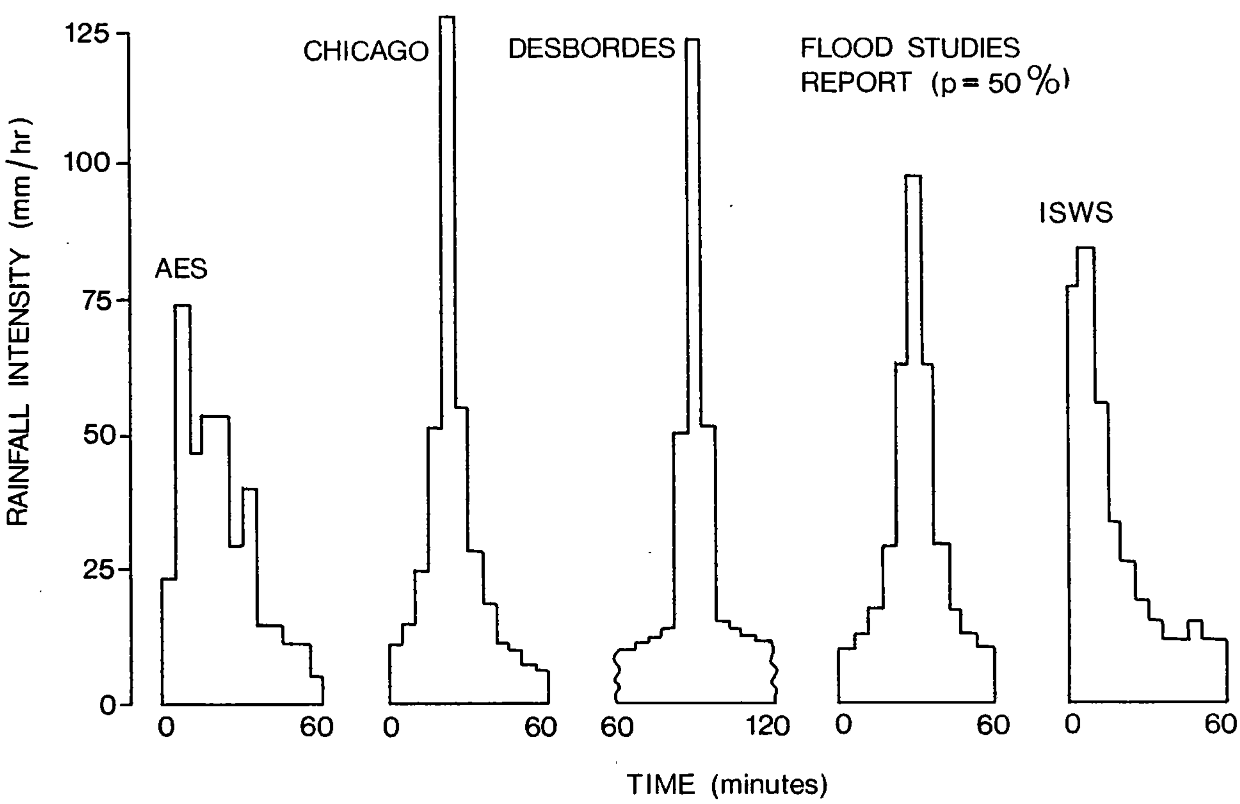


Fig. 4.7 Design storm hyetographs

mean value of all observed timings (Keifer and Chu, 1957), or as a mean value of timings in the predominant group of storms (Terstriep and Stall, 1974). In the latter case, storms are first classified into groups with advanced, centered and delayed profiles. After identifying the group with the largest population, the mean value of peak intensity timings is determined for this group and adopted for the design storm (Huff, 1967).

The temporal distribution of storm rainfall completes the definition of the design storm hyetograph. Again, a number of approaches are available and some of these are illustrated in Figure 4.7. It is apparent from this figure that storm profiles, derived by different methods from various rainfall records, vary significantly and, consequently, it is not advisable to transpose storm profiles among climatically different regions. Design storm profiles are sometimes derived by a national agency from the actual storms and the temporal distribution is then given in special tables (Natural Environment Council, 1975). The distribution used in the Illinois State Water Survey (ISWS) storm was derived by preparing graphs of the cumulative rainfall versus time (see Figure 4.6) for actual heavy storms and adopting the mean distribution of the predominant group of storm profiles (Terstriep and Stall, 1974). In the Chicago storm procedure, the peak intensity timing is determined first and intensities for other durations are positioned around the peak intensity (Keifer and Chu, 1957).

The latest research indicates that design storm distributions are best determined by fitting a selected distribution model to the recorded rainfall data and determining the model parameters by the method of moments (Marsalek and Watt, 1984). For this purpose, local rainfall records are discretized into individual events and only severe storms are retained for distribution analysis. The selection criteria can be based on the total rainfall depth which would correspond to a particular return period, e.g., two years. The reduced set of events is then discretized using a certain interval and a selected distribution is fitted to these data. In the absence of comprehensive evaluations and comparisons of various distributions, it is recommended to use the simpler ones, such as the triangular (Yen and Chow, 1980) or combined triangular/exponential (Hydrotek, 1985) distributions. The fitting of these distributions is done by the method of moments. The selected distribution is then applied to total rainfall and the storm hyetograph is produced. The fitted parameters can then be used in a regional analysis for interpolations and extrapolations of data.

4.6.5.2 Other types of synthetic design storms

A review of engineering applications revealed that various design storms have been applied (not always properly) to many types of drainage design problems, such as sizing of drainage pipes and ditches, comparisons of pre-development and post-development runoff peak flows, sizing of stormwater storage facilities, control of water quality of drainage discharge, and real-time control of combined sewer systems. In such applications, the storm characteristics to be considered include the total rainfall depth; the storm duration; the frequency of occurrence; the distribution of rainfall during the storm and the corresponding peak intensities for various durations; the peak intensity timing, and the rainfall antecedent to the intensity peak; the antecedent dry period; inter-event time; storm movement (the direction and velocity); and the storm development and decay. All these storm characteristics may affect the results of runoff quantity and quality computations and, consequently, the drainage design. Under such circumstances, the development of design storms is much more complicated than the procedures given in the preceding section. Techniques for the development of complex design storms were proposed for specific projects or locations and lack a more general verification. Consequently, such techniques are not included here and the reader is referred elsewhere (Kidd and Packman, 1980; Arnell et al., 1984).

4.6.5.3 Historical design storms

The discussion in the preceding sections indicates some problems with the development and application of synthetic design storms. Most of these problems concern the assignment of identical return periods to the storm and runoff event, definition of design storm parameters, and definition of antecedent conditions. To alleviate these problems, the use of actual historical rainfall data has been proposed (McPherson, 1975). Ideally, one should use a sufficiently long rainfall record to undertake continuous runoff simulation and to evaluate the drainage system performance. To reduce costs, continuous simulation is sometimes approximated by a series of single-event simulations for selected actual storms (Marsalek, 1978; Geiger et al., 1976). The selection of these storms, which effectively replace the rainfall record for a particular purpose, is based on ranking of all storms according to a particular storm parameter. Furthermore, one may have to estimate the initial conditions at the start of individual events and, if required, make adjustments to account for the initial conditions (Kidd and Packman, 1980). As an example, runoff peak frequency plots for simulations with actual storms on an urban catchment are given in Figure 4.8.

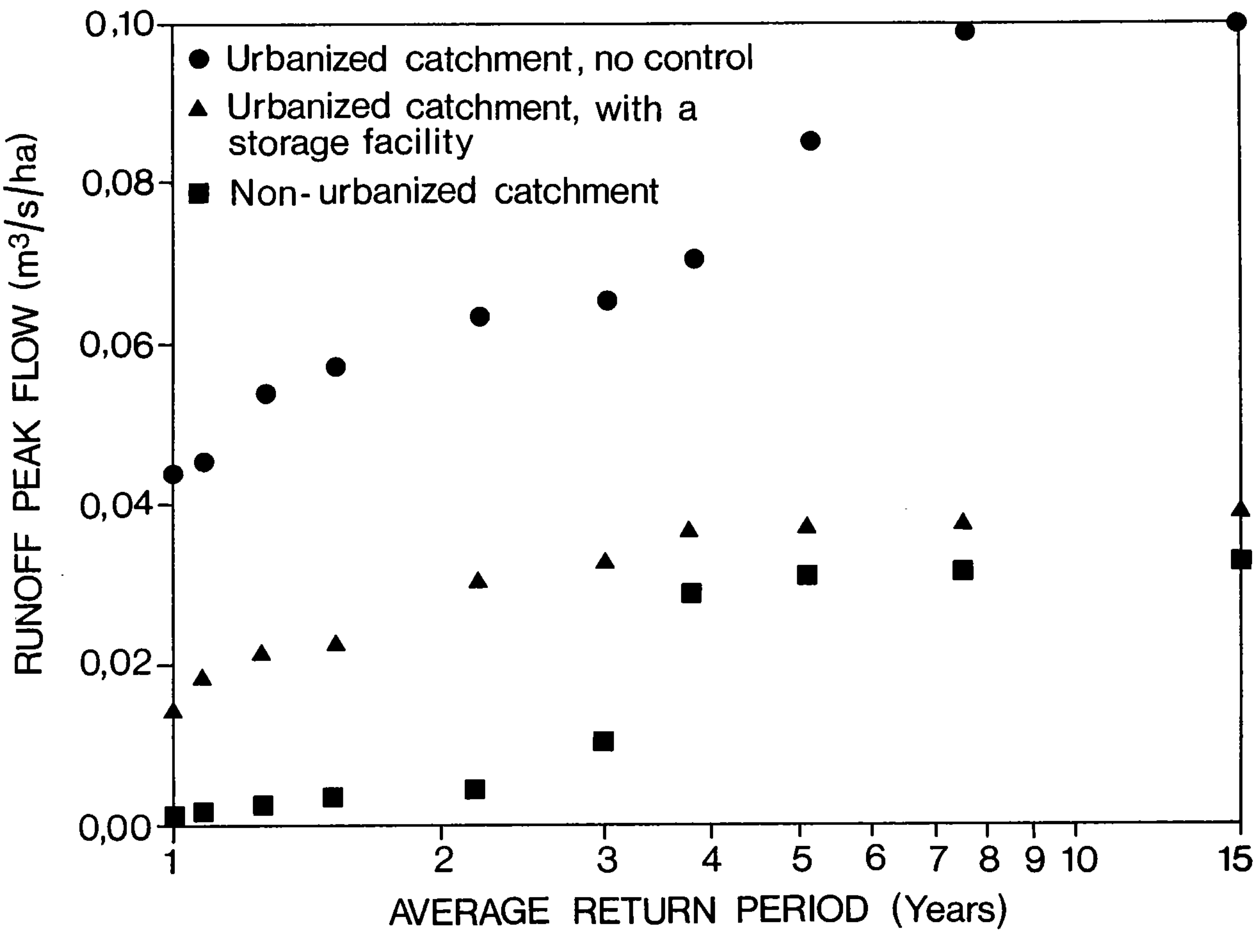


Fig. 4.8 Runoff peak frequency curves derived from simulations with actual storms

The literature on the use of actual storms in drainage design is fairly extensive and reference is made to several additional publications dealing with this topic (Marsalek, 1978; Arnell, 1982; Voorhees and Wenzel, 1984).

4.7 WATER QUALITY CONSIDERATIONS

When water quality considerations are included in the study objectives, it is necessary to consider the quantity and quality of various inflows to and outflows from the sewer system. Three sources of flow entering the sewer system are considered here - stormwater, dry weather flow, and infiltration and inflow of extraneous water into sewers. The uncontrolled outflow from sewers reaches the receiving waters in two forms - either as discharges of stormwater from storm sewers in separate sewer systems, or as combined sewer overflows from combined sewer systems. The following discussion deals with both the sources of pollution in sewer systems and the outputs from sewer systems. Additional information on water quality parameters and their monitoring is given in Chapters 3 and 5 of Volume II (Unesco, 1987).

4.7.1 Stormwater quality

It is generally recognized that urban runoff contributes to pollution of surface waters. The magnitude and rate of pollutant emissions in urban runoff are dependent on complex interactions between climatic processes, land use, and transport in sewer systems. The effect of urban runoff on water quality in receiving waters depends on the magnitude and rate of runoff pollutant emission and the characteristics and uses of the receiving waters. In order to evaluate the effect of urban runoff on receiving waters, it is required to identify both the runoff pollution characteristics and the receiving water characteristics and to establish a cause and effect relationship between the two. This section deals with procedures used to evaluate urban runoff pollution characteristics. It must be emphasized here that the urban runoff quality processes are very complex and not yet fully understood. At the present state of knowledge, the collection of actual water quality data in the field represents the best approach, if time and budget constraints allow this.

The processes controlling quality of stormwater are rather complex, as shown in Figure 4.9. The sources of pollution are numerous and their contributions are difficult to quantify. Consequently, the research into runoff quality processes is rather limited and more effort has been devoted to field observations of stormwater quality and creation of a data base. Extensive studies of urban runoff quality were reported in numerous countries and the sources of data from these studies are summarized in Appendix A. Caution is advised when transposing runoff quality data to other locations, because the sources of pollutants and their strength are likely to differ from place to place.

Typical concentrations of pollutants in urban runoff and combined sewer overflows were given earlier in Table 2.1 in Chapter 2. Another way of reporting runoff quality data is in the form of annual unit area loads for various land use. Examples of such loads from British (Ellis, 1986), Canadian (Dick and Marsalek, 1970; Marsalek, 1984) and Finnish (Melanen, 1981) studies are given in Table 4.5. Discussion of data in Tables 2.1 and 4.5 follows.

Table 4.5 Annual Unit Pollutant Loadings for Stormwater and Combined Sewer Overflows

	Annual Pollutant Loadings (kg/ha/yr)				
Source	Total Suspended Solids	BOD	COD	Total N	Total P
Runoff in storm sewers	100 - 6300	5 - 170	20 - 1000	2 - 12	0,2 - 2,2
Residential area runoff	600 - 2300	5 - 100	20 - 800	2 - 12	0,2 - 2,2
Commercial area runoff	100 - 800	40 - 90	100 - 1000	5 - 12	1,2 - 2,2
Industrial area runoff	400 - 1700	10 - 90	200 - 1000	5 - 10	1,0 - 2,1
Highway runoff	120 - 6300	90 - 170	180 - 3900	-	-
Combined sewer overflows	1200 - 5000	500 - 1300	500 - 3300	15 - 40	4 - 8

Data sources: Ellis (1986), Dick and Marsalek (1979), Marsalek (1984), and Melanen (1981).

The highest loads and concentrations in stormwater were observed for total suspended solids. The main sources of solids include air pollution, soil erosion, wear of solid surfaces, and use of antiskid materials. Suspended solids concentrations in stormwater can be higher than those in sanitary sewage.

Biochemical oxygen demand (BOD) loads and concentrations in stormwater are about an order of magnitude smaller than those in overflows. This is caused by low levels of biodegradable organics in runoff and possible interference of present toxic metals with this test. The main source of organics in overflows is sanitary sewage and solids scoured in sewers during high flows.

Chemical oxygen demand in stormwater is fairly high and exceeds BOD. The COD/BOD ratio for stormwater is generally much higher than that for sanitary sewage.

Total nitrogen and phosphorus occur in stormwater in concentrations much lower than those in sanitary sewage. Over extended time periods, these nutrients may contribute to eutrophication of receiving waters. Phosphorus loadings are generally more important, because the receiving waters are typically phosphorus limited. Annual loadings of N and P in combined sewer overflows exceed significantly those in stormwater.

Total coliform counts in stormwater reach levels up to 10^5. Such values are much lower than those corresponding to sanitary sewage (10^7 - 10^8, the most probable number/100 ml). Main sources of coliforms are cross-connections between storm and sanitary sewers, and bird and animal faeces deposited in urban areas.

Besides the above discussed conventional pollutants, many other constituents, some fairly persistent and toxic, have been found in stormwater. Among these, heavy metals are most common with the highest concentrations reported for lead, zinc and copper (Malmqvist, 1983; Ellis, 1986). Other substances found in small quantities include hydrocarbons, pesticides, and many other toxic substances (Marsalek, 1986).

Although field observations provide the best indication of stormwater quality, the costs of extensive data collection may be prohibitive in many cases. Consequently, field programmes are typically used either to produce annual pollutant loads for typical urban catchments, or to calibrate stormwater quality models. A brief description of both approaches follows.

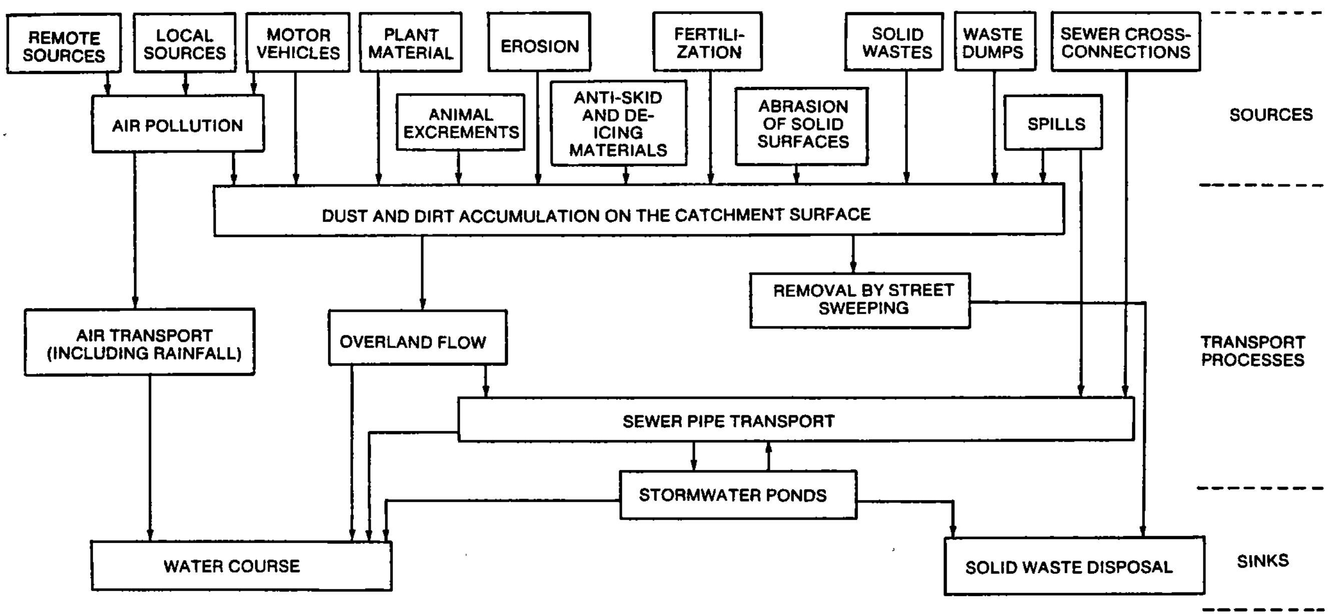


Fig. 4.9 Sources of pollutants in stormwater and pollutant pathways

4.7.1.1 Annual unit pollutant loading rates in stormwater

Annual unit pollutant loading rates in stormwater have been developed in several countries (Sullivan et al., 1977; Melanen, 1981; Sullivan et al., 1978) and can be used for an expedient evaluation of pollutant emissions in urban runoff. In general, the unit loading rates are expressed as a function of land use and some other factors, such as the annual precipitation, population density, and street cleaning. In a U.S. study (Sullivan et al., 1977), the pollutant rates in stormwater were expressed as

$$M_s = \alpha(i, j)\, R\, f(P_d)\, \gamma \tag{4.6}$$

where M_s = weight of pollutant (j) from land use (i) with storm sewers or unsewered per unit area per year, $\alpha(i, j)$ is a constant defined for a particular (i, j), R is the annual precipitation, $f(P_d)$ is a population density function, and γ is the street sweeping effectiveness ($0 < \gamma < 1$). Constants $\alpha(i, j)$ are available for BOD, suspended solids, volatile solids, nitrogen and phosphorus. Details of application of Equation 4.6 and numerical values of the parameters $\alpha(i, j)$ can be found in the original references (Heaney et al., 1977; Sullivan et al., 1977).

Examples of annual unit pollutant loads in stormwater were given earlier in Table 4.5 as ranges of values reported in several countries.

4.7.1.2 Modelling of stormwater quality

The annual unit pollutant loads which were discussed in the preceding section are useful for a preliminary evaluation of the magnitude of pollution in urban runoff. However, in detailed analyses of water quality in receiving waters, much shorter time intervals are considered. Although the selection of such time steps is not well understood, the time steps reported in the practice were as short as five minutes. Short-interval stormwater quality data can be produced by computer models discussed in this section. At the present stage of development such models have to be calibrated in order to obtain reliable results.

Various approaches to the modelling of stormwater quality have been proposed in the literature. These approaches vary from simple regression models to physically-based models. The latter models are preferred in practice, because they are applicable to various catchments and require less field data for calibration purposes. Among the deterministic models, perhaps the best known are the Storm Water Management Model (SWMM) of the U.S. Environmental Protection Agency (Huber et al., 1982) and the STORM model (U.S. Army Corps of Engineers, 1977). Both models are continuously updated and refined and have been applied in a number of countries. A brief description of basic concepts employed in SWMM and STORM follows.

Both models consider the urban surface, particularly the impervious parts, as the most important source of pollutants. During dry weather, dust and dirt accumulate on the catchment surface. The rate of accumulation in time is either linear or non-linear, as specified by the user. The amount of accumulated dust and dirt is reduced by street sweeping, which is assumed to be done at regular intervals and with a certain efficiency of removal. The composition of accumulated dust and dirt is specified by the user, using seven basic constituents - suspended solids, volatile suspended solids, BOD_5, COD, total coliform, total nitrogen, and total phosphorus. During a storm, pollutant accumulations are washed off the catchment surface at a rate which is directly proportional to the pollutant amount remaining on the surface. Another important source of solids is urban soil erosion. For this purpose, the Universal Soil Loss Equation (Wischmeier and Smith, 1958) is used to quantify the load of suspended solids contributed by soil erosion. The washed-off pollutants are then routed through sewers.

The success of stormwater quality modelling depends on the extent of catchment and calibration data and user's experience. Where only limited stormwater quality data are available, such data are probably best utilized in conjunction with a stormwater quality model.

A recent state-of-the-art review indicates that there are six fully operational urban runoff quality models available (Huber, 1986). All these models are capable of simulating accurately runoff hydrographs and pollutographs, if adequate calibration data are available. The methods of collection of such data are described in Chapters 3 and 5 of Volume II (Unesco, 1987).

4.7.2 Quality of combined sewer overflows

The composition of combined sewer overflows is generally derived from the composition of the dry weather flow, scour of sewer deposits, and composition of stormwater. Pollutant concentrations in overflows frequently exceed those in stormwater and dry weather flow, because of sewer sediment scour.

The composition of combined sewer overflows has been studied extensively in many countries, as shown in Appendix A. As an example of pollutant concentrations reported for overflows, some data were given earlier in Tables 2.1 and 4.5.

It is of interest to compare the composition of overflows to that of stormwater. With the exception of suspended solids, pollutant concentrations in overflows exceed those in stormwater several times. The main pollution problems caused by combined sewer overflows are those associated with high BOD's, high content of nutrients, and high bacteria counts which may pose a public health threat.

The earlier discussed techniques for evaluation of stormwater quality are applicable to combined sewer overlows as well. Annual unit pollutant loads in overflows may be calculated from Equation 4.6 by using proper constants $\alpha(i, j)$. Listings of such constants were published by Heaney et al. (1977).

The modelling concepts for stormwater are also employed in the modelling of overflows. In addition to runoff sources of pollution, dry weather flow is considered in quantities and composition specified by the model user. During low flows, sewage sediment is allowed to settle in sewers and may be resuspended during high flows. Pollutant mixing, decay, and routing are also considered (Huber et al., 1982).

4.7.3 Dry weather flow - quantity and quality

Drainage of many large metropolitan areas is provided by combined sewers. During dry weather, these sewers convey relatively low flows which are referred to as dry weather flows. During wet weather, combined sewers carry dry weather flow as well as surface drainage runoff (stormwater). Because of high runoff inflow, the sewer capacity may be exceeded and the excess flow leaves the sewer system as a combined sewer overflow. In order to estimate the composition of flows in combined sewers, one needs to know characteristics of both stormwater and dry weather flow. Stormwater characteristics were discussed in the preceding section and a brief discussion of dry weather flow follows. For a more detailed treatment of this subject, the reader is referred to standard handbooks (WPCF, 1970a; WPCF, 1970b).

4.7.3.1 Quantity of dry weather flow

Dry weather flow comprises liquid and water-carried wastes from residences, commercial establishments, industrial plants, and institutions, together with minor quantities of stormwater and groundwater that are not admitted intentionally.

The best estimates of dry weather flow quantity are obtained by direct flow measurements in the sewer network, or at a wastewater treatment plant. In the absence of such measurements, the quantity of dry weather flow can be estimated from metered water consumption, or calculated on the basis of population serviced with sewerage, population density, land use, tributary area and standard per capita sewage flows.

Apart from domestic wastes discussed above, industrial wastes have to be considered. Flows from areas zoned for industrial use will include process flows generated by wet industrial processes and domestic-type wastes from employees. Certain process flows are allowed by municipal ordinance to be discharged into public sewer systems and can affect significantly the quality and quantity of sewage flows in these sewer systems. For modelling purposes, it is desirable to collect field data on industrial wastes. When collecting data on the nature and character of industrial wastes, the following factors have to be considered: the volume of water, distribution in time (the periodicity of flow), the operating schedule of the plant, and the pollutional strength of these wastes. Additional details are given in standard handbooks (WPCF, 1970b; ATV, 1982a; ATV, 1982b).

In simulations of flows in combined sewers, variations in sewage quantities during the day and during the week need to be quantified. If no local data on such variations are available, one may use distributions reported in the literature (Huber et al., 1982). One example of such distributions used in the North American practice is given in Figure 4.10. Such distributions are used in conjunction with the locally derived mean flow.

4.7.3.2 Quality of dry weather flow

Quality of dry weather flow widely varies from municipality to municipality. It seems to be affected by the population density, the average family income of residents, the use of garbage grinders, industrial wastes and infiltration. The best estimates of dry weather flow quality are obtained by laboratory analysis of sewage samples, or from sewage treatment plant records. In the absence of such field data, estimates of dry weather flow quality are obtained either by transposition of data from elsewhere, or by calculations based on population and average daily per capita loads. Typical concentrations of various constituents in municipal sewage, as reported in the U.S.A., are listed in Table 4.6 below (Lager and Smith, 1974).

Table 4.6 Pollutant Concentrations in Municipal Sewage (Lager and Smith, 1974)

	Constituent Concentration (mg/l)											
	BOD_5		COD		Suspended Solids		Total Nitrogen		Total Phosphorus		Total Coliform (MPN/100 ml)*	
	Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range
Concentration in Municipal Sewage	200	100-300	500	250-750	200	100-350	40	-	10	-	5×10^8	-

* Most probable number/100 ml.

Quality of domestic sewage can be calculated from population, average daily loads per capita, and daily per capita sewage flows. Huber et al. (1982) list the following average daily constituent loads per capita: BOD_5 - 0,09 kg/capita/day, suspended solids - 0,1 kg/capita/day, and total coliforms - 200 x 10^9 MPN (most probable number)/capita/day. Average constituent concentrations can be calculated by dividing these loads by daily flows per capita. Additional loads of constituents are contributed by industrial wastes.

Finally, the strength of dry weather flow is somewhat diluted by infiltration into sewers which may be particularly large in older sewer systems.

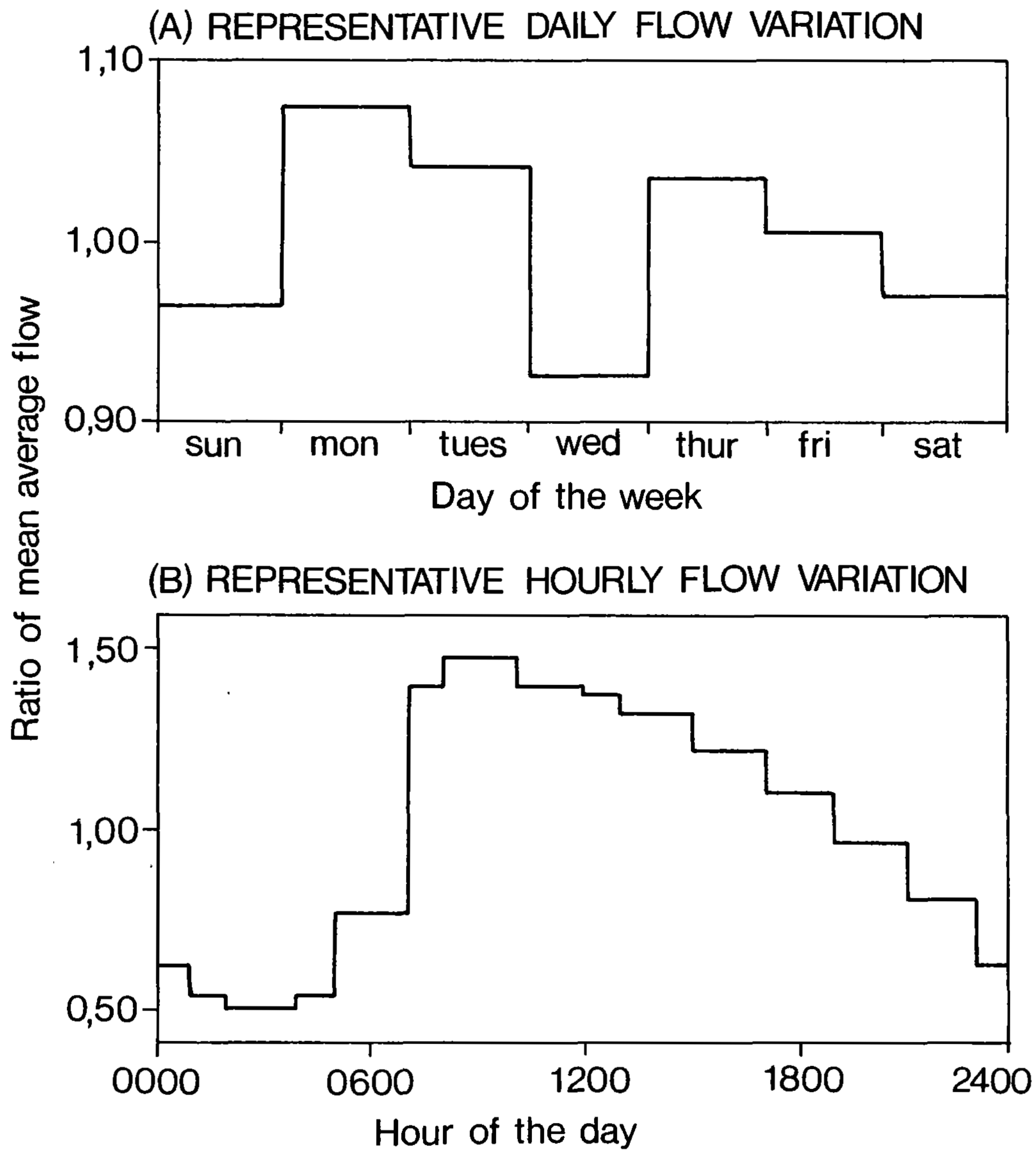


Fig. 4.10 Hourly and daily variations of domestic sewage flow

4.7.4 Infiltration and inflow into sewers

For the purpose of this manual, the following definitions of infiltration and inflow have been adopted (U.S. EPA, 1970):

"Infiltration" covers the volume of groundwater entering sewers and house connections from the soil, through defective joints, broken or cracked pipes, improperly made connections, manholes, walls, etc.

"Inflow" covers the volume of any kind of water discharged into sewer lines from such sources as roof leaders; cellar and yard area drains; foundation drains; commercial and industrial so-called clean water discharges; drains from springs and swampy areas; etc. It does not include, and is distinguished from, "infiltration" as previously defined.

Infiltration and inflow into sewers are widespread problems adversely affecting most of the existing collection systems and treatment plants. For detailed analysis of infiltration and inflow, the reader is referred to standard handbooks (WPCF, 1970b; U.S. EPA, 1970). A brief discussion of infiltration and inflow follows.

4.7.4.1 Infiltration into sewers

Excessive infiltration may become a serious problem in the design, construction, operation and maintenance of sewer systems. Infiltration is an important source of large volumes of flow in sanitary and combined sewer systems regardless of the size of the drainage area, types of sewer pipe construction, pipe and jointing materials, and types of soil formations in which the sewers are laid (U.S. EPA, 1970). Infiltration contributes significantly to hydraulic overloading of both collection systems and treatment plants. There are even cases where infiltration and inflow exceed the dry weather flow.

Among the adverse effects of infiltration and inflow, one can name the following:

- cost of treatment of infiltrated water;
- cost of construction of new relief sewers and treatment facilities to accommodate infiltration flows;
- reduced treatment plant efficiency;
- increased bypassing of treatment plants;
- higher pumping costs;
- basement flooding;
- cave-ins and structural failures of sewers and road structures resulting from soil washing into sewers; and,
- increased sewer maintenance costs resulting from soil deposits in sewers.

To avoid these costs, some degree of infiltration control needs to be exercised. In new systems, the infiltration control is achieved by adequate design, proper selection of materials of sewer construction, pipes, joints, and proper laying procedures and techniques. Correction of existing sewer infiltration can be accomplished by replacing defective components, sealing the existing openings, and building within the existing components (Lager and Smith, 1974).

During construction of new sewers, infiltration control can be achieved by proper consideration of sewer pipe material, selection of sewer joints, and construction consideration.

The important issues to consider in pipe material selection are structural integrity, wastewater characteristics, and local soil or gradient conditions. Combinations of these factors may make one material better suited than another, or preferable under certain conditions. Pipe resistance to infiltration should not be omitted from these considerations.

Good sewer joints are probably the most important single factor in the control of infiltration. A good joint must be watertight, resistant to root penetration, effects of soil, groundwater, and sewage, long-lasting, and flexible (Lager and Smith, 1974). New jointing methods which represent a vast improvement over the traditional methods include PVC and polyurethane joints, compression gaskets and chemical-weld joints.

Major infiltration problems occur in house lateral connections. These connections are made after the sewer pipes have been laid and tested. The connections are made in a crude way and are rarely inspected.

Sewer construction considerations include size of trenches, dewatering of trenches, pipe laying and assembly, backfilling, and post-construction performance tests.

In existing sewers, the correction of infiltration involves a lengthy systematic plan of action, which includes the following steps (Lager and Smith, 1974):

- identification of the drainage system;
- evaluation of the extent of infiltration (by field measurements, or by examining treatment plant records);
- survey of the sewer system and adjacent soils (smoke tests, soil conditions, groundwater conditions);
- economic and feasibility study to determine the most cost-effective locations for infiltration control;
- sewer cleaning (if required);
- photographic and television inspection; and
- restoration of the sewer system.

4.7.4.2 Inflow into sewers

Inflow sources generally represent deliberate connection of drains to a sewerage system. Such connections may be authorized and permitted; or they may be illicit connections made for the convenience of property owners and for solution of on-property problems, without consideration of their effects on public sewer systems (Lager and Smith, 1974).

The nature of inflow water and its effect on sewer systems are very much similar to those of infiltration water, as discussed in the preceding subsection.

Correction of inflow problems depends on regulatory and enforcement action on the part of city officials, rather than on construction measures. The effects of inflows into sewers can be greatly reduced by a variety of methods. For example, many authorities advocate the discharge of roof water into street gutter areas or onto lots where it can percolate into the soil. Discharging roof or areaway drainage onto the land or into street gutters reduces the immediate impact on the sewer system by providing for reduction of the flow volume and attenuation of the peak discharge. The use of pervious drainage swales and surface storage basins within urban areas allows the stormwater to percolate into the ground (Lager and Smith, 1974).

Depressed manholes with vented covers in street areas where runoff can pond over the cover should be repaired or the covers replaced with unvented covers. Finally, commercial and industrial water users should be encouraged to practice on-stream reclamation and reuse of the so-called "clean water" (mostly cooling waters) to control this source of inflows.

5 Quantity of stormwater

5.1 STORMWATER ANALYSIS

5.1.1 Scope of stormwater analysis

As explained in Chapters 2 and 4, the flow in an urban drainage system is composed of stormwater, infiltrated water, and domestic, commercial and industrial sewage. In the case of a separate sewer system, two sewers, one for dry weather flow and the other one for stormwater need to be designed. In the case of a combined system, only one common sewer is designed. While the dry weather flow can be quantified fairly easily, a reliable determination of stormwater flows is rather difficult.

Information on stormwater runoff quantity is needed for planning, design and operation of urban drainage systems. Runoff events can be characterized by individual parameters, such as the runoff volume, peak flow and duration, or by a complete runoff hydrograph showing the runoff flow rate variation in time. Such a hydrograph of course contains the earlier mentioned individual characteristics. Similarly, various methods of urban hydrology yield either specific characteristics of runoff events, e.g., the peak flow, or entire runoff hydrographs. Among the former methods, empirical formulae expressing the runoff peak flow as a function of the runoff coefficient, the total catchment area and rainfall intensity can be mentioned. The latter, hydrograph methods, comprise hydrologic methods using time-dependent transfer functions for rainfall-runoff relationship for the entire drainage area ("black box") and hydrodynamic methods employing the continuity and energy equations. Consequently, different methods describe the stormwater runoff process with different detail and accuracy. Because more detailed and complex approaches require greater detail in input data and hence greater costs, the selection of the method for calculation of stormwater runoff should be based on the study objectives, the complexity of the design problem and the data base available. Typical objectives of drainage studies were explained in Chapter 1, components of drainage systems were discussed in Chapter 3 and various supporting data or means of their collection are presented in Volume 2 (Unesco, 1987).

Rough estimates of stormwater quantity and quality may be adequate for the screening and planning of various design and operation alternatives, but more accurate data are required for the design of individual sewer system components. Thus, hydrologic methods are sometimes used in the planning phase and for expedient calculations of stormwater runoff required in real-time operation of drainage systems. On the other hand, hydrodynamic methods usually are applied in the design of drainage systems, particularly if the system is very complex and involves large or interconnected networks with many special structures. The design of smaller and simple (tree-type) drainage networks may still be done with coefficient methods or hydrologic methods. An overview of suggested applications of various stormwater runoff calculation methods to studies with certain objectives is given in Table 5.1. Of course, there are no rigid limits on the applicability of individual methods. Besides the calculation method, the choice of the rainfall input is of utmost importance and is dealt with in Chapter 4. Changes in stormwater runoff processes caused by snowfall and snowmelt are not addressed. Such problems are discussed elsewhere (Ven Te Chow, 1964; Toebes and Ouryvaev, 1970).

Sections 5.2 to 5.5 explain different basic formulae for calculation of stormwater runoff and provide values for key parameters needed in such calculations. The network design methods of Chapter 8 combine one or more of these runoff formulae in procedures for the design of drainage networks of various complexity. The most appropriate computational method can be chosen only if the assumptions, limitations and data input requirements of the basic approaches are well understood. For instance, the sophistication and accuracy of calculations by means of detailed methods are defeated if input requirements are not met with a detail and accuracy similar to that of the calculation procedure applied.

Table 5.1 Overview of applications of various urban stormwater runoff calculation methods

Runoff Attribute	Method	Study Objective	Data Requirement	Catchment Area*	Network Layout	Time Requirements and Application Costs
Runoff peak and volume	Coefficient methods	Screening, planning and design	Low	<300 ha	Simple tree-type	Low
Hydrograph	Hydrologic synthesis	Planning, design and operation	Low	<300 ha (sometimes larger)	Tree-type	Moderate
Hydrograph	Hydro-dynamic routing	Design	Extensive	Any size	Complex	Extensive

* Other considerations besides the area may also apply (e.g., storage in the system, catchment and sewer slopes).

5.1.2 Phases of runoff development

Urban areas comprise surfaces of widely varying characteristics. Typical examples of such surfaces are roofs, backyards, streets, sidewalks, parking lots, playgrounds, different kinds of vegetation covers, bare soils and rocks. The generation of runoff is strongly affected by the permeability of the surface cover, i.e., whether it is pervious or impervious. Pervious surfaces allow large losses due to infiltration into the ground and thereby reduce the rainwater supply available for runoff. Impervious covers, such as asphalt, concrete or a bare soil highly consolidated by traffic, contribute to high runoff rates because of low hydrologic losses. On the whole, the hydrological behaviour of urban drainage areas differs strongly from that of rural areas, because of high imperviousness and dense drainage network comprising paved gutters, sewers and natural streams. However, the basic physical phenomena governing the runoff process are the same for natural as well as urbanized areas.

When rain falls on an urban area, some rainwater falls on impervious surfaces and some on pervious surfaces with or without vegetal cover. At the start of rain, the rainwater falling on impervious areas is partly absorbed by wetting the surface, partly evaporated and partly trapped in surface depressions. After such depressions have been filled, water will overflow and form surface runoff. Water trapped in depressions on impervious surfaces will eventually evaporate after rainfall ceases.

On pervious surfaces, some rainwater is intercepted by vegetation and stored on leaves and stems and the rest reaches the pervious ground. At the start of rainfall, the rainwater reaching the pervious ground infiltrates into the soil. As rain continues, open spaces in the soil may be reduced by swell of colloidal materials and become filled with water. Consequently, the rate of infiltration is vastly reduced in comparison to the initial rate. When the rate of rainfall exceeds the infiltration capacity, water starts ponding on the surface and fills surface depressions. After such depressions have been filled, overflowing water forms surface runoff. After the rainfall ceases, water trapped in depressions evaporates and/or infiltrates into the soil.

Water which infiltrated into the soil may move laterally through the upper soil horizons towards the stream as the so-called interflow. Water percolating to the lower soil horizons reaches groundwater which is sometimes conceptually divided into two storage compartments - the active storage with subsurface (groundwater) flow and the inactive or deep groundwater storage. The description of water transport through the soil was included here only for completeness, because such transport processes are generally not considered in the analysis of stormwater quantity.

The transport of surface runoff in urban catchments starts with overland flow which drains into swales and gutters and those in turn drain into the drainage system through stormwater inlets. Inside the drainage system, stormwater is transported through open-channel or sewer conduit networks according to the hydraulic characteristics of such networks. The formation of runoff may be graphically presented by plotting hydrographs for a particular point on the catchment surface, at a stormwater inlet or in the sewer system, channel or stream which drains the catchment. Such hydrographs represent integrated effects of rainfall and catchment characteristics, such as the area, shape, surface cover, depression and infiltration capacities, land use, drainage patterns, surface and drainage slopes, and sewer, channel and stream characteristics. The magnitude of peak discharges and the shapes of these hydrographs are of interest for the layout of urban drainage systems.

The principles of stormwater runoff development and its possible superimposition with dry weather flow are summarized in Figure 5.1. In calculations, the rainfall-runoff process can be divided into three stages dealing with the determination of

- the rainfall excess
- surface runoff hydrographs at inlets to the drainage system (inlet hydrographs), and
- transport in the drainage system.

Some of the methods described in Sections 5.3 to 5.5 apply to both the determination of surface runoff and its transport in the drainage system.

5.2 RAINFALL EXCESS AND ABSTRACTIONS

5.2.1 Determination of rainfall excess

Hyetographs of actual or synthesized design storms, or rainfall intensities of certain magnitudes and recurrence frequencies are used as rainfall inputs in runoff calculations. The preparation of such inputs was discussed earlier in Chapter 4. The rainfall input can be divided into two parts, rainfall excess and rainfall abstractions. The former component represents that part of rainfall which is directly converted into runoff. The rainfall abstractions make up the remaining part of rainfall which does not contribute to runoff. Such abstractions comprise interception, evaporation, evapotranspiration, depression storage, and infiltration. They are also sometimes called losses, which is not a very appropriate term because no water is lost in the hydrological cycle. For individual rain storms, rainfall excess on impervious surfaces differs substantially from that on pervious surfaces. Within each surface category, there are additional differences in rainfall excess depending on detailed characteristics of the surface cover. As stated earlier, phenomena connected with snowfall and snowmelt are not addressed in the discussion that follows.

Rainfall excess $I_{ex}(t)$ is described by Equation 5.1 as rainfall $I(t)$ minus the rainfall abstractions which include interception $I_{in}(t)$, surface wetting $I_{we}(t)$, evaporation and evapotranspiration $I_{ev}(t)$, soil infiltration $I_{si}(t)$, and surface depression storage $I_{de}(t)$:

$$I_{ex}(t) = I(t) - I_{in}(t) - I_{we}(t) - I_{ev}(t) - I_{si}(t) - I_{de}(t) \tag{5.1}$$

All the terms in Equation 5.1 are generally expressed in units of rainfall intensity, such as mm/min, or mm/hr. Additional discussion of rainfall abstractions can be found in a hydrology handbook (Chow, 1964).

On impervious areas, the only abstractions taking place are $I_{we}(t)$, $I_{ev}(t)$ and $I_{de}(t)$. It should be further noted that evaporation during rainfall is normally negligible ($I_{ev}(t) \approx 0$) and that surface wetting and depression storage occur at the onset of rainfall and can be lumped together as the so-called initial abstraction $I_i(t) = I_{we}(t) + I_{de}(t)$. Thus for impervious areas, Equation 5.1 can be reduced to the following expression:

$$I_{exi}(t) = I(t) - I_i(t) \tag{5.2}$$

Although all the abstractions in Equation 5.1 generally apply to pervious surfaces, the initial and infiltration abstractions exceed the remaining ones by an order of magnitude and Equation 5.1 can be reduced, for practical applications to pervious surfaces, to the following form:

$$I_{exp}(t) = I(t) - I_i(t) - I_{si}(t) \tag{5.3}$$

Equations 5.2 and 5.3 are shown schematically in Figure 5.2. It should be noted that initial abstractions on pervious and impervious areas differ. Numerical values of initial abstractions which represent mostly depression storage are given in Section 5.2.2.

Rainfall excess can be calculated by considering the time-varying rainfall rates and abstractions, or by working with lumped estimates. The former approach is taken when using actual or synthesized storm hyetographs in conjunction with hydrologic or hydrodynamic methods

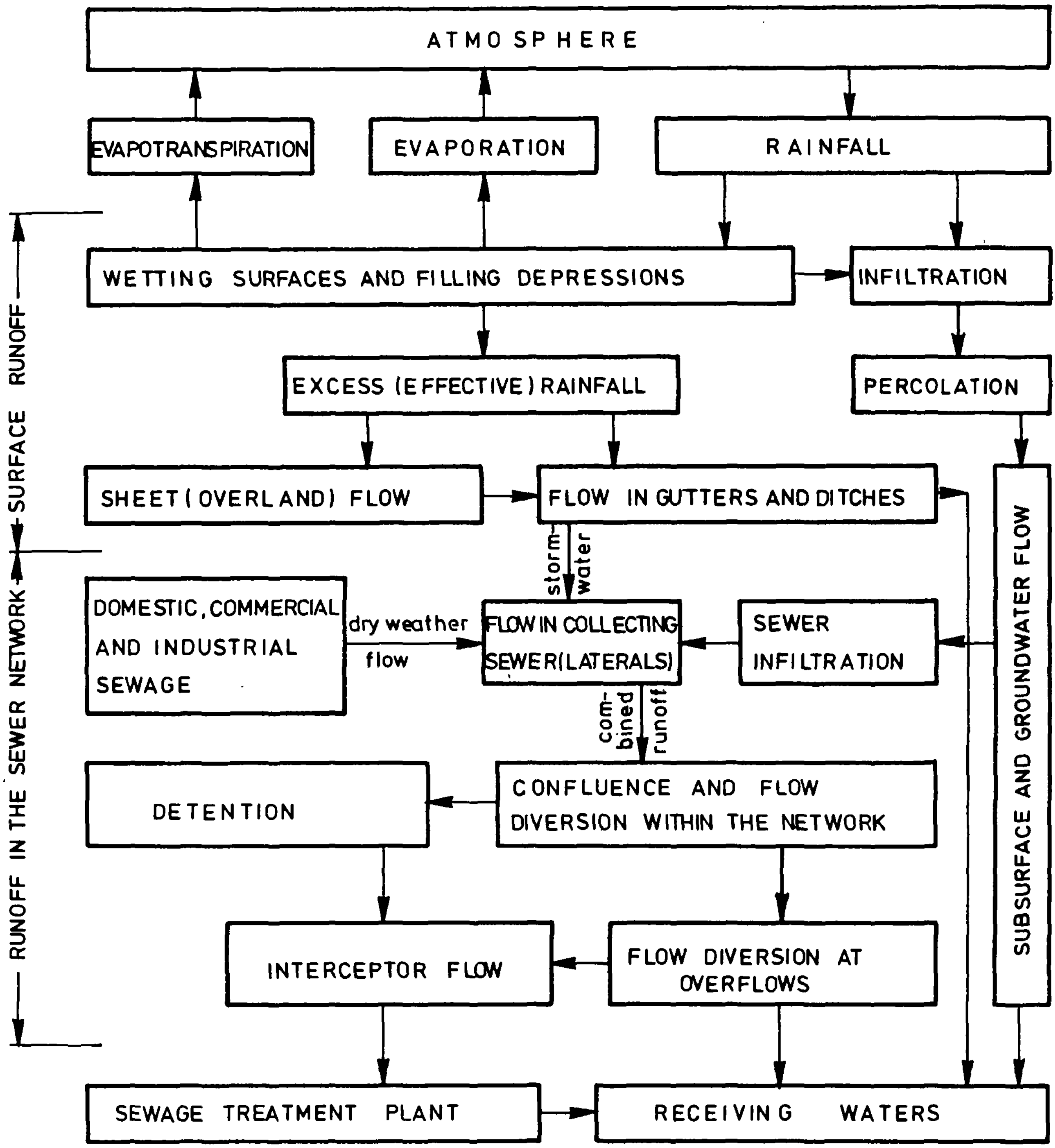


Fig. 5.1 Development of stormwater runoff and flow in combined sewer systems (after Geiger, 1984).

of runoff calculations. The latter, lumped approach, is used in coefficient methods for calculation of runoff peaks. Both approaches are shown schematically in Figure 5.3.

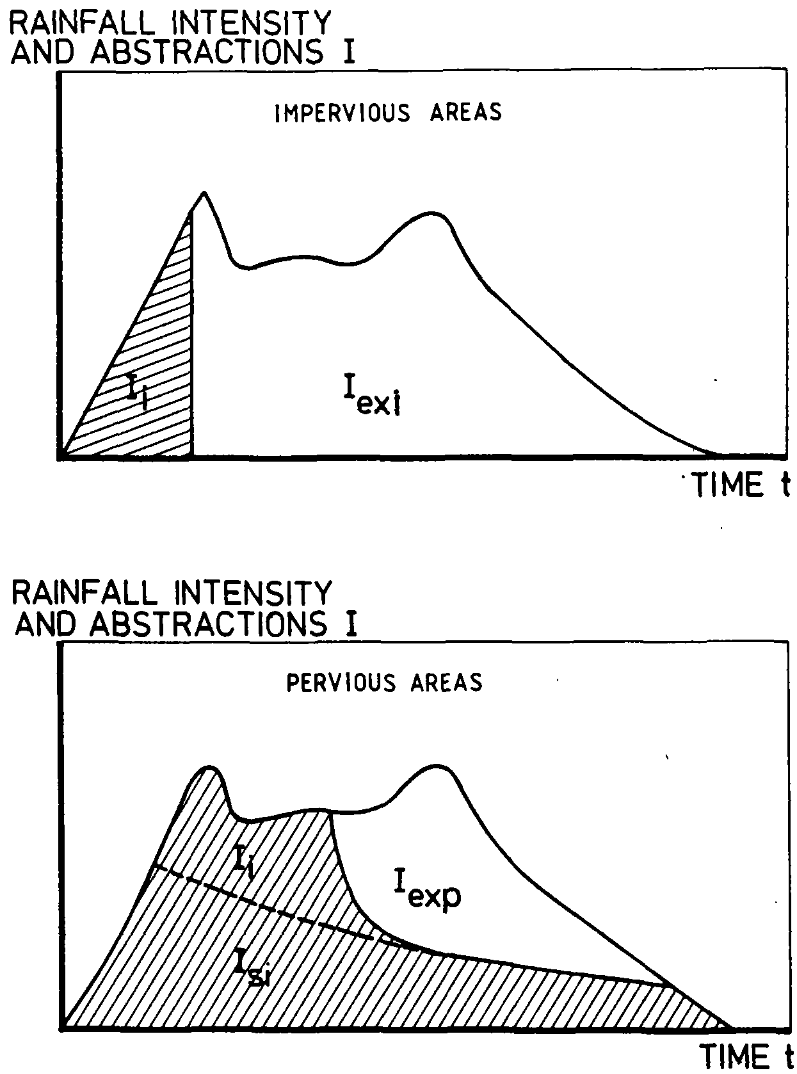


Fig. 5.2 Rainfall abstractions on impervious and pervious areas.

The calculation of time-varying abstractions can be done in two ways. In the first method, also referred to as the threshold method, the wetting interception and depression abstractions are combined together as the initial abstraction (see Figure 5.3a). In the second method, also known as the limiting value method, the wetting and interception abstractions are again combined as the initial abstraction, but the depression abstraction is considered to be an exponentially decaying function applied to the known total abstraction (i.e., the limiting value; see Figure 5.3b).

Lumped abstractions are reflected in the magnitude of the runoff coefficient and, for a constant runoff coefficient, rainfall excess represents a fixed fraction of rainfall (see Figure 5.3c). Finally, approximate estimates of rainfall excess on pervious areas can be obtained by subtracting a constant or time dependent value from the total rainfall (see Figure 5.3d).

Calculations of stormwater runoff peaks and volumes are usually not sensitive to rainfall abstractions on impervious areas. On the other hand, abstractions on pervious areas affect significantly both the runoff peak and volume. In the case of urban areas which comprise impervious and pervious subareas, runoff peaks are controlled largely by the impervious subareas whose sharp-peaked hydrographs precede somewhat attenuated hydrographs from pervious areas. This was confirmed by Kibler and Aron (1978) for watersheds with more than one-half of the total area developed. In such watersheds, flood peaks from moderate storms did not depend on infiltration abstractions or surface roughness of the pervious part of the watershed.

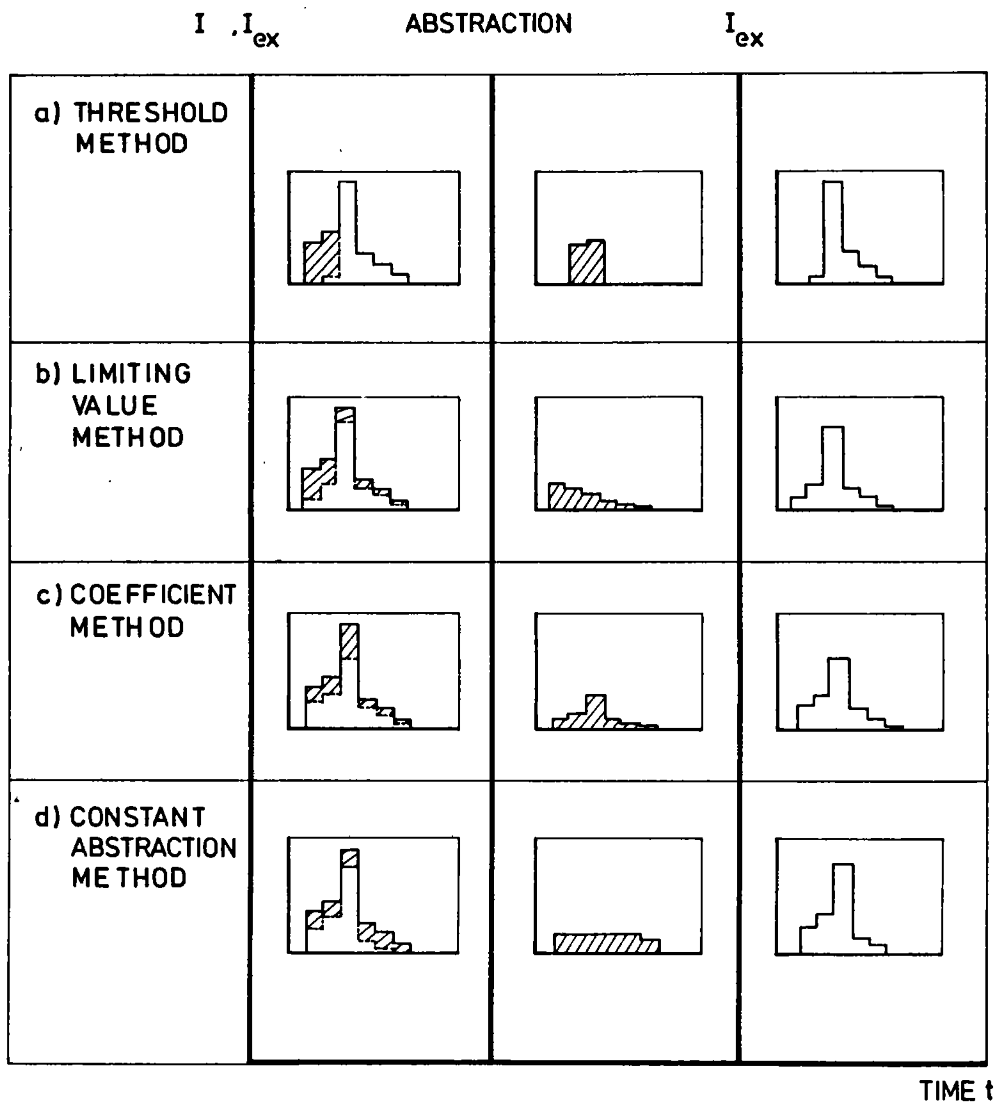


Fig. 5.3 Determining rainfall excess using various definitions for rainfall abstractions.

When runoff volumes are of importance, as in the case of the design of detention basins, runoff from pervious areas can have an appreciable effect and should be properly evaluated (Kibler, 1982). Erroneous assumptions about or errors in rainfall abstractions are directly reflected in calculated runoff peaks and volumes. Therefore, it is recommended to evaluate each specific design problem before deciding which abstractions should be considered.

5.2.2 Rainfall abstractions

5.2.2.1 Interception

Rainfall which is intercepted by vegetation prior to reaching the ground is referred to as interception loss. The amount intercepted is a function of species, age and density of the vegetation, character of the storm, and season of the year. It has been estimated that between 10% and 20% of the rainfall which falls during the growing season is intercepted and returned to the atmosphere by evaporation (Viessmann et al., 1972). Except for dense vegetation, such as a forest, water losses by interception during rain storms are not pronounced. Interception losses develop during the early portion of the storm and later the rate of interception rapidly approaches zero. Although there are formulae expressing interception as a function of rainfall and vegetation characteristics (Horton, 1939; Kittredge, 1948; Chow, 1964), it is a common practice to deduct the estimated interception from the storm rainfall as part of the initial abstraction.

5.2.2.2 Evaporation and evapotranspiration

The evapotranspiration process is highly complex and requires a large amount of input information for its formulation. This process is a function of the plant species, the vegetation density, season, soil characteristics, and meteorologic conditions such as wind speed, humidity, ambient temperature, cloud cover, and latitude. Evaporation occurs in the soil primarily at the soil-air interface. Transpiration is defined as vaporization of water at the surface of plant leaves after the soil water has been transported through the plant (Overton and Meadows, 1976).

For simplification, it is commonly assumed that water supply to the plant and soil surface is not limited which permits the treatment of evaporation at its potential rate. There are four basic approaches to the deterministic modelling of potential evapotranspiration:

- The energy budget approach,
- The water balance method,
- The aerodynamic approach, and,
- The combination of energy budget and aerodynamic approaches. Further details can be found in hydrological handbooks (Chow, 1964; WMO, 1966).

In calculations of stormwater runoff, evapotranspiration, which is negligible during rainfall, is typically neglected. Even in the Sahelian zone, with evaporation rates from 10 mm/day to 15 mm/day, the effects of evapotranspiration on runoff from rain storms are negligible.

5.2.2.3 Surface wetting and depression storage

The transition from wetting the surfaces to filling small voids or puddles with water is a continuous one. Therefore, wetting abstractions are frequently included in the depression abstractions, or both wetting and depression abstractions are combined and referred to as the initial abstraction. However, if separate values for wetting abstractions are needed, they may be estimated as 0,2 - 0,5 mm and 0,2 - 2,0 mm for impervious and pervious surfaces, respectively (Pieper, 1938; Braun, 1959).

Depression storage accounts for water which is trapped in small puddles and held until it infiltrates or evaporates. Since there are many sizes and shapes of depressions throughout an area, it is possible only to consider their gross effect. Linsley et al. (1949) reported that the water stored in depressions, at any given time after the beginning of rainfall, could be approximated by the following expression:

$$I_{de} = S_{de} \left(1 - e^{-\left(\frac{I_e}{S_{de}}\right)}\right) \qquad (5.4)$$

where S_{de} is the maximum storage capacity of depressions, and I_e is the rainfall minus infiltration, interception and evaporation.

For impervious urban areas, Kidd (1978) found a strong correlation between S_{de} and the surface slope, as shown in Figure 5.4. For horizontal or almost horizontal surfaces, depression storage values were reported by many authors in the literature. Such data are summarized in Table 5.2. Initial abstractions which represent mostly depression storage are frequently used in runoff calculations. A list of initial abstractions which depend on land use and surface cover permeability is given in Table 5.3 (Geiger and Dorsch, 1980). Such data can be used directly in Equation 5.2.

Since there is a finite upper limit on S_{de}, it follows from Equation 5.4 that the larger the storm rainfall, the less significant is depression storage in stormwater runoff calculations. Thus, when dealing with intense rain storms in urban areas, depression storage is not an important factor. On the other hand, for less intense storms on catchments with low imperviousness, depression storage would be an important factor. Further discussion of depression storage can be found elsewhere (Viessmann et al., 1972).

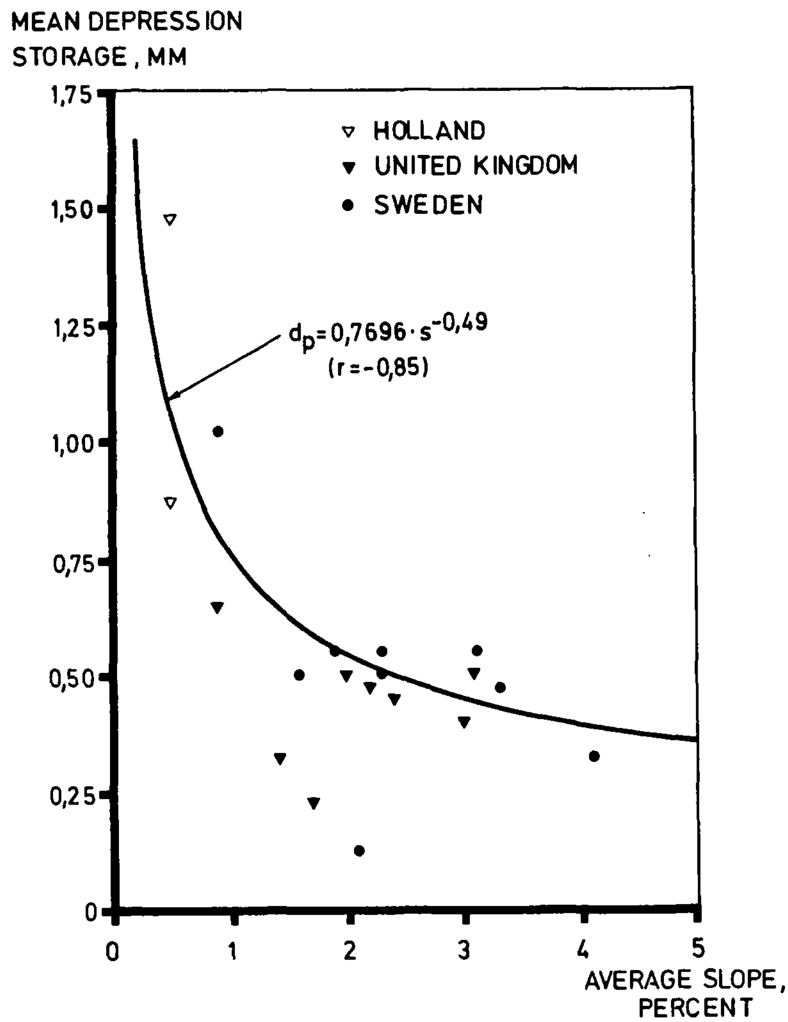


Fig. 5.4 Depression storage abstraction versus impervious area slope (after Kidd, 1978)

5.2.2.4 Infiltration

There are two basic conceptual approaches to infiltration, the soil-physics approach and the hydrological approach. The soil-physics approach relates the infiltration rate to such soil properties as soil conductivity, capillary tension, and moisture content. However, since such soil characteristics vary spatially and temporally and are difficult to measure, the application of this approach in design involves extensive judgement and requires a very large amount of input data. Rose (1966) and Hillel (1971) provided thorough descriptions of physical and chemical factors which may affect infiltration rates. Overton and Meadows (1976) presented a short derivation of the governing equations.

The hydrologic approach is parametric and utilizes lumped characteristics of the soil to simulate infiltration rates. The process parameters used in this approach should be determined by optimization. A comparison of hydrologic infiltration models was given by Overton (1964). The best-known and most widely used infiltration equation which uses the hydrologic approach was proposed by Horton (1939) in the form given by Equation 5.5 and illustrated in Figure 5.5.

$$f = f_c + (f_o - f_c)\, e^{-kt} \qquad (5.5)$$

where f is the infiltration rate at time t (minutes), f_o and f_c are the initial and final infiltration rates (mm/min), respectively, and k is an exponential decay coefficient (min^{-1}). The formula assumes an adequate supply of water so that the soil surface is flooded. This equation was first developed by Gardener and Widstoe (1921) as a special analytical solution of the Green and Ampt equation (Green and Ampt, 1911). Gardener related the k-value to individual soil hydrodynamic variables, while in the Horton equation (Equation 5.5), k is a lumped parameter for the soil hydrodynamic processes.

Table 5.2 Depression storage capacities as reported by various authors

Surface Type	Depression Storage in mm	Reference
Impervious Areas:		
Average surface structure	1,0 - 1,5	Fruehling (1903)
Average surface structure	1,0 - 2,0	Oechsner (1967)
Small areas	1,5 - 2,8	Viessmann (1968)
Smooth surface	0,5 - 0,7	Munz (1966)
Smooth asphalt paving	0,18	Pieper (1938)
Gravelled asphalt	0,52	Pieper (1938)
Concrete pavement	0,35	Pieper (1938)
Rough concrete pavement	0,55	Pieper (1938)
Cobble pavement	1,0	Pieper (1938)
Roofs, flat	2,5 - 7,5	
Roofs, sloped	1,0 - 2,5	
Pervious Areas:		
Bare clay	0,56 - 1,4	Horton (1939)
Bare clay	0,7 - 0,8	Hicks (1944)
Clayey soils with no or little vegetation	0,6 - 1,0	
Clayey soils with vegetation	1,5 - 4,0	Braun (1959)
Bare silty clay	2,0 - 3,0	Neal (1938)
Grass-covered silty clay	1,0 - 2,5	Sharp and Holtan (1940)
Bare clayey sand	3,0 - 4,0	Dvorak (1959)
Clayey-sandy humus with short grass	1,9	Schumann (1940)
Clayey sand with 25% grass cover	3,3	Reinhold (1955)
Clayey sand with 45% grass cover	4,6	Reinhold (1955)
Lawn grass	5,0 - 12,0	
Wooded areas and open fields	5,0 - 15,0	
Clay	2,5	Hicks (1944)
Loam	3,8	Hicks (1944)
Sand	5,0	Hicks (1944)

Table 5.3 Initial losses on impervious and pervious areas (after Geiger and Dorsch, 1980)

Land Use/Soil Surface Characteristics	Initial Loss in mm
Impervious Areas:	
Commercial	0,5 - 2,0
Residential	0,7 - 2,5
Industrial	1,0 - 3,0
Pervious Areas:	
Open space, uncultivated vegetation	10
Cultivated soil (corn, root plants, crops, viniculture, hop culture, etc.)	8
Playground	2
Protected green areas and slopes	5
Garden or meadow	5

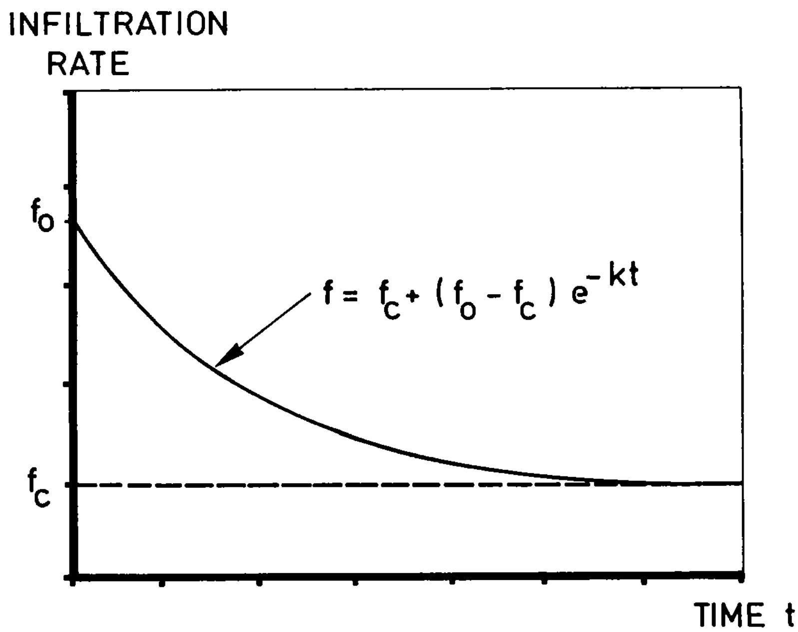


Fig. 5.5 Graphical presentation of Horton infiltration equation.

When applying Equation 5.5, some guidance in selection of parameter values can be found in the literature. Holtan et al. (1967) and Musgrave (1946) recommended some initial infiltration rates which are listed in Table 5.4. The final infiltration rates which were recommended by Chow (1964) are given in Table 5.5. It should be mentioned that some pavements allow infiltration at relatively low rates. Brick and concrete tiles exhibited infiltration rates of about 0,2 mm/min after one hour and up to 0,12 mm/min after 12 hours (van Dam and van de Ven, 1984). Although the last parameter in Equation 5.5, k, depends on soil characteristics, the vegetative cover and antecedent moisture conditions, it is frequently taken as 0,0697 min^{-1}.

Table 5.4 Initial infiltration rate f_o for various conditions

Condition	f_o in mm/min	Source
Soils without vegetation	0,27	Holtan et al. (1967)
Grassed and wooded areas	1,95	Holtan et al. (1967)
Clays without vegetation	0,60	Musgrave (1946)
Sandy soils with dense vegetation	1,80	Musgrave (1946)

Table 5.5 Final infiltration rate f_c for various soil types (after Chow, 1964)

Soil Type	f_c in mm/min
Heavy plastic clays and soils with high swelling	0 - 0,021
Clay loams, shallow sandy loams, soil low in organic matter, soils high in clay	0,021 - 0,064
Sandy loams and shallow loess	0,064 - 0,127
Deep sand, deep loess, aggregated silts	0,127 - 0,190

5.2.3 Lumped rainfall abstractions

5.2.3.1 Runoff coefficients

The runoff coefficient is based on an assumption that rainfall abstractions represent a fixed fraction of rainfall for any given drainage area, although in reality such abstractions (losses) vary in time as a result of changing rainfall, climate and surface conditions. Among the variables in the rational method described in Section 5.3.2, the runoff coefficient is the most difficult one to determine. The ratio of stormwater runoff volume to the total volume of rainfall for a particular area yields the average volumetric runoff coefficient. For the calculation of peak runoff, however, a peak runoff coefficient must be used. The peak runoff coefficient, C_p, is defined by Equation 5.6 below:

$$C_p = \frac{Q_p}{I_p} \tag{5.6}$$

where Q_p = peak runoff rate in litres/s•ha, and
I_p = peak rainfall intensity, in litres/s•ha.

It should be noted that both the peak runoff rate and rainfall intensity were given in litres per second per hectare. Other units are also used in practice as discussed in Section 5.3.2. The relationship between Q_p and I_p is shown in Figure 5.6.

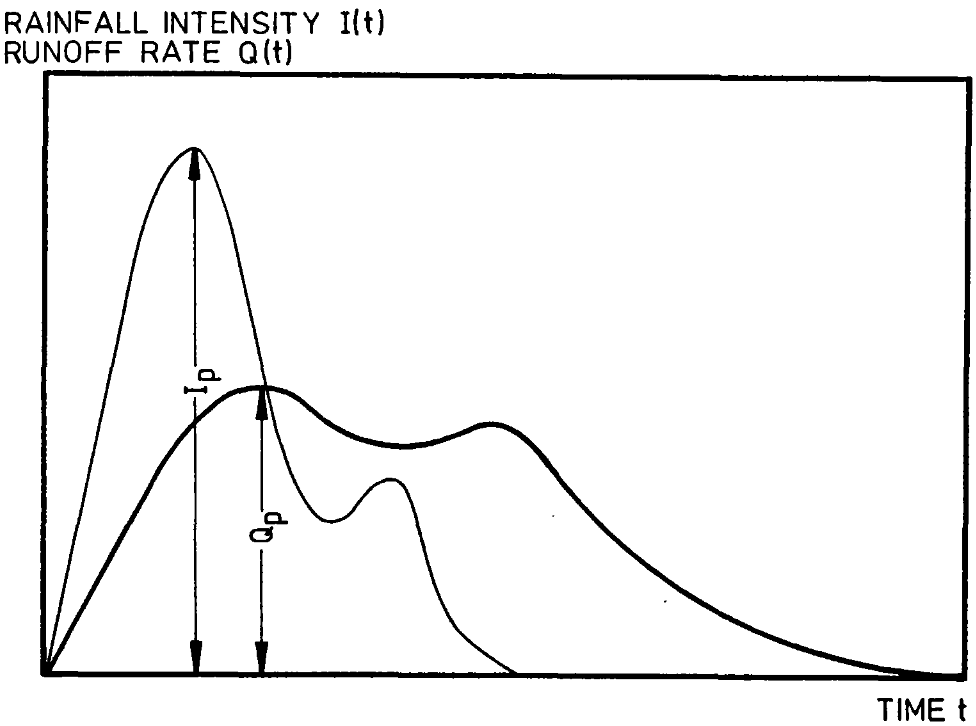


Fig. 5.6 Peak runoff coefficient: notation sketch.

In practical applications, it is common to work with overall runoff coefficients which are defined for various surface covers and are assumed to remain constant during the storm. It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in a drainage area. The peak runoff coefficient is the weighted average of the coefficients for individual subareas according to Equation 5.7:

$$C = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i} \tag{5.7}$$

This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for the entire area. Runoff coefficients for various surface covers currently used in practice are given in Table 5.6.

Table 5.6 Typical runoff coefficients for various surfaces and 2- to 10-year design return period (after WPCF, 1970a; ATV, 1982a; and CIEH, 1984).

Character of Surface		Runoff Coefficients
Pavements		
Asphaltic and concrete		0,70 - 0,95
Brick		0,70 - 0,85
Gravel and cobblestone		0,25 - 0,60
Sand and gravel sidewalks and roads		0,15 - 0,30
Tarred streets and sidewalks		
Slope	0 - 3%	0,85
	3 - 6%	0,90
	>6%	0,95
Lateral streets	0 - 5%	0,35
	>5%	0,50
Roofs		
Steep roofs		0,75 - 0,95
metal and slate		0,95
tiles and roofing felt		0,90
Flat roofs		0,50 - 0,75
Lawns		
Sandy soil:		
Flat	2%	0,05 - 0,10
Average	2 - 7%	0,10 - 0,15
Steep	7%	0,15 - 0,20
Heavy soil:		
Flat	2%	0,13 - 0,17
Average	2 - 7%	0,18 - 0,22
Steep	7%	0,25 - 0,35
Other surfaces		
Undeveloped land		0,05
Treated land (gardens)		0,10
Sport fields and playgrounds		0,20
Railway yards		0,20

The coefficients in Table 5.6 are applicable for storms with frequencies from 2 to 10 years. Less frequent, higher-intensity storms would require the use of larger coefficients, because infiltration and other losses would have a proportionally smaller effect on runoff. On the other hand, for storms with lower intensities, one could use smaller values. The above coefficients are based on an assumption that the design storm does not occur when the ground surface is frozen. The reported practice indicates that the composite runoff coefficients, selected on the basis of experience, produce acceptable results and require limited effort for evaluation of component factors.

5.2.3.2 Φ - Index

The Φ-index which is used as a bookkeeping method for pervious areas, accounts for interception, evaporation, wetting, depression and infiltration abstractions, in order to estimate a uniform abstraction rate from actual storm rainfall and runoff records. By subtracting the runoff volume from the total rainfall volume, the total volume of abstractions is obtained. If this volume is divided by the observed rainfall duration, a uniform abstraction (loss) rate is obtained, which is called the Φ-index. If the runoff measurements were taken in natural channels or streams, baseflow should be subtracted from the hydrograph.

Although the Φ-index method is a rather crude procedure, particularly because of the assumption of constant rather than decreasing abstraction rates, it may be often the best method available, particularly for larger watersheds. The W-index is a refinement of the Φ-index in that it excludes surface storage and retention (Chow, 1964).

5.3 SEPARATE CALCULATIONS OF RUNOFF VOLUME AND PEAK

5.3.1 Application of separate calculations

Calculation methods providing only the peak runoff rate or runoff volume may be derived from local data using generally applicable coefficient methods. While the methods derived from local observations are applicable only to the monitored area or very similar conditions, other coefficient methods are generally applicable. Such methods rely on gross assumptions for rainfall abstractions (Section 5.2.3) and on the choice of frequency and duration of the design rainfall (Chapter 4). Obviously, such assumptions result in considerable variations in the magnitude of design parameters. The simplistic scope of these methods permits and requires, to a large extent, subjective judgement in their application. This is especially true for the calculations of runoff peaks and volumes for different choices of design storm frequency. In calculations of runoff peaks, large errors can be caused by erroneous estimates of the time of concentration and the corresponding rainfall intensity. Because most urban drainage areas are fairly small, the times of concentration are relatively short and this contributes to relatively large errors in their estimates. Such short times then correspond to the steepest portion of the intensity-duration-frequency curves and this may further magnify the overall error in the calculated runoff peak. Although the runoff peak and volume are the key quantities for sizing sewers and detention basins, the restriction of the coefficient methods to the evaluation of individual single storm events is a severe limitation. Rather than providing a large drain capacity required to remove runoff without delay from the catchment area, it may be more economical to provide local detention storage which releases runoff over a longer period and reduces the requirements on drain capacity. Effective design of such integrated systems requires the use of complete runoff hydrographs and, therefore, it precludes the use of coefficient methods. Nevertheless for sewer sizing in small drainage areas, coefficient methods are widely used and useful, if applied correctly.

5.3.2 Runoff volume estimation methods

The runoff curve number method of the Soil Conservation Services (SCS) is a procedure frequently used for estimating runoff volume from storm rainfall. The SCS equation for predicting runoff volume reads as follows:

$$Q = \frac{(P-0{,}2S)^2}{P + 0{,}8S} \qquad \text{for } P > 0{,}2\ S \tag{5.8}$$

$$Q = 0 \qquad \text{for } P < 0{,}2\ S$$

where Q = total runoff volume (mm),
P = total storm rainfall (mm), and
S = potential maximum retention after runoff begins (mm).

In Equation 5.8, the term 0,2S represents the initial abstraction that accounts for all losses prior to the start of runoff and consists mainly of interception, infiltration, evaporation, and surface depression storage.

The potential maximum retention after runoff begins, S, is related to the soil and cover conditions in the watershed through the runoff curve number, CN, by the equation

$$S = \frac{25400}{CN} - 254 \tag{5.9}$$

The hydrologic soil cover complex CN accounts for hydrologic soil group, land use and treatment class, and hydrologic condition. Soils are classified into four groups (A,B,C & D) according to their minimum infiltration rate, which is obtained for a bare soil after prolonged wetting. Watershed cover conditions are described by land use and treatment class. Land use is characterized by the watershed urban and/or agricultural cover which comprises all vegetation, fallows, water surfaces, and impervious surfaces. Land treatment reflects agricultural practices, such as contouring, terracing, and management practices, for example, grazing control, crop rotation, and conservation tillage.

The hydrologic condition is a measure of the land use and treatment effects on runoff. A good hydrologic condition indicates that the soil has a relatively superior capacity for water infiltration. Examples of land use and treatment contributing to a good hydrologic condition include lawns or crop rotations which have a large amount of dense vegetation, land with a small percentage of impervious cover, practices leaving vegetative residue on the land surface, and practices leaving a rough surface. The CN is an indicator of the runoff potential for a unique hydrologic soil cover complex and ranges from 0 to 100. The CN's for cultivated and non-cultivated agricultural lands represent average watershed runoff conditions, when flooding occurs, and were determined by fitting a median curve to the measured rainfall and runoff data from small agricultural watershed areas. The CN's for developing and fully developed urban areas were obtained from comparisons and interpolations of data indicating the effects of imperviousness on runoff and runoff transport from impervious areas to the drainage system. Details of this method are given elsewhere (U.S. Department of Agriculture, SCS, 1975).

5.3.3 Peak runoff estimation methods

The best method for estimating peak runoff rates in ungauged catchments, for given recurrence intervals, is to develop prediction equations based on measurable storm and watershed characteristics. The development of these equations requires an extensive data base of measured peak runoff rates. The forms of these equations and techniques of their derivation are manifold. Popular techniques include linear and nonlinear regressions yielding expressions with independent variables raised to various powers. In other approaches, the independent variables are grouped to form dimensionless parameters raised to a power. The most common variables used in this approach are the drainage area, channel length, watershed slope, stream slope, watershed storage, mean annual precipitation, rainfall intensity for given duration and recurrence interval, bank-full discharge, rural peak runoff rate for a given recurrence interval, land use in the watershed, channel conditions, and percentage of the total area serviced by storm drains, curbs and gutters. These derived relationships can be presented in the form of equations or graphically. As an example of the graphical form, the method developed by Imhoff and Imhoff, (1985) is shown in Figure 5.7. In this method runoff peaks are estimated from the sewer length, flow velocity and runoff coefficient. For example, assuming a sewer length of 7,5 km, an average flow velocity of 2,0 m/s and a runoff coefficient of 15%, a peak runoff flow rate of 11 l/s•ha is obtained from Figure 5.7 (see the worked out example in this figure).

The above procedure is given here only as an example of derivation of design graphs from extensive local observations. Such design graphs are not generally applicable. The graphs in Figure 5.7 apply only to the conditions similar to those in the northwestern part of the Federal Republic of Germany. These conditions are characterized by certain surface covers and rainfall characteristics. The data base required for the derivation of peak runoff estimation methods must be sufficiently long to ensure that events of the magnitude of the desired design frequency are included in this base. In addition, the catchment conditions affecting storm runoff must remain more or less constant throughout the period of observations. Such restrictions, however, can be seldom met because of usual changes in urban development.

5.3.4 Peak runoff calculation by the rational method

In the rational method, the peak runoff rate is related to rainfall intensity by the formula

$$Q = k \cdot C \cdot I \cdot A \tag{5.10}$$

where
- Q = peak runoff rate in litres/s,
- C = runoff coefficient,
- I = average rainfall intensity in litres/s•ha for a duration equal to the time of concentration, T_C,
- A = drainage area in ha, and
- k = general factor for reduction of the runoff peak. If other units for Q and I are used, a unit conversion factor has to be included in Equation 5.10.

The underlying principle of the rational method is the assumption that, for rainfall of a constant intensity, the maximum discharge will occur at a basin outlet at the time when the entire area upstream of the outlet is contributing runoff. This time is referred to as the time of concentration which is defined as the time required for runoff to travel from the hydraulically most distant point in the basin to the outlet. For simplicity, the average flow velocity along this path is taken as 1 m/s. To account for the reduction in runoff peak because of the travel time and for spatial distribution of rainfall in larger catchments, the k-factor is added. Other key assumptions of the rational method include the following:

- The frequency or return period of the computed peak flow is the same as that of the design storm,
- The rainfall intensity is constant for the entire rainfall duration and spatially uniform for the whole area under analysis,
- The necessary basin characteristics can be identified, and
- The runoff coefficient does not vary during the storm.

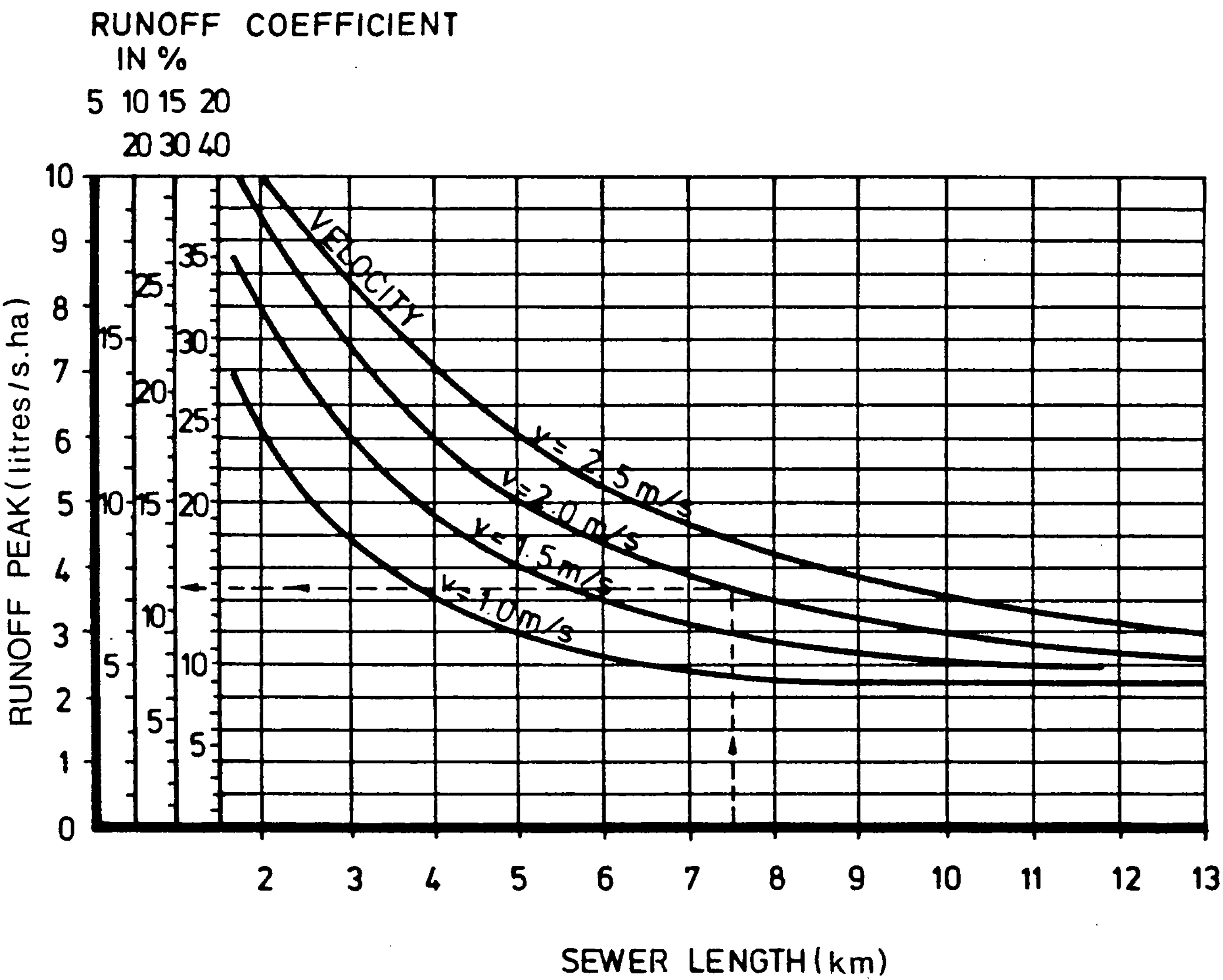


Fig. 5.7 Estimation of runoff peaks from sewer lengths, flow velocities and runoff coefficients (after Imhoff and Imhoff, 1985).

The contributing area at any point under consideration in a drainage system is the only element of the rational method subject to precise determination. Boundaries of the drainage area may be established by field surveys or from suitable maps or aerial photographs. The entire drainage area is subdivided into subareas, each of which drains into a particular inlet. This requires a preliminary layout of the system and tentative locations of inlet points. Rearrangements of the system layout or of inlet locations often become necessary as the design proceeds.

The determination of values of the coefficient C is difficult because this factor combines many variables including infiltration, ground slope, ground cover, surface depression storage, antecedent precipitation and soil moisture, shape of the drainage area, overland flow velocity, and others. A detailed discussion of this coefficient and its values for practical applications were given earlier in Section 5.2.3. As described in Chapter 4, the determination of the rainfall intensity I for drainage design involves considerations of an average frequency of rainfall occurrence, intensity-duration characteristics of rainfall for this selected frequency of occurrence, and an estimate of the time of concentration. As already discussed, all of these components depend on subjective judgement.

Traditionally, the use of the rational method has been recommended for small areas, usually less than 300 ha, with tree-type drainage networks. The application of the rational method in planning is further discussed in Chapter 8. In larger basins, the sewer or channel system is more complex and proper analysis of such systems usually requires the use of hydrograph methods which account for flow routing and channel storage effects (Kibler, 1982). In spite of the limitations of the rational method, long time experience with its use has resulted in practical definitions of its variables and its generalized representation of runoff is feasible if the designer uses judgement in evaluating the component factors. Thus, the rational method, if correctly used, yields satisfactory results for relatively small areas.

In different countries, various modifications of the basic rational method have been developed, accounting for slope and shape of the drainage area, the distribution of impervious areas within the catchment and the magnitude of the design rainfall intensity. Such modifications are usually reported in national handbooks, such as those prepared by WPCF (1970a) and ATV (1982a and 1982b). Because these modifications are tied to network calculations, they are further discussed in Chapter 8.

5.3.5 Substitution hydrographs derived by the rational method

The peak flow computed by the rational method may be considered as the peak of an equilateral triangular hydrograph with a base length equal to two times the time of concentration. This assumption holds true only if the duration of the rainfall is assumed equal to the time of concentration, which is the normal design assumption in the rational method.

If the rainfall duration is longer than the time of concentration, a trapezoidal hydrograph is obtained and its peak flow is again equal to CIA as previously derived. However, for a given frequency, the peak flow of the trapezoidal hydrograph is smaller than that of the triangular hydrograph because the intensity of rainfall decreases with the duration of the rainfall. Thus, for any rainfall frequency-intensity-duration curve described in Chapter 4, it is possible to draw a family of hydrographs, as shown in Figure 5.8, with increasing rainfall durations. The triangular hydrograph, shown in Figure 5.8 and derived for the duration of rainfall equal to the time of concentration, gives, as previously stated, the maximum runoff rate and is normally used for sizing sewer pipes and other conveyance elements. However, other hydrographs in Figure 5.8, which have a trapezoidal shape, produce greater volumes of runoff. Such volumes are equal to the area of hydrograph trapezoids.

Once such a family of hydrographs has been developed, the storage volume of a detention basin may be computed for an allowable outflow rate from the basin. However, this procedure is not recommended for general use, because superior hydrologic and hydrodynamic methods are available for hydrograph calculation, as discussed in the next section.

By plotting the allowable outflow hydrograph in conjunction with the family of runoff hydrographs, the required storage volume of the detention basin can be computed as the maximum area (storage volume) below one of the trapezoidal hydrographs and above the outflow hydrograph (the cross-hatched area in Figure 5.8).

Although the use of the rational method hydrograph procedure seems attractive because of its ease and simplicity, the results may be inaccurate. Consequently, the use of such a procedure should be limited to small catchments with areas of 4 ha or less (Wanielista, 1979). More reliable hydrograph procedures, whose applications require calculation efforts comparable to those of the rational method, are described in Section 5.4.

5.4 HYDROGRAPH CALCULATION BY HYDROLOGIC METHODS

5.4.1 Application of hydrologic methods

As previously mentioned, a serious limitation of the rational method is the fact that it yields only the peak flow. A major alternative is to calculate the entire runoff hydrograph and to account for flow detention. As shown in Section 5.3.5, such a hydrograph can be approximated by a procedure which adds more assumptions to the rational method. However, it is preferable to derive runoff hydrographs by hydrologic or hydrodynamic methods, particularly because these methods allow for verification of calculated hydrographs against the measured ones.

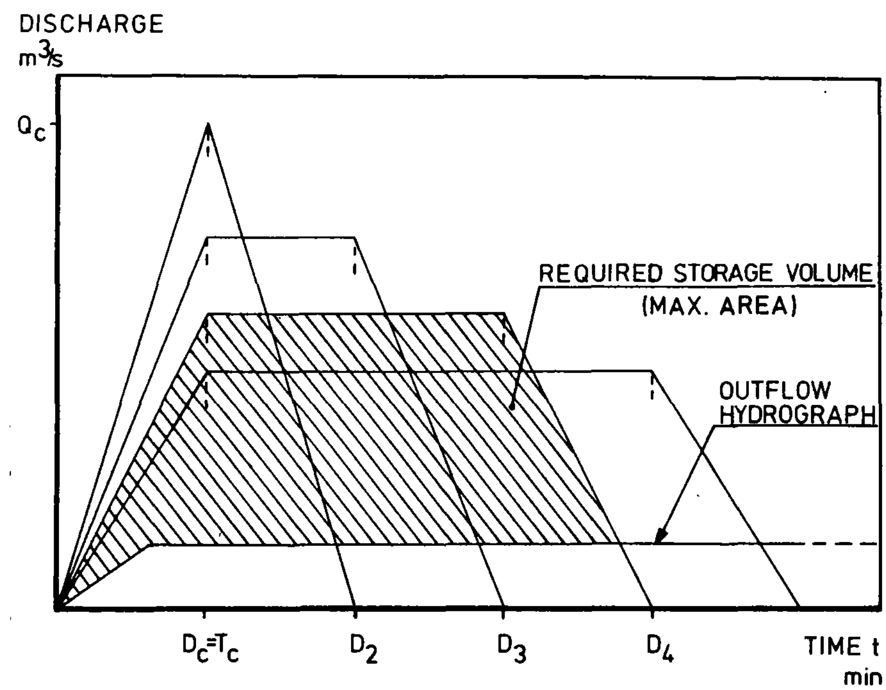


Fig. 5.8 Family of hydrographs developed from the rational method procedure (after Wanielista, 1979).

While the hydrodynamic methods require clear and detailed hydraulic boundary conditions, hydrologic methods are insensitive to changes in such conditions along the flow path, as long as the overall catchment and system characteristics remain constant in space and time. Therefore, surface runoff or combinations of runoff and sewer flow may be modelled with hydrologic methods, even though the flow path changes as water flows from roofs to drains, from sidewalks to gutters, and from gutters into sewers. Transfer functions or model parameters, which form a substantial part of hydrologic methods, reflect the characteristics of the total area. Therefore, the input requirements for hydrologic methods are much less detailed and intensive than those for hydrodynamic methods, and basically represent proper definitions of transfer functions and model parameters. The simplicity of the hydrologic methods results in limited mathematical efforts and low computing time requirements.

While for empirical formulae of the rational method type the design intensity represents the rainfall input, actual rainfall patterns or synthetic design storm hyetographs serve as rainfall inputs for hydrograph calculation methods. For derivation of design frequencies and design storms, the reader is referred to Chapter 4.

5.4.2 Conceptual models

The conceptual approach to the modelling of rainfall-runoff processes in a given catchment is mainly based on system analysis in which the catchment is considered as a system converting input, the rainfall, into an output, the runoff at the catchment outlet (see Figure 5.9).

Mathematically, the catchment is represented by a response function, h, transforming inputs, I's, into outputs, Q's. Thus, the conceptual modelling of the rainfall-runoff processes basically consists in identifying the function h. That may be accomplished in two ways, either by an analytical approach based on the system analysis theory or by the synthesis approach in which an a priori given function h is fitted to the rainfall, runoff and catchment data.

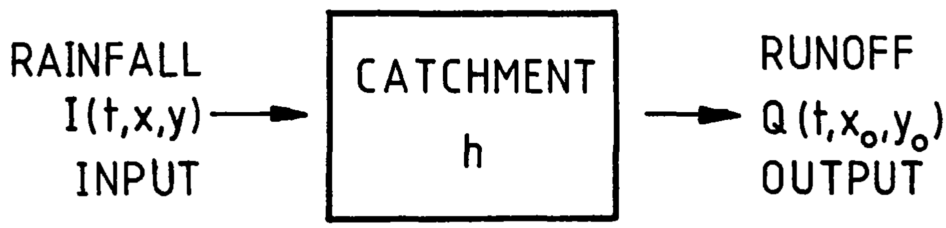


Fig. 5.9 Transformation of rainfall into runoff.

5.4.3 Analytical derivation of the transfer function

The analytical derivation of the transfer function is essentially based on the linear systems theory. The simplest form of the rainfall-runoff transformation is given by the classical convolution integral for a lumped, linear, time-invariant system written as follows:

$$Q(t) = \int_0^t I_{ex}(\tau)\, h(t-\tau)\, d\tau = \int_0^t I_{ex}(t-\tau)\, h(\tau)\, d\tau \tag{5.11}$$

For continuous rainfall excess $I_{ex}(t)$ and runoff $Q(t)$, the function h, which is called the transfer function, represents the system response to a Dirac impulse function. In hydrologic applications, this function h is called the Instantaneous Unit Hydrograph (IUH). Figure 5.10 shows the transformation described by Equation 5.11.

According to the linear system theory, some other forms of the convolution integral (Equation 5.12) may have to be used in special cases. For example, if the rainfall-runoff process varies in time and space, the corresponding system is considered to be time-varying and spatially distributed. Equation 5.11 is then rewritten in the following form:

$$Q(t) = \int_0^A \int_0^t I_{ex}(u, \alpha)\, h(u, t-u, A-\alpha)\, du\, d\alpha \tag{5.12}$$

where A is the catchment area. Usually, the rainfall input in the form of hyetographs is given in a discrete form, and consequently, Equation 5.11 is written as:

$$Q(j\Delta t) = \sum_{i=1}^{j} I(i\Delta t)\, H(t-j)\Delta t \tag{5.13}$$

where the function H is the system response to a unit volume input over the duration of Δt. The function H is known as the so-called unit hydrograph which was first derived by Sherman (1932) and further developed by many other researchers (Chow, 1964).

The derivation of the function H may be accomplished by various mathematical methods. However, the most widely used method is that based on the matrix form of Equation 5.13 which is given as Equation 5.14 below:

$$\begin{bmatrix} I_1 & & & \\ I_2 & I_1 & & \\ \cdot & I_2 & \cdot & \\ \cdot & \cdot & \cdot & I_2 \\ I_N & \cdot & \cdot & I_2 \\ & I_N & & \\ & & \cdot & \cdot \\ & & & I_N \end{bmatrix} \begin{bmatrix} H_1 \\ H_2 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ H_n \end{bmatrix} = \begin{bmatrix} Q_1 \\ Q_2 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ Q_m \end{bmatrix} \tag{5.14}$$

or as

$$IH = Q \tag{5.15}$$

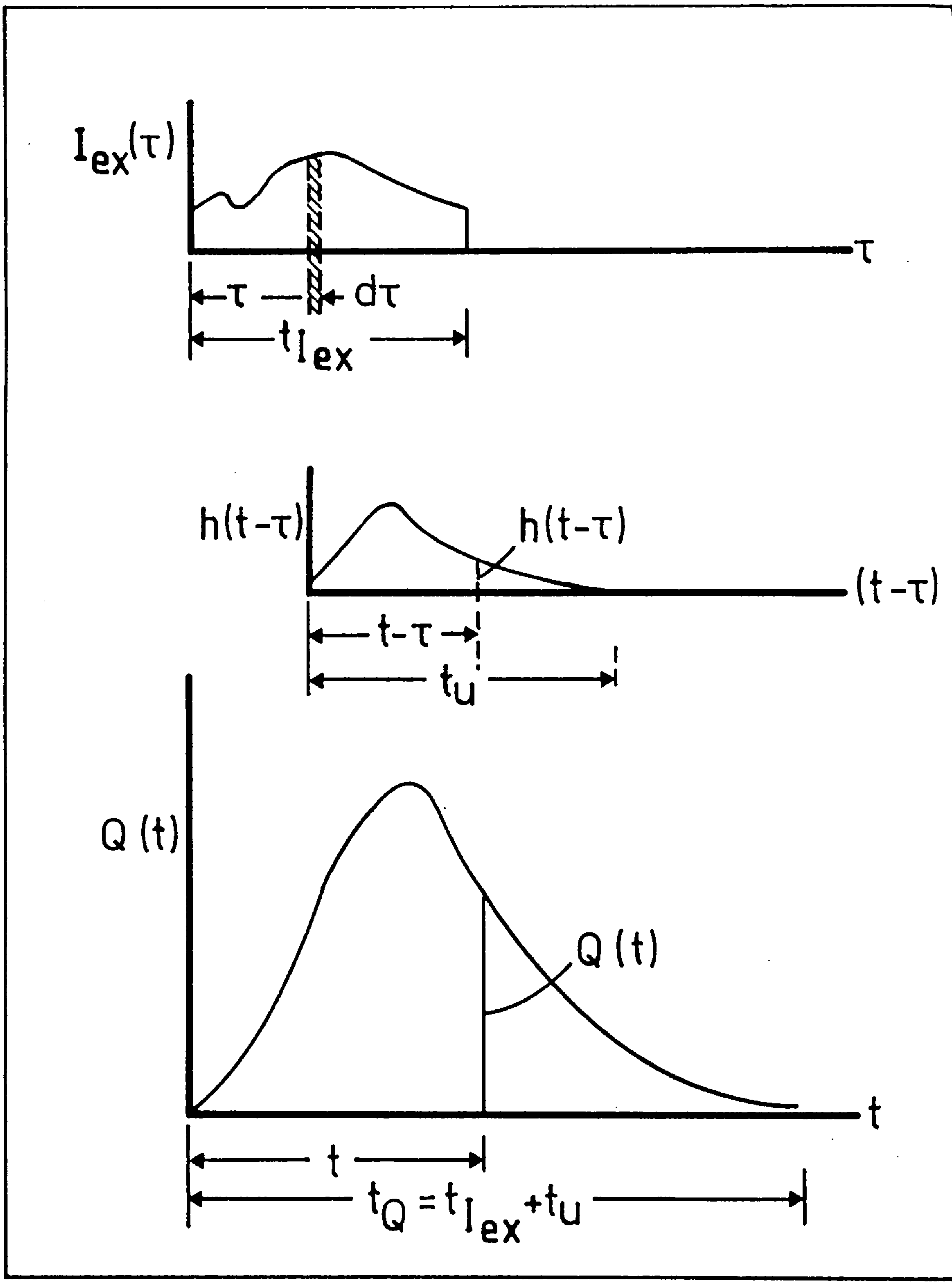


Fig. 5.10 Determination of storm runoff hydrograph by the unit hydrograph method.

In general the problem posed by the discrete convolution relation is over-determined because there are more equations than unknowns ($m > n$). A unique solution can be obtained by applying the least-squares criterion for minimization of the residual sum of squares. This is a widely used expedient and a variety of treatments of least-squares methods can be found in algebra or statistical handbooks. The use of the least squares method to derive a unit hydrograph from observed data often produces an unrealistic solution. A common fault is that the derived unit hydrograph is unstable and even contains negative ordinates. Of course the response of a catchment to a pulse of net rainfall should be smooth with usually just one maximum point and two inflexion points, one on the rising limb and one on the receding limb. The above anomaly stems from the fact that the unit hydrograph is defined in terms of a large number of parameters (ordinates) and consequently has the flexibility to approximate very closely the observed relationship between net rainfall and quick-response runoff. The least squares criterion ensures that this is done, and, in the absence of any restrictions on the relative values of the ordinates, the end product can be an unstable unit hydrograph.

Various modifications to the least-squares method have been proposed to ensure the stability of derived unit hydrographs. The restricted least-squares method attains a stable result by reducing the number of ordinates defining the unit hydrograph. A simpler alternative is to smooth out the derived unit hydrograph. Various methods for smoothing derived unit hydrographs and averaging techniques for individually derived unit hydrographs are discussed by Boorman and Reed (1981).

From numerical solutions of Equation 5.15, some researchers tried to derive empirical relationships for unit hydrographs in order to find some specific hydrograph patterns that could be used for ungauged catchments. This has been rarely successful and, therefore, the analytical approach is seldom used in urban hydrology.

5.4.4 Synthesis of the transfer function

In the approach based on synthesis, a given function h is applied to rainfall-runoff data and the function parameters are fitted to the data using some criteria for the goodness of fit. Many functions can be used for this purpose. In such procedures, the rainfall-runoff process is considered as a result of two elementary effects:

- Translation effects by means of which the input $I_{ex}(t)$ translates into $Q(t+T)$, where T is a translation (lag) time, and
- Storage effects described by a general storage equation in the form:

$$S(t) = \sum_{n=0}^{N} A_n \frac{d^nQ}{dt^n} + \sum_{m=0}^{M} B_m \frac{d^mI_{ex}}{dt^m} \tag{5.16}$$

where $S(t)$ is the water temporarily stored on the catchment and A_n and B_m are storage constants.

Thus, a conceptual rainfall-runoff model derived by the synthetic approach will comprise the following terms:

- The continuity equation, i.e.,

$$dS/dt = I_{ex}(t) - Q(t) \tag{5.17}$$

- Various equations describing the translation and/or storage effects. Although there are no restrictions on the variety of such conceptual models, only a few of them produced satisfactory results for urban catchments. These are discussed in the following sections.

5.4.4.1 Models considering translation effects only

The so-called isochrone model is the most widely used translation conceptual model. In this approach, each point in the catchment is assigned a travel time required by a water particle to reach the outlet of the catchment. By connecting all the points with equal travel times, the so-called isochrones are obtained. The maximum travel time is called the time of concentration. By integrating the areas contained by individual isochrones, a time-area diagram is constructed as shown in Figure 5.11. According to this figure, the area contributing runoff increases linearly with time until the rainfall duration equals (or exceeds) the time of concentration and the total area contributes runoff.

From the mathematical point of view, the isochrone method should be considered as a spatially-distributed, time-invariant, linear model. For a discrete input, the system equation may be written as:

$$Q(n\ dt) = \sum_{j=1}^{n} I_{ex\ j}\ A(n+1-j) \tag{5.18}$$

where A(n+1-j) is the ordinate of the time-area diagram (see Figure 5.11). Equation 5.18 may be written in a matrix form similar to that of Equation 5.14. The response function is given by the time-area diagram. Such a model can take into account the areal distribution of rainfall or runoff coefficients over the catchment. In that case, Equation 5.18 can be rewritten as:

$$Q(n\ dt) = \sum_{j=1}^{n} \bar{I}_{(j,\ A(n+1-j))}\ \bar{C}_{(j,\ A(n+1-j))}\ A_{(n+1-j)} \tag{5.19}$$

where $\bar{I}$ and $\bar{C}$ are the mean rainfall and mean runoff coefficient during the j-th time interval, respectively, and are related to the $A_{(n=1-j)}$ element of the area. It should be noted that the function h is not specified here, but simply given by the catchment characteristics. Consequently, this model, if adequate, could be used for ungauged catchments. On the other hand, when this model is fitted to a given data set, it can be used to identify runoff coefficients. However, the neglect of storage effects, inherent to this model, will generally lead to overestimates of runoff or runoff coefficients.

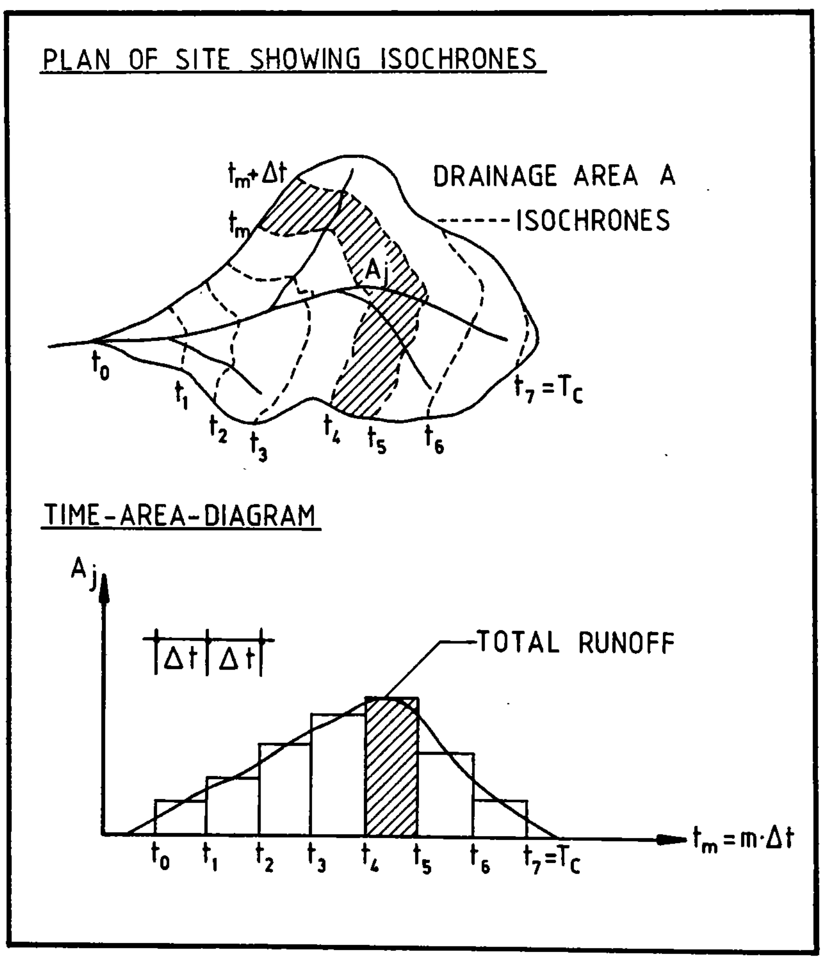


Figure 5.11 Isochrone method.

5.4.4.2 Models considering storage effects only

Among all the storage models that could possibly be derived from Equations 5.16 and 5.17, only a few have been widely used and successfully tested. Several such models are discussed below.

The single linear reservoir model uses a simplified form of Equation 5.16 in the form:

$$S = K\,Q \tag{5.20}$$

where K is the only model parameter and it is expressed in time units. The IUH of this model can be expressed as:

$$h(t) = (1/K)e^{-t/K} \tag{5.21}$$

A discrete integration of the model equation leads to the following explicit relationship:

$$Q(j\,\Delta t) = Q((j-1)\Delta t)\,e^{-\Delta t/K} + I_{ex}(j\Delta t)(1-e^{-\Delta t/K}) \tag{5.22}$$

The Muskingum linear storage model uses a modified version of Equation 5.16 in the form:

$$S = K[xQ + (1-x)\,I_{ex}] \tag{5.23}$$

where K and x are two model parameters. Usually, the Muskingum procedure equation is established by numerical integration of the differential equation derived from Equations 5.17 and 5.23, for constant K and x, in the following form:

$$Kx\,\frac{dQ}{dt} + Q = I_{ex} - K(1-x)\,\frac{dI_{ex}}{dt} \tag{5.24}$$

A discretized numerical integration of Equation 5.24 will generally lead to an explicit relationship written as:

$$Q(j\Delta t) = C_1\,Q((j-1)\Delta t) + C_2\,I_{ex}\,(j\Delta t) + C_3\,I_{ex}\,((j-1)\Delta t) \tag{5.25}$$

The cascade of linear storage reservoirs (Nash, 1957) is schematically shown in Figure 5.12. When all the reservoirs have the same K parameter, the IUH of this model is expressed as:

$$h(t) = \frac{1}{K(n-1)!}\left(\frac{t}{K}\right)^{n-1} e^{-t/K} \tag{5.26}$$

The cascade model has two parameters, K and n. In practical applications, n does not have to be an integer and the factorial (n-1)! in Equation 5.26 is then replaced by the Gamma function.

All the three models discussed are linear and this feature generally allows simple explicit calculations of the runoff Q(t). Some researchers also proposed various non-linear storage models arguing that the rainfall-runoff process is non-linear. The most widely tested and simplest non-linear storage model is described by a storage equation in the form:

$$S = KQ^n \tag{5.27}$$

This model has two parameters, K and n. From Equations 5.17 and 5.27, it is possible to derive a differential equation of the rainfall-runoff process, for K and n invariable in time, in the following form:

$$K\,n\,Q^{n-1}\,\frac{dQ}{dt} + Q = I_{ex} \tag{5.28}$$

For a given time interval, Equation 5.28 can be solved numerically using implicit methods.

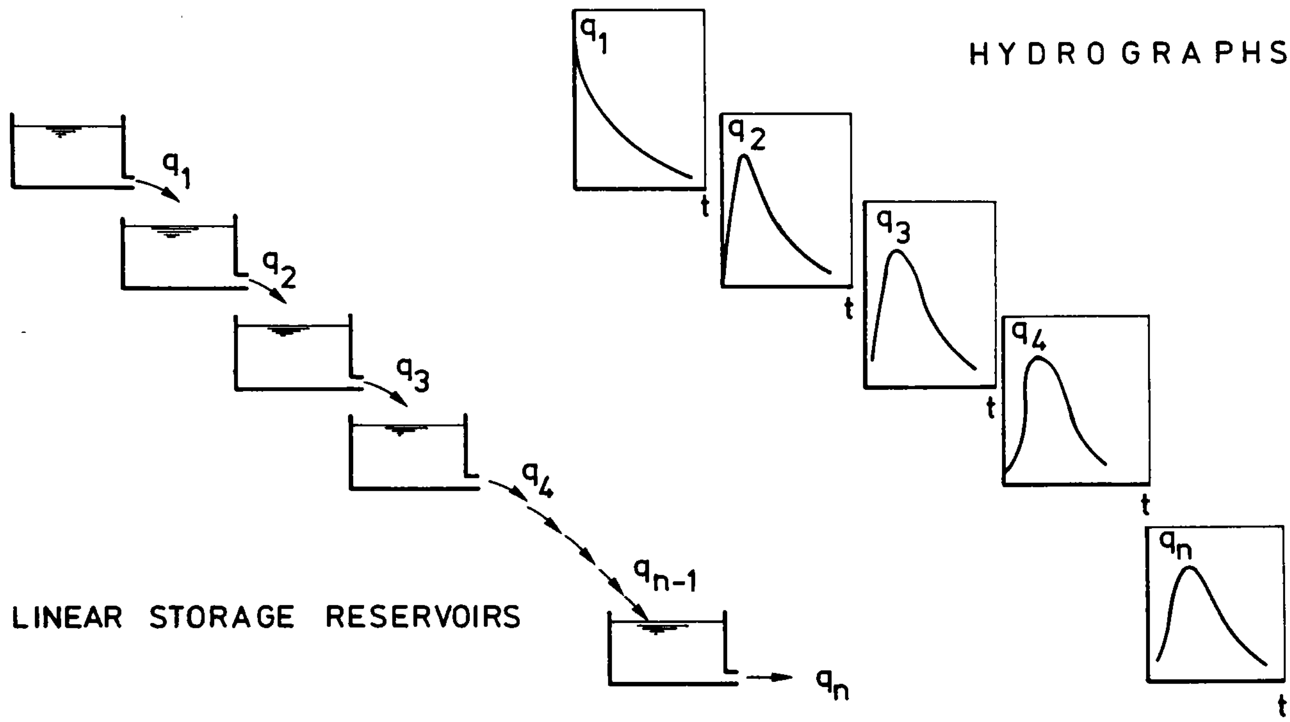


Figure 5.12 Routing of instantaneous inflow through a series of linear storage reservoirs.

5.4.4.3 Models considering both translation and storage effects

Although the combination of storage and translation effects should generally lead to unlimited modelling approaches, only a few such combinations have been tested against the actual data and evaluated for general applicability to the actual phenomena. Two such models are presented below.

The isochrone method (also referred to as a generalized rational method) can be combined with a linear reservoir. In this approach, the output Q(ndt) given by Equation 5.18 is routed through a linear reservoir (Equation 5.22).

The linear or non-linear reservoir models can be modified to account for translation effects. The corresponding storage equations are written for a linear reservoir as:

$$S(t) = K\, Q(t+\tau) \tag{5.29}$$

and for a non-linear reservoir as:

$$S(t) = K\, Q^n(t+\tau) \tag{5.30}$$

where τ is a parameter reflecting translation effects (Kidd, 1978).

5.4.5 Determination of parameters of conceptual models

Usually, the determination of conceptual model parameters is done by fitting the calculated runoff to the observed one using various fitting criteria, such as for example, the least-squares procedure. When the parameters are numerous, the fitting procedure becomes rather complex. It is then desirable to simplify the fitting procedure by determining one or more parameters from some hydrodynamic considerations, as done for example for the non-linear reservoir model, and apply the fitting procedure to the remaining parameters.

The fitting of a given model to a sample of rainfall-runoff data will produce a sample of numerical values of the model parameters. Except for the case of constant parameters, the resulting model with fitted parameters will be ineffective in predicting runoff from another sample of rainfall data, even if the model fitted well the initial data sample. Consequently, some researchers used multivariate analysis techniques to derive relationships between the parameter values and the catchment and rainfall characteristics that can be easily estimated. For example, for the linear reservoir model, Viessman (1968) established the following expression, for the parameter K, applicable to catchments with areas up to 4000 m^2, slopes from 1 to 8%, and various degree of imperviousness:

$$K = 0{,}525 \frac{(n\ L)^{0{,}66}}{S_o^{\ 0{,}33}} \tag{5.31}$$

where n is the Manning coefficient of roughness of the catchment surface, L is the maximum flow length (m), and S_o is the mean slope along the flow path.

Desbordes (1978) applied the linear reservoir equation to 21 U.S. and French urban catchments with areas up to 100 ha. In such cases, the transfer function describes both surface runoff and flow transport in sewers. A multiple regression of measured data yielded an equation for the parameter K in the following form:

$$K = \frac{5{,}1\ A^{0{,}18} \cdot T_E^{0{,}21} \cdot L^{0{,}15}}{S_o^{0{,}36} N_{TE}^{0{,}07} \cdot (1+\alpha)^{1{,}9}} \text{ (in minutes)} \tag{5.32}$$

where
- A = the catchment area (ha),
- T_E = the duration of the peak rainfall intensity (minutes),
- L = the length of the catchment (m),
- S_o = the slope, dimensionless,
- N_{TE} = the total rainfall (mm) for the duration T_E, and
- α = (area contributing runoff)/(total area).

Again, after including rainfall the model actually becomes nonlinear.

Further details on the determination of parameters of conceptual models, especially for linear reservoirs in series and non-linear single reservoirs, can be found in Chow (1964), Dyck (1980) and Kidd (1978).

However, it should be emphasized that such equations as 5.31 and 5.32 are statistically based and depend on the characteristics of the catchment sampled, the general climatic and urbanization characteristics, and other factors. Consequently, they should not be used without some testing of their applicability to other specific conditions.

5.5 HYDROGRAPH CALCULATIONS BY HYDRODYNAMIC AND SIMPLIFIED HYDRODYNAMIC METHODS

5.5.1 Application of hydrodynamic methods

While hydrologic methods use functions to transform rainfall patterns into discharge hydrographs and derive the corresponding water levels from steady-state flow relationships, hydrodynamic methods solve physically-based equations of conservation of mass and momentum using water level and discharge as dependent variables and time and space as independent variables. Geometrical data are used to describe effects of retention and only basic hydrologic parameters are needed to represent additional phenomena such as friction.

In hydrodynamic stormwater computations, the following three subsystems are commonly distinguished:

- surface runoff
- sewer flow, and
- streamflow or flow in other types of receiving waters.

In general, each of the above subsystems can influence only the downstream subsystems. Consequently, the subsystems can be treated one after another, proceeding in the direction of flow. Computational results for the upstream subsystem then serve as boundary conditions for the downstream subsystem. Because the first subsystem, surface runoff, is typically calculated by means of coefficient methods, hydrological methods or simplified hydrodynamic methods (e.g., the kinematic wave equation), the section on hydrodynamic methods deals mostly with sewer flow.

5.5.2 Hydrodynamic equations

Without significant simplifications, sewer flow can be considered as a horizontal one-dimensional unsteady gravity flow. Consequently, the so-called St. Venant equations of momentum and continuity describe accurately discharge, or velocity and depth, as functions of distance and time. In terms of discharge, the continuity equation can be written as

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{5.33}$$

and the momentum equation can be written as

$$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{1}{gA}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + \cos\theta\,\frac{\partial h}{\partial t} - (S_o - S_f) = 0 \tag{5.34}$$

In terms of velocity, Equations 5.33 and 5.34 can be written as

$$\frac{\partial h}{\partial t} + D\frac{\partial v}{\partial x} + v\frac{\partial h}{\partial x} = 0 \tag{5.35}$$

$$\frac{1}{g}\frac{\partial v}{\partial t} + \frac{v}{g}\frac{\partial v}{\partial x} + \frac{\partial}{\partial x}(h\cos\theta) - (S_o - S_f) = 0 \tag{5.36}$$

where
- Q = the discharge,
- v = the velocity,
- h = the depth measured normal to the channel bottom,
- S_o = the channel slope,
- θ = the angle between channel bottom and the horizontal,
- D = the ratio of flow area to water surface width (the mean hydraulic depth),
- x = the distance in the direction of flow, and
- S_f = the friction slope.

The transient nonuniform flow in a sewer pipe is shown in Figure 5.13.

In Equations 5.34 and 5.36, S_f is usually evaluated using the Manning or Darcy-Weisbach formulae, with S_f replacing S_o, and using the local values of discharge, velocity and depth. Both Manning and Darcy-Weisbach formulae are further discussed in Chapter 6.

5.5.3 Simplification of the hydrodynamic equations

Because of the complexity of the St. Venant equations, numerous simplifications are used in their applications. Table 5.7 summarizes various forms and simplifications of the momentum equation.

In Equations 5.36 to 5.40, $\partial v/\partial t$ is the local acceleration resulting from the change of velocity with time (transient flow), $\partial v/\partial x$ is the convective acceleration resulting from the change of velocity in flow direction (nonuniform flow) and $\partial(h\cdot\cos\theta)/\partial x$ is the change of water depth in flow direction. The simplest form of the momentum equation is the kinematic wave approximation which neglects all except the last two terms of the complete momentum Equation 5.36 (see Equation 5.40). If Equation 5.40 is combined with the continuity equation, the resulting calculation yields a routed hydrograph which theoretically has no attenuation of the peak. In such calculations, only hydrograph translation and some shape modification are achieved. Note, however, that the numerical procedure usually introduces some numerical attenuation. If in addition to using Equation 5.40 the velocity v in the continuity Equation 5.35 is kept constant, the runoff hydrograph is only translated and its shape remains unchanged.

The diffusion-wave approximation, Equation 5.39, incorporates the last three terms of Equation 5.36 and thereby permits hydrograph attenuation. The quasi-steady dynamic wave, Equation 5.38, adds the convective velocity term and leads to a more accurate description of the hydrograph movement which is similar to that in the complete momentum equation.

A complete analysis of the effects of neglecting individual terms of Equation 5.36 on the runoff hydrograph was given by Verworn (1980). His numerical tests were carried out for a 1000 m sewer pipe with a slope of 1%, a Strickler roughness coefficient of 40 $(m/s)^{1/3}$ (Manning n = 0,025 $(sm^{-1/3})$), a baseflow of 0,075 m^3/s and a free outflow. In these calculations, the pipe length was discretized into 20 m sections and a time step of 20 s was used. As a flow input, two hydrographs of different steepness, denoted as QZ in Figures 5.14 and 5.15, were used. The notation assigned to the routed hydrographs in Figures 5.14 and 5.15 corresponds to that listed for various simplifications of the momentum equation in Table 5.7.

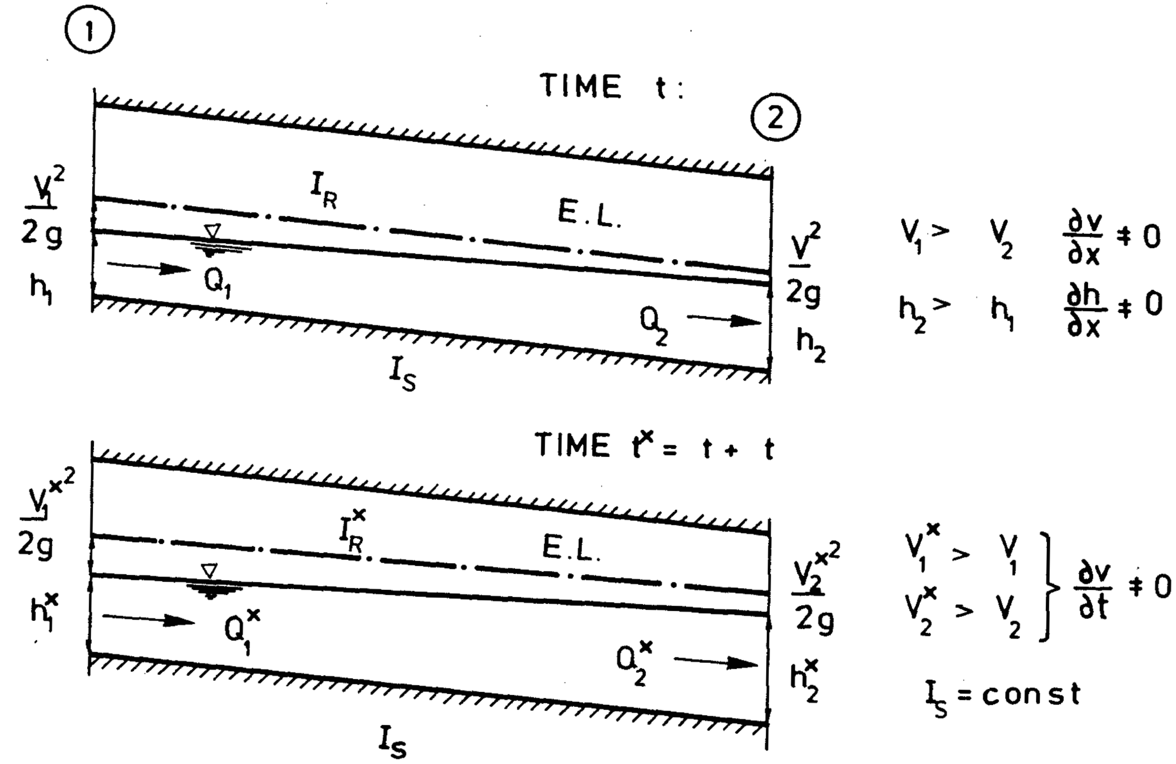


Fig. 5.13 Transient non-uniform flow in a sewer pipe.

Table 5.7 Simplifications of the momentum-equation

Case	Flow Mode	Equation	Equation Number
A	Transient-nonuniform (complete momentum equation)	$\frac{1}{g} \cdot \frac{\partial v}{\partial t} + \frac{v}{g} \cdot \frac{\partial v}{\partial x} + \frac{\partial}{\partial x}(h \cdot \cos\Theta) = S_o - S_f$	5.36
B	Transient-uniform	$\frac{1}{g} \cdot \frac{\partial v}{\partial t} + \frac{\partial}{\partial x}(h \cdot \cos\Theta) = S_o - S_f$	5.37
C	Simplified transient-nonuniform (quasi-steady dynamic wave)	$\frac{v}{g} \cdot \frac{\partial v}{\partial x} + \frac{\partial}{\partial x}(h \cdot \cos\Theta) = S_o - S_f$	5.38
D	Simplified transient uniform (diffusion wave)	$\frac{\partial}{\partial x}(h \cdot \cos\Theta) = S_o - S_f$	5.39
E	Kinematic wave $v = f(h)$	$0 = S_o - S_f$	5.40a
F	Kinematic wave $v = v_o$ = constant	$0 = S_o - S_f$	5.40b

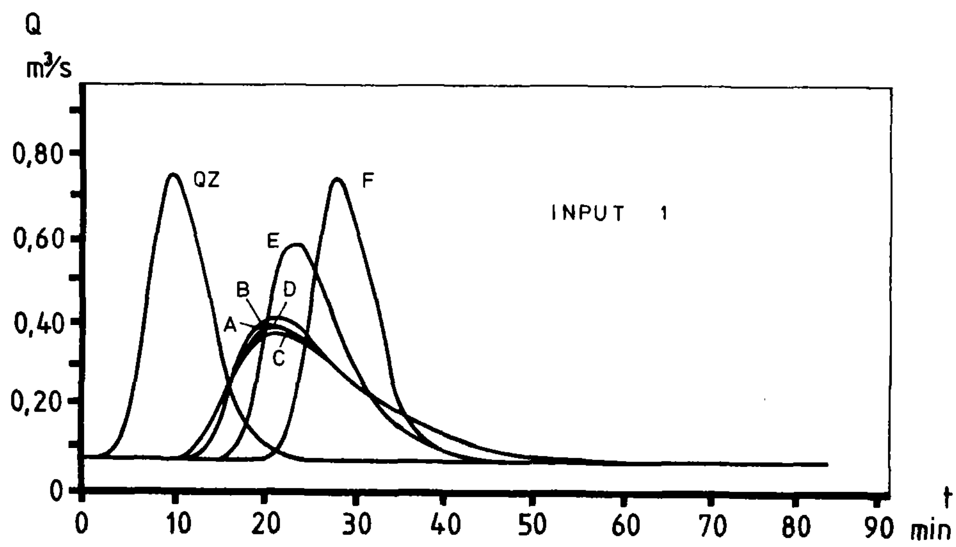


Fig. 5.14 Flow routing using full and simplified St. Venant equations: runoff hydrographs for input hydrograph No. 1 (after Verworn, 1980).

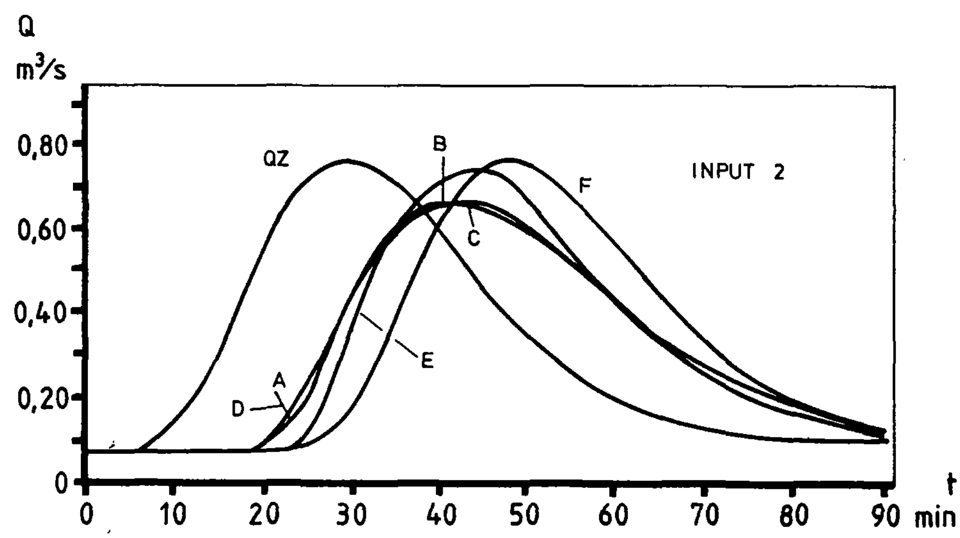


Fig. 5.15 Flow routing using full and simplified St. Venant equations: runoff hydrographs for input hydrograph No. 2 (after Verworn, 1980).

The magnitudes of deviations resulting from neglecting $\partial v/\partial t$, case B, or $\partial v/\partial x$, case C, from the results obtained by solving the complete St. Venant equations are similar. Case B results in slightly larger and case C in slightly smaller runoff peaks, respectively. Case D approximated best the exact solutions, case A, although both $\partial v/\partial t$ and $\partial v/\partial x$ were neglected. This close agreement can be explained by the fact that, in case D, both neglected terms are of a similar magnitude, but opposite signs. Consequently, a better fit is obtained by neglecting both terms rather than one of them. In comparison to case D, case E also neglects the term $\partial(h \cdot \cos\theta)/\partial x$ which is significantly large and its neglect leads to increased errors. For both inputs, case E produced higher runoff peaks than cases A, B and C. This is caused by neglecting the retention term. Finally, in case F, it is assumed that $v = v_o$ = constant, and consequently, the input hydrograph is simply translated from the inlet to the outlet. The lag time is equal to the time of travel. This case corresponds to the use of the rational method as a hydrograph method (see Section 5.3.5).

5.5.4 Solution methods for the hydrodynamic equations

There are four basic methods for solving the complete St. Venant equations:

- Analytical solutions which are feasible only for simplified systems. They may be used to verify numerical solutions and to study basic phenomena. Other than that, they have no practical applications.
- Transformation into characteristic equations which facilitates graphical or numerical solutions. For supercritical flow as well as for teaching purposes, this is a useful method for computing sewer flow. In general, the method of characteristics is considered to be less useful than the next two methods listed below.
- Finite difference solutions of the basic differential equations (5.33 and 5.34), and
- Finite element solution by using the Galerkin approach.

The last two methods may lead to either implicit or explicit equations. The former ones, linear or nonlinear, have to be solved simultaneously by iteration or direct-solution techniques, such as the Gauss algorithm. In the latter case, two explicit equations are solved for every point in time and space to obtain the two independent variables.

Although the finite element and finite difference methods can be shown to produce identical results, there are some basic differences between implicit and explicit numerical schemes. The implicit schemes are unconditionally stable, but they are not always consistent (i.e., do not converge to the proper solution). The implicit schemes may be more complicated to solve, but their high computational efficiency leads to reduced computer time requirements. The double sweep method is an important technique in this category (Cunge, 1961).

The explicit numerical methods are stable only if sufficiently small time steps are used. The selected time step should satisfy the Courant criterion written as

$$\Delta t < \frac{\Delta x}{v \pm \sqrt{gh}} \tag{5.41}$$

where Δt is the computational time step, Δx is the length step, v is the flow velocity, h is the depth of flow, and g is the acceleration due to gravity.

The major advantage of explicit schemes is their consistency of stable solutions. This somewhat compensates for lower sophistication and greater computer time requirements.

Numerous comparisons of various solution schemes have been carried out (Brandstetter, 1976) without proving any particular method clearly superior to others. The most important difference can be seen in the appropriate application of suitable models to the problem at hand.

5.6 COMPARISONS OF VARIOUS RUNOFF COMPUTATION APPROACHES

In summary, the estimation and coefficient methods of Section 5.3 yield only discrete values of the peak flow and runoff volume. Such results are adequate only for smaller areas with simple tree-type sewer networks. A comprehensive overview of the limitations of the rational method (Section 5.3.4) was given by McPherson (1969). When dealing with larger urban drainage areas and especially the design of storage facilities, the methods yielding runoff hydrographs, which were described in Sections 5.4 and 5.5, should be used. The hydrological synthesis and hydrodynamic calculations given in Sections 5.4 and 5.5 are preferable to hydrograph derivations based on coefficient methods (given in Section 5.3.5), in order to avoid the extensive and questionable assumptions required in the latter approach. Advantages and disadvantages of the individual methods were described in Sections 5.3.1, 5.4.1 and 5.5.1.

Comparative tests of various modelling approaches have been performed on hypothetical test catchments or sewer systems (Section 5.5.3) as well as on actual systems with observed field data. The comprehensive models further discussed in Chapter 8 comprise mostly the basic approaches and concepts given in Sections 5.2, 5.4 and 5.5. Further details of such models and their computational schemes are given in Chapter 8. Comparisons of runoff models have been conducted by two types of research groups:

- Model builders testing their own model and sometimes comparing it with other models (Watkins, 1962 and 1976; Metcalf and Eddy et al., 1971; Swinnerton et al., 1972; Papadakis and Preul, 1973; Terstriep and Stall, 1974; Watt and Kidd, 1975; Chow and Yen, 1975; Helliwell et al., 1976; Kidd, 1978; Geiger and Dorsch, 1980; Geiger, 1984).
- Independent teams evaluating one or more models (Terstriep and Stall, 1969; Waller et al., 1976; Stall and Terstriep, 1972; Heeps and Mein, 1973; Marsalek et al., 1975; MacLaren, 1975; Aitken, 1975; KWK/ATV, 1977; Abbott, 1978; Geiger, 1984).

The most comprehensive overview of comparative tests was provided by Colyer (1977). He found that peak discharge and runoff volume were best predicted by hydrologic methods (Section 5.4). The rational method (Section 5.3.5) has not performed well, partly because it is strictly a design method and some of its assumptions are inappropriate for reproducing actual events which are significantly smaller than the design event. While most of the models tested were acceptable in their calculation of time to peak, the simulated runoff volumes showed large variations in their accuracy. One of the later studies (Geiger, 1984) found that hydrograph calculations by hydrologic and hydrodynamic methods agreed well with field data from a 540 ha urban catchment as long as the sewer system was not surcharged. However, for comprehensive sewer networks with surcharging, the advantages of the hydrodynamic methods may be outweighted by numerical instabilities. When examining model comparisons, it should be realized that their results depend on the characteristics of the study area, the accuracy of field measurements, author's experience with individual models and, last but not least, the author's objectives or personal bias.

While applications of coefficient and hydrologic methods require very little data and computing time, hydrodynamic calculations are extremely time consuming in both data preparation and computing. On the other hand, proper definition of parameters for the coefficient and hydrologic methods requires much more experience than the preparation of data for hydrodynamic methods.

Finally, while coefficient methods can hardly be checked against field data, hydrologic and hydrodynamic calculations can be easily compared with measurements. Such measurements are to some extent necessary for successful application of hydrologic models. Furthermore, verifications of model results by field data increase the acceptance of model results and designs based on such calculations.

Intentionally, no recommendations to use a specific method are given, because the successful application of any method also depends on the availability of local data and the engineering experience. For instance, the use of the rational method by an experienced designer may produce better results than the application of a hydrodynamic method by inexperienced personnel.

6 Hydraulics of conduits and open channels

6.1 HYDRAULIC DESIGN OF DRAINAGE SYSTEMS

Drainage systems consist of various conveyance elements and special structures which convey runoff from the catchment surface to the point of disposal. Descriptions of such elements and structures were given in Chapter 3. In hydraulic design of drainage systems, it is required to determine design flows in all system elements using design parameters described in Chapter 4 and computational procedures given in Chapter 5. After such design flows have been established, it is possible to proceed with hydraulic design of individual conveyance elements and special structures. Towards this end, basic hydraulic principles required for sizing of conveyance elements, such as conduits and open channels, are discussed in this chapter. The hydraulic design of special drainage structures is discussed in Chapter 7 and special topics of network design are treated in Chapter 8.

A properly designed and functioning drainage system must effectively convey the maximum discharge, for which it is designed, transport solids suspended in runoff to minimize their deposits and the resulting odour nuisance, and avoid structural damages due to excessive flow velocities. A hydraulic design meeting such performance criteria is not an easy task considering large variations in drainage flows and their composition. It should be realized, however, that a cost effective drainage system cannot be designed without a thorough hydraulic analysis considering all drainage components as a system of interrelated elements.

Drainage systems consist of open channels and subsurface conduits. Thus from the hydraulics point of view, both open-channel flow and pressure flow are encountered in drainage design. Furthermore, the flow can be uniform or nonuniform, and steady or unsteady. The discussion that follows deals mostly with steady uniform flow and, to a lesser degree, with nonuniform flow. Selected aspects of unsteady flows in drainage systems are addressed in Chapter 8.

The material presented in this chapter starts with a review of fundamentals of hydraulic analysis, followed by a discussion of pressure flow design, open-channel design, velocity considerations, and nonuniform flow problems.

6.2 HYDRAULIC FUNDAMENTALS

6.2.1 Types of flow

Flow is considered to be steady if the flow rate at a point in a conveyance element remains constant with time and unsteady if it varies. Although the flow in drainage networks is in many cases unsteady, it is often acceptable to simplify the hydraulic analysis by assuming steady flow conditions. Note that the acceptance of such an assumption is also affected by the choice of the runoff computation method. Some methods assume a constant rainfall and, consequently, the calculated runoff and the flow in the drainage network are steady. Cases where an unsteady flow analysis needs to be applied are discussed in Chapter 8.

Steady open-channel flow is said to be uniform if the velocity and the depth do not change along the channel and nonuniform if the velocity or depth, or both change. Pressure flow is uniform if the flow area remains uniform along the conduit.

Other types of flow include laminar and turbulent flows, and subcritical, critical, and supercritical flows in open channels. The flow in drainage systems is always considered to be turbulent. A further discussion of the above types of open-channel flows is given later.

6.2.2 Basic principles

In most hydraulic designs of drainage systems, variation of flow characteristics across any section may be ignored and the analysis is then limited to the one-dimensional case, thus considering only changes of mean values in the direction of flow. Basic tools used in such an analysis include the continuity, momentum and energy principles. Such principles are briefly described below in forms suitable for practical design. For a detailed treatment of these subjects, the reader is referred to the literature (Chow, 1959; Rouse, 1961; Streeter, 1971).

For steady incompressible flow, the equation of continuity is expressed as

$$Q = A_1 V_1 = A_2 V_2 \tag{6.1}$$

where Q is the discharge, A is the cross-sectional area, V is the mean velocity, and the subscripts designate two sections as shown in Figure 6.1.

The momentum principle may be written as

$$\sum F_x = \frac{\gamma}{g} A \left[(\beta V_x)_2 - (\beta V_x)_1 \right] \tag{6.2}$$

where x represents an arbitrary direction, F_x is the summation of the x-components of external forces acting on the fluid body under consideration, Q is the discharge, γ is the specific weight, g is the gravity acceleration, β is the momentum correction factor (taken as $\beta = 1$ in sewer design work), V_x is the x-component of the mean velocity, and the subscripts 1 and 2 designate two sections.

Finally, the energy principle may be written as

$$\frac{\alpha V_1^2}{2g} + z_1 + y_1 + H_a = \frac{\alpha V_2^2}{2g} + z_2 + y_2 + H_L \tag{6.3}$$

where α is the kinetic energy coefficient, V is the mean velocity, y is the pressure head above the invert ($y = p/\gamma$, where p is the pressure and γ is the specific weight), z is the elevation above the datum, H_a is the energy gain between sections 1 and 2 (e.g., as produced by a pump), and H_L is the energy loss between the two sections (see Figure 6.1).

For the sizing of drainage elements, Equation 6.3 can be further modified by assuming uniform flow and evaluating energy losses. Two types of losses are distinguished - friction losses and form (minor) losses. Friction losses are caused primarily by viscous and turbulent shears along the boundary of the conduit. Form losses are caused by shear as well as pressure differentials caused by various hydraulic phenomena. Both types of losses are discussed in more detail later in this chapter.

6.2.3 Flow friction formulae

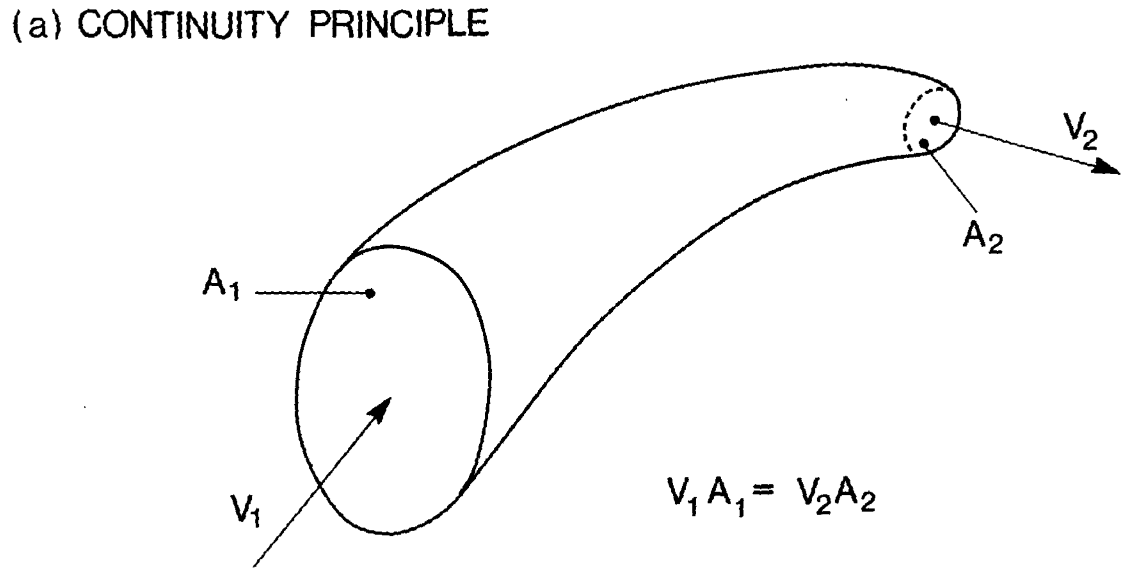
A wide variety of flow equations has been developed to overcome difficulties associated with the application of the energy principle to the solution of design problems. Most of these equations, which are applicable to both pipes and channels, are essentially empirical. Their applications are limited to steady uniform flow in which only friction losses are present.

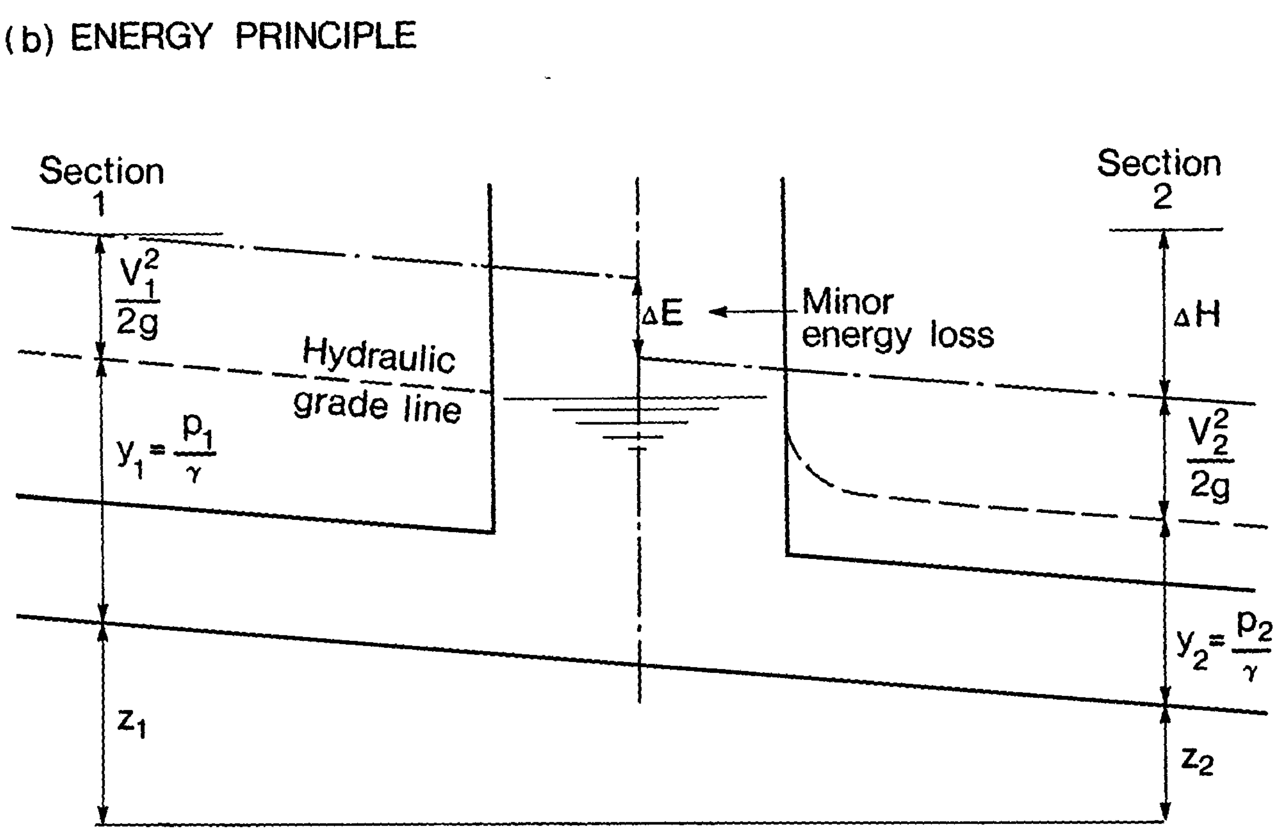
Over the years, many flow friction formulae have been proposed and recommended for various applications. A detailed treatment and comparisons of various formulae are beyond the scope of this report and the reader is referred to the literature on this subject (Chow, 1959; Henderson, 1966). For practical reasons, the discussion is limited to the Manning and Darcy-Weisbach formulae, which are widely used by practicing engineers and should be adequate for most drainage design problems.

6.2.3.1 Manning equation

The Manning equation is one of the oldest and simplest flow friction formulae. Because of recommendations by various authors and authorities (WPCF, 1970a), this formula is widespread in sewer design practice, particularly in North America. The Manning equation may be written, in metric units, as

$$V = \frac{1}{n} R^{2/3} S_e^{1/2} \tag{6.4}$$





$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + \Delta H$$

Fig. 6.1 Notation sketch for continuity and energy principles where V is the mean velocity, n is the Manning roughness coefficient, R is the hydraulic radius (equal to the flow cross-sectional area divided by the wetted perimeter), and S_e is the slope of the energy grade line (equal to the invert slope for steady uniform flow).

Typical values of the Manning roughness coefficient n for various materials are listed in Table 6.1. It should be noted that the values of n from 0.013 to 0.015 are most frequently used in sewer design.

Table 6.1. Manning's friction coefficient and values of effective absolute roughness for various conduit materials (after WPCF, 1970a)

	Manning's Coefficient n	Effective Absolute Roughness k (mm)
Closed Conduits		
Asbestos-cement pipe	0,011 - 0,015	0,3 - 3,0
Brick	0,013 - 0,017	1,5 - 6,0
Cast iron pipe		
Uncoated (new)	-	0,3
Asphalt dipped (new)	-	0,1
Cement-lined and seal coated	0,011 - 0,015	0,3 - 3,0
Concrete (monolithic)		
Smooth forms	0,012 - 0,014	0,3 - 1,5
Rough forms	0,015 - 0,017	1,5 - 6,0
Concrete pipe	0,011 - 0,015	0,3 - 3,0
Corrugated metal pipe (12 mm x 70 mm corrugations)		
Plain	0,022 - 0,026	30,0 - 60,0
Paved invert	0,018 - 0,022	9,0 - 30,0
Spun ashpalt lined	0,011 - 0,015	0,3 - 3,0
Plastic pipe (smooth)	0,011 - 0,015	0,3
Vitrified clay		
Pipes	0,011 - 0,015	0,3 - 3,0
Liner plates	0,013 - 0,017	1,5 - 3,0
Open Channels		
Lined channels		
Asphalt	0,013 - 0,017	-
Brick	0,012 - 0,018	-
Concrete	0,011 - 0,020	0,3 - 9,0
Rubble or riprap	0,020 - 0,035	6,0
Vegetal	0,030 - 0,040	-
Excavated or dredged		
Earth, straight and uniform	0,020 - 0,030	3,0
Earth, winding and fairly uniform	0,025 - 0,040	-
Rock	0,030 - 0,045	-
Unmaintained	0,050 - 0,140	-
Natural channels (minor streams, top width at flood stage less than 30 m)	-	30,0 - 90,0
Fairly regular section	0,030 - 0,070	-
Irregular section with pools	0,040 - 0,100	-

Two more points about Manning's n should be mentioned. Firstly, the coefficient varies with the depth of flow as demonstrated in Figure 6.2. Secondly, the coefficient slightly increases with the pipe size and this may have to be taken into account when dealing with large pipes. For practical purposes, the coefficient n is usually considered as a constant regardless of the depth of flow or the conduit dimensions. Applications of the Manning equation to drainage design are discussed later.

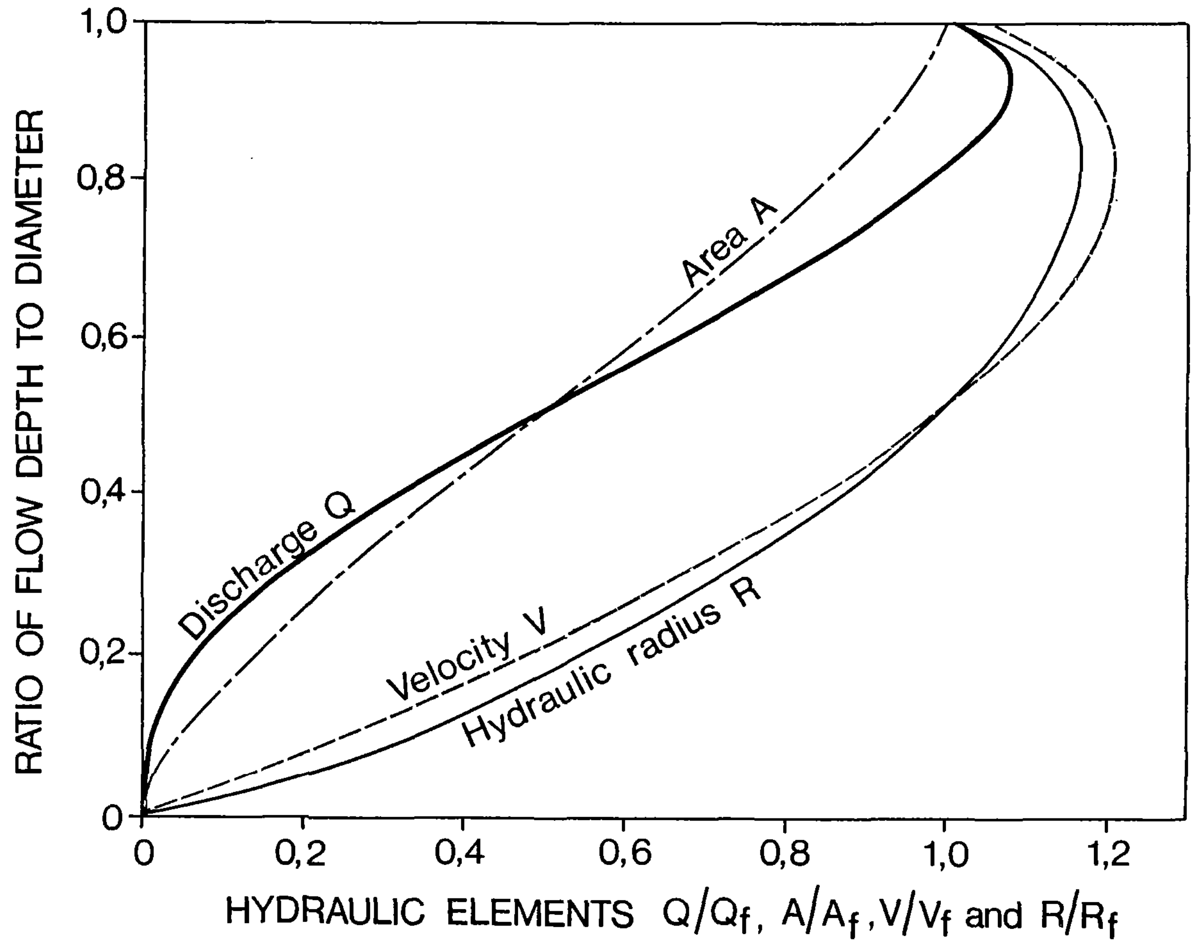


Fig. 6.2 Hydraulic elements for circular sewers

6.2.3.2 Darcy-Weisbach equation

The Darcy-Weisbach equation represents a basic approach to the formulation of flow equations. Its advantages include dimensional correctness and applicability to any fluid over a wide range of conditions. For open channels, this equation may be written as

$$S_e = \frac{f}{4}\frac{}{R}\frac{V^2}{2g} \tag{6.5}$$

where S_e is the slope of the energy grade line, f is a dimensionless friction factor, and R, V, and g have been defined previously.

For pressure flow, the equation is usually written as

$$h_f = \frac{f\,L}{D}\frac{V^2}{2g} \tag{6.6}$$

where h_f is the friction head loss, L is the length of the conduit, and D is the conduit (pipe) diameter.

It can be shown that the friction factor f may be expressed as

$$f = \phi \left(\mathrm{Re}, \frac{k}{D}\right) \tag{6.7}$$

where ϕ is a function, Re is the Reynolds number, and k is the effective absolute pipe roughness which is listed for various materials in Table 6.1. The Reynolds number used in this connection is expressed as

$$\mathrm{Re} = \frac{4\ R\ V}{\nu} \tag{6.8}$$

where ν is the kinematic viscosity (frequently taken as $\nu = 1{,}141 \times 10^{-6}\ m^2 s^{-1}$, for water at 15°C). For pressure flow in circular pipes, Equation 6.8 further simplifies to

$$\mathrm{Re} = \frac{D\ V}{\nu} \tag{6.9}$$

The relationship for the friction factor f described by Equation 6.7 has been experimentally studied by many researchers. As a result of these studies, various equations and graphs have been proposed for the general relationship in Equation 6.7. Among these, the most widespread is the Colebrook-White expression written as

$$\frac{1}{\sqrt{f}} = -2 \log \left(\frac{k}{3{,}7\ D} + \frac{2{,}51}{\mathrm{Re}\sqrt{f}}\right) \tag{6.10}$$

The main disadvantage of Equation 6.10 stems from the fact that it has to be solved by iterations. Such problems become relatively minor if one uses a programmable calculator to undertake these iterations. Furthermore, graphs and explicit approximations for the friction factor f have been developed to simplify calculations.

For design purposes, a graphical presentation of Equation 6.7 is particularly popular and a number of graphs have been prepared for this purpose. Among these graphs, the Moody diagram, which is shown in Figure 6.3, is particularly popular.

Further information on applications of the Darcy-Weisbach equation to drainage design and on explicit approximations for the friction factor f is given in Sections 6.3 and 6.4.

6.2.4 Form losses

Energy losses resulting from rapid changes in the direction or magnitude of the velocity are called form losses, or also minor or local losses. Under special circumstances, these losses may become fairly large and even exceed the friction losses.

Form losses are usually expressed in terms of the kinetic energy, in the following form:

$$h_L = K \frac{V^2}{2g} \tag{6.11}$$

where h_L is the form loss, and K is the energy head loss coefficient. The loss coefficient K varies depending on the magnitude of flow velocity and direction changes at various hydraulic structures.

For some design work, it may be more convenient to express the form loss in terms of the equivalent length of straight pipe, which is then added to the actual pipe length. The earlier discussed flow friction formulas can be used for this purpose.

Form losses usually occur in drainage systems at various transitions which connect conduits of different characteristics. For smaller conduits, transitions are typically sudden and are confined to manholes. For larger conduits, more gradual and hydraulically efficient transitions may be in order.

The consideration of form losses is important for the calculation of water surface profiles and hydraulic grade lines. A brief summary of common form losses follows; additional details can be found in hydraulic handbooks (Brater and King, 1976; WPCF, 1970a; Miller, 1971; Wright-McLaughlin Engineers, 1969).

Most of the losses discussed here apply to the pressure flow. In open channels the energy considerations are more complex and will be further discussed in the section on nonuniform flow problems.

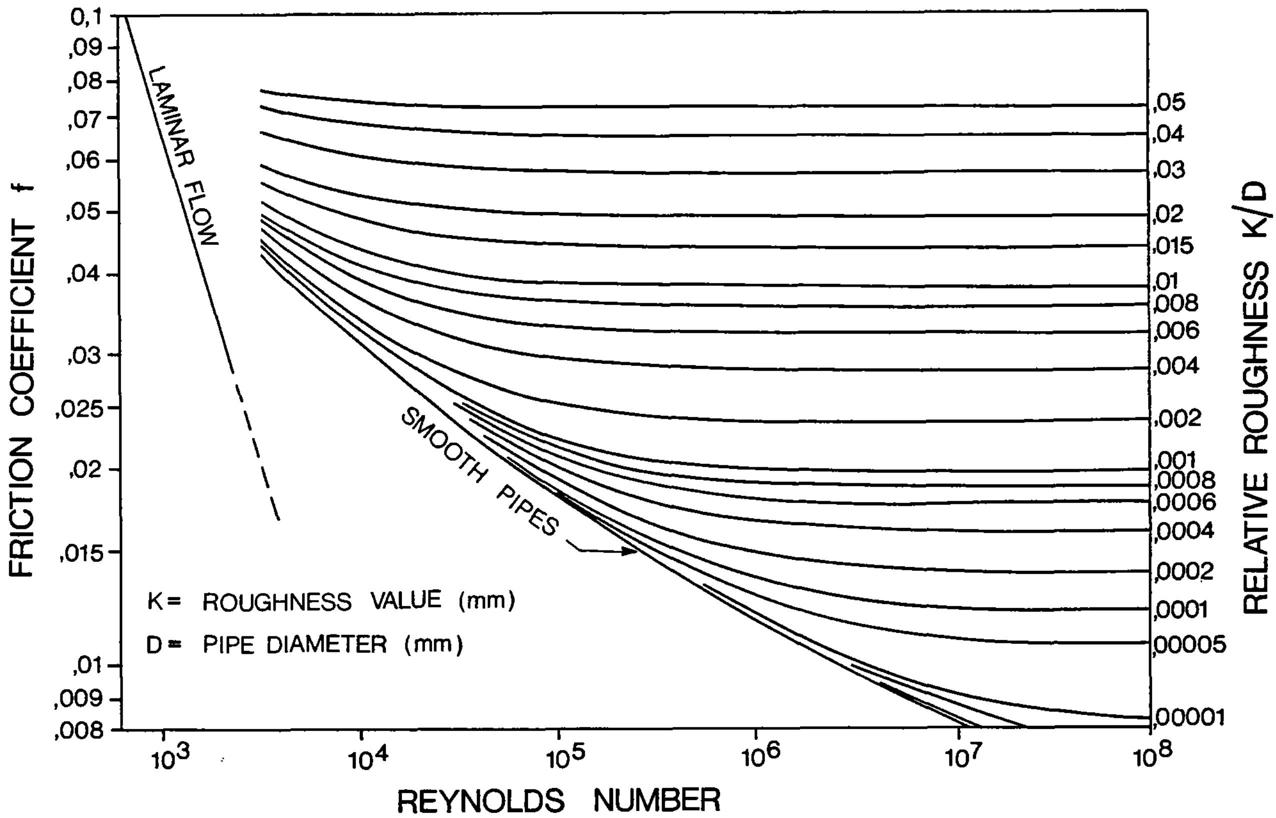


Fig. 6.3 Moody diagram: Friction factor versus Reynolds number and relative roughness

6.2.4.1 Entrance loss

The loss coefficient for the entrance loss varies from 0,2 for well-designed (rounded edge) entrance to 1,0 for inward projecting pipe without a headwall. For a square-edged entrance flush with a headwall, the loss coefficient equals 0,5 (Brater and King, 1976).

6.2.4.2 Contraction and expansion loss

For pressure flow, the loss due to a contraction can be expressed as

$$h_L = K \frac{V_2^2}{2g} \left[1 - \left(\frac{A_2}{A_1}\right)^2\right]^2 \tag{6.12}$$

where V_2 is the downstream velocity, A_1 and A_2 are the flow areas upstream and downstream of the contraction ($A_1 > A_2$), and the coefficient K is equal to 0,5 for a sudden contraction and to about 0,1 for a well-designed gradual contraction (WPCF, 1970a).

The expansion loss in pressure flow can be expressed as

$$h_L = K \frac{V_1^2}{2g} \left(1 - \frac{A_1}{A_2}\right)^2 \tag{6.13}$$

where V_1 is the upstream velocity and K is equal to 1,0 for a sudden expansion and to about 0,2 for a well-designed transition (WPCF, 1970a).

Somewhat similar expressions have been derived for subcritical open-channel flow and well-designed transitions (no wall deflecting more than 12.5°) (American Iron and Steel Institute, 1980):

Contraction Loss ($V_2 > V_1$) $$h_L = 0,1\ (\frac{V_2}{2g} - \frac{V_1}{2g}) \qquad (6.14)$$

Expansion Loss ($V_1 > V_2$) $$h_L = 0,2\ (\frac{V_1^2}{2g} - \frac{V_2^2}{2g}) \qquad (6.15)$$

For supercritical flow, the evaluation of losses is more complicated and additional factors, such as standing waves, need to be considered. Details can be found elsewhere (Chow, 1959).

6.2.4.3 Manhole loss

Manhole losses are probably the most common form losses in sewer systems. Basically, a manhole loss represents a combination of the expansion and contraction losses. For straight-flow-through manholes, the loss coefficient varies from 0.05 to 0.35 depending on the manhole geometry (Archer et al., 1978; Marsalek, 1984). Losses at other types of junction manholes are much more complex and generally depend on both junction geometry and flow conditions. A detailed discussion of such losses can be found in the literature (Marsalek, 1985; Wright-McLaughlin Engineers, 1969).

6.2.4.4 Bend loss

The sewer pipe alignment frequently changes. Such changes in the alignment, bends, can be either gradual as in the case of curved sewers, or sudden. Sudden bends are located at manholes fitted with deflectors.

Values of the bend loss coefficient are given in Figure 6.4 (American Iron and Steel Institute, 1980) for various bend arrangements and angles of deflection. For deflection angles smaller than 40°, one may also use the following formula (Wright-McLaughlin Engineers, 1969):

$$h_L = 0,25 \sqrt{\frac{\gamma}{90°}\ \frac{V^2}{2g}} \qquad (6.16)$$

where γ is the deflection angle.

6.3 PRESSURE FLOW

Calculations of pressure flows in drainage pipes are based on the continuity and energy equations. In a general case, all the energy losses need to be evaluated and substituted into the energy equation (Equation 6.3). Various formulae for evaluating form losses were presented in Section 6.2.4; the procedures for the calculation of friction losses and the sizing of pipes flowing full are given below.

In the hydraulic drainage design, the designer is usually given the design discharge, then selects a pipe material which determines the pipe roughness, and needs to determine either the pipe diameter D, for a chosen slope S, or the pipe slope for a chosen diameter. In both cases, the choices of D and S have some economic implications. The larger the pipe diameter, the higher the cost of the pipe. The steeper the slope, the smaller the pipe diameter required, but the excavation costs may increase. Further information on these aspects and the minimum-cost drainage system design can be found elsewhere (Tang et al., 1975).

To solve the hydraulic design problem, both the Manning and Darcy-Weisbach equations can be used. The earlier presented forms of these equations, Equations 6.4 and 6.6, are cumbersome to use in the actual design work and, consequently, various more convenient forms of these equations and design aids have been developed for practical applications.

The Manning equation forms a basis for numerous design aids. For circular pipes flowing full, it is convenient to express the Manning formula in one of the following forms:

$$Q = \frac{0,31}{n} D^{8/3} S_e^{1/2} \qquad (6.17)$$

$$D = (\frac{3,2\ Q\ n}{D_e^{1/2}})^{3/8} \qquad (6.18)$$

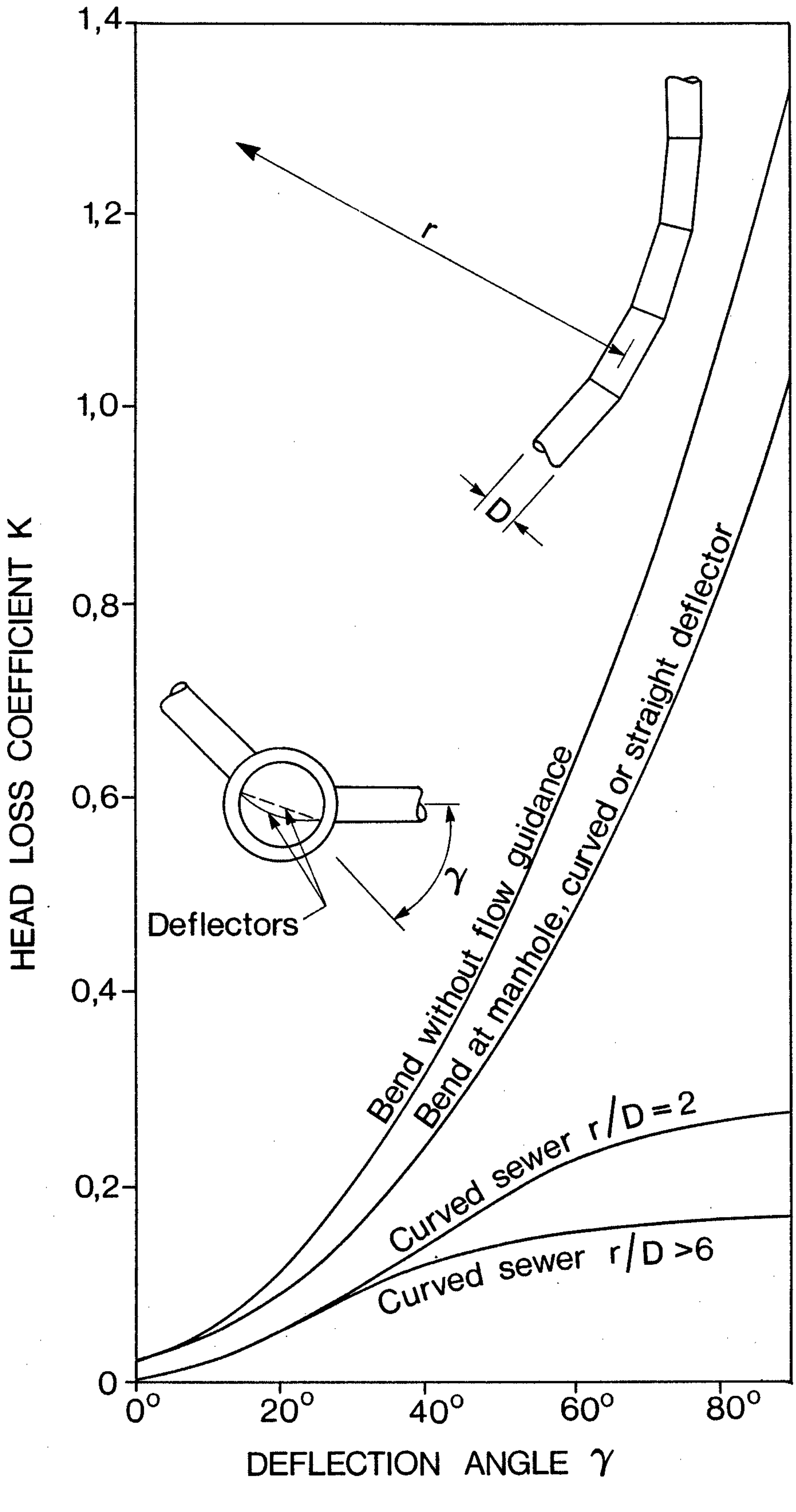


Fig. 6.4 Sewer bend head loss coefficient

$$S_e = \left(\frac{3,2\ Q\ n}{D^{8/3}}\right)^2 \tag{6.19}$$

where Q is the pipe discharge, n is the Manning roughness coefficient, D is the pipe diameter, and S_e is the energy grade line slope.

Basic design problems can be readily solved using Equations 6.17 to 6.19. Furthermore, various nomographs have been prepared for applications of the Manning formula in sewer design. Two such nomographs have been included in Chapter 7. The first one provides a graphical solution for Equations 6.17 to 6.19. Thus, if any three of the four variables (Q, n, D, and S_e) are given, the fourth variable can be determined from the nomograph.

Where a large number of repetitious calculations is to be made, it is advantageous to prepare a graph of pipe discharges for various slopes, pipe diameters, and a chosen value of pipe roughness. An example of such a graph is given in Chapter 7.

The Darcy-Weisbach equation, with the Colebrook-White function for the friction factor, is rather cumbersome to use in actual design. Consequently, various design aids and procedures have been developed to simplify the solution of design problems, using both numerical and graphical solutions.

Numerical solutions of the Colebrook-White function (Equation 6.10) are based on approximate explicit expressions for the friction factor f. Although such approximations slightly deviate from the exact iterative solution of Equation 6.10, the resulting errors are small, less than several percent. In this report, explicit expressions proposed by Barr (1975) have been adopted. Further information on other approaches can be found elsewhere (Li, 1974; Swamee and Jain, 1976).

Considering the pipe diameter D, the discharge Q, and the energy grade line slope S_e as the main design variables, Barr (1975) derived the following explicit expressions for these variables:

$$\frac{0,9\ Q}{(S_e\ g)^{0,5}\ D^{2,5}} = -\ 2\ \log \left\{\frac{1}{2,415\ [Q^{0,4}/k\ (S_e\ g)^{0,2}]^{0,95}} + \frac{3,046}{[Q^{0,6}\ (S_e\ g)^{0,2}/\nu]^{0,931}}\right\} \tag{6.20}$$

$$\frac{0,9\ Q}{(S_e\ g)^{0,5}\ D^{2,5}} = -\ 2\ \log \left\{\frac{1}{3,7\ D/k} + \frac{1,775}{(S_e\ g)^{0,5}\ D^{1,5}/\nu}\right\} \tag{6.21}$$

$$\frac{0,9\ Q}{(S_e\ g)^{0,5}\ D^{2,5}} = -\ 2\ \log \left\{\frac{1}{3,7\ D/k} + \frac{4,1365}{(Q/\nu D)^{0,89}}\right\} \tag{6.22}$$

where k is the effective absolute roughness.

As an alternative to the numerical solutions, various graphical solutions of the pipe design problem, based on the Colebrook-White function, have been developed (Barr, 1978; Li, 1974). Such solutions are not very convenient to use and it would appear that, considering the present availability of electronic calculators, the graphical solutions may have outlived their usefulness.

Where a large number of pipe sizing calculations is to be made, design charts, based on the Colebrook-White function, may expedite such calculations. Towards this end, extensive design charts have been prepared by Ackers (1969) and Kirschmer (1966).

After both the friction and form losses have been evaluated, the designer plots the energy and hydraulic grade lines for the proposed system and checks that the hydraulic grade line is not above any service connections, in which surcharge conditions would create unacceptable flooding or structural damages.

6.4 OPEN-CHANNEL FLOW DESIGN

Many drainage elements, including storm sewers, are often designed for open-channel flow conditions. In the first phase of design, the dimensions of drainage elements need to be determined and this is usually done by assuming steady uniform flow in the drainage network. Under this assumption, the slope of the energy and hydraulic grade lines is the same as the channel bottom slope and the depth of flow is the so-called normal depth. For these conditions, the earlier discussed flow friction formulae may be applied to design problems.

It is unlikely that in any drainage system the uniform flow could prevail throughout the system. Rather, there will be interconnected reaches of uniform and nonuniform flows. Consequently, once the elements of the drainage network have been tentatively sized for uniform flow, it is necessary to analyze the actual flow conditions in the system. Towards this end, water surface profiles are calculated in important elements of the drainage system and, if required, adjustments in the element dimensions are made.

The discussion in this section is limited to uniform flow. Selected aspects of nonuniform flow will be discussed later in Section 6.6.

The Manning equation may be used for hydraulic design of open channels in several ways. For channels of an arbitrary cross-section, the discharge may be expressed from the Manning equation as

$$Q = \frac{1}{n} A R^{2/3} S_e^{1/2} \tag{6.23}$$

where A is the flow area and other quantities are as defined previously. Equation 6.23 is not in a form convenient for solution of design problems, because it must be solved by successive approximations. To remedy this, Chow (1959) introduced a term $A R^{2/3}$, which depends only on the depth of flow and channel geometry, as the section factor for uniform flow computations and rearranged Equation 6.23 as follows

$$A R^{2/3} = \frac{n Q}{S_e^{1/2}} \tag{6.24}$$

Equation 6.24 is very useful for computations of uniform flow. If the discharge, slope, and roughness are known, this equation gives the section factor $AR^{2/3}$ and hence the normal depth for a particular cross-section. For a known slope, roughness, depth, and hence the section factor, the normal discharge can be calculated. In order to simplify calculations, dimensionless graphs showing the relationship between the depth and the section factor $AR^{2/3}$ have been produced by Chow (1959) for rectangular, trapezoidal, triangular, and circular channels. Such graphs are given in Chapter 7.

The earlier discussed nomographs for the pressure flow in circular pipes have also some application to flow problems in open channels. In particular, the nomograph for the solution of the Manning equation may be used to determine the flow velocity in an open channel by substituting 4R (i.e., four times the hydraulic radius) for the pipe diameter D. The calculation is then completed by multiplying the velocity by the flow area.

Flows in circular pipes flowing partly full can be determined from the full-pipe flow, which can be obtained from the nomograph in Chapter 7, and the graph of hydraulic elements for circular sewers (Figure 6.2). The latter graph relates the discharge of a partly-filled pipe to the full-pipe capacity.

The Colebrook-White equation may be also used in open-channel design. Again, the pipe diameter is replaced by 4R and the friction factor f can be determined from Figure 6.3 or Equation 6.10. The flow velocity is then calculated from Equation 6.5 and used further to calculate the discharge.

It should be stressed again that the uniform flow design has to be further verified for the actual flow conditions. The occurrence of non-uniform flow in some reaches of the drainage system is likely and may result in modifications of the initial design. Computations of nonuniform flow problems are discussed in Section 6.6.

6.5 VELOCITY CONSIDERATIONS

Various computations discussed so far dealt only with hydraulic capacities of drainage elements. In practical sewer design, such computations are subject to two constraints - suspended solids should be transported under most flow conditions and velocities should not exceed certain limits to avoid structural damage.

The problem of self-cleansing velocities in sewers has been studied by many researchers. Using Shields' results, Camp (1946) developed the following equation for the velocity required to transport sediment in conduits flowing full:

$$V = \sqrt{\frac{8\,B}{f}\, g(s-1)\, D_g} = \frac{R^{1/6}}{n}\sqrt{B(s-1)\, D_g} \tag{6.25}$$

where s is the specific gravity of the particle, D_g is the particle diameter, B is a dimensionless constant (B = 0,04 for inception of the particle motion, and B = 0,8 for adequate self-cleansing), f is the friction factor, n is the Manning roughness coefficient, and g is the gravity acceleration.

Equation 6.25 indicates that the velocity required to transport materials in sewers is primarily dependent on the particle size and specific weight, and only slightly dependent on the conduit shape and the depth of flow reflected by the hydraulic radius to the one-sixth power.

Difficulties in achieving self-cleansing velocities in sewers at all times arise from the fact that flows in sewers vary and, consequently, the depth of flow and velocity vary as well. Even though a sewer may be designed to be self-cleansing at a particular discharge, it is not necessarily self-cleansing at lower discharges. The range of flow variations depends on the type of sewer. The least flow variations are encountered in sanitary sewers, in which the self-cleansing velocities (about 0,5 to 1,0 m/s) should be achieved for the mean flow (WPCF, 1970a). In storm sewers, the minimum flow approaches zero and the self-cleansing velocities can not be achieved under such conditions. It is suggested that storm sewers should achieve self-cleansing velocities during moderate storms with flows considerably less than the design flow (WPCF, 1970a). Such moderate flows could be taken as flows corresponding to a storm with the monthly occurrence (i.e., the storm, which occurs on the average twelve times a year). The most difficult is self-cleansing of combined sewers. It is rarely possible to design combined sewers with adequate self-cleansing velocities at the minimum dry-weather flow, if the sewer capacity must be adequate for the stormwater runoff. Consequently, combined sewers are often subjected to the sediment deposition during dry weather periods and these deposits are then flushed during wet weather periods. Such sewer flushing contributes to large pollution shock-loading of the receiving water body.

Camp's work (Equation 6.25) was further extended by Fair et al., (1966) to flows at less than the full depth. The resulting relationships for various hydraulic elements are as follows:

$$\frac{S}{S_f} = \frac{R_f}{R} \qquad \frac{V_s}{V_f} = \frac{n_f}{n}\left(\frac{R}{R_f}\right)^{1/6} \qquad \frac{Q_s}{Q_f} = \frac{n_f}{n}\frac{A}{A_f}\left(\frac{R}{R_f}\right)^{1/6} \tag{6.26}$$

where V_s is the self-cleansing velocity, Q_s is the discharge at the self-cleansing velocity, and the subscript f refers to the full-pipe flow. The relationships given by Equations 6.26 are presented graphically in Chapter 7. It is of interest to note that the ratio S/S_f remains practically constant for sewer flows between the half and full depth. For flows smaller than half-full, the sewer slope must be increased to maintain self-cleansing velocities.

For simplicity, it is sometimes specified to keep the minimum velocities in sanitary sewers in excess of 0,5 - 1,0 m/s. The corresponding minimum slopes vary from 0,0022 (for D = 0,305 m) to 0,0008 (for D > 0,68 m). Similar values could be also used for other types of sewers.

Although it is desirable to transport flows in sewer systems rather swiftly, harmful high velocities leading to erosion of sewers must be avoided. For clear water in hard-surfaced conduits, the limiting velocity is fairly high. Velocities between 3,6 to 6,0 m/s have been found harmless to concrete channels (WPCF, 1970a). Erosion of pipes may result from much lower velocities if the water carries sand or other gritty material. For continuous flow in sanitary sewers, the limiting velocity of 3 m/s is recommended. In storm sewers, flows are intermittent and the maximum design velocities could be allowed to exceed the limit of 3 m/s. However, velocities in excess of 6 m/s should be avoided even for the design flow (WPCF, 1970a).

6.6 NONUNIFORM FLOW PROBLEMS

Conditions of steady nonuniform flow exist when a constant discharge is conveyed by transport elements with varying cross-sections and slopes. Such conditions are frequently encountered in drainage design and need to be properly analyzed. In the analysis of nonuniform flow, it is essential to establish first the normal and critical depths in various reaches of the drainage system. In the next phase, the water surface profile is calculated starting from the control sections with the known depth of flow and proceeding in the upstream or downstream direction according to the flow regime.

The selected nonuniform flow problems discussed in this section include the critical flow, drawdown and backwater curves, and the hydraulic jump.

6.6.1 Critical flow

The critical flow is defined as the state of flow at which the specific energy is at its minimum for a given discharge. The critical flow is characterized by a condition that its Froude number is equal to unity and this condition may be written as

$$Fr = \frac{V}{\sqrt{g\ y_m}} = 1 \tag{6.27}$$

where y_m is the hydraulic mean depth equal to A/T, A is the flow area, and T is the width of the water surface.

In drainage design, the flow is generally turbulent and for such a flow, two flow regimes can be distinguished - the flow is said to be subcritical when Fr is less than unity, and the flow is supercritical, when Fr is greater than unity. In subcritical flow, gravity waves can propagate upstream, but in supercritical flow, they propagate only downstream. The propagation of gravity waves then determines the direction in which the water surface profile calculations proceed. The starting point for these calculations is a control section with a known flow depth and, fairly often, the control section is chosen as a section with the critical depth of flow. It is therefore of interest to determine the critical depth in individual reaches of the drainage system using the following expression:

$$Fr = \frac{Q^2\ T}{g\ A^3} = 1 \tag{6.28}$$

Equation 6.28 is solved by trial and error by evaluating the left-hand side for various depths of flow. When this term equals unity, the critical depth is obtained.

To expedite calculations of critical flow, Chow (1959) introduced the section factor for critical flow computations defined as

$$Z = \frac{Q}{\sqrt{g}} \tag{6.29}$$

where Z is a function of the depth and depends on the channel cross-section. Chow determined values of Z for various cross-sections and these are shown in Chapter 7 in the form:

Trapezoidal sections $\qquad y_c/b = f(Z/b^{2,5}) \qquad (6.30)$

Circular sections $\qquad y_c/D = f(Z/D^{2,5}) \qquad (6.31)$

where y_c is the critical depth, and b is the width of the channel bottom. Thus for a given discharge, the section factor is computed from Equations 6.29 to 6.31, the value of $Z/b^{2,5}$ (or $Z/D^{2,5}$) is entered in Figure 7.5 and the value y_c/b (or y_c/D) is read from the graph. A similar procedure is followed when the critical depth is given and the discharge is to be determined. For rectangular channels, the critical depth can be expressed directly from Equation 6.28 as

$$y_c = g^{1/3} q^{2/3} \tag{6.32}$$

where q is the unit discharge ($m^3/s/m$). For circular channels, the critical depth can also be determined from the nomograph given in Chapter 7.

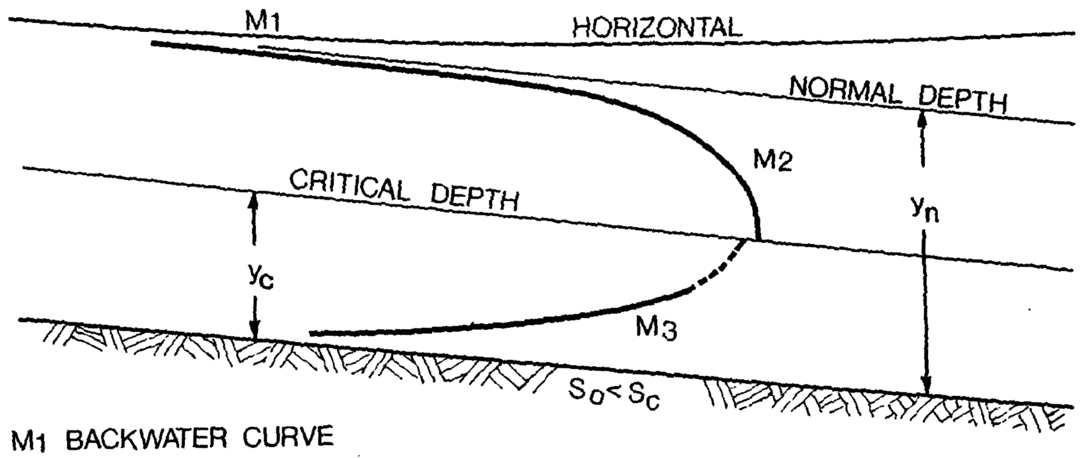
6.6.2 Drawdown and backwater curves

The design of some drainage channels and conduits may be affected by the shape of drawdown and backwater curves and, consequently, it is essential to determine the shape of these curves. In the case of drawdown curves, it may be possible to reduce costs by reducing the size of the conduit as the depth gradually decreases in the direction of flow. For backwater curves, it is of interest to find the increases in the depth and the distance upstream to which the backwater curve extends.

A general analysis of flow profiles for the gradually varied flow, which is characteristic for drawdown and backwater curves, is beyond the scope of this report and can be found elsewhere (Chow, 1959; Henderson, 1966). For drainage design, the discussion may be limited to the six most common flow profiles shown in Figure 6.5. Engineering calculations of these profiles are usually based on the following equation (Chow, 1959):

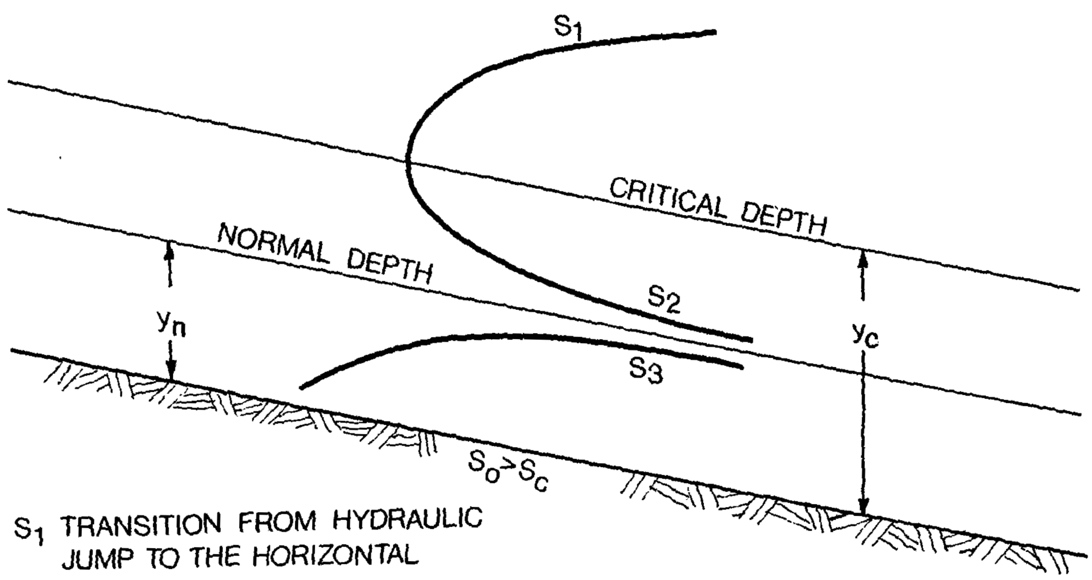
$$x = \frac{\left(y_1 + \frac{V_1^2}{2g}\right) - \left(y_2 + \frac{V_2^2}{2g}\right)}{S_e - S_o} \tag{6.33}$$

(a) Mild slope ($y_n > y_c$)



M_1 BACKWATER CURVE
M_2 DRAWDOWN CURVE
M_3 STREAM BELOW A SLUICE

(b) Steep slope ($y_n < y_c$)



S_1 TRANSITION FROM HYDRAULIC JUMP TO THE HORIZONTAL

S_2 DRAWDOWN CURVE WHERE STEEP SLOPE CHANGES TO STEEPER SLOPE

S_3 TRANSITION FROM STEEP SLOPE TO MILDER STEEP

Fig. 6.5 Classification of flow profiles where x is the distance between the two sections of the flow, y is the flow depth, S_o is the bottom slope, S_e is the average slope of the energy grade line between the two sections, and subscripts 1 and 2 refer to the two sections between which the calculation is carried out.

There is a large number of methods for the computation of the gradually varied flow and a detailed analysis of many of these methods can be found elsewhere (Chow, 1959). In engineering applications, two methods are particularly popular - the direct step method and the standard step method. A brief outline of both methods follows.

The direct step method is one of the simplest step methods and is applicable to prismatic channels. In principle, it is based on the following equation:

$$\Delta L = \frac{\Delta(y + h_v)}{\bar{S}_e - S_o} \tag{6.34}$$

where ΔL is the length step (measured in the direction in which the calculations are proceeding), y is the depth of flow, h_v is the velocity head, $\bar{S}_e$ is the average slope of the energy grade line (over the length step), S_o is the invert slope, and $\Delta(y+h_v)$ is the change in specific energy along the length step. For calculations of the slope S_e, the earlier discussed Manning equation may be used, or any other uniform flow formula. Note that, in the direct step method as well as in the standard step method discussed later, the step calculations should be carried upstream if the flow is subcritical and downstream if the flow is supercritical.

Another stepwise method for calculations of the gradually varied flow is the standard step method, which is based on the following equation:

$$h_f = 0{,}5\ (S_{e1} + S_{e2})\ \Delta L \tag{6.35}$$

where h_f is the friction loss between the two end sections, S_e is the energy grade line slope, ΔL is the length between the two end sections, and subscripts 1 and 2 refer to the respective end sections.

The standard step method has more flexibility and may be used for both uniform and nonuniform channels. On the other hand, it requires a trial and error solution. Further details of this method can be found elsewhere (Chow, 1959).

6.6.3 Hydraulic jump

Besides the gradually varied flow discussed in the preceding section, the rapidly varied flow may be also encountered in the drainage design. The most common case, the hydraulic jump, is discussed below.

The hydraulic jump is a phenomenon through which a flow abruptly changes from supercritical flow at a relatively low depth to subcritical flow at a greater depth. The hydraulic jump is sometimes used to dissipate excess energy of flow entering the drainage system, or to avoid scour of earthen channels. The problems connected with incorporating the hydraulic jump in the design include the determination of the location of the jump, flow depths upstream and downstream of the jump, and the head loss due to the jump.

The simplest case which is discussed here represents a hydraulic jump in a slightly sloping rectangular channel. In this case, the jump characteristics may be described by the following equation (Chow, 1959):

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8Fr_1^2} - 1\right) \tag{6.36}$$

where y_1 and y_2 are the depths before and after the jump, respectively, and Fr_1 is the Froude number of the upstream flow ($Fr_1 = V_1/\sqrt{g y_1}$).

The head loss, ΔH, due to the jump, can be expressed as

$$\Delta H = \frac{(y_2 - y_1)^3}{4\ y_1\ y_2} \tag{6.37}$$

For nonrectangular channels, the calculations become more tedious and the reader is referred to standard texts on open-channel flow (Chow, 1959: Henderson, 1966).

7 Design of components of drainage systems

7.1 INTERFACE BETWEEN HYDROLOGIC AND HYDRAULIC DESIGNS

Various components of drainage systems, such as conduits, catch basins, drop structures, inlets, overflow structures, culverts, siphons, and storage facilities, were described in Chapter 3. Basic hydraulic principles used for design of conveyance elements were presented in Chapter 6 and this chapter reviews some special hydraulic aspects to be considered in detailed design of drainage system components. It assumes that design flows are known and were determined by the methods discussed in Chapter 5. Structural and geotechnical designs are not discussed here, although such aspects are sometimes very important. The interface between hydrologic modelling and hydraulic design is often iterative because the flow magnitude is influenced by hydraulic characteristics of the outlet, storage outflow relations, velocities in the conveyance system, etc. As an example, in underground sewer design with the rational method, flow computations and pipe selection should be interfaced with constraints for cover, slopes and velocities as prescribed by municipal criteria. Minimum velocities are required for self-cleansing and maximum permissible velocities are determined by the materials. The use of corrugated steel pipes instead of concrete conduits changes the time of concentration and modifies the flow.

Another example of interface between hydraulic design and hydrologic analysis is the determination of storage requirements. The minimum storage is given by an optimum control operation with a gate which releases all the inflow when it is smaller than the maximum permissible release rate and maintains a constant outflow when the inflow reaches this specified maximum discharge. Operated gates, however, are rarely used in drainage systems and the discharge of most outlets varies with head according to relations which are quite different from one technical alternative to another. Note that for orifices and conduits, the discharge varies with the square root of the head while, on the other hand, for weirs and culverts with inlet control, the discharge is a function of head to the power of 1,5. Selection of the outlet type has to consider local configuration, type of embankment, available head, probability of clogging, etc. These non-hydraulic factors, therefore, may change the volume of required storage.

Hydraulic design of many drainage structures, such as overflow chambers, may become quite complex. In this case, municipal engineers may require advice or assistance from hydraulic specialists. It should be also mentioned that some network design methods discussed in Chapter 8 integrate the computational aspects of hydrologic and hydraulic design in a single design tool, a comprehensive computer method.

7.2 PRACTICAL DESIGN OF CONVEYANCE ELEMENTS

7.2.1 Sizing of conveyance elements

Basic hydraulics principles used in the sizing of conduits and channels were discussed in Chapter 6. Practical aspects are described in this section. Frequent designs of conveyance elements are usually done with simplified methods or nomograms such as those based on the Manning formula and shown in Figures 7.1 and 7.2. Practical computations usually assume that the roughness coefficient, n, does not vary with the depth of flow (see Table 6.1). The error introduced by this assumption is small. For a variable n, velocities do not reach a maximum for the relative depth of 0,8. Although the formula of Colebrook-White is more complex than that of Manning, it can be easily used with the nomogram in Figure 7.3 (see also Chapter 6).

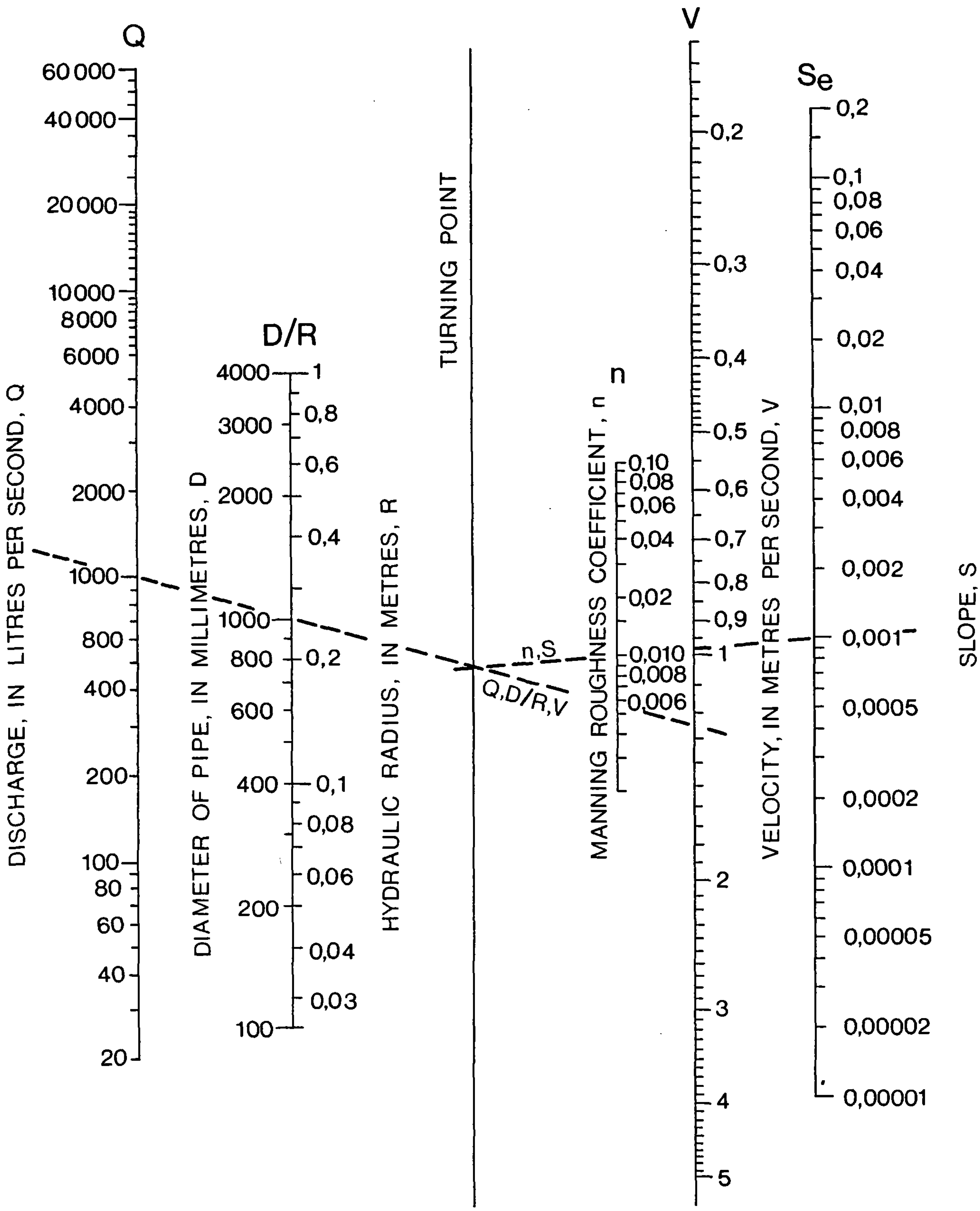


Fig. 7.1 Nomogram for application of Manning formula

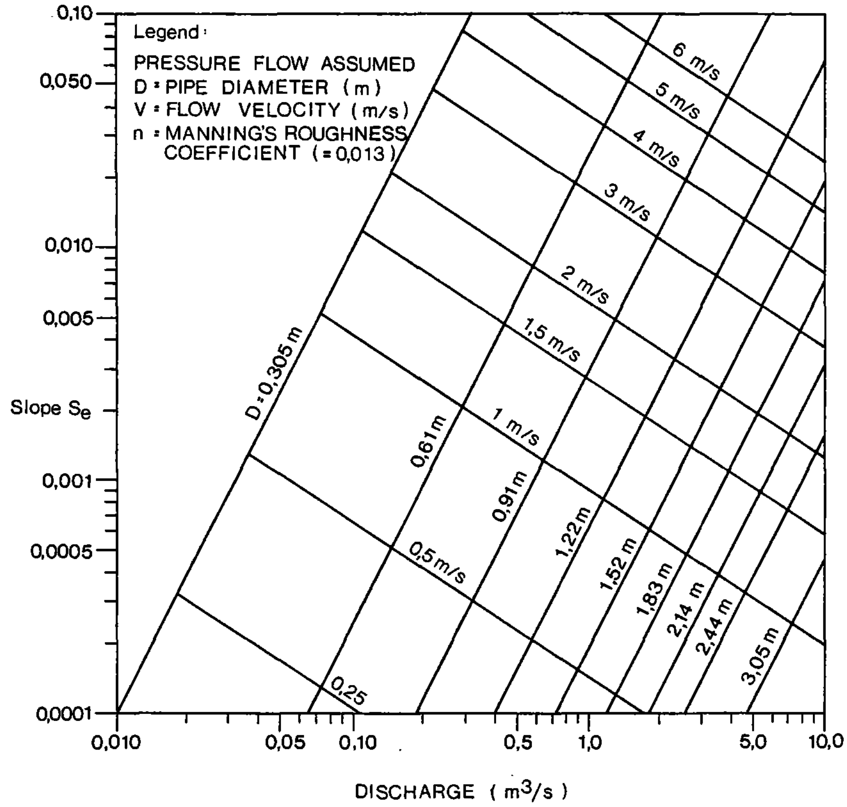


Fig. 7.2 Pipe flows calculated by Manning formula

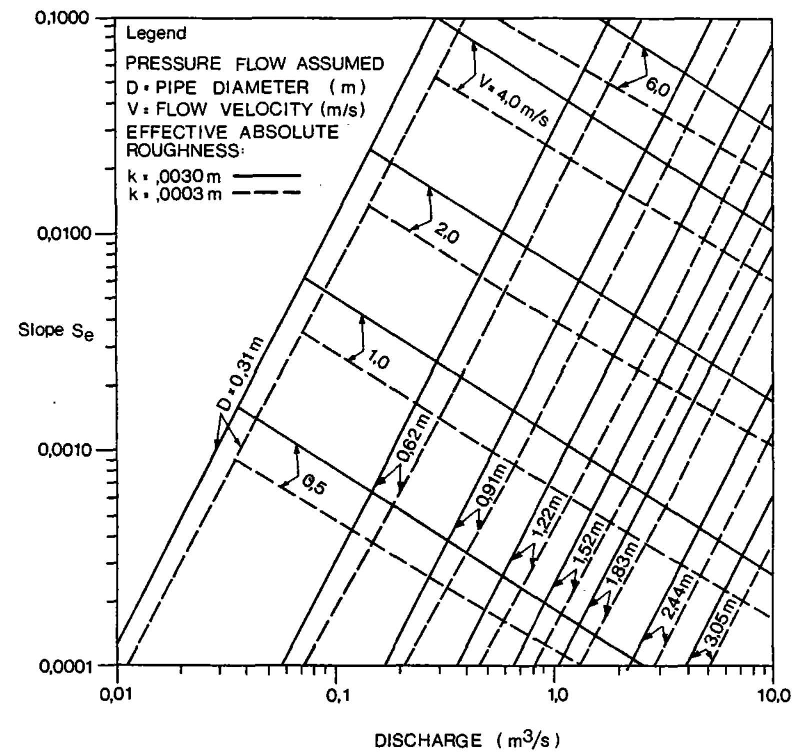


Fig. 7.3 Pipe flows calculated by Colebrook-White equation

Finally, nomograms for normal and critical depths in open channels with trapezoidal and circular cross-sections are shown in Figures 7.4 and 7.5.

Underground storm sewers for minor systems have been designed in more recent projects with circular cross-sections. If a large flow, e.g., in a large watershed or major system, is collected in an underground storm sewer, the height of the sewer can be reduced by using rectangular or composite cross-sections. These cross-sections have very low flow velocities during frequent storms. In order to provide self-cleansing at such low flows, it is possible to use a small narrow collector cunette, in the bottom. In older installations, egg-shaped cross-sections have been used for the same reason. Nomograms for these cross-sections are available in various handbooks of hydraulic design.

7.2.2 Self-cleansing velocities

Although comprehensive information about the concentration and grading of sediments in sewers and storm drains is lacking, it is well known that silt, sand and coarser solids enter storm drainage systems in appreciable quantities. If they are not transported through the system by the flow, a maintenance operation is required to remove them.

Various criteria have been proposed for self-cleansing conditions in conduits and they are frequently expressed as a limiting velocity (see Chapter 6). An example of a graph of self-cleansing velocities in sewers flowing partly full is given in Figure 7.6. Recent theoretical works and experiments suggest, however, that a constant value of the limiting velocity does not adequately describe the flow capacity to transport sediment in pipes flowing partly full. Ackers (1978) showed that for a certain diameter, the velocity required to avoid accumulation of sediments varies significantly with sediment concentration. He derived the relationships expressing the diameter and the velocity as a function of discharge and sediment concentration in the following form:

$$D = C_1 Q^{0,43} \tag{7.1}$$

$$V = C_2 D^{0,325} \tag{7.2}$$

where D is the required diameter (m),
V is the required velocity (m/s),
Q is the rate of discharge (m^3/s), and
C_1 and C_2 are coefficients, shown in Table 7.1, which depend on the sediment concentration and the degree of pipe cleanliness.

Table 7.1 Recommended coefficients for required diameter and velocity for self-cleaning action (after Ackers, 1978).

Sediment Concentration (mg/l) x 10^6	Clean Pipe		Sediment Bed Thickness = 0,1 Pipe Diameter	
	C_1	C_2	C_1	C_2
100	1,36	0,62	1,46	0,56
200	1,26	0,74	1,36	0,65
500	1,16	0,90	1,23	0,83

As discussed earlier in Chapters 2 and 3, many drainage systems in developing countries are built as open-channel networks. Design criteria used for the design of open channels in a drainage project in Africa are summarized below:

i) Whenever possible, it is preferred to avoid supercritical flow by reducing the slope or changing the cross-section. If supercritical flow cannot be avoided, the channel depth should account for a possible hydraulic jump and could be set equal to the specific energy head. The channel layout should avoid bends in reaches with supercritical flow. In such reaches, a rectangular cross-section is preferred, at least for frequent storms. If the rectangular channel of width, B_o, is combined with a trapezoidal channel, the base width of the trapezoid should be greater than B_o.

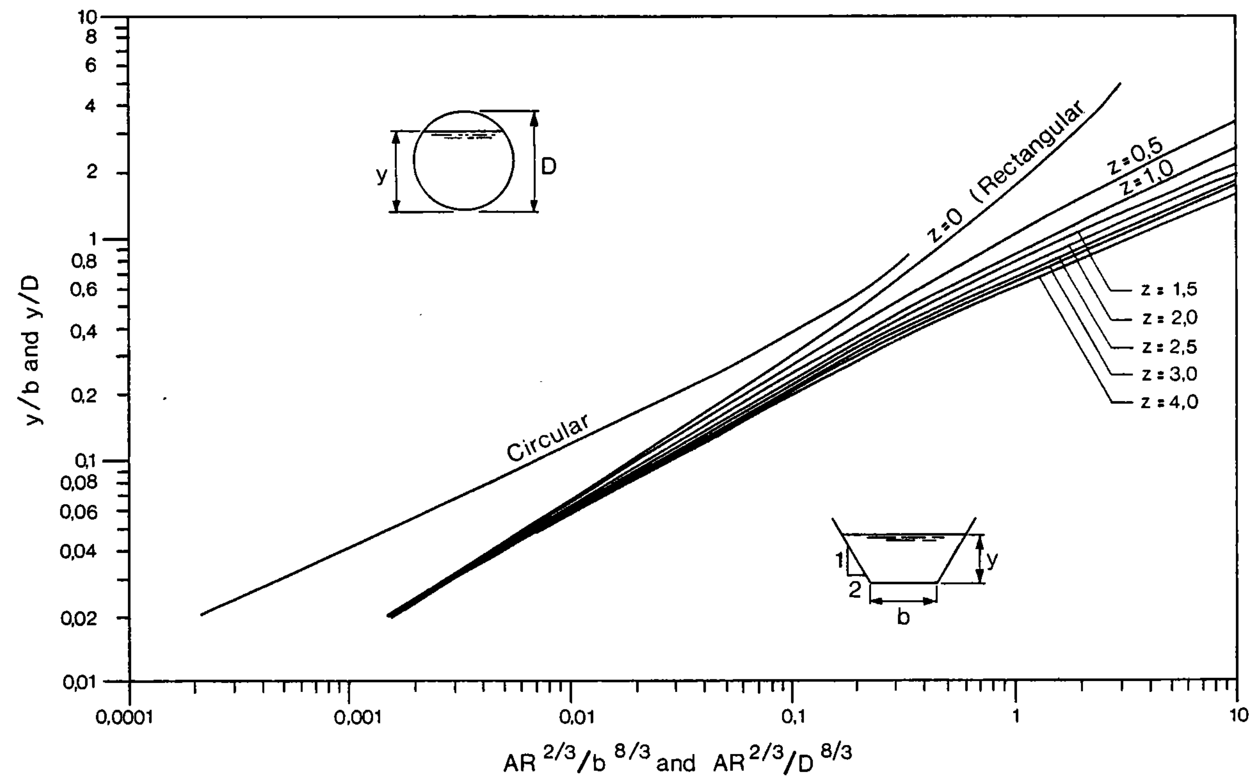


Fig. 7.4 Nomogram for determining the normal depth in various open channels (after Chow, 1959)

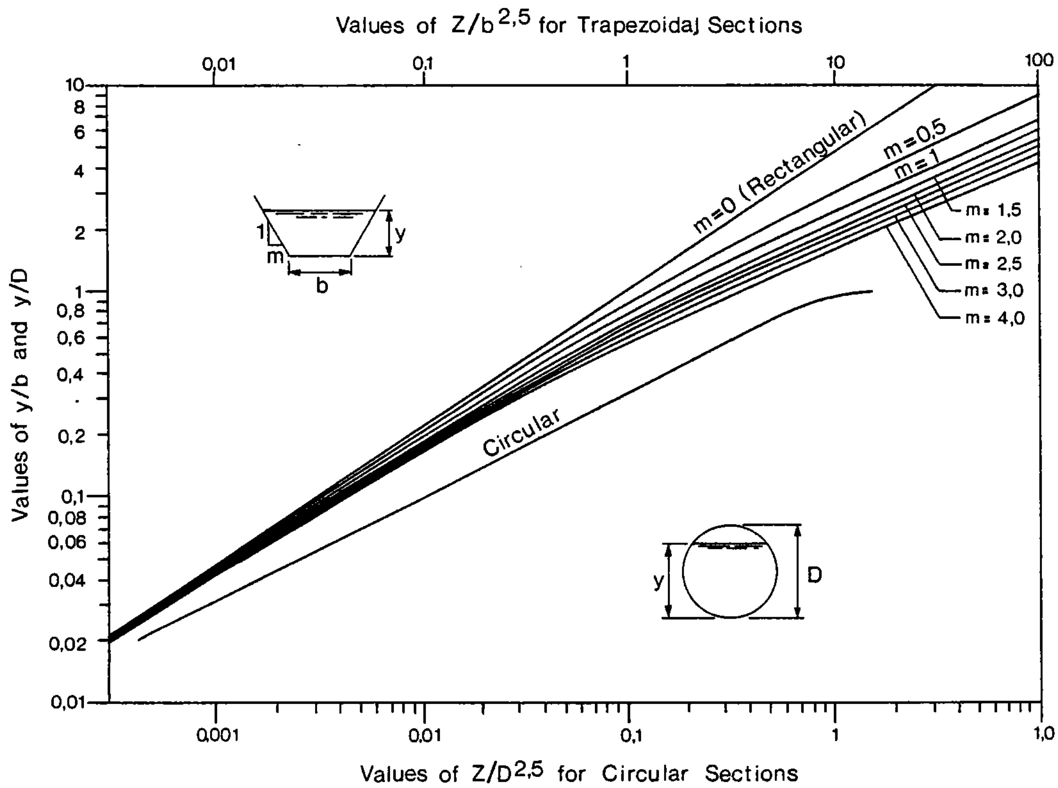


Fig. 7.5 Nomogram for determining the critical depth in various open channels (after Chow, 1959)

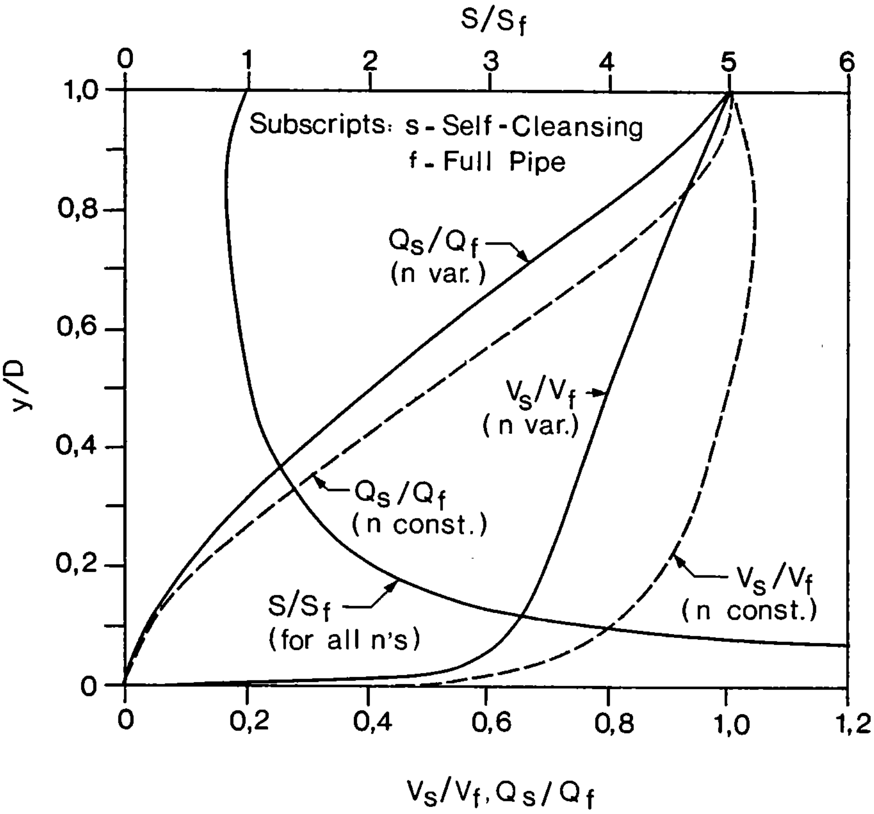


Fig. 7.6 Hydraulic elements of self-cleansing sewers (after WPCF, 1970a)

ii) Critical flow should be avoided, because it is unstable. It is preferable to maintain subcritical flow in channels characterized by Froude numbers smaller than or equal to 0,8, where the Froude number is defined by Equation 6.27.

iii) Maximum velocities in earth channels should be controlled to avoid erosion. For guidance, the following maximum permissible velocities are suggested:
Sandy soil V_{max} = 0,75 m/s
Loess V_{max} = 1 m/s
Sandy clay compacted V_{max} = 1,5 m/s

iv) To ensure sediment transport and self-cleansing of channels, the following minimum velocities are suggested:
Lined channels V_{min} = 1 m/s
Earth channels V_{min} = 0,65 m/s

v) For small open channels, the optimum cross-section for maximum conveyance can be selected as a half square or a half equilateral hexagon.

vi) Where sand traps are provided, flow velocities should be smaller than 0,3 m/s (for sand particle diameter greater than 0,2 mm).

vii) The minimum freeboard in channels meeting the condition (ii) above and for supercritical flow should be 0,25 m and 30% of the depth, respectively. Underground sewers should be designed for free flow and a maximum flow depth of 0,7 of the cross-section height.

viii) Slabs covering open channels should be checked for uplift caused by a small surcharge.

7.3 MANHOLES AND CATCH BASINS

The hydraulic design of sewer networks is based on equations of mass continuity and energy conservation. The latter equation requires consideration of two types of head losses - skin friction losses in sewer pipes and form losses at various appurtenances and special structures, such as manholes. While skin friction losses are caused primarily by viscous and turbulent shears along the conduit boundary, form losses may be caused by shear as well as pressure differentials caused by flow separation, changes in flow alignment, and drag on flow obstructions. In some cases, form losses at junctions may be fairly large, in comparison to friction losses, and junctions then act as bottlenecks which seriously limit the capacity of the sewer system. Under such circumstances, the sewer system becomes surcharged and this condition may lead to basement flooding or sewage overflows. It is therefore desirable to include minor losses in the hydraulic design of sewers. This is done sometimes by increasing the roughness coefficient of sewer pipes, or by compensating for manhole and junction head losses by invert drops. Magnitudes of head losses at common junction manholes are given below. For further details, the reader is referred to the literature (Marsalek, 1985).

Minor losses are generally expressed in the following form:

$$\Delta E = K\, V_o^2/2g \tag{7.3}$$

where ΔE is the minor head loss, K is the head loss coefficient and V_o is the mean velocity in the manhole outfall. Coefficient K attains widely ranging values depending on manhole geometry and flow characteristics. Some basic K values are given below.

For straight-flow-through manholes, the head loss coefficient varies from 0,05 to 0,35. The lower value corresponds to open-channel flow through a junction with U-shape benching (i.e., a channel) and the higher one corresponds to pressure flow through a junction without benching.

Larger head losses are observed at junctions with bends. For a 90° bend, the head loss coefficient varies from 0,9 to 1,9, where the lower value corresponds to pressure flow at junctions with a U-shape benching and the higher value corresponds to the case without benching (Marsalek, 1985). For deflection angles smaller than 90°, approximate K-values can be obtained by interpolation of data for 0° and 90°.

Head losses at junctions of a main with a lateral or two opposed laterals depend on both the junction geometry and the flow division between the main and the lateral, or both laterals, respectively. At junction of a main and a lateral, the highest K's were found for designs without benching and almost all of the flow passing through the lateral. K's for the main and lateral were 1,7 and 1,9, respectively. Such values could be reduced by about one-third by installing a curved U-shape benching at the junction. Junctions of two opposed laterals behaved similarly and K-values as high as 1,9 were observed (Marsalek, 1985).

Recently, more attention was also given to the design of catch basins to improve their retention of sediments. An experimental study, conducted by Lager et al. (1976), compared various catch basins in terms of sediment retention and recommended the design shown in Figure 7.7. The dimension from the outlet pipe crown to the street or inlet grade is primarily determined from structural considerations, as it contributes little to the hydraulic performance. The design in Figure 7.7 has proven to be very efficient for coarse material removal but with respect to fine material its performance is less satisfactory.

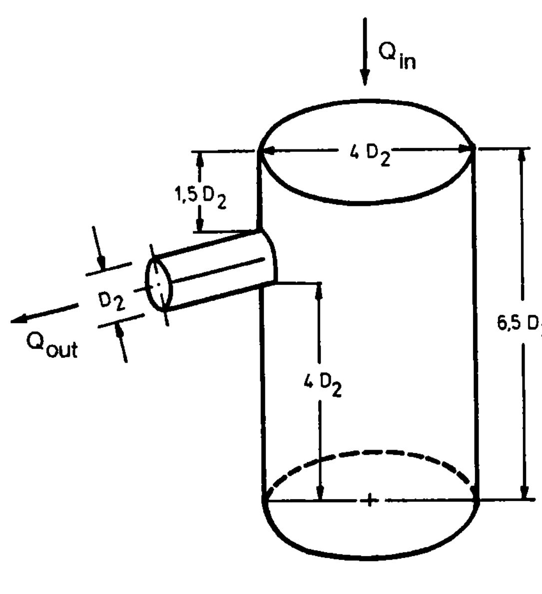


Fig. 7.7 Catch basin design with high retention of sediment

7.4 DROP STRUCTURES

Drop structures are designed to dissipate excessive flow energy and, at the same time, to prevent air entrainment. Air entrainment may cause air pockets, surges and flow instabilities. For high heads, special air separators have been designed. Computations of energy dissipation can be facilitated by nomograms or simplified relations.

As an example for design of the drop structure shown in Figure 7.8, it is possible to use the nomogram shown in the same figure. In a trial and error computation, as a first step, a height of the sill, p, is assumed and the difference in energy head, T_o (metres), is read on the dotted line. With the first value, T_o, and specific flow, q (1/s), the values of the contracted and tailwater depths, h_c and B, respectively, are determined from the nomogram. The height of the sill, p, can then be computed as $p = B - h_k$. A new difference in head, T_o, will result and is entered into the nomogram. The correct sill height is usually obtained in no more than two or three trials. Figure 7.8 also gives a relation for the length of the chamber based on the equation of the jet.

7.5 PRINCIPLES OF INLET DESIGN

An important aspect of proper operation of urban drainage is an adequate interception of runoff by the minor drainage system. At the same time, the capacity of inlets must also be matched with conduit capacity. This requires proper analysis of both the gutter flow and inlet operation. The gutter flow can be evaluated numerically, or as an alternative, it is possible to use the nomogram for triangular gutters given in Figure 7.9.

The maximum permissible flow depths or spreads across the pavement are generally specified, for storms with various frequencies of occurrence, by municipal drainage criteria. The first restriction, the flow depth, avoids flooding by overland runoff and the second one, the spread, maintains a part of the road open to traffic. The flow depth or spread are controlled by the placement and capacity of inlets and are important mainly for highway drainage design. Typical design considerations include the determination of inlet interception flow for a given gutter flow and inlet type and geometry, and the determination of inlet spacing for a given interception capacity. As shown for grate inlets with longitudinal bars in Figure 7.10, carry-over flow may occur in three ways:

i) flow between the curb and the first slot,
ii) flow outside the last slot (towards the pavement crown),
iii) carry-over flow across the grate itself.

The spread of the maximum gutter flow is generally larger than the inlet width. The first of the above flows is relatively small. Theoretical and experimental considerations show that it is uneconomical and/or unfeasible to eliminate entirely the carry-over flow caused by flows next to the curb and outside the last slot.

In general practice, if the carry-over flow from the first inlet does not exceed about 25% of the total gutter flow, the total flow tends to be intercepted by the time the third or fourth inlet is reached (assuming uniform flow, continuous grade and uniformly-spaced inlet series).

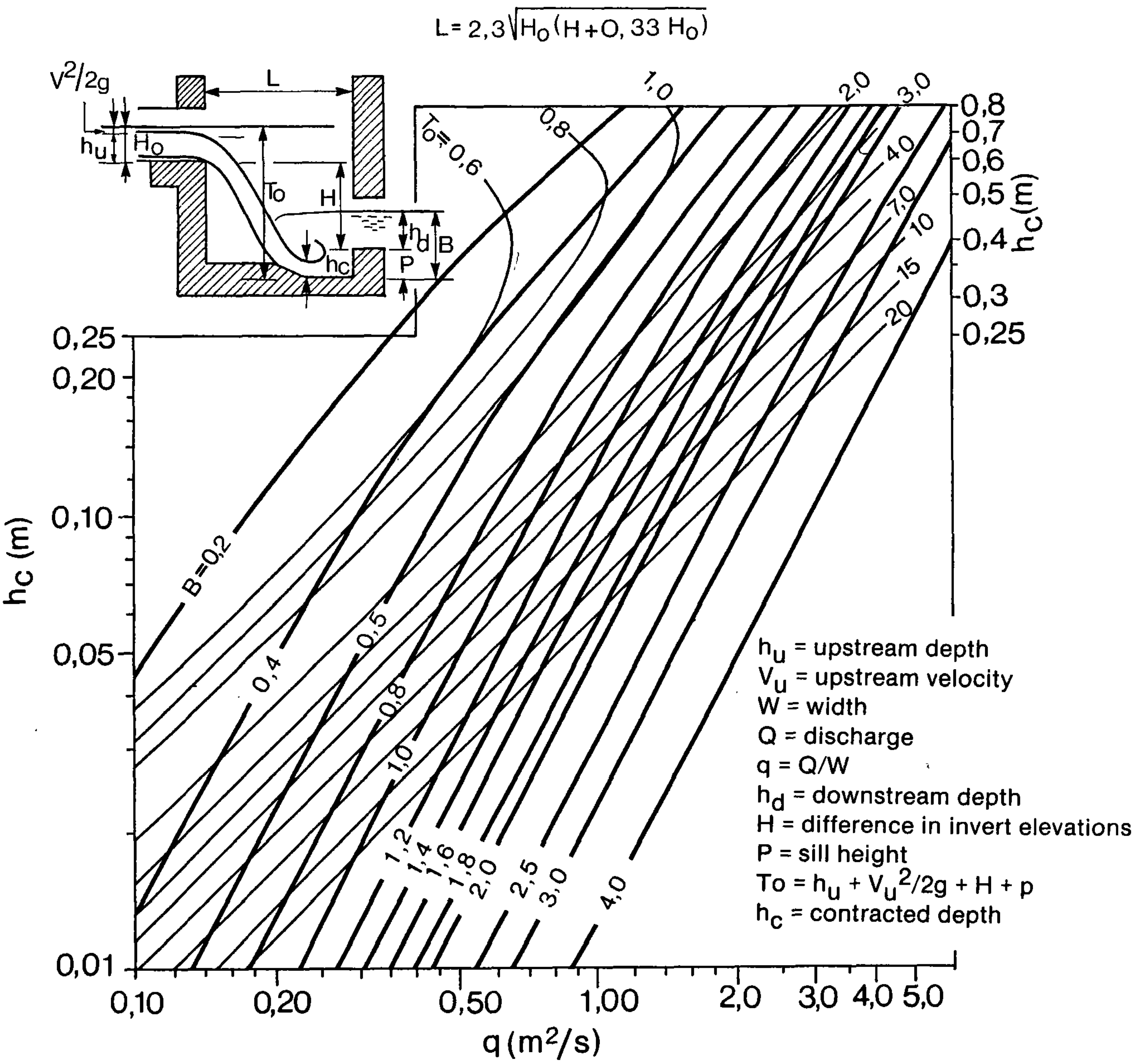
As shown in Figure 7.11, simple experimental diagrams can be developed for direct computation of intercepted flow by a given type of inlet and street cross-section (Townsend et al., 1981). Other numerical methods based on extensive hydraulic laboratory tests were presented by Marsalek (1982).

Storm sewer surcharge can be avoided by restricting the inflow of runoff into the conduit, in terms of maximum free flow capacity. Figure 7.11 shows a diamond-shaped inlet restrictor applied in several Canadian and U.S.A. projects. Inlet restrictor design is usually combined with the analysis of depth of street flows during major storms. Such computations can be done by hand or using a dual drainage computer model.

7.6 OVERFLOW STRUCTURES

The basic hydraulic considerations in design of storm sewage overflow structures in combined sewers are listed below. It should be mentioned that overflow structures are sometimes used even in storm sewers for partial treatment of stormwater discharges.

i) The overflow should come into operation only when inflow exceeds a prescribed discharge to the interceptor and treatment plant (Q_{tp}).
ii) The intercepted flow should not increase significantly as the inflow Q_{in} into the structure rises above Q_{tp}. If $Q_{tp\ max}$ is the maximum flow discharged into the interceptor for the maximum inflow $Q_{in\ max}$, it is possible to define a separation coefficient, α_{sep}, as



EXAMPLE

Given: h_u = 1,2m; V_u = 2 m/s; Q = 4,8 m^3/s; W = 2m; h_d = 1m; H = 2,8m

Try P = 1m

Calculate To = 1,2 + 2^2/19,62 + 2,8 + 1 = 5,2m

Determine from the chart B = 1,9m

Calculate P = B — h_d = 1,9 — 1,0 = 0,9m

Calculate new To = 5,1 — correct value

Determine h_c = 0,27 from the chart

Fig. 7.8 Nomogram for design of energy dissipators at pipe outlets (after Dumitrescu, 1970)

EQUATION

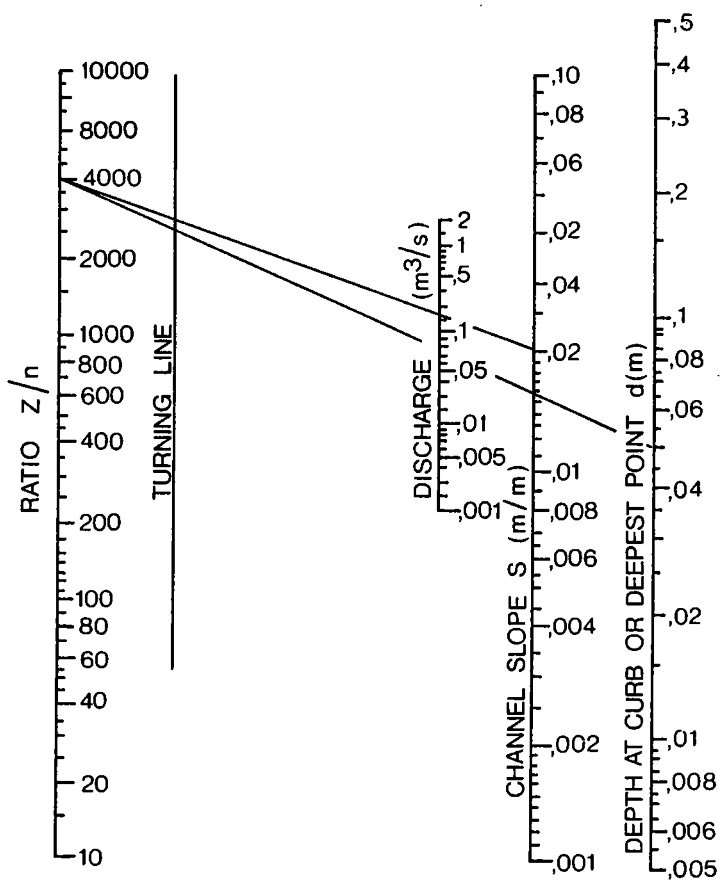
$Q = 0{,}375 \ S^{1/2} d^{8/3}$

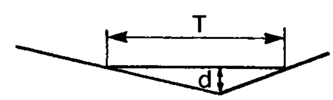
Q = triangular channel discharge
Z = reciprocal of crossfall
n = Manning's roughness coefficient
S = channel slope; d = flow depth

INSTRUCTIONS

1. Connect Z/n ratio with slope S; intersect with turning line, connect with depth d to obtain discharge Q.

2. For shallow V-shaped channel shown below use Nomograph with Z=T/d.





EXAMPLE:

Given : S = 0,02, Z = 50
n = 0,013, Z/n = 3846
d = 0,05

Find : Q = 0,070 m^3/s

Fig. 7.9 Nomogram for flow in triangular channels

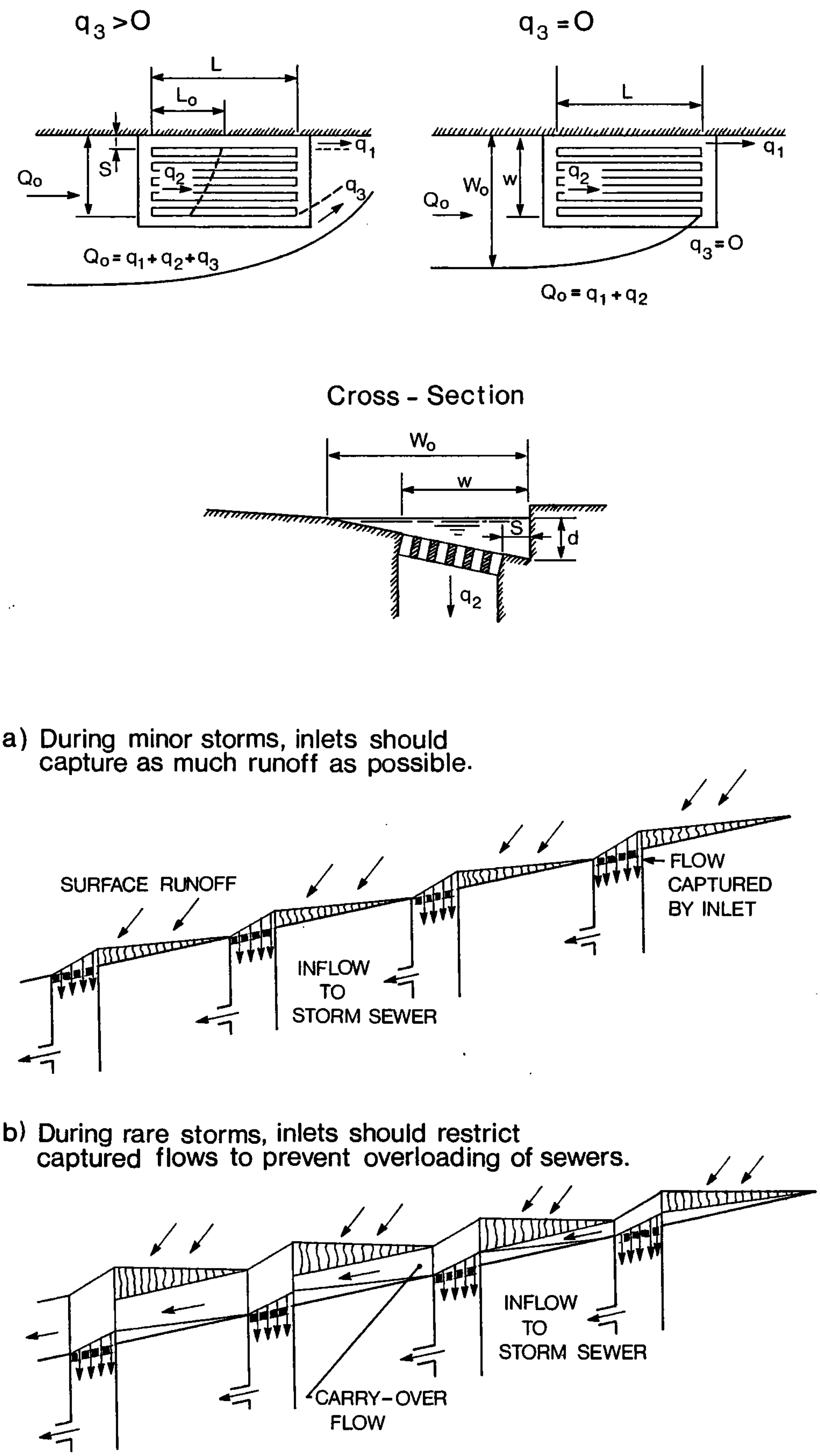


Fig. 7.10 Operation of sewer inlets

$$\alpha_{sep} = \frac{Q_{tp\ max}}{Q_{tp}} \tag{7.4}$$

An ideal case would correspond to $\alpha_{sep} = 1$. This can be achieved only by appropriate operation of a control gate. For other considerations in the design of overflow structures, see also Chapter 3.

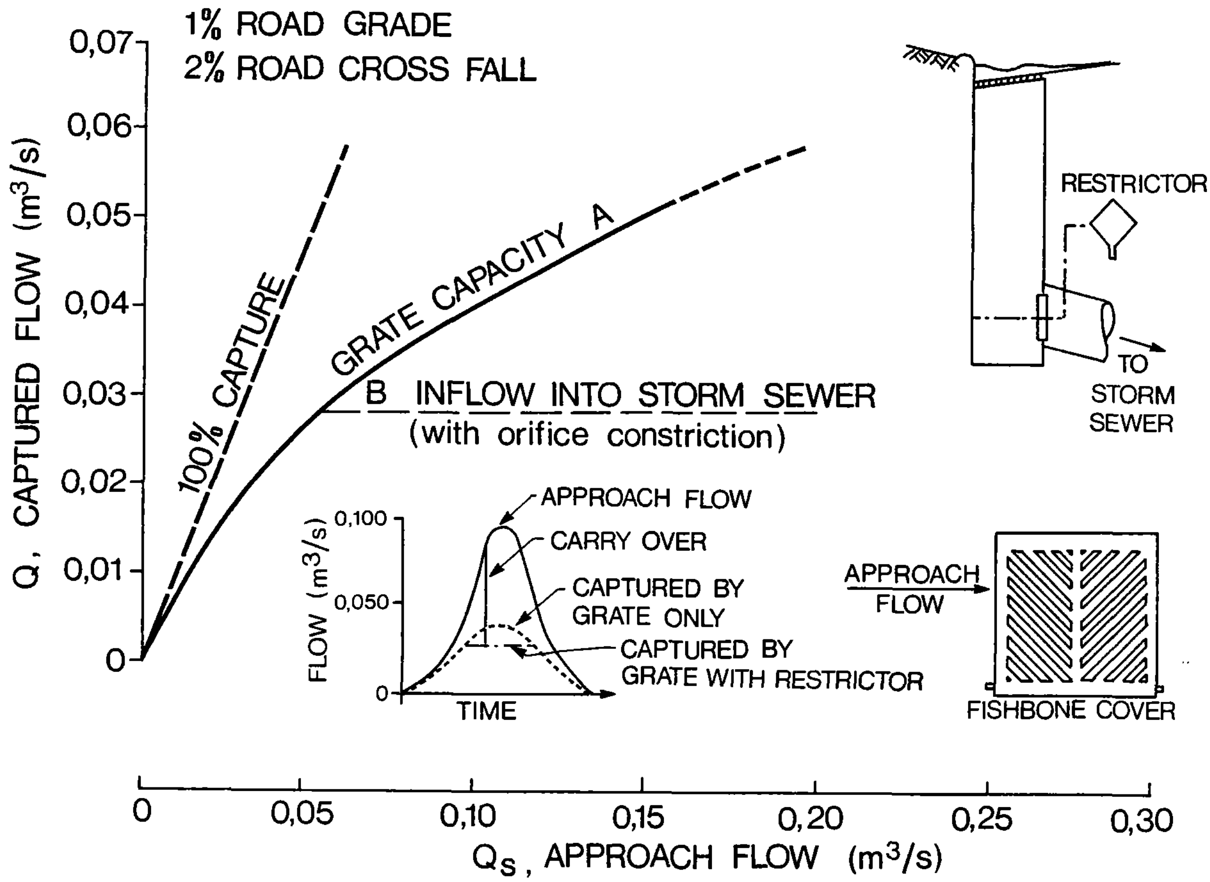


Fig. 7.11 Hydraulic performance of diamond-shaped inlet flow restrictors (after Townsend et al., 1981)

Besides hydraulic considerations, the design of overflow structures should also consider control of pollution in overflows and maintenance of these structures. Towards this end, it is recommended to maximize the amount of pollutants retained in the system and transported towards the treatment plant. This condition will minimize the load of pollutants in overflows. Furthermore, the overflow chamber should be self-cleansing. It should be designed so as to minimize turbulence and the risk of blockage and should require the minimum amount of maintenace.

The above conditions can be met by practical recommendations from model tests. An example of a stilling-pond overflow chamber, which was studied in a scale model, is shown in Figure 7.12 (Balmforth, 1982). This structure consists of a chamber in which dry-weather flow passes through a V-shaped channel to the outlet pipe. The discharge of stormwater is normally controlled by a weir, which is set at such a level that the sewer upstream of the chamber fills before the weir overflows and the volume stored in this way is discharged to the treatment plant after the storm has passed. A scum board which is located a short distance upstream of the weir prevents floating matter from passing over the weir. The outlet to the treatment plant interceptor is designed as a throttle pipe, which limits the rate of flow to a specific value at the moment when the weir begins to overflow. Ideally, this throughflow should remain constant. As the head over the weir increases, a further flow increase is inevitable but can be minimized by a careful design of the throttle pipe.

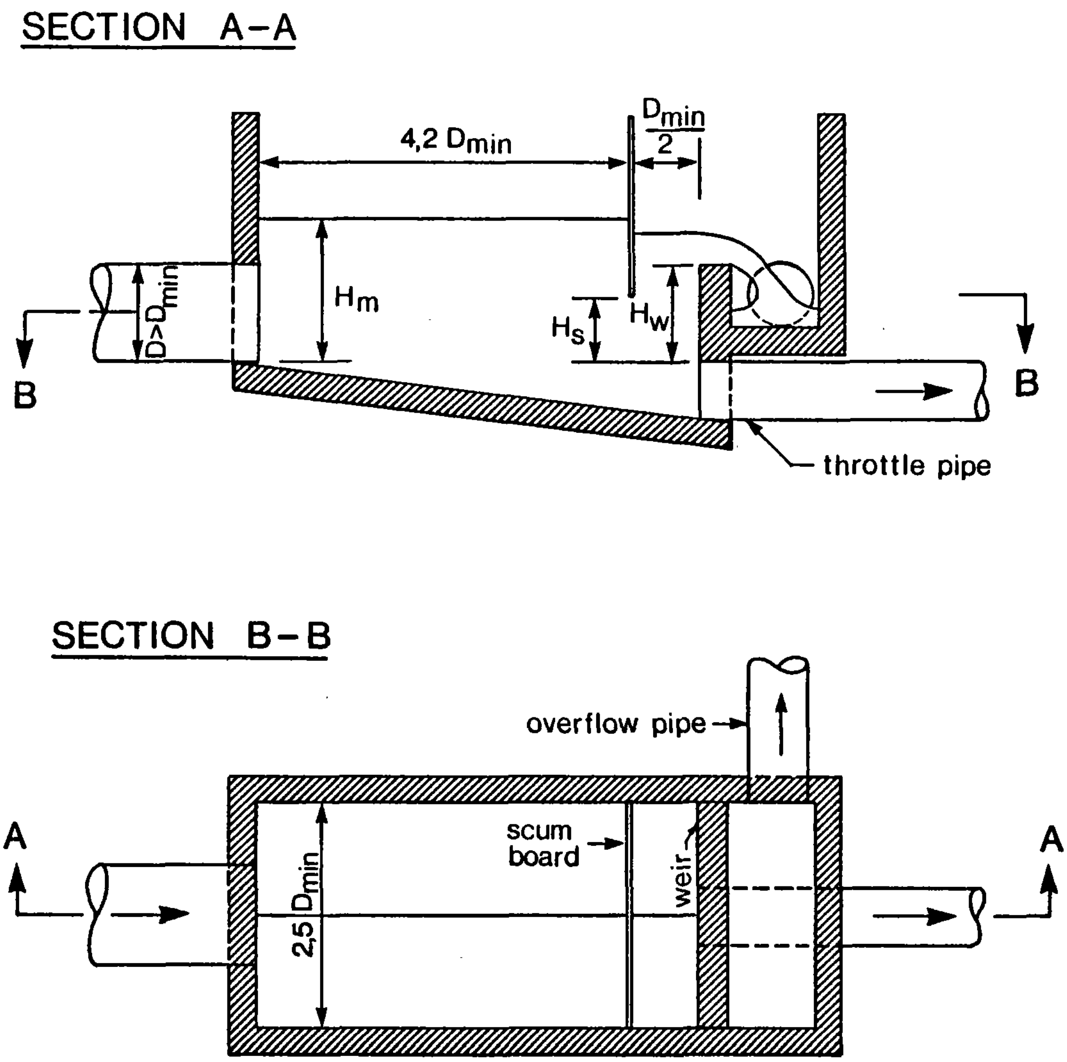


Fig. 7.12 Stilling-pond overflow chamber (Balmforth, 1982)

Model studies reveal that floating matter tends to remain in quiescent zones, at the surface, near the upstream corners of the chamber. A drop in the efficiency of settling occurred when the Froude number of the incoming flow, Q, exceeded the value of 0,60. Hence, the minimum diameter of the inlet pipe was recommended as

$$D_{min} = 0,165\ Q^{0,4} \tag{7.5}$$

Experiments indicate that the numerical coefficient in Equation 7.5 varies slightly with the height of the weir. Table 7.2 gives recommended dimensions of the overflow chamber shown in Figure 7.12.

Table 7.2 Recommended dimensions for stilling ponds (Balmforth, 1982)

Height of Weir Crest H_w/D	Minimum Diameter of Inlet Pipe $D_{min}\ Q^{0,4}$	Top Water Level H_m/D_{min}	Height of Scum Board H_s/D
0,9	0,165	1,60	0,5
1,0	0,161	1,70	0,6
1,2	0,157	1,85	0,8

It is also recommended to design the V-channel with the side slopes of 45° and the throttle pipe with a minimum diameter of 225 mm or greater.

During dry-weather flow, the throttle pipe runs partly full, and the velocity at this stage should facilitate self-cleansing of the structure.

Flow patterns at regulators with lateral weirs may be quite complex, and detailed hydraulic analyses or model tests are recommended for important structures. For typical, simpler structures, Jensen (1982) derived design graphs from analytical considerations.

7.7 CULVERTS

Although the primary purpose of culverts is to convey water under roadways, if properly designed, they may also be used to restrict flow and reduce downstream storm runoff peaks. The design of a culvert is influenced by the purpose, hydraulic efficiency, and costs of the proposed structure and also by the topography of the proposed culvert site. If there is a sufficient head of water, the choice of the inlet may not be critical. For limited heads and where erosion or sedimentation is a problem, a poor inlet design may severely restrict culvert capacity. A culvert can operate under inlet or outlet control. Inlet control conditions occur when the discharge capacity of the culvert is controlled at the culvert entrance by the depth of headwater, type of inlet edge, entrance geometry, barrel shape and cross-sectional area. Outlet conditions do not affect hydraulic performance of inlet-controlled culverts.

Outlet control refers to that condition where the capacity of the culvert is controlled at the outlet and depends on all the hydraulic phenomena taking place upstream of the outlet.

Some typical water surface profiles in culverts are shown in Figure 7.13.

In the case of inlet control, the tailwater level will be relatively low so that the culvert runs partly full for some or all of its length. If the slope of the bed is supercritical, the critical flow depth will be established at the inlet and, from there on, the depth of flow will diminish (case A). Case B shows the situation where the inlet is submerged by a high headwater level and the tailwater is very low. The critical depth could be induced at the inlet either by a steep downstream slope or a high headwater level. Case C, where a hydraulic jump occurs, is possible for high tailwater levels. In that case, it is recommended to ensure stable flow conditions in the culvert by installing a ventilation device. In each of the inlet-control cases, the barrel size downstream of the inlet could be reduced without affecting the culvert discharge. Conversely, if the inlet conditions were improved, the capacity of the culvert for any limiting headwater could be increased.

For very high tailwater and headwater levels, the culvert is completely drowned and the discharge is controlled by the difference between the entrance and exit water levels. This is a form of outlet control (case D). Another outlet control condition is found when the pipe capacity limits the culvert discharge and/or increases the headwater level. Consequently, the control is transferred to the barrel (case E).

The last two cases describe the pipe flow, in which the head available is used primarily to overcome conduit friction.

In some cases, culverts fail to perform as intended because of high tailwater levels which create backwater conditions. In general, the normal depth of flow in the downstream channel, at a discharge equal to the culvert capacity, should be computed to check if the water surface elevation is below the invert of the culvert exit.

For outlet control conditions, the outlet flow may be either a free flow with a critical depth at the outlet, or a pressure flow, if the outlet is submerged.

Culvert flow computations are frequently conducted by means of nomograms, such as those shown in Figures 7.14 and 7.15 (Portland Cement Association, 1964).

For inlet-controlled culverts, the required head is quite sensitive to the inlet shape. Operation of existing culverts can be improved by using an improved hydraulic shape. Studies have also been conducted with hooded inlets which eliminate vortices for submerged flow conditions.

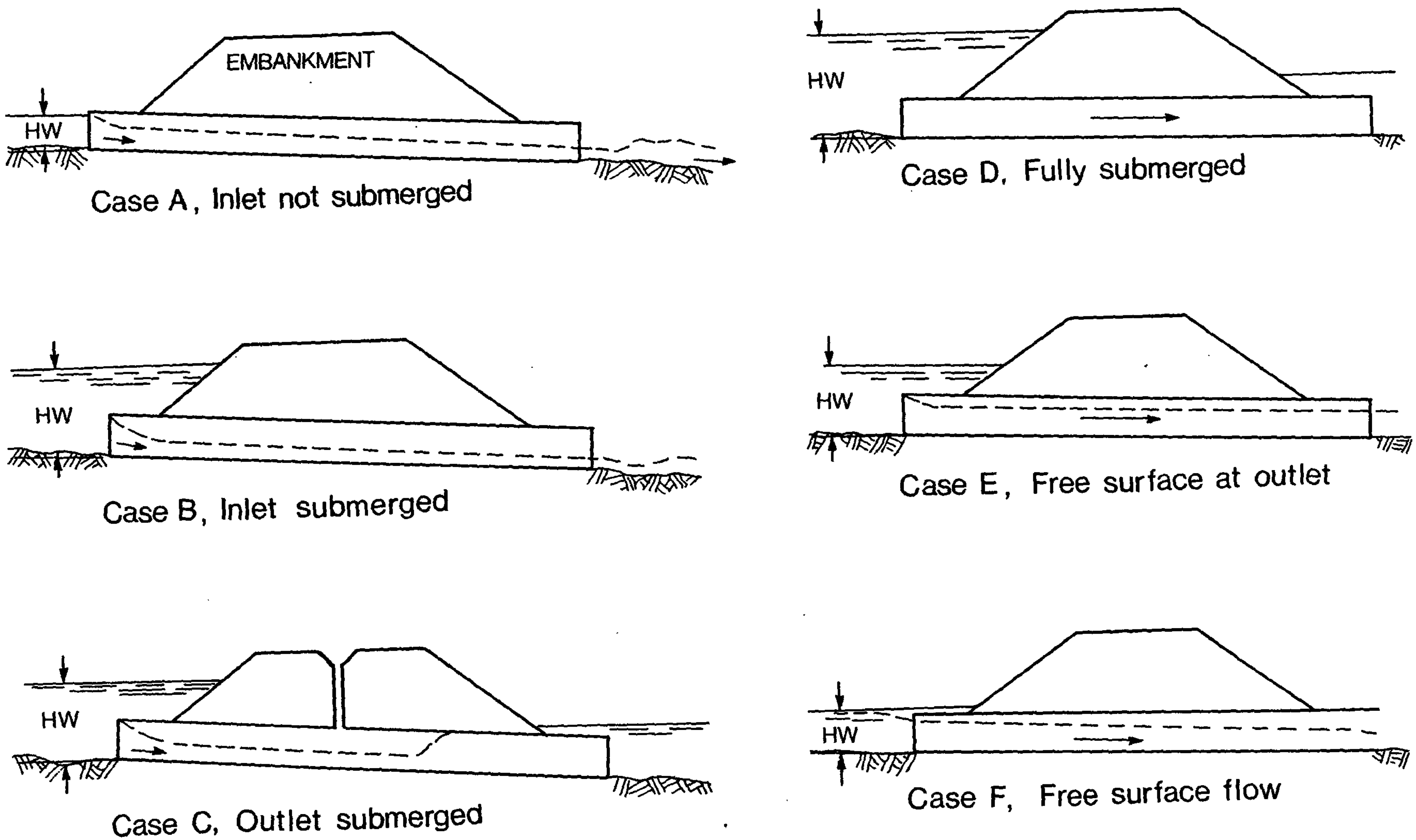


Fig. 7.13 Typical water surface profiles in culverts

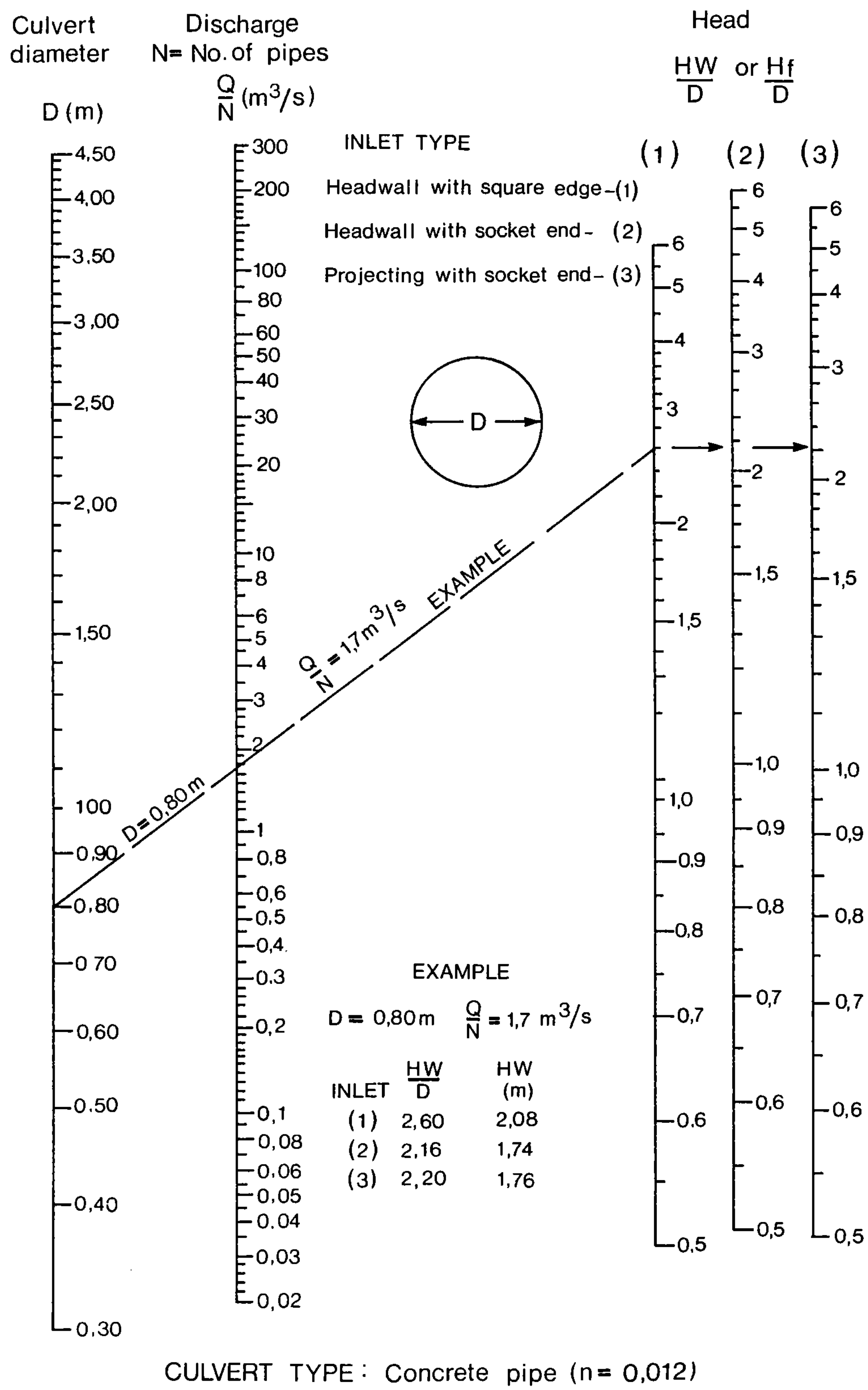


Fig 7.14 Nomogram for design of culverts with inlet control (Portland Cement Association, 1964)

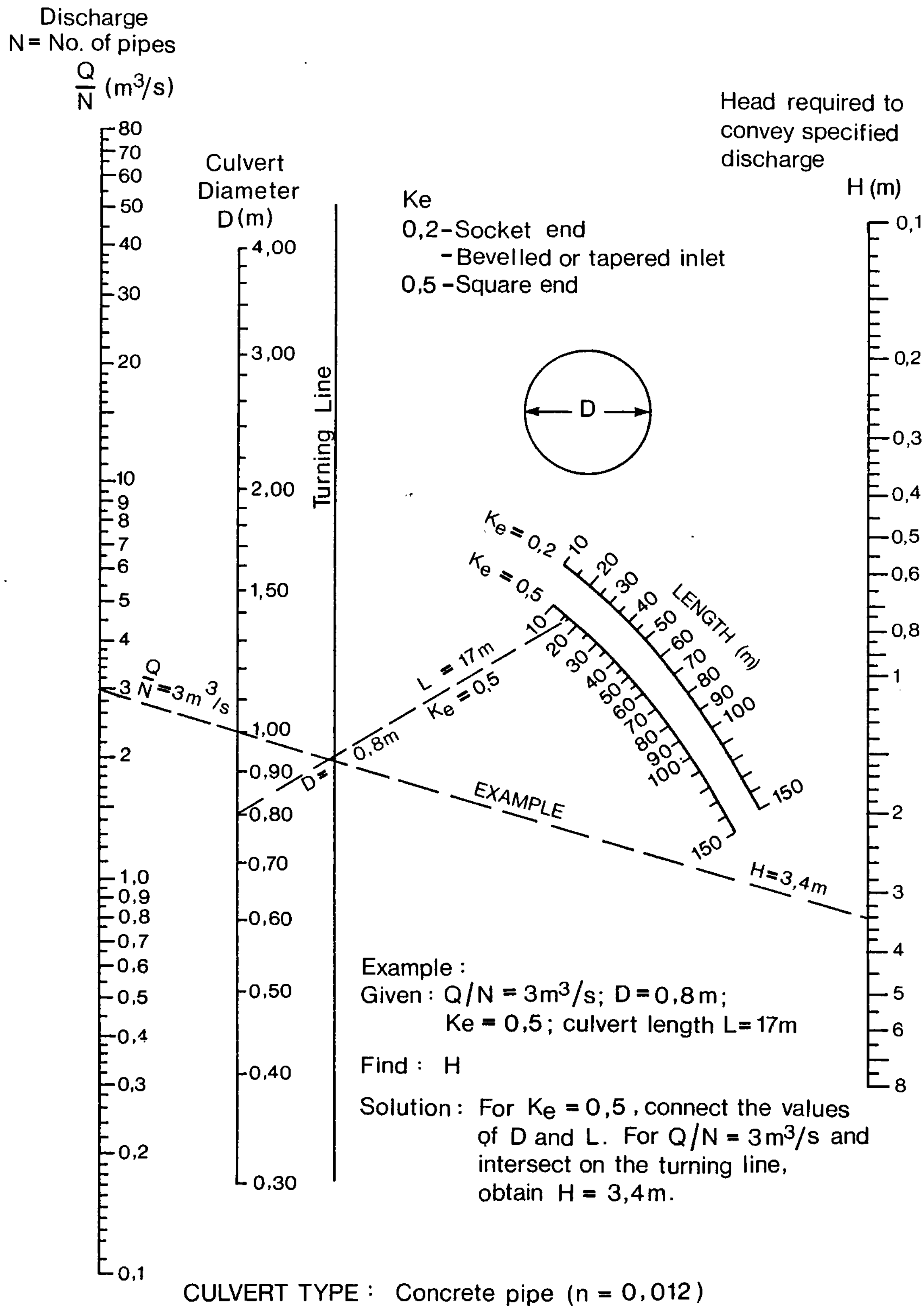


Fig 7.15 Nomogram for design of culverts with outlet control (Portland Cement Association, 1964)

7.8 OUTLETS OF SEWERS AND CULVERTS

Bank and stream bed erosion can be controlled by means of properly designed outlet structures. Such structures are designed as outlets with end walls and wing walls, aprons, or as submerged outlets with impact stilling basins (Figure 7.16).

The design of riprap downstream of outlets involves the selection of a rock size large enough so that the force attempting to overturn individual rocks is smaller than the gravitational force holding the rocks in place. Because riprap is graded, the design procedures must also consider an appropriate gradation of particle sizes so that erosion of smaller particles on the surface will leave an armored channel that is stable. Finally, the design procedures must also include a methodology for selecting appropriate underlying filters so that water flowing beneath the riprap will not erode the base material.

Free-springing jets occur when the culvert outlet height is greater than the tailwater depth which is a common case in steep terrain. The erosive potential of such jets is very high, particularly if the depth of flow in the receiving channel is shallow. In general, free-springing jets or partially submerged jets are acceptable only in highly stable receiving channels or for infrequent high-discharge operation.

Supported-jet outlets are those in which the downward diffusion of the jet is prevented by either a natural or man-made floor or apron. This type of operation will allow the flow from the outlet to spread and distribute itself across the width of the discharge channel. It is necessary to provide an apron of sufficient length to fully distribute the flow before it reaches the natural channel. The approximate length of apron can be expressed as

$$L_a = 0{,}66\ V_o\ D \tag{7.6}$$

where L_a is the length of the apron, V_o is the outlet velocity (m/s), and D is the outlet diameter (m).

If the tailwater covers the outlet apron and partially submerges the outlet, it is often desirable to provide end walls and wing walls. Wing walls may be built at any angle as long as the total angle of the flare is greater than a design value (usually 22°) and the apron covers the bottom of the enclosed area.

Generally, the width of the transition zone is increased in order to allow formation of a hydraulic jump as the tailwater depth increases. In some cases, even if the tailwater is too low to cause a hydraulic jump, it is desirable to use the hydraulic jump for energy dissipation. The necessary depth can be provided either by depressing the outlet apron, or by providing a broad-crested weir, at the end of the apron, which would create a pool of sufficient depth to meet the tailwater requirements for the formation of the hydraulic jump. The method of depressing the apron seems preferable, because there is less scouring at the end of the apron. To ensure the formation of the hydraulic jump on the apron, the use of baffle blocks is sometimes recommended.

In some cases, culverts operate with submerged outlets. Submerged jets discharged from such outlets remain concentrated and do not readily dissipate their energy. For jets supported by a bottom plane, velocities remain fairly high over the length of 6,2 D. Past this region, apron velocities for outlets with inverts located at the apron level may be calculated as

$$V = 6{,}2\ \frac{V_o D}{L} \tag{7.7}$$

where V is the maximum velocity (m/s) at the distance L (m) from the outlet, D is the outlet diameter, and V_o is the outlet velocity (m/s).

If protection by riprap or a long apron is not feasible, a compact solution may be achieved by using an impact stilling basin in which the flow strikes a vertical baffle and thereby loses its energy (Figure 7.16b). The baffle partly deflects the flow against the floor and partly, by a horizontal extension, against the incoming flow (see Figure 7.16b). Although no submergence of the outlet by tailwater is required for good operation, its performance improves with outlet submergence because of reduced outlet velocities. The design of the impact stilling basin depends only on the discharge and is independent of outlet velocity and pipe size as long as outlet velocities do not exceed 9 m/s (U.S. Department of the Interior, Bureau of Reclamation, 1960).

7.9 DESIGN OF STORAGE FACILITIES

In studies of runoff control by storage, the designer usually knows the inflow hydrograph which was determined by the hydrologic methods discussed in Chapters 5 and 8. Two types of design problems are usually encountered: the verification of an existing facility and the sizing of a new reservoir.

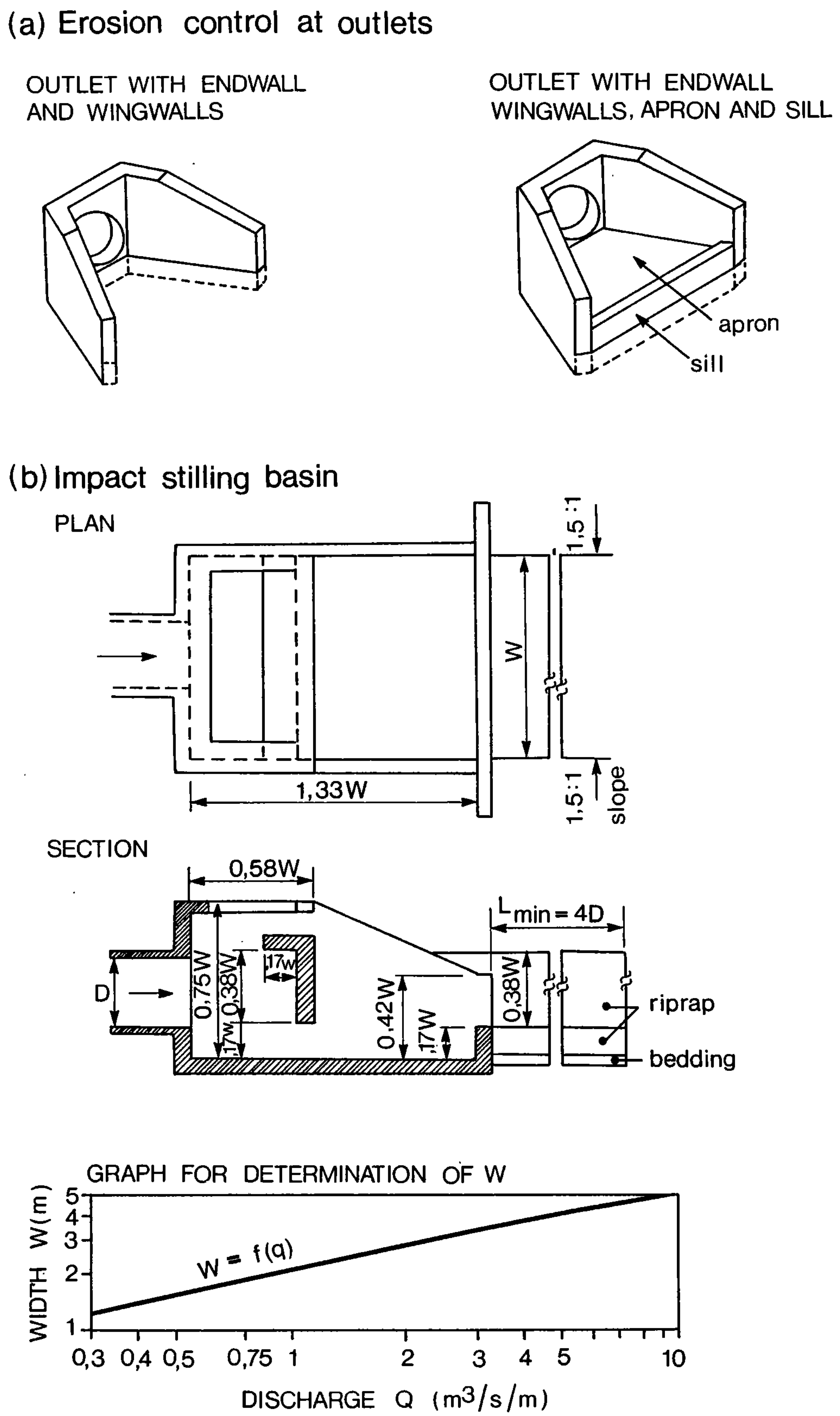


Fig. 7.16 Erosion control at outlets and design of impact-type stilling basins (after U.S. Department of the Interior, Bureau of Reclamation, 1960)

In general (see Figure 7.17), the change in storage, ΔS, during a time interval, Δt, is given by

$$\Delta S = \frac{I_2 + I_1}{2} \Delta t - \frac{O_1 + O_2}{2} \Delta t \qquad (7.8)$$

where I is the inflow rate, O is the outflow rate and subscripts 1 and 2 refer to the beginning and end of Δt, respectively. Equation 7.8 can be reduced to the form

$$\frac{\Delta S}{\Delta t} = I - O \qquad (7.9)$$

which is known as the continuity equation for reservoir routing. For practical calculations, Equation 7.8 is rearranged as

$$(S_2 + \frac{O_2}{2} \Delta t) = (S_1 - \frac{O_1}{2} \Delta t) + \frac{I_2 + I_1}{2} \Delta t \qquad (7.10)$$

where $S_2 - S_1 = \Delta S$.

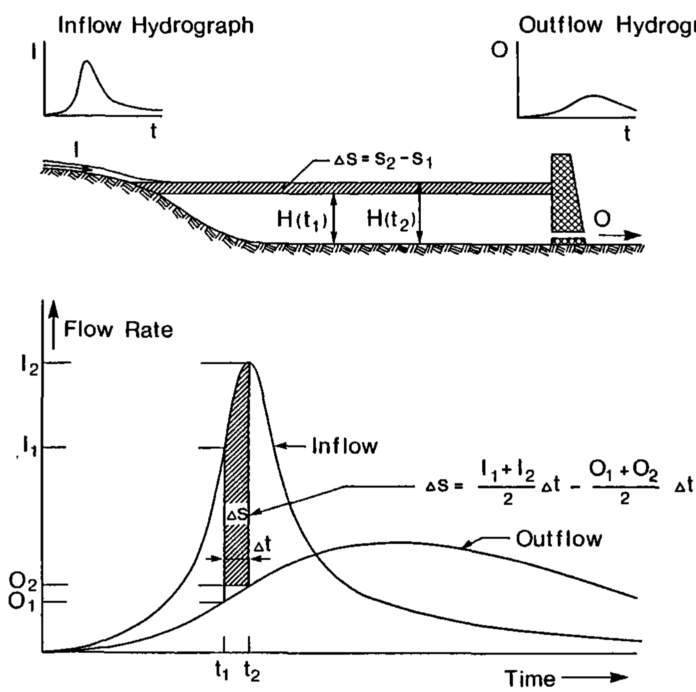


Fig. 7.17 Flow routing through a reservoir

In the verification case, the outlet characteristics are usually known as a relationship between the outflow, O, and elevation, Z. On the other hand, the reservoir geometry gives a relationship between Z and the storage, S. It is, therefore, easy to express the outflow as a function of storage in the form

$$O = f\,(S + \frac{O}{2} \Delta t) \qquad (7.11)$$

which can be plotted. At each time step, the right-hand terms in Equation 7.10 are known and a tabular computation gives the left term for which the outflow value, O_2, can be determined from Equation 7.11.

For design purposes, several trial and error computations with various storage volumes are required, until the maximum outflow, O_{max}, is achieved. Its magnitude is usually known from an analysis of downstream conditions or as a pre-development value.

The maximum depth in the reservoir is not known and should be first assumed. For this depth and Q_{max}, it is required to size an outflow structure and calculate the head vs. outflow relationship and the storage vs. outflow relationship. Once this is determined, the initially estimated dimensions are verified according to the previously described procedure. If the required storage volume or maximum outflow is different from the initially assumed value, another trial is necessary. For many outlets, the discharge varies with the head.

It is also recommended to verify the reservoir operation for rainfalls with different durations and return periods. A reservoir for peak flow shaving designed for a given maximum inflow may not be efficient for larger or much smaller peak inflows. If peak shaving is required for different return periods, it may be necessary to design the facility as a multiple outlet storage.

Other simplified methods are also used. One of them is based on hydrographs derived from the rational method (Section 5.3.5). By approximating the release rate, it is possible to find approximate storage volumes for different durations of a uniform rainfall. The maximum storage is given by a critical rainfall duration.

7.10 ECONOMIC ANALYSIS

The return period of a design storm should be based in principle on a cost-benefit analysis, detailed or implicit. The return period of design storms recommended in design criteria for various municipalities varies significantly. As discussed in Chapter 4, minor systems in many residential areas in North America are designed for flows corresponding to 5- to 10-year storms without surcharge. Downtown areas have greater return periods (25 years). In some countries in Europe, however, the return period is based on surcharge below the street level for a one-year storm.

This wide variation shows that the sizing of drainage systems is frequently based on tradition, available funds, political pressures, etc.

Although an economic analysis is in principle possible, it is rarely used for minor systems. Attempts have been made, however, to use this approach for major drainage systems, mainly in connection with flood damage reduction programs. Several agencies in the U.S.A. and Canada have developed standardized relations for estimation of damages, in terms of flood elevation, for various types of homes. It is, therefore, possible to find the damage for floods with different frequencies. Additional general flood protection benefits to the entire community, from traffic improvements, etc., are considered as a percentage of this value. For a given flood control alternative, the affected area and annual damage will be reduced and the difference between the two values with and without control gives an annual benefit. Annual costs are given by financial analysis methods which consider annual costs of repayment of the investment. Such costs vary with the discount rate and operation maintenance.

Comparisons of the cost-benefit ratio, or of the net benefits, are useful for decision-makers. Selection of the preferred scheme, however, accounts frequently for social, environmental and political factors which are more difficult to quantify.

8 Network design methods

8.1 INTRODUCTION

This chapter gives information concerning the most widely used methods for the design of sewer networks, starting with the simplest ones and proceeding to the more complex ones. As stated earlier in Chapter 1, it should also make designers aware not only of the accuracy and the limits of these methods, but also of the needs or opportunities for their use. Using such information, the designer should be able to select a good tool, if not the best one, according to design objectives, and the available data, computer facilities and resources. Some technical aspects of these design tools were also discussed in Chapters 5, 6 and 7.

After more than ten years of a worldwide intensive research effort in urban hydrology, many new computational tools have been developed. Most of them are based on more or less sophisticated modelling of the urban water cycle. Some of them are well known; they have been applied successfully in many practical studies and are freely available, because they have been developed by means of public fundings. An engineer with access to good computer facilities should normally choose one of these tools, according to his design objectives, and the available resources. However, it should be borne in mind that a proper use of such a new method or tool inherently implies a good knowledge of the detailed operations which the method or tool perform. In other words, the engineer should have a sufficient knowledge of the hydrological and hydraulic processes simulated by the tool he is planning to use. If not, the advantages of using a new procedure may be illusive and such an application may turn out to be time consuming, expensive and possibly produce even less accurate results than a well-known and well-applied simpler method.

This is probably the reason why the rather old, approximate and simple, so-called rational method, and its variants, remain so popular in developed countries where hydrological simulations are not limited by the lack of computer facilities. To obtain full benefits from using new design methods, interested practitioners should consider training in urban hydrology and water management modelling. In turn, their thorough knowledge of practical drainage design should be of great interest in improving modelling tools produced by researchers. Practitioners are best qualified to identify the shortcomings of models encountered in practical applications and also to find ways for making models attractive for their colleagues. A good example of such fruitful co-operation is the U.S. Storm Water Management Model (SWMM) User's Group whose formation was partly prompted by the fact that the SWMM is a rather complex and sophisticated model which should never be used without a careful inspection of its applicability or its usefulness in a given design situation (U.S. EPA, 1977; U.S. EPA, 1978-1982).

In the chapter that follows, some examples are given only for the simplest methods. Other examples and detailed information on the most comprehensive and sophisticated models can be found in the referenced handbooks and papers. Frequent references are made to Chapters 3, 4 and 5, which provide background information for this chapter. Finally, Chapters 1 and 2 provide descriptions of general problems and concepts which are pertinent to network design.

8.2 PRELIMINARY ANALYSIS

A preliminary analysis is a very important first step of the design procedure. It involves considerations of the basic approaches to urban drainage which were developed in Chapter 2. After this first step, the engineer should be able to choose an appropriate method for his design problem.

Some items of the preliminary analysis are quite obvious and these are summarized below.

- The design objectives which, as stated in Chapter 1, can be classified into various categories, such as screening, planning, analysis, design and operation. The precision levels, the time and space intervals used to describe the rainfall-runoff process and drainage simulations may vary from one objective to another and so may vary the design methods or models used.
- The accuracy of data describing the existing urban area and its drainage network, and the accuracy of projections of the future development and its connection to the existing drainage network.
- Local constraints on the drainage area given by frequent flooding, quality aspects of runoff, existing sewers, and the available space for drainage or treatment facilities.
- Constraints imposed by receiving waters - runoff quality or quantity, sewage overflows, downstream receiving water uses, etc.
- The available data, such as rainfall data, runoff coefficient estimates, runoff quality data and existing sewers geometry.
- Local constrains for choosing a design method, according to the level of knowledge of rainfall-runoff processes, computer facilities available for simulations of these processes, technical guidelines and regulations, etc.

Of course, many of the above items should be examined when using any design method. A special attention should be paid to the design parameters as described earlier in Chapter 4. Among these parameters, the design return periods of elements of the drainage system are probably most dependent on designer's experience reflecting local socio-economic conditions. As said in Chapter 4, the choice of the design return period should be based on the least-cost concept, but that theoretical concept is rarely effective in practice because of large uncertainties in the estimated failure costs and in probabilistic distributions of runoff peak discharges and volumes. From a general point of view, it should be recognized that the construction costs of storm sewer systems are often barely sensitive to the design return period regardless of the design method used (Section 4.1).

This discussion of preliminary analysis, given as introductory remarks to the design methods for sewer networks, shows that engineers should systematically test the sensitivity of the models or formulae they use, in order to identify the most sensitive parameters having the greatest influence on the calculations and which, therefore, should be estimated with the greatest precision level. These sensitivity tests are very helpful for choosing an appropriate method according to the conditions of the design problem and may save a lot of time. It should be noted that sensitivity analyses are not specific to sewer network design, but they generally apply to any engineering technique which involves many uncertainties.

8.3 METHODS YIELDING PEAK FLOW FOR SINGLE EVENTS

These methods have been most widely used and will be probably used for a long time to come because they require limited input data and less calculations than storm runoff models. Their use is limited to quick screening of objectives, and the planning and design of tree-type storm sewer systems or small drainage structures (like culverts) in rather small watersheds. They can be classified into two broad types: General empirical formulae and the rational method with its derivatives.

8.3.1 General empirical formulae

From a general point of view, these formulae give the peak discharge at the outlet of a given watershed as a function of the following parameters or variables:

- Watershed characteristics: Area, mean slope, main channel length, etc.
- Rainfall characteristics: Mean intensity for a given duration, rainfall depth for a given duration, return period of the mean intensity, etc.
- Land use and surface cover: Urban, pavement, lawn, forests, etc.

These formulae have been established for specific design objectives in various locations, according to the available experimental data. Although many such formulae are available (Chow, 1964; Rawls et al., 1980), they should not be used outside of their experimental domains. The simpler ones give only a specific peak runoff rate per unit of catchment area (see Chapter 5.3.3). Some of them represent empirical probability distributions of peak runoff, in which the distribution parameters have been expressed as a function of some variables mentioned above. The more complex formulae have been established using correlation or multivariate analysis. Although these formulae can give quick runoff estimates for a given area, they cannot be used without experimental support reflecting local conditions.

8.3.2 Rational method and its modifications

The rational method is probably the most popular method for designing storm sewer systems. It has been applied all over the world and many refinements of the method have been produced. Although the rational method incorporates some empirical aspects and its applications require a great deal of judgement and experience, it has been shown that it is founded on a theoretical basis and a well accepted hypothesis. This made the method transferable from one country to another.

8.3.2.1 General formula

The rational method can be written in a general form as

$$Q_p(T) = k \cdot C \cdot iM(t_c,T) \cdot A \cdot F \tag{8.1}$$

where $Q_p(T)$ is the peak runoff, for a return period T, at the design point in the drainage system,
- C is the runoff coefficient,
- $iM(t_c,T)$ is the mean rainfall intensity, with a return period T, for a duration equal to the so-called time of concentration,
- A is the area of the watershed upstream of the design point,
- k is a unit conversion factor depending on the units used, and
- F is a correction factor taking into account such effects as spatial rainfall distribution, storage in sewers, watershed shape and slope, etc. This correction factor is not used in all formulations of the rational method.

The rational method formula looks very simple and that is the reason why it remains so popular. However, it is founded on some implicit hypotheses which indicate its limitations.

The first hypothesis assumes that Q_p is produced by the mean rainfall intensity of duration equal to the time of concentration, t_c, and so it is independent of the temporal distribution of instantaneous intensities over that duration. This means that the rainfall-runoff process is assumed to be a linear process. If it were non-linear, the rational method could lead to large errors. Consequently, the method should not be used in the following cases:
- Watersheds with important storage effects, such as detention basins, flood plains, strong backwater effects in flat areas or caused by submerged outlet conditions.
- Watersheds with strong variations in the areal distributions of land slopes or land use.

The second hypothesis is a probabilistic one because it assumes that the peak runoff Q_p has the same return period as the mean rainfall intensity. This means that the rainfall-runoff process is not random and the runoff coefficient C is not a random variable. On the other hand, C should be a random variable in natural watersheds where the runoff coefficient depends on antecedent rainfall conditions. Assuming the return period of $iM(t_c,T_1)$ as T_1 and the return period of $C(T_2)$ as T_2, the peak discharge $Q_p(T')$, given by Equation 8.1, will generally have an unknown return period T', which can be estimated only by statistical analysis of observed data. However, as T_1 increases, the error in T' becomes smaller and, for saturated conditions, Equation 8.1 may lead to correct estimates. For such conditions, the infiltration rates remain approximately constant (see Chapter 5.2.5) and so does the runoff coefficient. But it is not generally possible, without experimental data, to give an a priori lower limit of T_1, for which the runoff process becomes deterministic. Although many modified rational method formulae taking into account these effects have been proposed for natural watersheds (Chow, 1964), they should not be extrapolated or transposed.

As a consequence of the rational method hypotheses, the use of the method should be limited to small homogeneous urban watersheds and the design of simple tree-type storm sewer systems with free outfalls and no special hydraulic structures (e.g., detention basins). In such conditions, the method gives good estimates of peak runoff as shown experimentally by comparison of the results given by the rational method and much more complex simulation models (Wisner, 1981). However, it is not easy to give precise limits on the method applicability, because they may depend on local drainage practices. Examples of such limits are given below.

For the total watershed area, the upper limit for the applicability of the rational method should be between 100 and 300 ha. When comparing practices in various countries, it was noted that this upper limit varies widely. For example, this limit is 10 - 50 ha in the United Kingdom, 200 ha in France, 1500 ha in the U.S.A., and 4500 ha in the Soviet Union. Such upper limits for the rational method applicability depend on catchment homogeneity, experimental data and analysis, and local refinements of the method.

The rational method is generally applicable to the catchments with a minimum imperviousness from 20% to 30%. The selection of this minimum value further depends on local practices for drainage of natural pervious areas in towns (such as open spaces or private gardens), the return periods under consideration (usually less than 10 years), climatic conditions, vegetation, soils characteristics, etc.

8.3.2.2 Simple applications of the rational method

The rational method may be used successfully for design of small hydraulic structures, such as small bridges or culverts, if only the peak runoff is necessary for such design and the watershed upstream of the design point falls within the general application limits of the method.

Assuming that the local IDF curves are known (see Section 4.6.1), the designer has to estimate C and t_c. Usually, a mean value of C is estimated by weighting the individual C_i values of homogeneous subareas A_i according to the following expression:

$$C = \Sigma\, C_i A_i \,/\, \Sigma\, A_i \tag{8.2}$$

For each subarea with homogeneous land use, the C_i values may be estimated from local experience or well-known coefficient tables (see Section 5.2.3). If there are important pervious areas connected to the drainage system, the designer should estimate the runoff coefficient C_p for these areas according to the selected design return period. Usually, a rough estimate of the mean runoff coefficient C of the area upstream of the design point can be given as

$$C(T) = (1 - IMP) \cdot C_p(T) + IMP \cdot C_{imp} \tag{8.3}$$

where IMP is the imperviousness measured from aerial photographs or field investigations, $C_p(T)$ is the runoff coefficient of the pervious areas and C_{imp} is the runoff coefficient of impervious surfaces, such as roofs, street, and parking lots. Moreover, the designer should only consider the pervious and impervious areas which are directly connected to the sewer system. If a and b are the fractions of the total directly-connected impervious and pervious areas, respectively, then Equation 8.3 can be written as

$$C(T) = b \cdot (1 - IMP) \cdot C_p(T) + a \cdot IMP \cdot C_{imp} \tag{8.4}$$

In some cases, a and b are much smaller than 1, and further depend on the return period T. In typical European urban conditions, b is often very small and may be neglected for the design periods usually considered. However, the situation may be different in developing countries with significant pervious open spaces without dense vegetation cover.

For C_{imp}, a value of 0,9 is generally used. Because C_p depends greatly on local climatic conditions, local soil coverages and characteristics, and local practies, it is not easy to give a default value for C_p. For example, in developed countries of Europe and North America, such a default value will range from 0,1 to 0,3 for T values from 5 to 10 years. However, the designer must remember that in these countries the soil vegetation cover is often dense. On the other hand, for bare soils with slopes up to 5%, the C_p value may exceed 0,6. As the calculated peak runoff is directly related to the mean C value, the designer should pay a special attention to its estimate.

The estimate of t_c is not as easy as it seems. Usually, t_c is taken as the sum of the overland flow time, t_o (or the "time to entry" or the "inlet time"), and the time of travel, t_d, in sewers or the main channel. While t_d can be estimated using the flow formulae given in Chapter 6, other methods are needed to compute t_o. For small fully-sewered areas, some drainage authorities specify t_o as a constant typically ranging from 5 to 15 minutes. In more complex situations, it is recommended to use the kinematic wave formula in the following form (Ragan and Duru, 1972):

$$t_o = 6.9\, L^{0.6}\, n^{0.6}\, i^{-0.4}\, S^{-0.3} \tag{8.5}$$

where t_o is in minutes, L is the travelled length (m), n is the Manning's roughness coefficent, i is the rainfall intensity (mm/hr), and S is the slope (m/m). Other formulae for t_o and t_c were discussed and evaluated by McCuen et al. (1984).

For quick estimates or for comparison of various development scenarios (planning purposes), a rough estimate of t_c can be obtained by dividing the length of the main drainage route by the mean estimated flow velocity. In many cases the Q_p value is less sensitive to t_c than to C.

The rational method remains of great interest for quick comparison of development schemes, or for evaluation of the effects of urbanization shown in the example below. As shown in Figure 8.1, a natural watershed that may be divided into two subareas A_1 and A_2 is assumed. The catchment data given in Table 8.1 describe both pre- and post-urbanization conditions. Detailed calculations are given in Appendix B and their results are summarized in Table 8.2. They lead to the following observations:

Firstly, for natural conditions, it should be noted that the peak runoff from the whole watershed, at outlet 0, is greater than Q_{p1} or Q_{p2} but smaller than $(Q_{p1} + Q_{p2})$. In general, the usual calculations by the rational method should lead to the following relationships:

$$Q_p \geq \text{Max } (Q_{pj}) \tag{8.6}$$

$$Q_p \leq \Sigma \; Q_{pj} \tag{8.7}$$

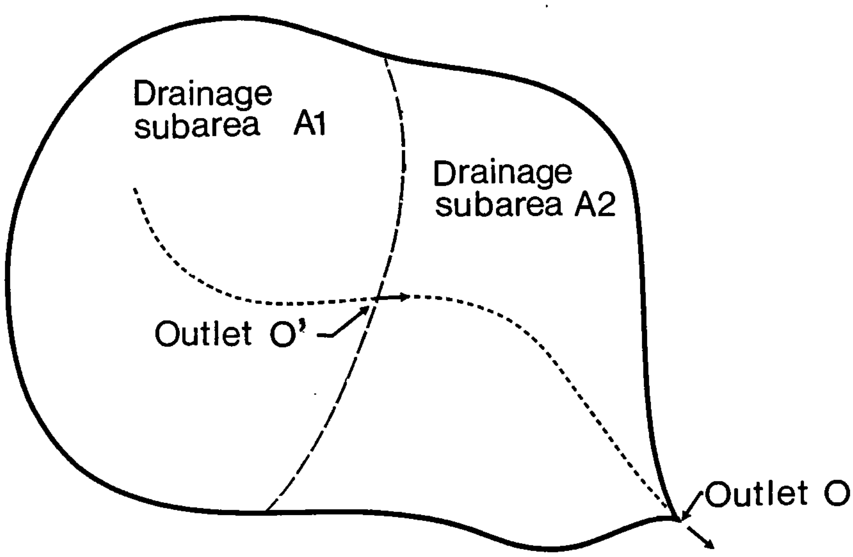


Fig. 8.1 Catchment used for demonstration of applicability of the rational method

Table 8.1 Watershed characteristics

Sub-area	Pre-urban		Post-urban	
	A_1	A_2	A_1	A_2
Area (ha)	40	60	40	60
Channel length (m)	1200	1600	1200	1600
Channel flow velocity (m/s)	1,2	0,8	2,0	1,4
Overland flow length (m)	300	400	300	400
Overland flow velocity (m/s)	0,3	0,2	0,6	0,4
Runoff coefficient	0,3	0,2	0,6	0,6

Some "partial-area" effects, however, may lead to results which do not satisfy Equations 8.6 and 8.7. In that case, it may be concluded that the application limits of the rational method have been exceeded because of strong non-homogeneities in the subcatchments. That is to say that another method must be used in order to get more precise results.

Table 8.2 Peak runoff (m^3/s) for various development scenarios

Peak Runoff	A_1, A_2 Natural	A_1 Urban A_2 Natural	A_1 Rural A_2 Urban	A_1, A_2 Urban
Q_{p1}(0')	2,6	6,8	2,6	6,8
Q_{p2} (0)	1,9	1,9	7,6	7,6
Q_{p1+2} (0)	3,8	5,8	7,7	12,7

Secondly, it should be noted that urbanization results in strongly increased peak runoff values - by 160% for A_1, 300% for A_2 and 230% for the entire watershed. Moreover, when only a part of the watershed is urbanized and the subareas A_1 and A_2 are fairly non-homogeneous, in terms of C-values and drainage velocities, the results seem erroneous, especially in the case when A_1 is fully urbanized. The total peak runoff at the outlet of 5,8 m^3/s is smaller than Q_{p1} at 0' (6,8 m^3/s). Equation 8.6 is no longer satisfied. This is a proof of the rational method deficiency in this case, even if the watershed is rather small. To avoid that, the designer should consider the value of 6,8 m^3/s for the outlet 0, if there are no important storage effects between points 0' and 0.

8.3.2.3 The rational method applications in design of sewer networks

As stated before, the rational method is the most widely used method for the design of storm sewers. Many countries, including some developing ones, have produced guidelines, manuals and handbooks giving more or less detailed or standardized information on the use of this method (WPCF, 1970a; ATV, 1982a; Ministere de l'Interieur, 1977; Rawls et al., 1980; V.A.V. 1976).

Having limited the use of the rational method to simple tree-type sewer networks, the design principles are the same for any form of this method. Such principles are summarized below.

- The drainage area is first subdivided into subareas with homogeneous land use according to the existing or planned development.
- For each subarea, the designer estimates the runoff coefficient C_j and the corresponding area A_j.
- The layout of the sewer system is then drawn according to the topography, the existing or planned streets and roads, and local design practices.
- Inlet points are then defined according to the detail of design considerations. For main sewers, for example, the outlets of the earlier mentioned homogeneous subareas should serve as inlets to the sewers. In that case, the A_j-areas are typically 1 to 10 hectares or more, and should be serviced by a secondary sewer system, implicitly taken into account in the calculations. On the other hand in very detailed calculations, all the inlet points should be defined according to local design practices. The drainage areas per inlet should be much smaller, often less than 1 hectare.
- After the inlet points have been chosen, the designer must specify the drainage subarea for each inlet point A_{in} and the corresponding mean runoff coefficient C_{in}. If the drainage subarea for a given inlet has non-homogeneous land use, a weighted coefficient may be estimated using Equation 8.2 (or 8.3 and 8.4).
- In practical applications, a simple tree-type layout scheme of a sewer network is drawn as shown in Figure 8.2. The inlet points are numbered from the most upstream parts of the watershed to the final outlet. The trunk sewer lengths and mean invert slopes are estimated. In a preliminary step of the design procedure, a first estimate of the invert slopes can be obtained by using the mean slopes of the soil surface.
- The runoff calculations are then done by means of the general Equation 8.1 for each inlet point, proceeding from the upper parts of the watershed to the final outlet (point 8 in Figure 8.2). The peak runoff which is calculated at each point is then used to determine the diameter of the downstream trunk sewer using a hydraulic formula for pipes flowing full (see Section 6.3).

The main problem in the above computations is to estimate the time of concentration t_{ck} at the k-th inlet point (McCuen et al., 1984). For the most remote subareas, inlets 1, 5 and 6 in Figure 8.2, the time of concentration is approximately equal to the travel time of overland flow. A fairly arbitrary value is often specified for t_c, ranging from 1 to 15 minutes, according to local drainage and design practices. In other cases, t_o may be estimated from Equation 8.5 or as shown in Section 5.3.4.

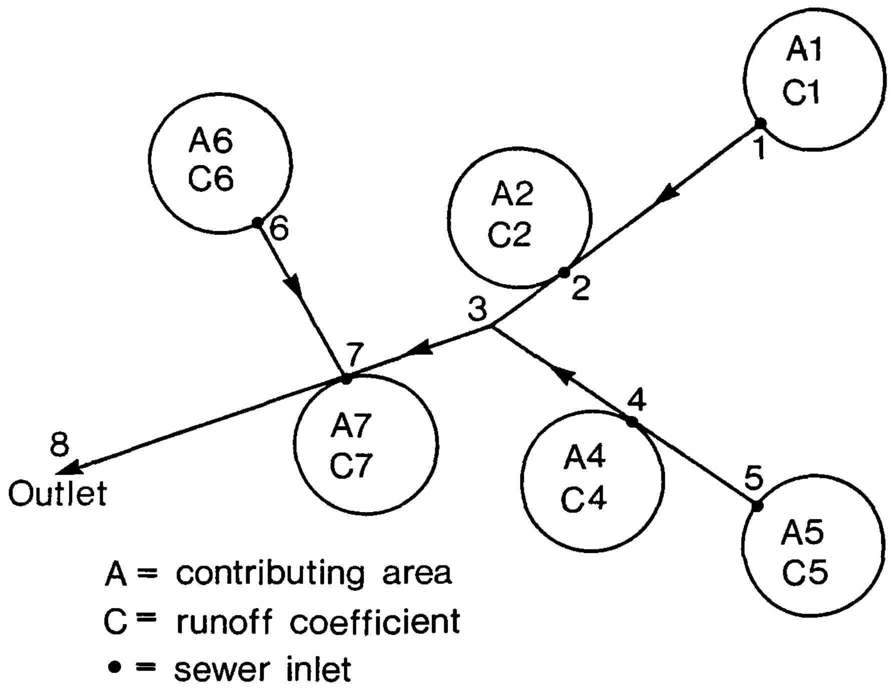


Fig. 8.2 Tree-type scheme used in the application of the rational method

As the calculations progress downstream, the times of concentration at any inlet point must be estimated along all the possible flow paths and the longest time is generally retained in the calculations. The reader should note that the longest time is not necessarily obtained for the longest flow path. For example, at inlet point number 7, the various flow times to that point are as follows:

$t_{o1} + t_{d1,2} + t_{d2,3} + t_{d3,7}$
$t_{o2} + t_{d2,3} + t_{d3,7}$
$t_{o5} + t_{d5,4} + t_{d4,3} + t_{d3,7}$
$t_{o4} + t_{d4,3} + t_{d3,7}$
$t_{o6} + t_{d6,7}$
t_{o7}

When reaching the inlet point, the designer must note that the times of flow in sewer trunks, such as $t_{d1,2}$ between points 1 and 2, are normally unknown, because the pipes upstream of point 7 are not flowing full for the design rainfall corresponding to the sewer trunk between points 7 and 8. For small sewer systems, the designer should use the flow velocities calculated at preceeding steps for full-flowing pipes. The calculations are thus explicitly defined by proceeding from the upper parts of the watershed to its outlet. Such a procedure should lead to a small overestimation of the calculated peak. For very large sewer systems, the peak runoff at a design point must be calculated by a trial and error procedure which is summarized in five points below.

- Obtain the first estimate of the time of concentration at the design point by assuming that the upstream pipes are flowing full.
- Read the first estimate of the design rainfall intensity from the IDF curves for the selected design return period.
- Calculate peak runoffs at the inlet points upstream of the last point under design.
- Estimate travel times for each partially-filled sewer trunk using tables or graphs (see Chapter 6).
- Produce a new estimate of the time of concentration and repeat the whole procedure until the estimated and calculated concentration times, for the design point, are approximately equal.

When the above refinement is implemented in hand calculations, such calculations become rapidly awkward as the area upstream of the design point becomes larger. Of course, the procedure may be simply computerized using small programmable calculators.

After the first calculation of the peak discharge has been completed, the final design procedure begins. In some cases, because of constraints on flow velocities in sewers, the initially chosen sewer slopes, which are equal to soil surface slopes, have to be modified. New slopes have to be chosen according to velocity limitations and the above procedure must be repeated in order to check the earlier selected pipe diameters.

To avoid the trial and error procedure, some methods use modified forms of Equation 8.1. Usually, such forms use an explicit formulation of the time of concentration at the point under design. In France, for example, such a modified form of the rational method is known as the Caquot's formula (Ministère de l'Intérieur, 1977) in which the time of concentration is given as

$$t_c = m \cdot S^c \cdot A^d \cdot Q_p^f \tag{8.8}$$

where S is the mean slope along the path corresponding to t_c, and m, c, d, and f are parameters defined from observations. For French design conditions, the parameter values are 0,5, - 0,41, 0,51 and - 0,29, respectively.

The reader should also note that if the designer uses the longest flow times along various flow paths upstream of the design point, some partial-area effects are encountered as the area gets larger, and such effects should be examined. These partial-area effects are caused by the opposite influences of the increasing area, when proceeding downstream, and the decreasing design rainfall intensity with the increasing t_c. So, in some cases, because of non-homogeneous slopes and land use, it may happen that an intense storm of a duration shorter than the time of concentration, for a particular design return period, may produce a higher peak runoff, even if only a part of the area upstream of the design point is contributing runoff to that peak. Such a situation should be checked at the junction of two main sewers, at the outlet from a large subarea with relatively large concentration time and in other similar cases.

The reader will find an example of the rational method calculations for the sewer system shown in Figure 8.2 in Appendix C.

In many forms of the rational method, a correction factor F is used (see Equation 8.1). Such a coefficient is evaluated from local land use and observations, and should not be extrapolated or transposed without a careful inspection of its influence. For rather small watersheds, say less than 100 hectares, this coefficient may be about 1 and the magnitude of the correction that it would introduce would be far below the uncertainties inherent to the rational method.

8.3.2.4 Concluding remarks

As said before, the rational method is a very simple method that may be used for small homogeneous urban watersheds (less than 100 to 300 ha) with a simple tree-type sewer system having a free outfall and no special hydraulic structures. If limited to those cases, the rational method will produce fairly good results which are comparable to those produced by much more sophisticated models (Wisner, 1981) and sufficiently accurate for design of small sewer systems.

8.4 SEWER DESIGN USING HYDROGRAPH MODELS

For the peak-flow, single-event methods, such as the rational method, all hydrological and hydraulic processes of the urban water cycle are generalized in synthesized formulations, which have limited application domains. On the other hand urban hydrological models attempt to simulate individual processes of the urban hydrological cycle. These simulations deal with water quantity and/or quality aspects.

From a general point of view, the rainfall-runoff processes may be divided into some principal components that should be properly modelled (see Chapter 5). Such components comprise the following phenomena:

- Design storm modelling,
- Rainfall abstractions,
- Rainfall-runoff modelling at inlets,
- Flow routing through sewer trunks,
- Flow routing through special hydraulic structures such as detention basins, sewer junctions, sewer drops, etc.

While the first three components belong to the hydrological analysis, the last two belong to hydraulic analysis. For each of the above mentioned five principal components, various modelling approaches are possible according to such factors as the design requirements, available data, accuracies required, available computer facilities, and the minimum time and space intervals required.

Usually, a hydrograph model for network design comprises linked submodels, each of which simulates one of the principal process components. Because of various modelling approaches to individual components and various linkages of these components, many urban hydrological models have been developed all over the world during the last 15 years. For such a large number of models, it is not easy to give an objective evaluation. It should be also noted that designers experienced in urban hydrological modelling are capable of developing their own linkages of the existing submodels.

The designer should be aware of the fact that the most sophisticated tool is not always the best tool for his problem and that in newly developed packages, all the submodels should have comparable accuracies. For example, it is counter-productive to route runoff hydrographs through sewers using a highly accurate hydrodynamic model, if such hydrographs were produced from hydrological input data containing very large errors. The accuracy of such simulations should not be judged by the precise method used for the hydrograph routing.

Before describing some existing well-documented simulation models, some remarks on the modelling of the above mentioned principal components are given.

8.4.1 Design storm modelling

This component has been already discussed in Chapter 4 on design parameters. The reader will find in that chapter many good references related to various approaches to the design storm concept. It should be recognized that design storms or actual rainfall events are usually needed in order to produce complete runoff hydrographs at any point in the sewer system and thereby to produce more comprehensive results when simple peak-runoff methods are inadequate (e.g., in a system with storage effects).

The design storm modelling may lead to various submodels according to the phenomenon to be simulated. Three types of such submodels are further discussed below.

Single-event design storms are used for design (or analysis) of storm sewer systems. Synthesized storms or actual events of certain characteristics can serve for this purpose. At the beginning of the project, the appropriate design storm is generally not known and the designer should experiment with various selected storms, in order to study their effects on the whole sewer system. The design storms are usually discretized into very short time intervals (e.g., 5 minutes or even less).

Continuous rainfall data are also used in the planning or analysis of storm sewer systems. Such rainfall series may be observed data generally discretized into fairly long time intervals (e.g., one hour or even longer), or they may be produced by a stochastic model of rainfall time series with time intervals often longer than one day.

When using continuous runoff simulation in conjunction with long rainfall records discretized in short time intervals, it is possible to undertake statistical analysis of simulated runoff parameters, such as runoff peaks and volumes, in order to estimate their frequencies of occurrence independently of the statistical distribution of the rainfall characteristics. Such a procedure represents the best approach but it requires good rainfall data and good computer facilities. At the same time, it is rather time consuming and expensive. For longer return periods, say more than 5 years, the results produced are often close to those obtained with a simple synthetic design storm or an actual design storm.

The rainfall spatial distribution is sometimes simulated for fairly large watersheds, say larger than 1000 to 5000 ha. At present such simulation models are scarce, because they cannot be developed without extensive records from rather dense rain gauge networks. Simpler models use reduction coefficients applied to the point rainfall at the centre of the storm. They are generally presented in the following form:

$$i(d,t) = c(d) \cdot iM(t) \tag{8.9}$$

$$i(A,t) = c(A) \cdot iM(t) \tag{8.10}$$

where d is the distance from the centre of the storm cell to a given point in the watershed, A is the area around the centre of the storm cell, iM(t) is the rainfall intensity at the centre of the storm cell over the duration t, i(A,t) is the mean rainfall depth over the area A, i(d,t) is the rainfall intensity at a distance d from the centre of the storm, and c is a reduction coefficient (see also Figure 4.4 in Chapter 4). Some research is being done to develop more precise forms of Equations 8.9 and 8.10 from radar rainfall data.

8.4.2 Rainfall abstractions

Rainfall abstractions were discussed in Chapter 5 where numerous approaches to quantification of such abstractions were described.

Rainfall abstractions are used to define the net storm hyetographs which serve as inputs to other submodels. It appears that a sophisticated modelling of abstractions is needed only when there are natural pervious areas, producing significant runoff, connected to the sewer system. For such areas, it is difficult to specify values of the probable abstractions under design conditions.

In many densely urbanized areas, the pervious zones are isolated from the sewer system. Simple runoff coefficients (see Section 5.2.3) can then be used to describe abstractions. In catchments with extensive pervious areas, local observations may be needed in order to quantify the abstractions.

8.4.3 Rainfall-runoff modelling

The modelling concepts of rainfall-runoff processes were also discussed in Chapter 5. Many models of the rainfall-runoff processes have been produced using analyses with various detail. For example, some models deal only with overland flow in highly urbanized small catchments (less than one hectare, for example). Other models deal with rainfall-runoff processes in large watersheds (more than 50 hectares, for example). Such runoff models consider implicitly the existing natural or man-made drainage.

The first modelling approach should be quite empirical. In the next stage, a more sophisticated approach, such as hydrological methods, should be used. Many such methods are based on the linear reservoir theory. The simplest model in this category can be expressed as

$$dS(t)/dt = In(t) - Q(t) \tag{8.11}$$

$$S(t) = K \cdot Q(t) \tag{8.12}$$

Equation 8.11 is the continuity equation, in which In(t) is the excess rainfall, Q(t) is the discharge at the catchment outlet, and S(t) is storage on the catchment surface and in its drainage system. Equation 8.12 is the storage equation in which K is the only parameter of the model. The K values can be defined from observations (see Section 5.4.5). For urbanized watersheds researchers have established empirical equations giving design K-values expressed as functions of the catchment and rainfall characteristics (Desbordes, 1978; Rao et al., 1972).

The linear reservoir model is very simple and can be easily operated. Assuming empty sewers at the beginning of the storm, the discretized solution of Equations 8.11 and 8.12 is given by

$$Q(n \cdot dt) = Q((n-1) \cdot dt) \cdot e^{-1/K} + In(n \cdot dt) \cdot (1-e^{-1/K}) \tag{8.13}$$

where K is given in dt units, and dt is the time interval used in computations.

Many other simple models could be also used, such as the generalized rational method in the form of time-area or isochrone line methods, or models combining two well-known processes, storage and translation (see Section 5.4).

8.4.4 Hydraulic routing through sewers and special structures

The hydrographs calculated in the preceding step are then routed through sewers. Again, many routing methods with various accuracy or detail can be used. They were summarized in Section 5.5.

The simplest routing techniques are based on translations of the inlet hydrographs to the next inlet or junction point. The translation velocity can be defined as, for example, the flow velocity corresponding to the mean flow discharge of the total inlet hydrograph, or a mean velocity corresponding to a part of the inlet hydrograph around the peak discharge, or some weighted-average velocity defined as

$$V_{ik} = \Sigma\, Q_m V_m / \Sigma\, Q_m \tag{8.14}$$

where Q_m is the discharge of the m-th discretized part of the hydrograph, V_m is the corresponding flow velocity, and V_{ik} is the weighted translation velocity of the hydrograph between point i and k.

Some other simple techniques are based on storage routing similar to that described above for the rainfall-runoff proccess. In that case, the inflow hydrograph at a given point is considered as the storage input, and the storage output is the hydrograph at the downstream section of the sewer trunk. Linear and non-linear storage equations have been adopted for such flow routing using various approaches. One of the frequently used methods is the so-called Muskingum method described by the following equation:

$$S(t) = K [xQ(t) + (1-x) I(t)] \tag{8.15}$$

where $I(t)$ is the inflow, $Q(t)$ is the outflow, and K and x are parameters. Equation 8.15 is associated with the continuity Equation 8.11. Under specific conditions, numerical integration of the resulting differential equation leads to good results.

The earlier discussed simple hydrograph translation techniques may be used for conditions characterized by relatively steep slopes and absence of significant backwater effects. Storage routing techniques give good results for rather flat sewer systems. For systems with significant backwater effects or in order to get very accurate results, the use of the Saint-Venant hydrodynamic equations may be required (see Section 5.5). However, the reader should be aware of the fact that flow routing by means of complete hydrodynamic equations requires large computer facilities. Moreover, for fast-varying transient flows, even the complete one-dimensional Saint-Venant equation is not always adequate.

For surcharged sewers, the common routing techniques must be modified. A few models simulate such conditions using conceptual approaches with limited experimental support data (Price and Kidd, 1978).

Some models also contain procedures for flow routing through special hydraulic structures, such as overflow structures, detention basins, etc. Most of these procedures represent conceptual approaches involving continuity equations and functional equations of the hydraulic controls and structures. In many cases, specific subroutines must be added to a general package in order to account for hydrograph routing through special hydraulic structures.

8.4.5 Runoff quality modelling aspects

Runoff quality modelling which was briefly discussed in Section 4.7 is a rather new aspect of urban hydrology. It depends greatly on local conditions. Runoff quality models are relatively few and represent highly empirical approaches requiring calibration data for successful application (Torno et al., 1986; Geiger, 1984).

Runoff quality models deal with some selected pollutants, such as suspended solids or biochemical oxygen demand (BOD), and use semi-empirical equations to calculate the rates at which a given pollutant is washed off from the watershed through the sewer network. Sophisticated quality models, some of which are discussed later, comprise a number of submodels. Each submodel simulates a particular component of the total process, e.g., transport of pollutants through sewer networks, accumulation and erosion of pollutants in the watershed, sedimentation in sewers, special processes in local hydraulic structures or treatment facilities, etc.

Great caution is urged when using the existing runoff quality models without local calibration data, because they have been developed for specific land use and drainage practices. Even then, the general modelling concepts may be of some interest to the designer, because he can use them to build his own appropriate models.

8.4.6 Network design using comprehensive models

The preliminary analysis described in Section 8.3.2.3 for the rational method applications must still be done when using comprehensive models. Thus, the drainage area is first subdivided into subareas with homogeneous land use, and the proposed structures or the existing sewer system are drawn in a more or less detailed manner, depending on the design objectives.

According to the requirements of the model used, the designer will first assemble and carefully check all the required data on design rainfall, sewer geometry, hydraulic roughness, runoff coefficients, and rainfall abstraction parameters. Generally the data set will be much larger than that required for the rational method. In many cases, the data required will not be available and the designer will have to use default values given in the user's manual of the chosen model. If these default values seem unsuitable for the design conditions, the designer should test the model sensitivity to these values, using their probable range. If the model is too sensitive to such parameters, it should not be used and the designer should select another model of a similar complexity, or even a simpler one (e.g., as simple as the rational method).

The first review and analysis of the required and available data is very important and should not be attempted without a detailed user's manual. From a general point of view, the designer should not operate any comprehensive model without careful inspection of its detailed user's manual describing, with a sufficient detail, the operations which are performed.

After the preliminary analysis, and according to the design objectives, sewer network complexity, available data, computer facilities, and other factors, the designer should be able to choose an appropriate model from those now available all around the world. Of course, it is not possible to describe here, in detail, all the existing comprehensive models. Consequently, only the best known and publicized ones are mentioned. A distinction should be made between proprietary and non-proprietary models. The former ones are distributed by private firms, usually for a fee, and their descriptions may be less detailed than those of the non-proprietary models distributed by public agencies.

Some comparative studies have been done for the best known runoff models (Brandstetter, 1976). However, the results of such studies become quickly outdated, because many models are being continuously improved. For the four levels of stormwater management problems indicated in Table 8.3, the main characteristics of 38 applicable models are summarized in Table 8.4. Both tables were adopted from a report (U.S. EPA, 1977) giving summarized descriptions of some of the models mentioned in Table 8.4 and about 250 referenced papers dealing with urban hydrology and water management modelling. The reader should find reading this report beneficial even if some information presented is already out of date. Furthermore, some recent models, developed after the publication of the above report, are not included in Table 8.4. Among such models, it is possible to name the Wallingford procedure (National Water Council, 1981; Price and Kidd, 1978) developed in the United Kingdom by the Hydraulics Research Station and the Wallingford Institute of Hydrology, the RERAM model (Desbordes, 1978; Service Technique de l'Urbanisme, 1980) developed in France for the Ministries of Interior and Equipment by the Laboratory of Mathematical Hydrology of the Montpellier University of Science, and the IMPSWMM Urban Drainage Modelling Procedures of the University of Ottawa (Wisner, 1984) developed in Canada. All the above models can be characterized as design/analysis models.

Table 8.3 Levels of analysis for stormwater management (U.S. EPA, 1977)

Analysis Level	Model Type	Model Complexity	Purpose of Model	Model Characteristics
I	Desk-top	Low to medium	Problem assessment, preliminary planning alternative screening.	No computers required. Equations or nomograms based on statistical analyses of many years of records.
II	Continuous simulation	Low to medium	Problem assessment, planning, preliminary sizing of facilities (particularly storage) alternative screening, assessment of long-term impacts of proposed designs.	Programs of a few hundred to a few thousand statements, use many years of rainfall records with hourly time steps, may include flow routing and continuous receiving water analysis.
III	Single event	Medium to high	Analysis for design, detailed planning.	Programs with over 10,000 statements, higher modelling precision, deal with rainfall, sewer flow routing, and possibly the receiving waters. Short time steps and simulation steps, few alternatives are evaluated.
IV	Operational	Medium	Real-time operation of sewerage systems.	Use telemetered rainfall data and feedback from sewer system sensors to make continuously short-term predictions of the system response and control decisions during storms

It should be recognized that even models of high complexity (e.g., SWMM and HSPF) employ, specially in water quality modelling, many empirical relationships which are related to specific land use and types of urban development and may differ from one country to another. So the designer should first carefully study the user's manual before using a given modelling package.

It is not possible to give here detailed information on all the currently used or available models. The reader will find many good papers in the above mentioned references or elsewhere (Geiger, 1984; U.S. Army Corps of Engineers, 1977). Among all the existing models,

Table 8.4 Characteristics of planning, design, and operational models in order of increasing complexity (U.S. EPA, 1977)

Model		Catchment Hydrology						Sewer Hydraulics						Wastewater Quality						Miscel-laneous					
Origin	Acronym	Multiple catchment inflows	Dry-weather flow	Input of several hyetographs	Snowmelt	Runoff from impervious areas	Runoff from pervious areas	Flow routing in sewers	Upstream and downstream flow control	Surcharging and pressure flow	Diversions	Pumping stations	Storage	Dry-weather quality	Stormwater quality	Quality routing	Sedimentation and scour	Quality reactions	Wastewater treatment	Receiving water quality simulation	Receiving water flow simulation	Can choose time interval	Design computations	Applied to real problems	Computer program available
LEVEL 1 (Desk-top)																									
University of Florida	SWMM Level 1		x			x	x						x	x	x				x				x		
URS Research Company	-					x		x							x	x									
EPA-MERL	-		x			x	x						x	x	x			x	x	x	x		x	x	
LEVEL II (Continuous)																									
University of Massachusetts	-	x	x			x		x	x																x
Chicago Sanitary District	FSP	x	x	x	x	x	x	x					x								x	x		x	x
University of Illinois	ISS	x	x		x	x	x	x	x		x	x	x									x	x		x
Metcalf & Eddy	Simplified SWMM	x			x	x	x						x		x				x	x	x	x		x	x
Norwegian Water Research	NIVA	x	x					x	x	x	x	x	x	x	x	x			x			x	x		
Corps of Engineers	STORM				x	x	x				x		x		x			x						x	x
City of Chicago	CHM-RPM	x	x	x		x	x	x		x	x			x	x				x		x	x	x	x	x
Hydrocomp	HSP	x	x	x	x	x	x	x	x		x		x	x	x	x		x		x	x	x		x	
Dorsch Consult	QQS	x	x	x		x	x	x	x	x	x	x	x	x	x	x			x	x	x	x		x	
LEVEL III (Single event)																									
Queen's University	QUURM	x				x	x	x																	
University of Nebraska	HYDRA	x		x		x	x	x														x			
Wilsey and Ham	WH-1	x				x	x	x														x	x	x	
Colorado State University	-	x	x			x		x	x																x
University of Massachusetts	-	x	x			x		x	x																x
University of Cincinnati	UCUR	x				x	x	x													x	x			x
British Road Research Lab.	RRL	x	x	x		x		x													x	x		x	x
Illinois State	ILLUDAS	x	x	x		x	x	x					x									x	x		x
Chicago Sanitary District	FSP	x	x	x	x	x	x	x					x								x	x		x	x
CH2M-Hill	SAM	x	x	x		x	x	x		x	x	x										x			
MIT - Resource Analysis	MITCAT	x	x	x	x	x	x	x		x	x		x								x	x		x	
University of Illinois	ISS	x	x		x	x	x	x	x		x	x	x									x	x		x
Norwegian Water Research	NIVA	x	x					x	x		x	x	x	x	x	x			x			x	x	x	
City of Chicago	RPM	x	x			x	x	x		x	x			x	x				x		x	x	x	x	x
SOGREAH	CAREDAS	x	x			x	x	x	x	x	x	x	x			x				x	x	x		x	
Hydrocomp	HSP	x	x	x	x	x	x	x	x		x		x	x	x			x		x	x	x		x	
Battelle Northwest	BNW	x	x	x		x	x	x		x	x		x	x	x	x			x			x	x	x	x
Watermation	CSM	x	x	x		x	x	x	x	x	x	x	x	x	x	x			x			x	x	x	
EPA	SWMM	x	x	x		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Dorsch Consult	HMV	x	x	x		x	x	x	x	x	x	x	x	x	x	x			x	x	x	x		x	
Water Resources Engineers	STORMSEWER	x	x	x		x	x	x	x	x	x	x	x	x	x	x		x			x	x		x	
LEVEL IV (Operational)																									
Minneapolis - St. Paul	UROM-9	x	x	x		x	x	x			x											x		x	x
Seattle Metro	CATAD	x	x	x		x	x	x	x	x	x	x	x									x		x	x
Battelle Northwest	BNW	x	x	x		x	x	x		x	x		x	x	x	x			x			x	x	x	x
Watermation	CSM	x	x	x		x	x	x	x	x	x	x	x	x	x	x			x			x	x	x	
Hydrocomp	HSP	x	x	x	x	x	x	x	x		x		x	x	x	x		x		x	x	x		x	

x Characteristic included in model.

two are probably best known, because they were among the first models introduced and have been improved continuously by users. The first one is the STORM model (Storage, Treatment, Overflow and Runoff Model) of the U.S. Corps of Engineers (U.S. Army Corps of Engineers, 1977). The original version was completed in January 1973 by Water Resources Engineers, Inc. (Walnut Creek, California). It has since been revised and expanded a number of times. The major processes modelled by STORM are shown in Figure 8.3. It was designed primarily to evaluate stormwater storage and treatment facilities required to abate stormwater and combined sewage overflows. The model continuously simulates the time-varying flow of wastewater and five conservative quality constituents, from many urban and non-urban land uses, within a single catchment, without routing them through the sewer network. Although the model considers the flow diversion to the treatment plant, it does not simulate pollutant removals or associated costs. It is a rather simple tool for planning purposes.

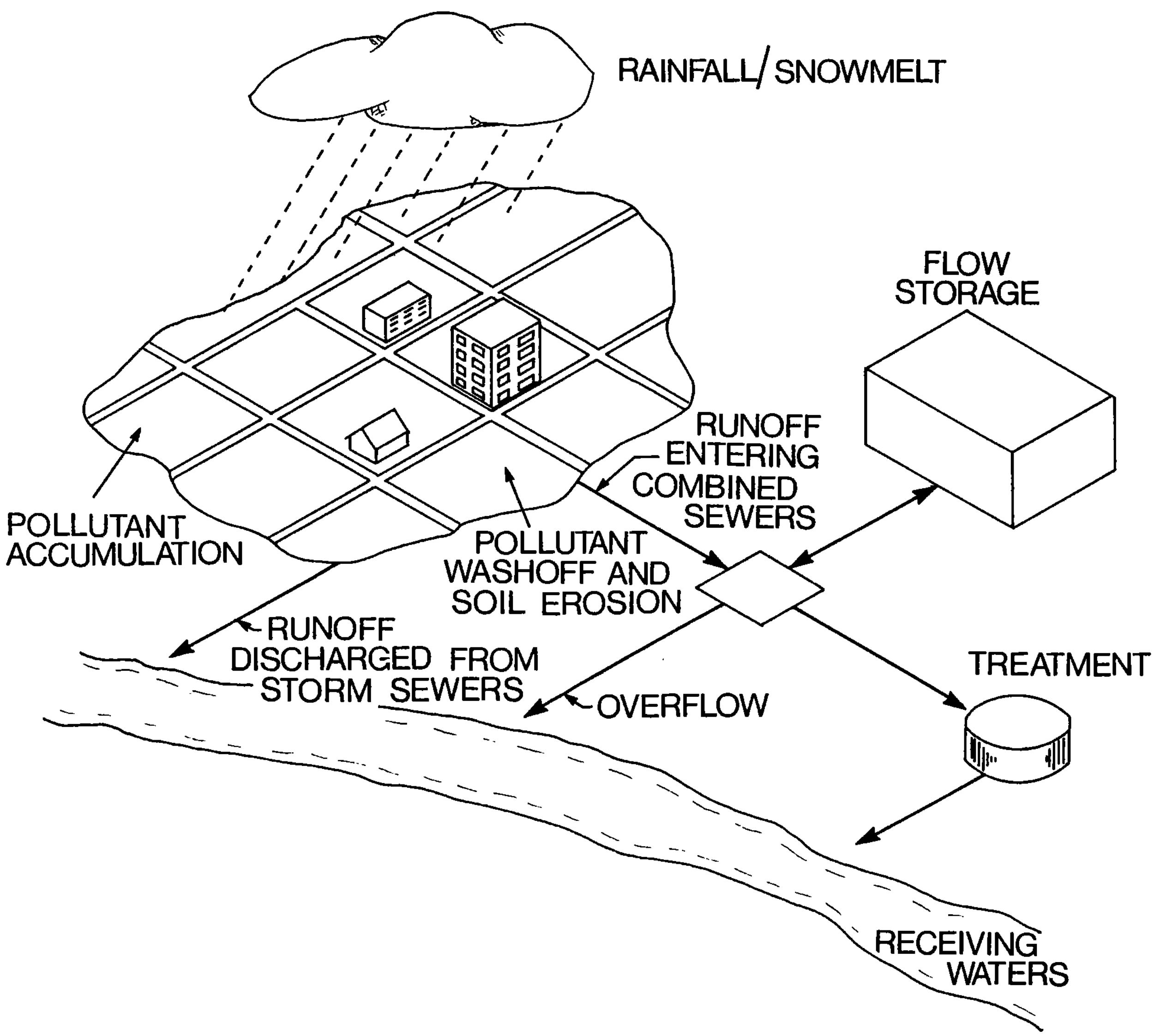


Fig. 8.3 Conceptual representation of the urban system used in the STORM model

The second model is the Storm Water Management Model (SWMM) of the U.S. Environmental Protection Agency. The model was originally developed by Metcalf and Eddy Inc., the University of Florida and Water Resources Engineers in 1971, and it has been subsequently improved by the University of Florida (U.S. EPA, 1977; Huber et al., 1982). A somewhat dated overview of the SWMM structure is given in Figure 8.4 which depicts the Canadian version of SWMM (Environment Canada, 1976).

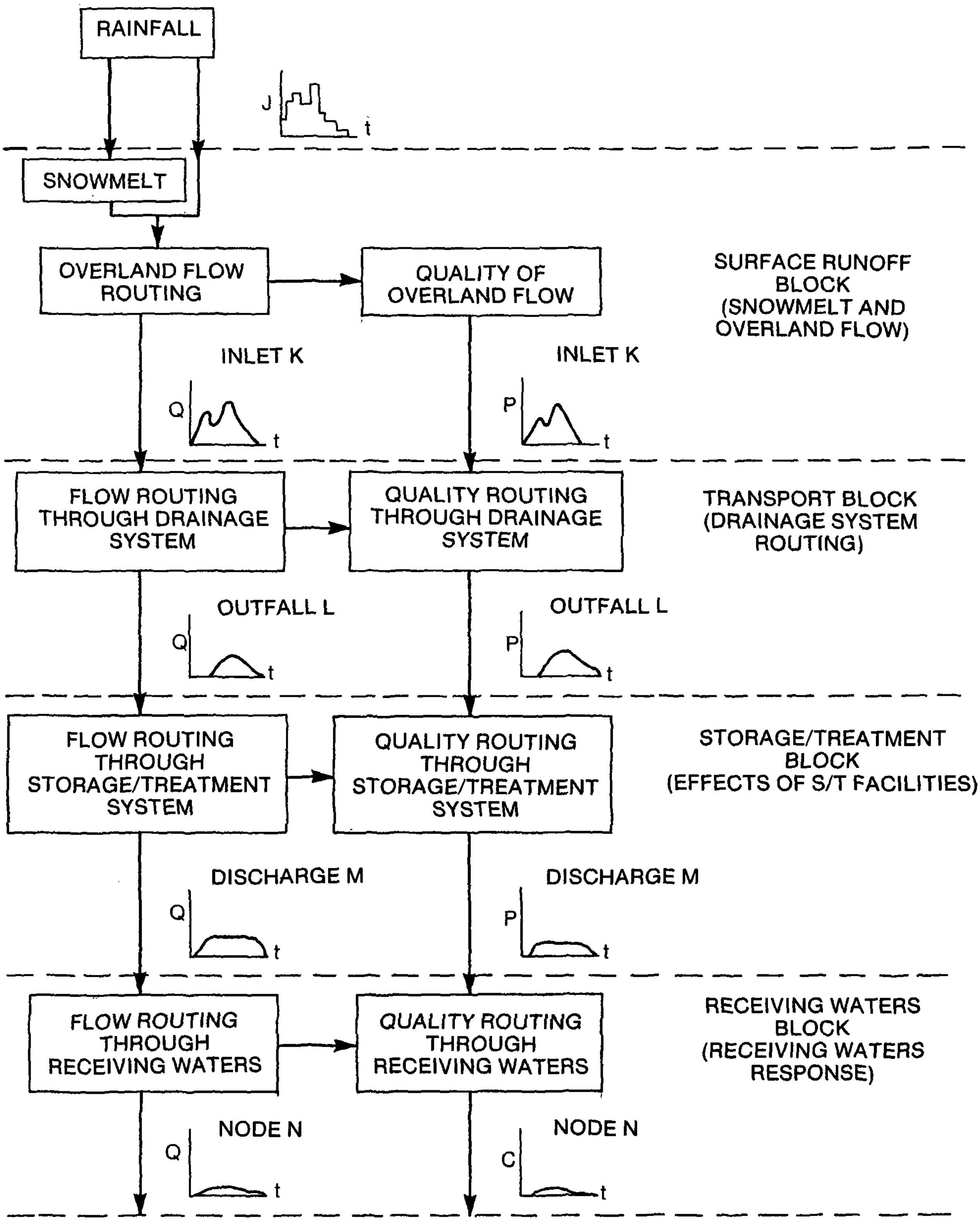


Fig. 8.4 Overview of the Stormwater Management Model (SWMM)

The model program, consisting of more than 12,000 statements, is one of the most detailed and comprehensive mathematical procedures for the simulation of single urban runoff events, storm sewer discharges, and combined sewer overflows. It includes numerous water quality constituents, pumping stations, diversion structures, sedimentation and scour, treatment and storage with associated costs, and two-dimensional receiving water quantity and quality simulations. The data requirements are very large and include data on the catchment physiography, the system structures, information related to the system maintenance, the dry weather flow and receiving waters, and numerous rainfall records, runoff hydrographs and combined flow quality measurements. The most recent SWMM version can be operated in the continuous simulation mode and the model versatility and flexibility was greatly enhanced by letting the user choose from many optional modelling approaches including pressure flow routing (Huber et al., 1982).

Beyond the pioneering aspect of the SWMM, the reader should note that the first version has been continuously improved by the users meeting regularly since 1977. The U.S. EPA has published the proceedings of these meetings, and the designer can find valuable information in such publications (U.S. EPA, 1978-1982). Similar meetings are organized also in other countries (Great Britain, France and Scandinavian countries) with regard to some national models. However, most of the earlier mentioned models are pertinent to the developed countries conditions and should not be exported without adjustments for the conditions encountered in developing countries.

Presently, most of the existing models are still under study and development. For example, some models are being expanded to include continuous simulation and various new water quality submodels. Many research efforts deal with receiving waters quality modelling (Medina, 1979; Torno et al., 1986) which may gain more prominence in the future. It should be recognized, however, that the design of sewer networks by means of comprehensive models, such as those discussed above, reflects socio-economic conditions which vary from one country to another. In developing countries, the socio-economic conditions are probably of even greater importance. They should be first carefully analyzed and the appropriate network design methods should be derived from that analysis. Under such conditions, an appropriate network design model should be preferably established in these countries rather than imported from any developed country without modifications.

9 Organization and administration of urban drainage projects

9.1 DRAINAGE AS PART OF THE URBAN WATER SYSTEM

Good water management in urban areas depends on many aspects including political goals, laws and regulations, water management institutions, experienced staff receptive to innovations and modern techniques of water control, up-to-date knowledge and availability of information required. Urban water management is inter-linked with the development of water resources in the urban area and, consequently, with the planning, in space and time, of facilities of the water system on the whole, including water supply and sanitation services. Therefore, the guidelines for organizational and administrative aspects given in this chapter are not restricted only to urban drainage projects. The case studies and references given deal with institutional requirements of urban water management. They will make the responsible engineer aware of economic, financial, institutional and managerial implications of designing an urban drainage system as part of the urban water system.

9.2 PLANNING AND ORGANIZATIONAL ASPECTS

The set-up of urban drainage projects needs careful preparation, not only in the technical and economic sense, but also concerning the planning and organization of the project during and after its implementation. In many cases, urban drainage systems are interrelated with other components of the water system of the city and its surroundings, such as water supply, sanitation and transport facilities. The following examples illustrate this interdependency.

One of the provincial government policies in Ontario (Canada) is that every municipality is required to develop master drainage plans for all watersheds within the municipal boundaries. It states (Ministry of the Environment and Environment Canada, 1980) that "the purpose of this policy is to foster master drainage planning for rapidly developing municipalities to ensure that stormwater drainage systems are developed in a manner compatible with watershed needs, to identify existing water quality and flooding problems, and to avoid future problems."

In Nigeria, various institutions are engaged in the planning and implementation of sewerage, drainage and waste disposal services. In 1976, a streamlining of the organizational pattern for these services was introduced. In the beginning, this resulted in a division of responsibilities for sewerage, drainage, and waste collection and disposal among the Urban Development Board, the Ministry of Works and Housing and the Municipal Council of the Metropolitan Area, respectively. In 1978, a proposal for integration of these three sectors, sewerage, drainage and waste disposal, was formulated in the Master Plan Report (Kano, 1978). The basic principles for service integration and organization were followed by an action programme of organizational and managerial inputs required for a successful implementation of the Master Plan. Since the above basic principles are of general interest, their modified text is given below.

In the legal, administrative and socio-economic context, the organization and management of sewerage, drainage and waste disposal services can best be met within the framework of a public board or agency. The following principles should be observed in any reform of the administrative structure and functions aimed at the improvement of efficiency and reliability of performance.

- Clear identification and subsequent grouping of required activities (for instance, conservation and cleaning service, operation of sanitary sewers and sewage treatment and disposal facilities, waste collection and disposal) in logical coherent groups of activities and in a functional hierarchy of such groups.

- Drawing up organizational structures, in which the functions of different agencies are clearly defined, the division of work is logical and the lines of authority as well as duties and responsibilities are understood by all concerned.
- Every important function must be clearly attributed to one agency or administration. No single function or two similar functions shall be attributed to two different agencies or administrations.
- Designing a system of coherent objectives and priorities which are periodically reviewed.
- Enhancing adequate internal and external delegation of power, so that timely decisions are made and actions taken at the lowest possible level, where and when needed. Maximum autonomy at least for current tasks (recruitment, training, promotion and payment of personnel, purchase of equipment, commercial activities, land acquisition) should be implemented.
- No division of responsibilities between planning, design and execution of construction works, on one hand, and operation and maintenance, on the other.
- *Combining, to the maximum possible extent, similar activities dealing with sewerage and* water supply in one single organization.
- Choice of a structure which allows for a future streamlining on a nation-wide level.

The following may be defined as the main service functions:
- Preparation and enforcement of sanitary regulations,
- Planning, conceptual layout and design,
- Construction, construction supervision and implementation,
- Financial and commercial management,
- Operation, maintenance and repair, and
- Stock-keeping and materials handling.

For these functions, precise operational procedures with the following major features have to be prepared:
- Adequate cost and funding programme, annual forecasts of income and expenditure on operations and investment, authority and ability to impose and collect fees and charges, from which allocations are made for operation, maintenance and financial reserves;
- Work procedures and forms to enable a better and more equitable performance for quantity and quality of service, programme for effective appraisal of results (audits, inspections, statistical studies and reports), comprehensive record keeping;
- Regular review of inventory levels for materials, regular inventory and re-evaluation of the physical plant.

Services should be available to the greatest possible number of citizens without unbearable financial charges to them. Generation of income from service charges can only be raised step by step to such levels, which will fully cover operation and maintenance costs.

The Master Plan Report recommends that the long-term objective should be to establish a multifunctional unit responsible for sewerage, drainage and waste disposal according to Figure 9.1. The level of autonomy should gradually evolve towards a larger and well-defined independence. The proposed unit should consist of three main functional departments, i.e., Administration, Finance and Accounting, and Engineering Operation, Maintenance and Quality Control, and one staff unit.

Principles of efficient operation favour the establishment of an integrated organizational unit with a certain amount of autonomy. Therefore, collection, treatment and disposal of wastewater (sewerage) and stormwater (drainage) as well as domestic and industrial solid wastes *should ideally be executed by one single authority. Joint responsibility for waste collection* and drainage is particularly important.

Given the scarcity of qualified technical and adminstrative personnel, the organizational unit should be integrated into an existing organization rather than creating a completely new organization.

Good co-ordination of all activities concerning the three sectors and all other urban development activities should be assured via a comprehensive Urban Development Plan and by permanent close contact with urban planning and development activities (Kano, 1978).

The Institution of Water Engineers and Scientists (1983) proposed a tentative outline of the functions in water management organizations (see Figure 9.2) which is similar to the organizational chart given in the Kano Master Plan. Figure 9.2 shows the technical, financial and administrative functions in water supply and sanitation services. For a detailed description of these funtions, reference is made to the above-mentioned report. This report also concluded that in recent years, the implementation of a growing number of large projects has led to increased attention to their management. This is a welcome trend because any large project under implementation puts additional marked strain on the organization and management of the sponsoring agency. Poor project management can greatly add to that strain. At the same

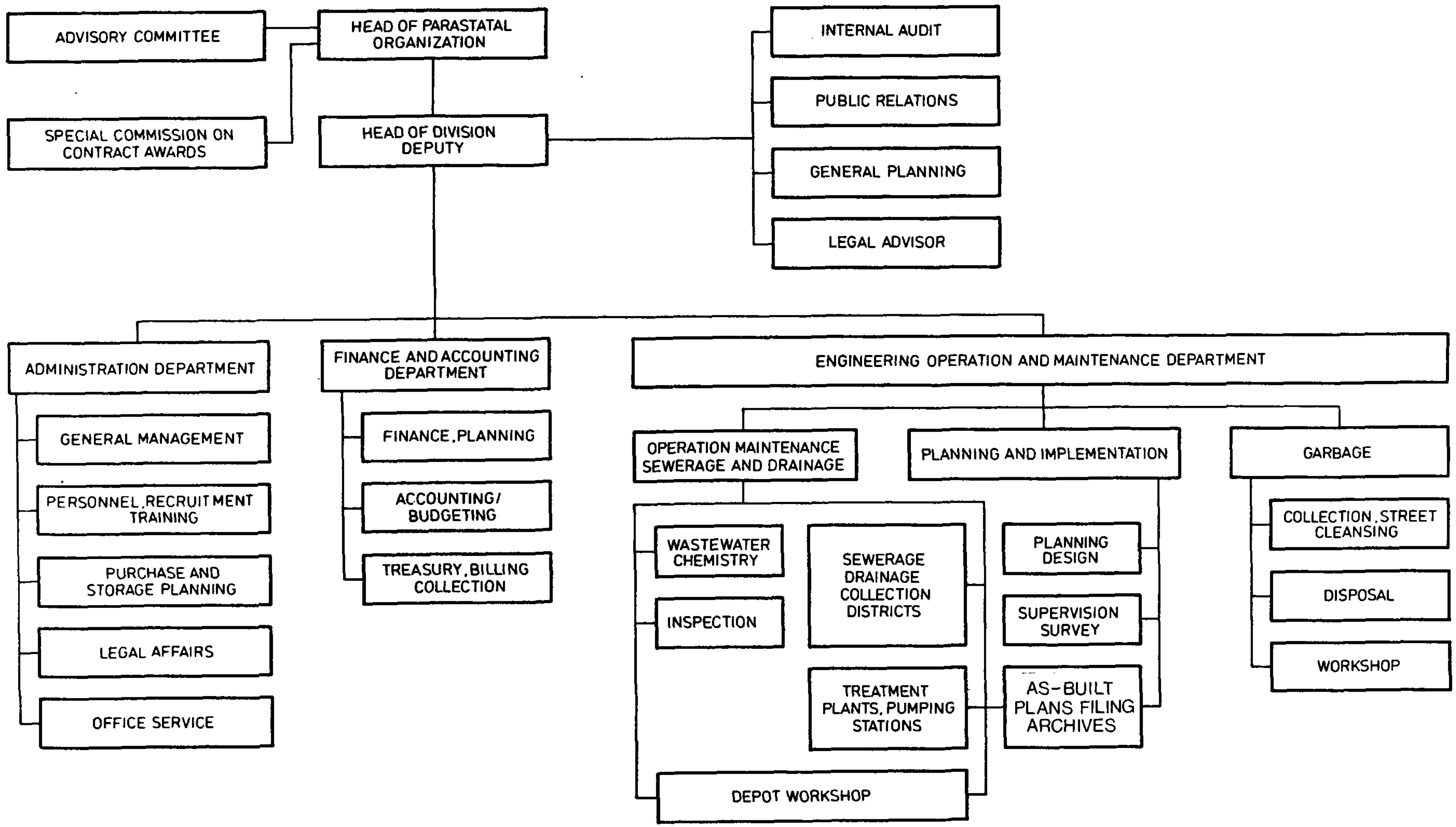


Fig. 9.1. Proposed organizational chart of an unit responsible for sewerage, drainage and waste disposal (Kano, 1978)

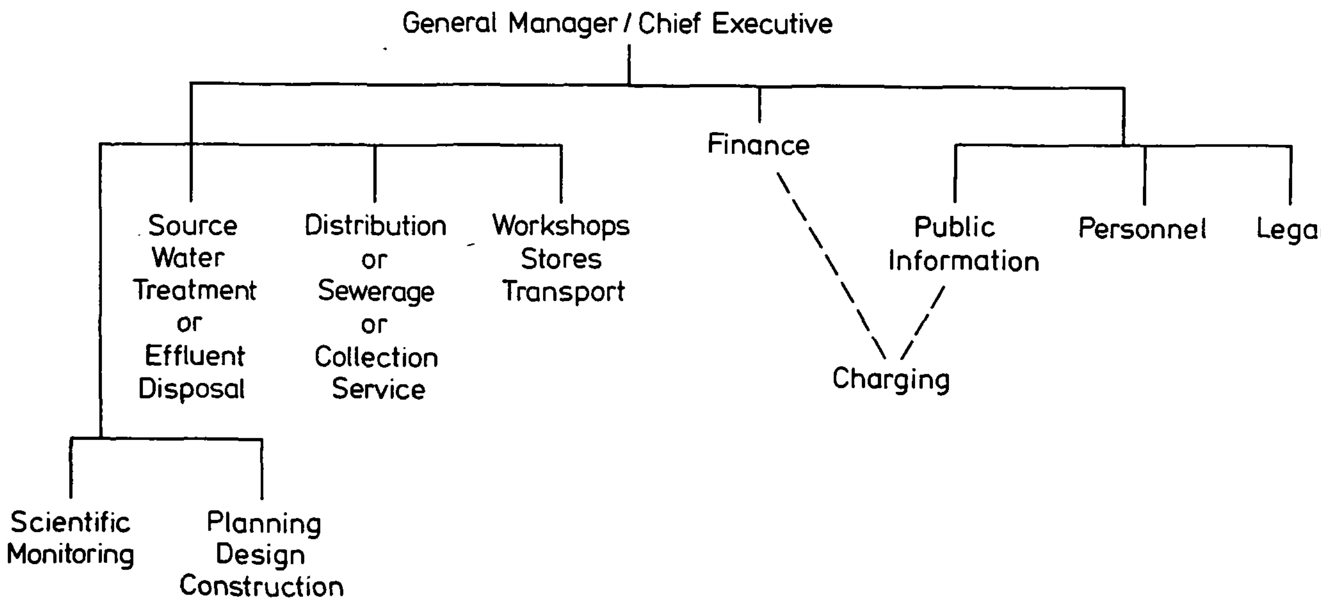


Fig. 9.2 Functions of an urban water management organization (Inst. of Water Engineers and Scientists, 1983)

time, the project produces little or no benefit for the agency until it is substantially or fully completed and provides additional services to water supply or sanitation system users.

Even though the scope of this chapter does not allow detailed discussion of project management, two related points of special importance to the sponsoring agency are made below. These points apply if the main design and construction efforts are provided by outside contractors and consultants.

(1) The project work is liable to disrupt the regular operational work of the agency. The best method of guarding against these effects usually is to separate very clearly executive responsibility for project management from responsibility for the continued management of normal operations. This is in line with the information given in the right-hand part of the chart in Figure 9.1 showing separate blocks for "operation and maintenance" and "planning and implementation", under one leading responsibility for engineering, operation and maintenance, as recommended in The Kano Master Plan Report.

(2) Having pointed out the potential strains of project execution, it is important to emphasize that such strains will not cease as soon as construction is completed.

The planning procedure of water resources management in the Province of Gelderland, the Netherlands, presented in Figure 9.3, shows the interdependency between the conditions of the management plan and the development and formulation of implementation goals and measures, which may lead to the selection of the optimal implementation plan. In this respect, general planning of water management in the Federal Republic of Germany (FRG) is brought to the attention. Already in 1957, the FRG water management law required water management plans for river basins or other territorial jurisdictions of the country. These plans include forecasts of the state of water resources, provisions for flood protection and conservation of water quality. Such planning is clearly linked with the planning of physical facilities (Philippi, 1974; Klosterkemper, 1980).

The connection between the development of urban water resources and research and engineering design is illustrated in Figure 9.4 which also indicates the relation between the decision makers and other agencies (Notodihardjo and Zuidema, 1982).

As the attitude to water problems is changing and the hydrological data needed for reliable water resources planning are to a great extent generated by means of various hydrological models, careful attention has to be given to the specification of required data (Gottschalk, 1982). Examples of such data are given in Table 9.1. Additional information on hydrological

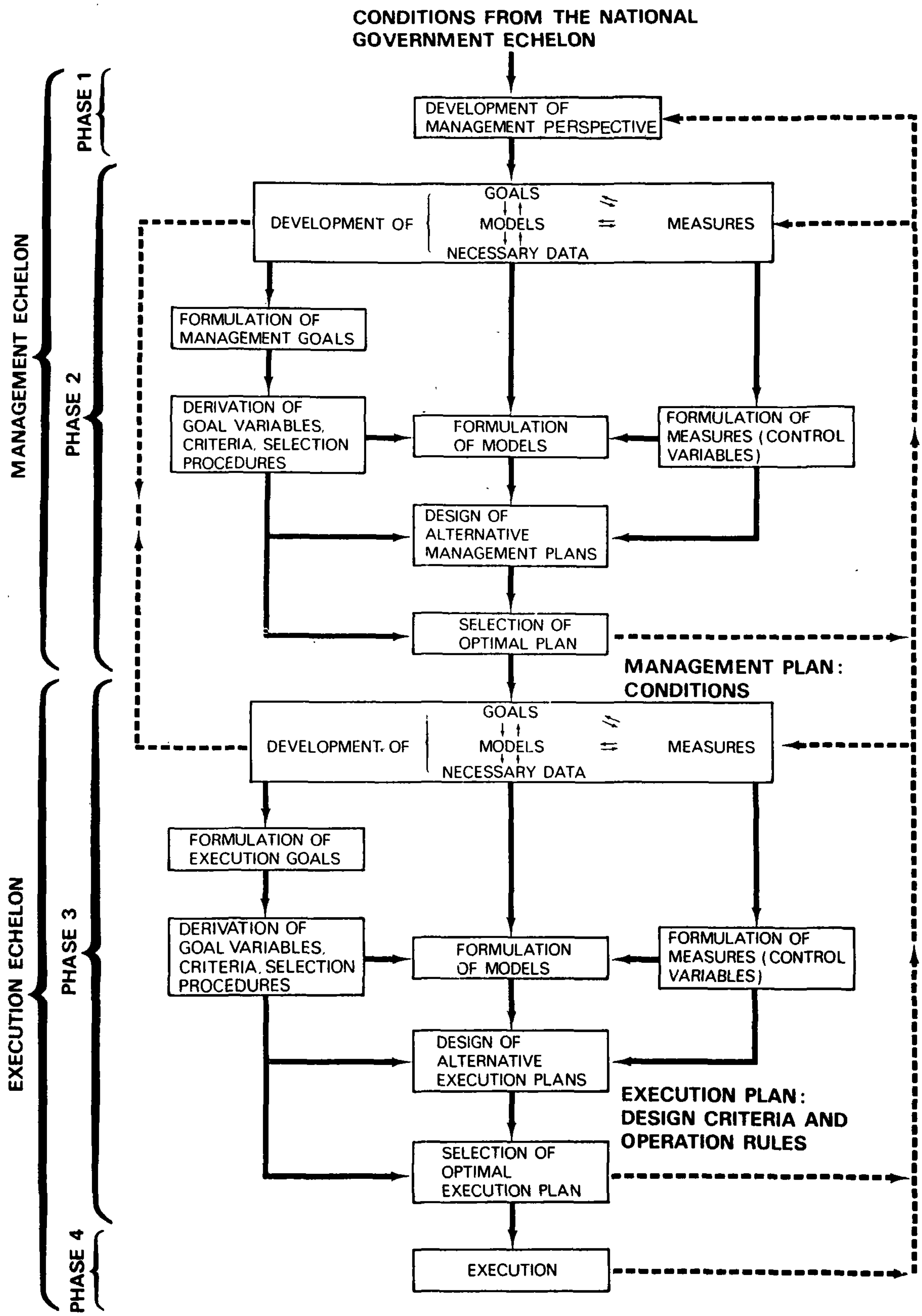


Fig. 9.3 Elaboration of the hierarchical planning procedure of water resources management (Unesco, 1979)

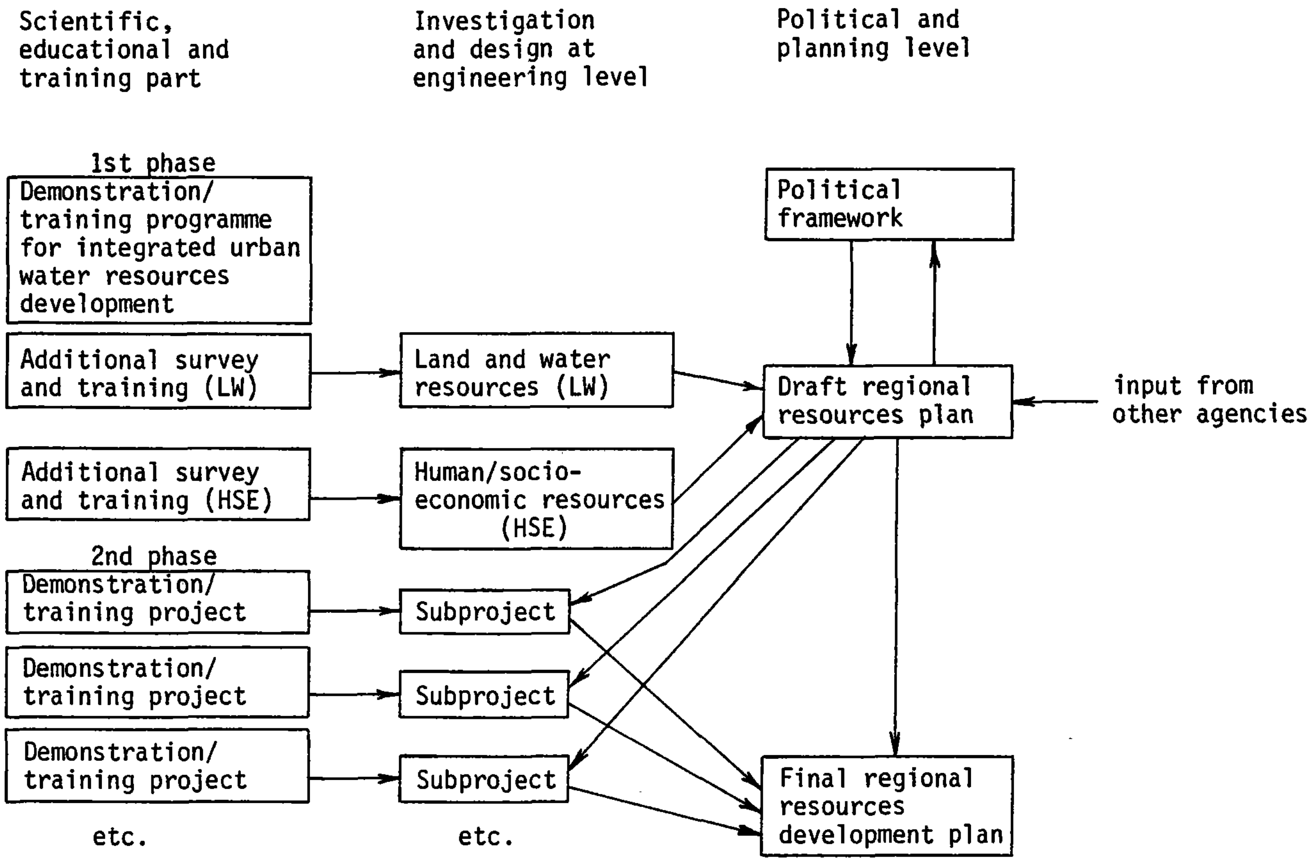


Fig. 9.4 Demonstration/training programme for integrated urban water resources management

and ancillary data was given earlier in Chapter 4 (Design Parameters) and the collection of hydrometeorological and ancillary data is described in Chapters 2 to 4 of Volume II (Unesco, 1987).

Once all the data required have been assembled, it is possible to proceed with actual design. If the considerations are restricted to storm sewers, the design procedure for sewers may be considered as consisting of four distinct steps shown in Figure 9.5 (Hengeveld and de Vocht, 1982). The first step of this procedure specifies essential boundary conditions of the design. This will be further elaborated on in Section 9.3.

It can be concluded that the organization and administration of urban drainage projects cannot be considered apart from regional water management and its legislative aspects and political framework. In such considerations, the watershed is often the preferred regional unit, though the metropolitan area may be the best adminstrative unit, as concluded by McPherson (1978).

9.3 SOCIO-ECONOMIC AND FINANCIAL ASPECTS

Ideally, the choice of an appropriate design frequency, as given in Figure 9.5, and discussed in Chapter 4, should be based on an economic analysis in which all the benefits of the project, including the costs of the damage prevented, are balanced against the costs of construction (Hengeveld and de Vocht, 1982). The project benefits comprise the so-called tangible benefits, such as property damage prevention, which can be readily quantified, and the intangible benefits, such as prevention of general inconvenience, transport delays and injury to health, which are much more difficult to assess (see Table 9.2). Benefits of urban drainage and flood control projects must be measured in terms of objectives of these projects. Traditionally, such objectives are considered to be flood protection, economic objectives other than reduction in property damage, and amenity objectives. The realizations of these objectives are usually measured as shown in Table 9.3 (Grigg, 1977). Not all benefits are financial and recipients of such benefits include property owners, public, developers and business groups.

Table 9.1 Examples of data required for the use of hydrologic models in planning (Gottschalk, 1982)

Surface Water	Meteorology
Stream discharge Lake and reservoir level Water temperature Sediment concentration Chemical and biological properties	Precipitation Temperature Humidity Barometric pressure Radiation Sunshine duration Evaporation Snow depth and water content Interception
Watershed Characteristics	**Soil and Ground Water**
Topography (drainage area and land slopes) Geology Soil type Geomorphology (drainage pattern) Vegetation Land use	Water level Temperature Chemical properties Storage coefficients Permeability Moisture content

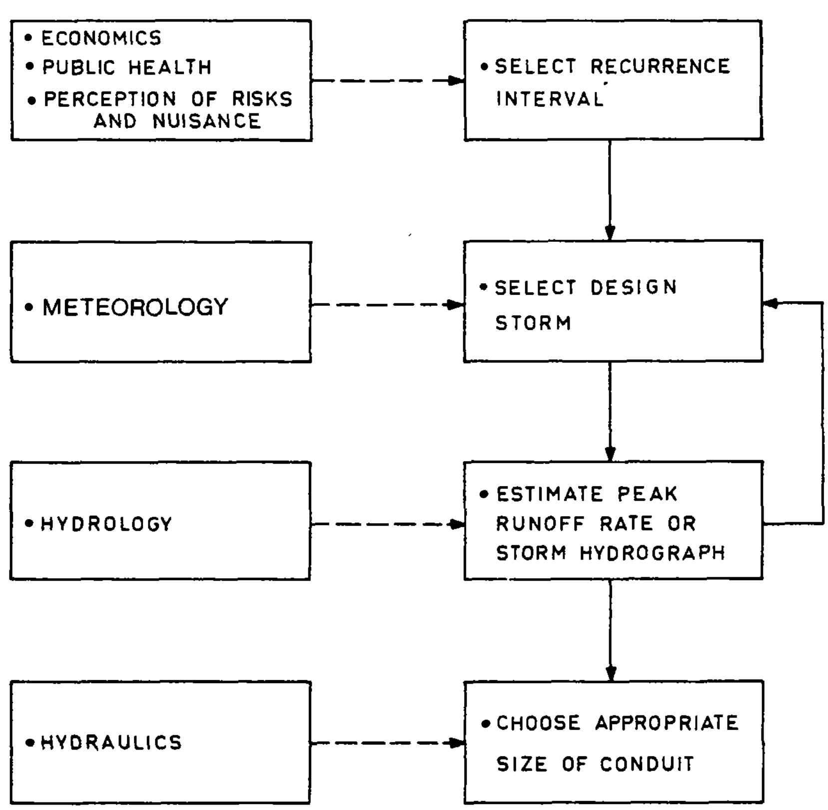


Fig. 9.5 Design procedure for stormwater systems

Table 9.2 Inventory of potential costs and benefits of urban drainage and flood control projects (Grigg, 1977)

Benefits	Costs
Reduced flood damage to public and private facilities	Construction costs
Land value enhancement	Land acquisition costs
Reduced liability to upstream land owners	Costs of non-structural programmes, including food plain zoning
Reduction in traffic delays	Evacuation and emergency programme costs
Reduced income, rental, sales and production losses	Administration costs
Reduced clean-up and maintenance costs	Insurance subsidy costs
Reduced emergency relief costs	Increased reconstruction costs due to the magnitude and extent of flood damage
Increased possiblities for recreation opportunities	Environmental and social costs
Reduced inconvenience	
Increased sense of security	
Alleviation of health hazards	
Improved aesthetic environment	
Reduced risk to life	
Improved water quality	

Table 9.3 Measures of urban drainage and flood control benefits (Grigg, 1977)

Project Objectives	Measures
Protective	
Minimize property damage	Average annual property damage
Eliminate life losses	Expected loss of lives
Alleviation of health hazards	Absence of hazards
Reduction of traffic hazards	Presence (absence) of hazards
Other Economic	
Improve land values	Measured land values
Reduction in maintenance	Expected maintenance budget
Reduction in liability	Presence (absence) of potential liability
Amenity	
Aesthetic improvements	Scale of aesthetic value
Recreational	Quantity of recreational opportunities
Convenience	Travel time, cleaning bills, etc.

Various techniques for evaluation of urban water projects are in use by national and local agencies. Such techniques are not necessarily incompatible among themselves, but they do approach socio-economic considerations from different angles. There is a trend towards the blending of river basin planning and urban planning, which results in confusion regarding the methodology for evaluation and interpretation. National and local authorities must reconcile these two forms of planning, and research in the social sciences is needed to develop improved concepts for precise evaluation.

Large drainage and sewerage schemes should be subject to socio-economic evaluation of both tangible and intangible benefits. Small schemes probably do not justify the effort needed, but should be designed as minimum cost solutions meeting the prescribed standards. The authorities responsible for setting drainage standards should re-examine those standards in the light of recent socio-economic changes in metropolitan regions (Lindh, 1979; Unesco, 1977) and with due attention to the regional context (Lindh, 1985).

Optimization of the layout plans of sewer networks is one field which has been explored so far only to a small extent. Walters (1982) developed a dynamic programming approach as a basis for robust and practicable computer programs allowing the designer to produce networks which are 5% to 15% cheaper than the typical manually designed networks. It is expected that in the near future the availability of low cost computing will make these models more attractive as design tools. A schematic flowchart for the network layout program is given in Figure 9.6.

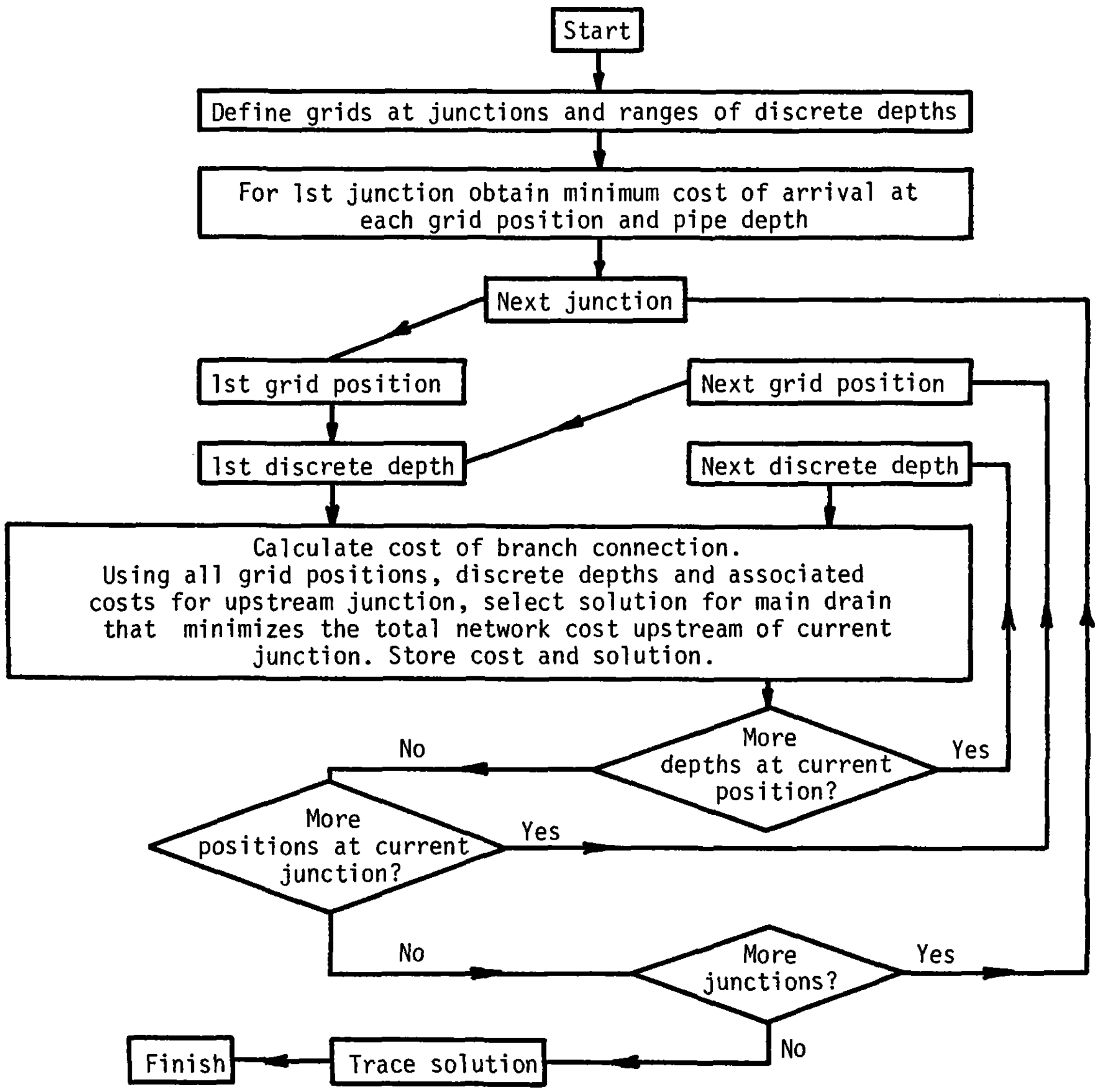


Fig. 9.6 Flowchart of Multicon programme for optimization of sewer network layouts

In the new towns of Lelystad and Almere, the Netherlands, the choice between a combined and a separate sewer system has been based on extensive studies dealing with costs, water quality, implementation and maintenance. The arguments leading to the preference of a separate system were as follows:

- The uniformity of the system for the whole city, which will facilitate good management and control during the implementation of the system, spread over about 20 years, and which will simplify the management after implementation.
- The separate system allows more flexibility than the combined system, during both the design and implementation.
- Lower design costs of the separate system.
- Lower energy-consumption of the separate system.

There were no differences in the total investment costs of the water systems (open waterways, sewers, subsurface drains) employing either sewer system, nor in pollutant loads reaching the receiving water (van Dam et al., 1981; de Jong and Smid, 1981).

During various stages of urban planning in the Netherlands (structure plan, land-use development plan, project plan), an overall cost-benefit analysis is carried out, including the proof of the viability of the development plan and compilation of the budget on the basis of the design. Generally, this requires proper procedures for cost-benefit analysis, budgetting and financial monitoring (van Dort et al., 1977; Unesco, 1979).

When the budget is being compiled, expenditures for site preparation and services, up to but excluding construction, can be subdivided as follows:

- Land acquisition
- Soil improvement and drainage
- Pavement
- Sewer systems
- Parks and recreational facilities
- Street lamps and fire hydrants
- Preparation and supervision
- Bridges, culverts, etc.
- Plan development costs
- Cost of work outside of the plan area
- Financing interest payments

The costs of five urban development plans were examined by van Dort et al. (1977) who reported that the costs of sewer systems amounted to 12% of the total site preparation and services costs. The cost of soil improvement and drainage, sewer systems, and bridges and culverts amounted to 32% of the total site preparation and services costs.

9.4 EFFECTIVE URBAN WATER ORGANIZATIONS

The criteria for effective urban water organizations were studied by Vlachos (1982). His findings are presented below.

Crucial factors for the success of any water resources project include proper water management organization, technological innovation, and efficient allocation and use of available resources. Other considerations, including national growth policies, environmental concerns and emerging policies for conservation of natural resources also call for more comprehensive, holistic or integrated planning.

The positive intangible benefits of water resources to community development, which have always been tacitly recognized, need to be articulated in specific terms. Towards this end, water, as an organizing concept, can play an active role in guiding and stimulating growth, in providing new standards and evaluation criteria, and in strengthening its potential as an additional means for achieving larger social goals. At the same time, water becomes part of a broader policy which recognizes a necessary balancing of three important dimensions (Vlachos, 1982):

- Efficiency, or the growth in material development so that a solid basis of economic sufficiency may be maintained;
- Equity, or fair access of resources and consumption to different segments of the population; and
- Effectiveness, or the overall significance of any project or policy vis-a-vis the pursuing of certain larger social goals.

In recent years, the increasing significance of ecological and social aspects of water resources development contributed to the broadening of their planning space. Diverse questions of social policy and equity in water management have emphasized an increased pre-occupation with expanded time horizon, the search for a higher resolution in any project effects, and multi-disciplinary integration. U.S. experience indicates that four sources of judgement contribute to a better planning effort and assessment of alternatives (Vlachos, 1982):

- Standards and criteria as developed in laws, guidelines and similar documents;
- Interpretations of courts, of elected officials and legal precedents;
- Professional judgements and disciplinary inputs; and
- Public participation and involvement.

It is hoped that in the junction of all these sources a set of criteria would be developed for appropriate or balanced decisions concerning facets of the urban water management system. Such an approach would be technically sound, economically viable, legally pertinent, socially acceptable, and politically feasible. Note that behind these abstract principles is the common sense for incorporating into urban hydrology systems the intangible dimensions of a harmonious collective life. A broader perspective in urban water management will make it possible to account for the variety of environments affected by urban water by considering more disciplines, by incorporating an extended horizon in planning and, by concerning planners with the far-reaching consequences of human actions in the surrounding environment (Vlachos, 1982).

References appearing in the text

Abbot, J. 1978. Testing of several runoff models on an urban watershed. New York, ASCE Urban Water Resources Research Program (Technical Memorandum No. 34).

Ackers, P. 1957. A theoretical consideration of side weirs as stormwater overflows. Proc. Inst. Civ. Engrs., Vol. 6, No. 6, p. 250-269.

Ackers, P. 1969. Charts for the hydraulic design of channels and pipes. Wallingford, Hydraulics Research Station (Hydraulics Research Paper 2).

Ackers, P. 1978. Urban drainage: the effects of sediment on performance and design criteria. Proc. Int. Conf. on Urban Storm Drainage, Southampton, p. 535-545.

Ackers, P.; Birkbeck, A.E.; Brewer, A.I.; Gameson, A.L.H. 1967. Storm overflow studies using crude sewage. Proc. Symp. on Storm Sewage Overflows, Inst. of Civ. Engrs., London, p. 63-78.

Aitken, A.P. 1975. Hydrologic investigation and design of urban stormwater drainage systems. Canberra, Australian Water Resources Council (Tech. Paper No. 10).

Alley, W.M. 1977. Guide for collection, analysis, and use of urban stormwater data. New York, ASCE (A conference report).

American Iron and Steel Institute. 1980. Modern sewer design (Canadian edition). Washington, DC.

APWA. 1981. Urban stormwater management. Chicago (Special Report No. 49).

Arnell, V. 1982. Rainfall data for the design of sewer pipe systems. Goteborg, Chalmers University of Technology (Report Series A:8).

Arnell, V.; Harremoes, P.; Jensen, M.; Johansen, N.B.; Niemczynowicz, J. 1984. Review of rainfall data application for design and analysis. Water Sci. and Tech., Vol. 16, No. 8/9, p. 1-45.

Archer, B.; Bettes, F.; Colyer, P.J. 1978. Head losses and air entrainment at surcharged manholes. Wallingford, Hydraulics Research Station (Report IT 185).

ATV. 1978. Bauwerke der Ortsentwaesserung (Urban drainage structures). St. Augustin, FRG, Gesellschaft zur Foerderung der Abwassertechnik (Arbeitsblatt A 241).

ATV. 1981. Grundlagen der Kanalnetzberechnung-Niederschlagauswertung (Basic sewer network calculations and rainfall analysis, report of the ATV Working Group No. 1.2.1), Korrespondenz Abwasser, No. 10, p. 737-744.

ATV. 1982a. Lehr- und Handbuch der Abwassertechnik (Textbook and handbook of wastewater disposal). Vol. 1: Wasserguetewirtschaftliche Grundlagen, Bemessung und Plannung von Abwasserableitungen (Water quality management, planning and design of urban drainage systems). Berlin-Muenchen, Verlag von Wilhelm Ernst & Sohn.

ATV. 1982b. Lehr- und Handbuch der Abwassertechnik (Textbook and handbook of wastewater disposal). Vol.2: Entwurf und Bau von Kanalisationen und Pumpwerken (Design and construction of sewer systems and pumping stations), Berlin-Muenchen, Verlag von Wilhelm Ernst & Sohn.

ATV. 1983a. Empfehlungen zur Niederschlagsanswertung-Starkregen in Abhaengigkeit von Wiederkehrzeit und Dauer (Guidelines for rainfall analysis - heavy rains, their frequency and duration). St. Augustin, FRG, Gesellschaft zur Foerderung der Abwassertechnik (Arbeitsblatt A 121).

ATV. 1983b. Richtlinien fuer die Bemessung und Gestaltung von Regenentlastungen in Mischwasser Kanaelen (Guidelines for planning and design of combined sewer overflows). St. Augustin, FRG, Gesellschaft zur Foerderung der Abwassertechnik (Arbeitsblatt A128, 4th edition).

Balmforth, D.J. 1982. Improving the performance of stilling pond storm sewage overflows. Proc. First Int. Sem. on Urban Drainage Systems, University of Southampton, Southampton.

Barr, D.I.H. 1975. Two additional methods of direct solution of the Colebrook-White function, Proc. Inst. Civ. Engrs., Vol. 59, Part 2, p. 827-835.

Barr, D.I.H. 1978. A new approach to fully graphical (dimensional) solution of the Colebrook-White function. J. Inst. of Water Engrs. and Scientists, Vol. 32, No. 1, p. 19-30.

Bischofsberger, W. 1982. Wasserguetewirtschaft und Gesundheitsingenieurwesen-Skriptum (Water resources management and sanitary engineering - lecture notes). Muenchen, Lehrstuhl fuer Wasserguetewirtschaft und Gesundheitsingenieurwesen der Technischen Universitaet Muenchen.

Blau, E. 1981. Ein Beitrag zur Versickerung des von Dachflaechen abfliessenden Niederschlags (A contribution to infiltration of roof runoff). Korrespondenz Abwasser, Vol. 28, No. 10, p. 716.

Boorman, D.B.; Reed, D.W. 1981. Derivation of a catchment average unit hydrograph. Wallingford, Institute of Hydrology (Report No. 71).

Brandstetter, A. 1976. Assessment of mathematical models for storm and combined sewer management. Cincinnati, Ohio, U.S. EPA (Report EPA-600/2-76-175a).

Brandstetter, A.; Field, R.; Torno, H.C. 1976. Evaluation of mathematical models for the simulation of time-varying runoff and water quality in storm and combined sewerage systems. Proc. Conf. on Environmental Simulation, Washington, DC, U.S. EPA (Report EPA-600/9-76-016), p. 548-552.

Brater, E.F.; King, H.W. 1976. Handbook of hydraulics, New York, McGtaw-Hill Book Company.

Braun, M. 1959. Beitrag zur Beurteilung und Ermittlung von Oberflaechenabfluessen auf unbebauten Siedlungsflaechen. Braunschweig, FRG, Braunschweig Technical University (Dissertation).

Brunner, P.G. 1975. Die Verschmutzung des Regenwasserabflusses im Trennverfahren (Stormwater pollution in storm sewer systems). Muenchen, Technische Universitaet Muenchen (Berichte aus Wasserguetewirtschaft und Gesundheitsingeneurwesen, No. 9).

Camp, T.R. 1946. Design of sewers to facilitate flow. Sewage Works J., Vol. 18, No. 3.

Chow, V.T. 1959. Open-channel hydraulics. New York, McGraw-Hill Book Company.

Chow, V.T. 1964. Handbook of applied hydrology. New York, McGraw-Hill Book Company.

Chow, V.T.; Yen, B.C. 1975. Urban stormwater runoff - determination of volumes and flow-rates. Washington, DC, U.S. EPA (Report No. EPA-600/2-76-116).

CIEH. 1984. Conception generale des systèmes d'assainissement urbain dans le contexte africain, Aspect technique. Ouagadougou, Burkina Fasso.

Colyer, P.J. 1977. Performance of storm drainage simulation models. Proc. Inst. Civ. Engrs., Vol. 63, Part 2, p. 293-309.

Colyer, P. 1982. Money down the drain. Water, No. 1.

Cunge, J.A. 1961. Étude d'un schèma de différences finies appliqué à l'intégration numèrique d'un certain type d'équation hyperbolique d'écoulement. Grenoble, Facutly of Science of Grenoble University (Thesis).

Cvjetanovic, B. 1971. Sanitation versus vaccination. In: Strategy of cholera control. Geneva, WHO (Paper BD/Cholera 171.5).

Dallaire, G. 1976. Controlling erosion and sedimentation at construction sites. Civil Engineering, Vol. 47, No. 10, p. 73-77.

de Jong, J.; Smid, J.E. 1981. Choice of the drainage system in the new town of Almere-Buiten (in Dutch) (Rioolstelselkeuze Almere-Buiten). H_2O, Vol 14.

Desbordes, M. 1978. Urban runoff and design storm modelling. Proc. Int. Conf. on Urban Storm Drainage, Southampton, p. 353-361.

Desbordes, M.; Masson, J.M.; Raous, P.; Trevisiol, Y. 1981. Definition de la pluie de projet de la région I - Paris - Monsouris. Montpellier, Universite des sciences et techniques du Languedoc (Note LHM 12/81).

Dick, T.M.; Marsalek, J. 1979. Importance des charges unitaires de polluants dans le ruissellement urbain d'eaux pluviales. Eau du Québec, Vol. 12, No. 4, p. 262-267.

Diniz, E. 1980. Porous pavement: phase I. Design and operational criteria. Washington, DC, U.S. EPA (Report No. EPA-600/2-80-135).

Dronkers, J.J. 1969. Tidal computations for rivers, coastal areas, and seas. J. Hyd. Div., ASCE, Vol. 95, No. HY1, p. 29-77.

Dumitrescu, D. 1970. Manualul inginerului hidrotechnician (Hydraulic engineering handbook). Bucuresti, Editura Technica.

Dvorak J. 1959. Bestimmung des Oberflaechenabflusses von geneigten Bodenflaechen. Deutsche Gewasserkundliche Mitteilungen, Vol. 3, No. 4, p. 74.

Dyck, S. 1980. Angewandte Hydrologie (Applied hydrology). Muenchen, Verlag von Wilhelm Ernst & Sohn.

Ellis, J.B. 1986. Pollutional aspects of urban runoff. In: Torno, H.C.; Marsalek, J.; Desbordes, M. (eds.). 1986. Urban Runoff Pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10, p. 1-38.

Environment Canada. 1976. Storm water management model study. Vol. I, Ottawa, Environment Canada (COA Research Report No. 47).

Evans, J.G. 1979. Drainage problems of a South Pacific Island. Proc. WED Conf., Loughborough, England.

Fair, G.M.; Geyer, G.C.; Okun, D.A. 1966. Water supply and wastewater removal. New York, John Wiley & Sons.

Field, R. 1986. Urban stormwater runoff quality management: low-structurally intensive measures and treatment. In: Torno, H.C.; Marsalek, J.; Desbordes, M. (eds.). 1986. Urban runoff pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10, p. 677-700.

Frederick, M.R.; Markland, E. 1967. The performance of stilling ponds in handling solids. Proc. Symp. on Sewage Overflows, Inst. of Civ. Engrs., London, p. 51-62.

Fruehling, A. 1902. Die Entwaesserung der Staedte: Handbuch der Ingenieurwissenschaften (Drainage of cities: engineering handbook). Leipzig, 4th edition.

Gardener, W.; Widstoe, J.A. 1921. The movement of soil moisture. Soil Science, Vol. II, p. 215-232.

Geiger, W.F. 1983. Progress in urban hydrology in the Federal Republic of Germany since 1979. Proc. Int. Symp. on Urban Hydrology, New York, ASCE, p. 54-78.

Geiger, W.F. 1984. Combined sewage quantity and quality - a contribution to urban drainage planning (in German). Muenchen, Technische Universitat Muenchen (Berichte aus Wassergewirtschaft und Gesundheitsingenieurwesen).

Geiger, W.F.; Dorsch, H.R. 1980. Quantity-quality simulation (QQS): a detailed continuous planning model for urban runoff control. Vol. I, Model description, testing and application. Washington, DC, U.S. EPA (Report EPA-600/2-80-011).

Geiger, W.F.; La Bella, S.A.; McDonald, G.C. 1976. Overflow abatement alternatives selected by combining continuous and single event simulations. Proc. Nat. Symp. on Urban Hydrology, Hydraulics and Sediment Control, Lexington, Kentucky, University of Kentucky, p. 71-79.

Goforth, G.F. 1983. Stormwater hydrological characteristics of porous conventional paving system. Washington, DC, U.S. EPA (Report No. EPA 600-600/2-83-106).

Gottschalk, L. 1982. Data sources in water management. In: Water Resources Engineering Studies, dedicated to Gunnar Lindh. Lund, Sweden, Department of Water Resources Engineering, University of Lund (Report 3061).

Green, W.H.; Ampt, G. 1911. Studies of soil physics I. The flow of air and water through soil. J. Agri. Sci., Vol. 4, p. 1-24.

Grigg, N.L. 1977. Benefits and costs of urban drainage and flood control projects. In: Effects of urbanization and industrialization on the hydrological regime and on water quality. Proc. of IAHS-Unesco Amsterdam Symp., IAHS-AISH Publication No. 123, p. 558-564.

Gumbel, E.J. 1958. Statistics of extreme values. New York, Columbia University Press.

Hahn, H.H. 1982. Die spezifischen Aufgaben des Ingenieurs der Siedlungswasserwirtschaft in Entwicklungslaendern. Eine Einfuehrung. In: Water Supply and Wastewater Treatment, Hahn, H.H. (ed.), Karlsruhe, Institut fuer Siedlungswasserwirtschaft, Universitat Karlsruhe, p. 1-18.

Heaney, J.P.; Huber, W.C.; Medina, M.A.; Murphy, M.P.; Nix, S.J.; Hasan, S.M. 1977. Nationwide evaluation of combined sewer overflows and urban stormwater discharges. Vol. II, Cost Assessment and Impacts. Washington, DC, U.S. EPA (Report EPA-600/2-77-064).

Heeps, D.P.; Mein, R.G. 1973. An independent evaluation of three urban stormwater models. Clayton, Australia, Monash University (Civ. Eng. res. report No. 4).

Helliwell, P.R.; Kidd, C.H.R.; Lowing, M.J. 1976. Estimation of the above-ground runoff hydrograph for storm sewer design purposes. Proc. Nat. Symp. on Urban Hydrology, Hydraulics and Sediment Control, Lexington, Kentucky, University of Kentucky, p. 81-89.

Henderson, F.M. 1966. Open channel flow. New York, The Macmillan Co.

Hengeveld, H.; de Vocht, C. 1982. Role of water in urban ecology. Urban Ecology, Vol. 6, Nos. 1-4.

Hicks, W.J. 1944. A method of computing runoff. Trans Am. Soc. Civ. Eng., Vol. 109, Paper No. 2230, p. 1217.

Hillel, D. 1971. Soil and water: physical principles and processes. New York, Academic Press.

Holtan, H.N.; England, C.B.; Allen, W.H. 1967. Hydrologic capacities of soils in watershed engineering. Proc. Int. Hydrology Symp., Vo. 1, Fort Collins, Colorado, p. 218-226.

Horton, R.E. 1939. Analysis of runoff - plot experiments with varying infiltration capacity. Trans. American Geophysical Union, Vol. 20, p. 693.

Huber, W. 1986. Deterministic modeling of urban runoff quality. In: Torno, H.C.; Marsalek, J; Desbordes, M. (eds.). 1986. Urban runoff pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10, p. 167-242.

Huber, W.C.; Heaney, J.P.; Nix, S.J.; Dickinson, R.E.; Polman, D.J. 1982. Stormwater management model, users manual, version III. Cincinnati, U.S. EPA (Project No. CR-805664).

Huff, F.A. 1967. Time distribtuion of rainfall in heavy storms. Water Resources Res., Vol. 3, No. 4, p. 1007-1019.

Hydrotek, Inc. 1985. Urban design storms for Canada. Toronto, Atmospheric Environment Service, Environment Canada.

Imhoff, K.; Imhoff, K.R. 1985. Taschenbuch der Stadtentwaesserung (Handbook of urban drainage, 26th edition). Muenchen, Oldenbourg-Verlag.

Institution of Water Engineers and Scientists. 1983. Water supply and sanitation in developing countries. London, 31-33 High Holborn.

James, L.D. 1965. Using a computer to estimate effects of urban development on flood peaks. Water Resources Research, Vol. 1, Part 2, p. 223.

Jensen, M. 1982. Hydraulisk beregming af overlob med stort kammer (Hydraulic calculations of overflows with storage chambers). Lyngby, Dept. of San. Engng., Technical University of Denmark (Report LtH 82-01).

Kalbermatten, J.M.; DeAnne, S.J.; Gunnerson, C.G.; Mara, D.D. 1982. Appropriate sanitation alternatives - a planning and design manual. Baltimore and London, The Johns Hopkins University Press (World Bank studies in water supply and sanitation #2).

Kano. 1978. Kano sewerage and drainage project. Master Plan Report, Nigeria.

Keifer, C.J.; Chu, H.H. 1957. Synthetic storm pattern for drainage design. J. of Hyd. Div., ASCE, Vol. 83, No. HY4, p. 1332-1-1332-25.

Keller, F.J. 1962. Effect of urban growth on sediment discharge, Northwest Branch Anacostia River Basin, Maryland. U.S. Geological Survey, Short papers in geology and hydrology (Professional Paper No. 450-C).

Kibler, D.F. 1982. Urban stormwater hydrology. Washington, DC, U.S. AGU (Monograph No. 7).

Kibler, D.F.; Aron, G. 1978. Effects of parameter sensitivity and model structure in urban runoff simulation. Proc. Int. Symp. on Urban Hydrology, Hydraulics and Sediment Control, Lexington, Kentucky, University of Kentucky.

Kidd, C.H.R. 1978. Rainfall-runoff processes over urban surfaces. Proc. of Int. Workshop held at the Institute of Hydrology, Wallingford, U.K.

Kidd, C.H.R.; Packman, J.C. 1980. Selection of design storm and antecedent condition for urban drainage design. Wallingford, U.K., Institute of Hydrology (Report No. 61).

Kirschmer, O. 1966. Tabellen zur Berechnung von Entwasserungsleitungen nach Prandtl-Colebrook (Tables for sewer sizing according to Prandtl-Colebrook). Heidelberg, FRG, Strassenbau, Chemie und Technik Verlagsgesellschaft mb.H.

Kittredge, J. 1948. Forest influences. New York, McGraw-Hill Book Company.

Klosterkemper, H. 1980. Die Organisation der Wasserwirtschaftsverwaltung in Nordrhein-Westfalen (Organization of water resources management in North-Rhine Westphalia). In: Wasserwirtschaft in Nordrhein-Westfalen, Muenchen, Verwaltungs-Verlag-Gmbh & Co. K.G.

Kourany, M.; Vasquez, M. 1969. Amer. J. Trop. Med. and Hyg., Vol. 18, No. 6, p. 936-941.

Krauth, K.H. 1979. Der Regenabfluss und seine Behandlung beim Mischverfahren (Stormwater runoff and its treatment in combined sewer systems). Muenchen, Oldenbourg-Verlag (Stutgarter Berichte zur Siedlungswasserwirtschaft, Report No. 66).

KWK; ATV. 1977. Vergleichende Untersuchungen von elektronischen Kanalnetzberechnungen (Comparison of computerized sewer network calculation methods). Hamburg, Verlag Paul Parey (Schriftenreihe des Kuratoriums fuer Wasser und Kulturbanwesen No. 31).

Lager, J.A.; Smith, W.G. 1974. Urban stormwater management and technology: an assessment, Cincinnati, Ohio, U.S. EPA (Report No. EPA-670/2-74-040).

Lager, J.A.; Smith, W.G.; Tchobanoglous, G. 1976. Catchbasin technology overview and assessment. Cincinnati, Ohio, U.S. EPA (Contract No. 68-03-0274).

Lautrich, R. 1980. Der Abwasserkanal, Handbuch fuer Plannung, Ausfuehrung und Betrieb (Handbook for planning, construction and operation of sewers). Hamburg, Verlag Paul Parey (4th edition).

Li, W.H. 1974. Direct determination of pipe size. Civil Engineering (ASCE), Vol. 44, No. 6.

Lindh, G. 1979. Socio-economic aspects of urban hydrology, Paris, The Unesco Press (Studies and reports in hydrology No. 27).

Lindh, G. 1985. The planning and management of water resources in metropolitan regions. Lund, Sweden, University of Lund (A paper prepared for IASH Symposium in Sao Paolo on Water resources management in urban areas).

Linsley, R.K.; Kohler, M.A.; Paulhus, J.L.H. 1949. Applied hydrology. New York, McGraw-Hill.

MacLaren, J.F. Ltd. 1975. Review of Canadian design practice and comparison of urban hydrologic models. Ottawa, Environment Canada (COA research report No. 26).

Malmqvist, P.A. 1983. Urban stormwater pollutant sources. Goteborg, Dept. of San. Engng., Chalmers University of Technology.

Mara, D. 1982. Appropriate technology for water supply and sanitation. Sanitation alternatives for low-income communities - a brief introduction. New York, The World Bank.

Marsalek, J. 1978. Research on the design storm concept. New York, ASCE Urban Water Resources Research Program (Techn. Memorandum No. 33).

Marsalek, J. 1982. Road and bridge deck drainage systems. Toronto, Ministry of Transportation and Communications (Report No. 228).

Marsalek, J. 1984. Caractérisation du ruissellement de surface issu d'une zone urbaine commerciale. Sciences et Techniques de l'Eau, Vol. 17, No. 2, p. 163-167.

Marsalek, J. 1985. Head losses at selected sewer manholes. Chicago, APWA (Special Report No. 52).

Marsalek, J. 1986. Toxic contaminants in urban runoff: a case study. In: Torno, H.C.; Marsalek, J.; Desbordes, M. 1986. Urban runoff pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10, p. 39-58.

Marsalek, J.; Dick, T.M.; Wisner, P.E.; Clarke, W. 1975. Comparative evaluation of three urban runoff models. Water Resources Bull., Vol. 11, No. 2, p. 306-328.

Marsalek, J.; Rossmiller, R.L.; Urbonas, B.; Wenzel, H.G. 1983. Annotated bibliography on urban design storms. New York, ASCE Urban Water Resources Research Program.

Marsalek, J.; Watt, W.E. 1984. Design storms for urban drainage design. Canadian J. of Civ. Engng., Vol. 11, No. 3, p. 574-584.

McCuen, R.H. 1982. A guide to hydrologic analysis using the SCS methods. Englewood Cliffs, N.J., Prentice Hall, Inc.

McCuen, R.H.; Wong, S.L.; Rawls, W.J. 1984. Estimating urban time of concentration. J. of Hyd. Engng., ASCE, Vol. 110, No. 7, p. 887-904.

McPherson, M.B. 1969. Some notes on the rational method of storm drainage design. New York, ASCE Urban Water Resources Research Program (Tech. Memorandum No. 6).

McPherson, M.B. 1975. Special characteristics of urban hydrology. In: Prediction in catchment hydrology, Canberra, Australian Academy of Science, p. 239-255.

McPherson, M.B. 1978. Urban runoff control planning. Washington, DC, U.S. EPA (Miscellaneous Report Series No. EPA-600/9-78-035).

McPherson, M.B. 1979. International symposium on urban hydrology. New York, ASCE Urban Water Resources Research Program (Tech. Memorandum No. 38).

Medina, M.A. 1979. Level III receiving water quality modeling for urban stormwater management. Cincinnati, Ohio, U.S. EPA (Report No. EPA-600/2-79-100).

Melanen, M. 1981. Quality of runoff water in urban areas. Helsinki, Water Research Institute, National Board of Waters (Publication No. 42), p. 123-190.

Metcalf and Eddy, Inc.; University of Florida; Water Resources Engineers. 1971. Environmental Protection Agency stormwater management model. Washington, DC, U.S. EPA (Report No. 11024).

Miller, D.S. 1971. Internal flow: a guide to losses in pipe and duct systems. Cranfield, U.K., The British Hydromechanics Association.

Ministere de l'Interieur. 1977. Instruction technique relative aux réseaux d'assainissement des agglomérations. Paris, Imprimerie Nationale (Circulaire n° 77284 INT).

Ministerie van verkeer en waterstaat. 1985. Alternatieven voor de waterbeheersing in stedelijke gebieden (Alternatives for water control in urban areas; in Dutch). Lelystad Rijksdienst voor de ijsselmeerpolders (Report 38 abw/cdc).

Ministry of the Environment and Environment Canada. 1980. Proposed model policies for urban drainage management. Ottawa, Environment Canada (COA Research Report No. 102).

Ministry of Transportation and Communications. 1982. Drainage manual. Toronto, Ministry of Transportation and Communications.

Moore, M.A.; De la Cruz, E.; Vargas-Mendez, E. 1965. Amer. J. Epidemiology, Vol. 82, No. 2, p. 162-184.

Munz, W. 1966. Die Wirkung verschiedener Gewaesserschutzmassnahmen auf den Vorfluter. Zeitschrift fuer Hydrologie, Basel, Birkhaeuser-Verlag.

Musgrave, G.W. 1946. Report of committee on infiltration. Trans. Am. Geoph. Union, Vol. 27, No. 5, p. 726.

Nash, J.E. 1957. The form of the instantaneous unit hydrograph Int. Assoc. on Scientific Hydrology, Publ. 45, Vol. 3, p. 114-121.

National Water Council. 1981. Design and analysis of urban storm drainage: the Wallingford procedure. Wallingford, U.K., Hydraulic Research Station.

Natural Environment Research Council. 1975. Flood studies report (four volumes). London, Natural Environment Research Council.

Neal, J.H. 1938. The effect of the degree of slope and rainfall characteristics on runoff and soil erosion. Columbia, Mo., Miss. Agr. Exp. Station (Res. Bull. 280).

Neumann, W. 1976. Der Oberflaechenabfluss in staedtischen Einzugsgebieten. Muenchen, Technische Universitaet Muenchen (Berichte aus Wasserguetewirtschaft und Gesundheitsingernieurwesen No. 11).

Notodihardjo, M.; Zuidema, F.C. 1982. Integral quantitative and qualitative planning and management of water resources in the river basins in the Bergan-Kerto-Susila region, Indonesia. Proc. Exeter Symp., IAHS Publ. No. 135, p. 203-209.

Oechsner, P. 1967. Die Berechnung der Regenwassermenge auf Grad der Haeufigkeit der Intensitaet und der Versickerungswerte. Muenchen, Oldenbourg-Verlag, Stuttgarter Berichte zur Siedlungswasserwirtschaft (No. 22).

Oluwande, P.A. 1979. Development of the aquaprivy for urban sanitation. Proc. WED Conf., Loughborough, England.

Overton, E.D. 1964. Mathematical refinement of an infiltration equation for watershed engineering. U.S. Dept. of Agriculture, Agri. Res. Station, p. 41-99.

Overton, E.D.; Meadows, M.E. 1976. Stormwater modeling. New York, Academic Press.

Papadakis, C.N.; Preul, H.C. 1973. Testing of methods for determination of urban runoff. J. Hyd. Div., ASCE, Vol. 99, No. HY9, p. 1319-1335.

Pescod, M.B. 1982. Objectives of water management in developing countries. In: Hahn, H.H. (ed.). 1982. Water supply and wastewater treatment. Karlsruhe, Institute fuer Siedlungswasserwirtschaft, Universitat Karlsruhe.

Philippi, R. 1974. Moeglichkeiten und Grenzen des Wasserwirtschaftlichen Rahmenplannung (Possibilities and limits of the general water management planning. Wasser und Boden, p. 310-312.

Pieper, K. 1938. Regenabfluss von Strassenflaechen. Dresden, Technical University (Dissertation).

Poertner, H.G. 1973. Better storm drainage facilities at lower cost. Civil Engineering (ASCE), Vol. 43, p. 67.

Portland Cement Association. 1964. Handbook of concrete culvert pipe hydraulics. Chicago, Portland Cement Association.

Price, R.K.; Kidd, C.H.R. 1978. A design and simulation method for storm sewers. Proc. Int. Conf. on Urban Storm Drainage, Southampton, p. 327-337.

Proctor & Redfern, Ltd.; MacLaren, J.F., Ltd. 1976. SWMM Study, Vol. I. Ottawa, Environment Canada (COA Research Report No. 47).

Puech, C.; Gonni, D.C. 1984. Courbes hateur de pluie-duree-frequence Afrique de l'oust et centrale. Ouagadougou, Comite Interafricain d'Etudes Hydrauliques (CIEH) (Serie Hydrologie, BP. 369).

Pugsley, W.I. (ed.). 1981. Flood hydrology guide for Canada. Toronto, Atmospheric Environment Service, Environment Canada (Publ. No. CLI3-81).

Ragan, R.M.; Duru, J.O. 1972. Kinematic wave nomograph for times of concentration. J. of Hyd. Div., ASCE, Vol. 98, No. HY10, p. 1765-1771.

Rameshan, S.; Sarma, P.B. 1978. Urban hydrological modelling and catchment research in India. Paris, The Unesco Press.

Rao, R.A., Delleur, J.W.; Sarma, B.S.P. 1972. Conceptual hydrologic models for urbanizing basins. J. Hyd. Div., ASCE, Vol. 98, No. HY7.

Rawls, W.J.; Stricker, V.; Wilson, K. 1980. Review and evaluation of urban flood flow frequency procedures. Beltsville, Maryland, Science and Education Administration, U.S. Dept. of Agriculture.

Reinhold, F. 1955. Zur Ermittlung von Abflussmessungen aus Niederschlagsbeobachtungen. Die Wasserwirtschaft, Vol. 45, No. 5, p. 113.

Rose, C.W. 1966. Agricultural physics. New York, Pergamon Press.

Rouse, H. 1961. Fluid mechanics for hydraulic engineers. New York, Dover Publications.

Rybaczynski, W.; Polprasert, C.; McGarry, M. 1982. Appropriate technology for water supply and sanitation - low-cost technoloy options for sanitation. A state-of-the-art review and annotated bibliography. New York, Int. Dev. Centre and World Bank.

Sanders, R.J.; Watford, J.J. 1974. Public utilities. New York, International Bank for Reconstruction and Development, International Development Assoc. (Report No. RES 2).

Sartor, J.D.; Boyd, G.B.; Agardy, F.J. 1974. Water pollution aspects of street surface contaminants. J. Water Poll. Control Fed., Vol. 46, No. 3, p. 458-467.

Schilling, W. 1986. Urban runoff quality management by real-time control. In: Torno, H.C.; Marsalek, J.; Desbordes, M. (eds.). 1986. Urban Runoff Pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10, p. 765-818.

Schliessmann, D.J. 1959. Diarrhoeal disease and the environment. Bull. WHO, Vol. 21, p. 381-386.

Schliessmann, D.J.; Atchley, F.O.: Wilcomb, M.J.; Welch, L.F. 1958. Relationship of environmental factors to the occurrence of enteric diseases in areas of eastern Kentucky. Public health monograph. Washington, DC, U.S. Dept. of Public Health, Education and Welfare (Monograph No. 54).

Schneider, W.J.; Rickert, D.A.; Spieker, A.M. 1973. Role of water in urban planning and management. Washington, DC, U.S. Geological Survey (Circular 601-H).

Schumann, A. 1940. Regenabfluss in Strasseneinschnitten. Dresden, Technical University (Dissertation).

Service Technique de l'Urbanisme. 1980. Le modele RERAM: guide de l'utilisateur. Paris, Ministere de l'Urbanisme et du logement.

Sharp, A.L.; Holtan, H.N. 1940. A graphical method of analysis of sprinkled-plat hydrographs. Trans. Am. Geoph. Union, Vol. 21, p. 558.

Sharpe, D.E.; Kirkbride, T.W. 1959. Stormwater overflows: the operation and design of a stilling pond. Proc. Instn. Civ. Engrs., Vol. 13, p. 445.

Sherman, L.K. 1932. Stream flow from rainfall by the unit graph method. Eng. News-Record, Vol. 108, p. 501-505.

Sieker, F.; Linde, B. 1982. Versickerung von Niederschlagwasser (Stormwater infiltration). Korrespondenz Abwasser, Vol. 11, p. 760.

Smisson, B. 1967. Design, construction and performance of vortex overflows. Proc. Symp. on Storm Sewage Overflows, Inst. of Civ. Engrs, London, p. 99-110.

Sokolov, A.A.; Rantez, S.E.; Roche, M. 1976. Floodflow computation. Paris, Unesco Press (Studies and reports in hydrology No. 22).

Stall, J.B.; Terstriep, M.L. 1972. Storm sewer design - an evaluation of the RRL method. Washington, DC, U.S. EPA (Report No. EPA-R2-72-068).

Streeter, V.L. 1971. Fluid mechanics (5th edition). New York, McGraw-Hill Book Company.

Sullivan, R.H.; Manning, M.J.; Kipp, T.M. 1977. Nationwide evaluation of combined sewer overflows and urban stormwater discharges, Vol. III - characterization. Cincinnati, U.S. EPA.

Sullivan, R.H.; Hurst, W.D.; Kipp, T.M.; Heaney, J.P.; Huber, W.C.; Nix, S. 1978. Evaluation of the magnitude and significance of pollution loadings from urban stormwater runoff in Ontario. Ottawa, Environment Canada (COA Reseach Report No. 81).

Swamee, P.K.; Jain, A.K. 1976. Explicit equations for pipe-flow problem. J. Hyd. Div., ASCE, Vol. 102, No. HY5, p. 657-664.

Swinnerton, C.J.; Hall, M.J.; O'Donnell, T. 1972. A dimensionless hydrograph design method for motorway stormwater drainage systems. J. Inst. Highw. Engrs., Vol. 19, No. 11, p. 2-10.

Tang, W.H.; Mays, L.W.; Yen, B.C. 1975. Optimal risk-based design of storm sewer networks. J. Env. Engng. Div., ASCE, Vol. 101, No. EE3, p. 381-398.

Tay, J.H.; Chan, Y.K. 1984. Water quality study of an urban reservoir. Proc. Int. Conf. on Urban Storm Drainage, Goteborg, p. 811-819.

Terstriep, M.L.; Stall, J.B. 1969. Urban runoff by RRL method. J. Hyd. Div. ASCE, Vol. 95, No. HY6, p. 1809-1834.

Terstriep, M.L.; Stall, J.B. 1974. The Illinois urban drainage area simulator, Urbana, Ill., Illinois State Water Survey (Bull. 58).

Toebes, C.; Ouryvaev, V. 1970. Representative and experimental basins. Paris, Unesco Press (Studies and reports in hydrology No. 4).

Torno, H.C.; Marsalek, J.; Desbordes, M. 1986. Urban runoff pollution. Heidelberg, FRG, Springer-Verlag, NATO ASI Series G, Vol. 10.

Townsend, R.D: Wisner, P.E.; Moss, D.J. 1981. Inlet control devices for storm sewer catch basins. Proc. Canadian Hydrology Symp., Ottawa, Nat. Research Council.

Unesco. 1977. Research on urban hydrology. Vol. 1, Paris, Unesco Press (Technical Papers in Hydrology No. 15).

Unesco. 1978. Research on urban hydrology. Vol. 2, Paris, Unesco Press (Technical Papers in Hydrology No. 16).

Unesco. 1979. Impact of urbanization and industrialization on water resources planning and management. Report of the Zandvoort Workshop. Paris, Unesco Press (Studies and reports in hydrology No. 26).

Unesco. 1980. Casebook of methods of computation of quantitative changes in the hydrological regime of river basins due to human activities. Paris, Unesco Press (Studies and reports in hydrology No. 28).

Unesco. 1987. Manual on drainage in urbanized areas. Vol. II, Data collection and analysis for drainage design. Paris, Unesco Press (Studies and reports in hydrology No. 43).

U.S. Army, Corps of Engineers. 1977. Guidelines for calibration and application of STORM. Davis, California, Hydrologic Engineering Center (Training document No. 8).

U.S. Department of Agriculture, Soil Conservation Service. 1972. National engineering handbook. Washington, DC.

U.S. Department of Agriculture, Soil Conservation Service. 1975. Urban hydrology for small watersheds. Washington, DC, SCS (Technical Release No. 55).

U.S. Department of the Interior, Bureau of Reclamation. 1960. Design of small dams. Washington, DC, U.S. Dept. of the Interior.

U.S. EPA 1970. Control of infiltration and inflow into sewer systems. Washington, DC, U.S. EPA (Report No. 11020EFF).

U.S. EPA. 1974. Urban stormwater management and technology. Cincinnati, Ohio, U.S. EPA (Report No. EPA 67012-74/070).

U.S. EPA. 1977. Urban stormwater management and technology update and user's guide. Cincinnati, Ohio, U.S. EPA (Report No. EPA 600/8-77-014.)

U.S. EPA. 1978. Proceedings stormwater management model (SWMM) user's group meeting. Washington, DC, U.S. EPA (Report No. EPA 600/9-78-019).

U.S. EPA. 1979a. Proceedings stormwater management model (SWMM) user's group meeting. Washington, DC, U.S. EPA (Report No. EPA 600/9-79-003).

U.S. EPA. 1979b. Proceedings stormwater management model (SWMM) user's group meeting. Washington, DC, U.S. EPA (Report No. EPA 600/9-79-026).

U.S. EPA. 1980. Proceedings stormwater management model (SWMM) user's group meeting. Washington, DC, U.S. EPA (Report No. EPA 600/9-80-017).

U.S. EPA. 1982. Proceedings of stormwater and water quality management modeling user's group meeting. Washington, DC, U.S. EPA (Report No. EPA 600/9-82-015).

V.A.V. 1976. Anvisningar For Beraknong Av Allmanna Avloppsledningar (in Swedish). Stockholm, V.A.V. (Report No. P28).

van Dam, C.H.; van de Ven, F.H.M.; van de Uunk, E.J.B.; Heijnis, J.D. 1981. Kwaliteits- en kostenaspecten van de rioolstelselkeuze; ervaringen in Flevoland (Water quality and cost aspects of urban drainage systems in Flevoland). H_2O, Vol. 14, No. 24.

van Dam, C.H.; van de Ven, F.H.M. 1984. Infiltration in the pavement. Proc. Int. Conf. on Urban Storm Drainage, Goteborg, p. 1019-1028.

van Dort, J.A.; Kievit, J.L. de; Knook, J.B.J.; Riede, P.A. 1977. Hydrologie. De toepassing bij planning en ontwikkeling van begieden op lokaal niveau (The role of water in municipal development plans). Grontmij-Agricultural and Civil Consulting Engineers, De Bilt, the Netherlands.

Verworn, W. 1980. Hydrodynamische Kanalnatzberechnung und die Auswirkungen von Vereinfachungen der Berechnungsgleichungen. Hannover, Institut fuer Wasserwirtschaft, Hydrologie und Land wirtschaftliche Wasserbau, Technical University of Hannover (Heft 47).

Viessmann, W. Jr. 1968. Runoff estimation for very small drainage areas. Water Res. Research, Vol. 4, No. 1, p. 87.

Viessmann, W. Jr.; Baugh, T.E.; Knapp, J.W. 1972. Introduction to hydrology. New York, Intext.

Vlachos, E. 1982. Socio-cultural aspects of urban hydrology. In: Laconte, D.; Haimes, Y.Y. (eds.). 1982. Water resources and land-use planning: a systems approach. The Hague, Martinus Nighoff Publishers, p. 255-273.

Voorhees, M.L.; Wenzel, H.G. 1984. Urban design storm sensitivity and reliability. J. of Hydrology, Vol. 68, p. 39-60.

WHO. 1976. Community water supply and excreta disposal in developing countries. Review of Progress. Geneva, World Health Statistics Report, Vol. 29, No. 10, p. 544-631.

WMO. 1966. Measurement and estimation of evaporation and evapotranspiration. Geneva (Technical Note No. 83).

WMO. 1974. Guide to hydrological practices. Geneva (WMO 16).

WPCF. 1970a. Design and construction of sanitary and storm sewers. Washington, DC, WPCF (Manual of Practice No. 9).

WPCF. 1970b. Operation of wastewater treatment plants. Washington, DC, WPCF (Manual of Practice No. 11).

Waller, D.H.; Coulter, W.A.; Carson, W.M.; Bishop, D.G. 1976. Urban drainage model comparison for a catchment in Halifax, Nova Scotia. Ottawa, Environment Canada (COA Research Report No. 43).

Walters, G.A. 1982. Optimizing the plan layout of storm sewer networks. In: Urban drainage systems. Proc. of the First Int. Sem., London, Pitman Advanced Publishing Program.

Wanielista, M.P. 1979. Stormwater management, quantity and quality. Ann Arbor, Ann Arbor Science.

Wasmer, H.R. 1979. Wastes management as a world problem. Proc. WED Conf., Lougborough, England.

Watkins, L.H. 1962. The design of urban sewer system. London, H.M. Stationery Office (Road Res. Techn. Paper 55).

Watkins, L.H. 1976. The TRRL hydrograph method of urban sewer design adapted for tropical conditions. Proc. Inst. Civ. Engrs., Vol. 61, Part 2, p. 539-566.

Watt, W.E.; Kidd, C.H.R. 1975. QUURM - a realistic urban runoff model. J. Hydrology, Vol. 27, p. 225-235.

Wischmeier, W.H.; Smith, D.D. 1958. Rainfall energy and its relationship to soil loss. Trans. AGU, Vol. 3S, No. 2.

Wisner, P.E. 1981. Critical review of the rational method. Session 3, WMO/Unesco Regional Training Seminar on Urban Hydrology, Manilla.

Wisner, P.E. 1984. Experience with the implementation of stormwater management models. Proc. Int. Conf. on Urban Storm Drainage, Goteborg, p. 467-474.

Wright-McLaughlin Engineers. 1969. Urban storm drainage criteria manual. Denver, Colorado, Wright-McLaughlin Engineers.

Yen, B.C.; Chow, V.T. 1980. Design hyetographs for small drainage structures. J. Hyd. Div., ASCE, Vol. 106, No. HY6, p. 1055-1076.

Yen, B.C.; Tang, W.H. 1976. Risk-safety factor relation for storm sewer design. J. Env. Engng., ASCE, Vol. 102, No. EE2, p. 509-514.

Abbreviations used in the list of references

AGU	American Geophysical Union
APWA	American Public Works Association
ASCE	American Society of Civil Engineers
ATV	Abwassertechnische Vereinigung
CIEH	Comite Interafricain d'Etudes Hydrauliques
COA	Canada-Ontario Agreement (on Great Lakes Water Quality)
EPA	(U.S.) Environmental Protection Agency
IAHS-AISH	International Association of Hydrological Sciences - Association Internationale des Sciences Hydrologiques
KWK	Kuratoriums fuer Wasser und Kulturbanwesen
NATO	North Atlantic Treaty Organization
STORM	Storage, Treatment and Overflow Model
SWMM	Storm Water Management Model
TRRL	Transport and Road Research Laboratory
UNESCO	United Nations Educational, Scientific and Cultural Organization
WHO	World Health Organization
WMO	World Meteorological Organization
WPCF	Water Pollution Control Federation

Appendix A

INTERNATIONAL LIST OF URBAN TEST CATCHMENTS

Almost all the methods available for storm runoff calculations employ parameters which may need to be calibrated against field data. Local values of such parameters, which depend on local meteorological, hydrological, and geophysical conditions, seldom exist. Furthermore, the existing data always reflect the past or existing pre-development conditions, but the planning of urban drainage is usually done for some future post-development situation. Therefore, it may be even impossible to collect local parameter values and it becomes necessary to adopt the values of parameters from other similar catchments. Such transposed values may reflect site-specific conditions, or much more general conditions typical for the whole country or continent. Quite often, the transposed parameter values are called default values.

The collection of urban field data is time consuming and requires extensive experience. Furthermore, it may be difficult to collect reliable data when the catchment characteristics, such as, e.g., land use, change during the data collection programme, as it often happens in urban areas. This problem is often encountered in the case of runoff pollution data. In spite of the aforementioned problems, it is still advisable to obtain local data, because they are useful for better understanding of local hydrological processes, verification of various assumptions made in calculations, and improvement of accuracy of and confidence in computed results.

In order to make the reader aware of sources of urban hydrological data, a list of urban hydrological test catchments has been compiled and presented in this Appendix. It is also hoped that the information presented will lead to a better understanding of typical parameter values used in hydrological calculations and to the sharing of the experience gained in data collection under similar meteorological, hydrological, land use, and institutional conditions. This list of catchments includes references to the institutions which were responsible for data collection programmes. By contacting these institutions, the reader may obtain information on such programmes. To assist the reader in the selection of sources of information pertinent to his study, the list of catchments also contains their basic characteristics including the catchment surface cover, drainage concept, land use and the phenomena monitored. The meteorological and general hydrological conditions should be judged from the catchment location.

For brevity, a number of abbreviations has been used in the catchment list. Such abbreviations are explained below.

Column (5)	Land use	res.	residential
		comm.	commercial
		ind.	industrial
Column (6)	Sewerage concept	sep.	separate system
		comb.	combined system
		mix.	mixed drainage
Column (8)	Phenomena monitored	P(n)	precipitation (number of stations)
		R(n)	runoff quantity (number of stations)
		PQ	precipitation quality
		RQ	runoff quality

Although every effort has been made to make the list of urban test catchments as complete as possible, its extent depended on co-operation of volunteers from various countries and, consequently, it is impossible to judge the completeness of the list in terms of both the countries listed or the catchments reported for individual countries. The names of co-operators participating in this survey and the countries for which they supplied the required information are given below. Their co-operation is gratefully acknowledged.

Co-operator:	Provided data for:
Mr. A. Van der Beken	Belgium
Mr. J.P. Lahaye	Benin, Burkina Faso, Mali, Niger and Togo
Mr. J. Marsalek	Canada
Mr. Y.W. Kang	Republic of China
Mr. P. Urcikan	Czechoslovakia
Mr. M. Jensen	Denmark
Mr. M. Melanen	Finland
Mr. M. Desbordes and G. Jacquet	France
Mr. W.F. Geiger	Federal Republic of Germany
Mr. O. Starosolszky	Hungary
Mr. F. Calomino and Mr. A. Paoletti	Italy
Mr. T. Sueishi	Japan
Mr. E. Schultz and Mr. van de Ven	The Netherlands
Mr. P Blaszczyk	Poland
Mrs. M.R. Matos	Portugal
Mr. S. Lyngfelt, Mr. J. Niemczynowicz and Mr. G. Westerstrom	Sweden
Mr. R. Gloor and Mr. B. Huber	Switzerland
Mr. T. Tingsanchali	Thailand
Mr. R.K. Price	United Kingdom
Mr. C. Maksimovic	Yugoslavia

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
<u>BELGIUM</u>									
Brussel-Oefenplein	20	50	2-3	campus	sep.	closed conduits	P(1), R(1), RQ	since 1983 (4 yr)	Vrije Universität Brussel Laboratory of Hydrology Pleinlaan 2, B-1050 Brussel
<u>BENIN</u>									
Cotonou-Bassin A	112	measure-ments in progress	flat	tradit. housing	comb.	open channels	P(10),R(1)	77 - 79 (22 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadougou, Burkina Faso
-Bassin B	43	- " -	flat	tradit. housing	comb.	open channels	P(9),R(1)	77 - 79 (22 mo)	- " -
-Bassin F	47	- " -	flat	public build., res.	comb.	open channels	P(3),R(1)	77 - 79 (22 mo)	- " -
<u>BURKINA FASO</u>									
Ouagadougou - St. Julien	48,2	10	gently sloped	tradit. houses	comb.	open channels	P(3),R(1)	77 - 79 (18 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadougou, Burkina Faso
-Rue Destenave	173	25	flat	tradit. houses, airport	comb.	open channels	P(8),R(1)	77 - 79 (18 mo)	- " -
-Av. de la Li-berté	520	25	flat	tradit. houses, public build.	comb.	open channels	P(8),R(1)	77 - 79 (18 mo)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
CANADA									
Toronto - Barrington	23	18	-	res.	sep.	closed conduits	P,R, RQ	2 yr	Borough of East York, Municipal Office 550 Mortimer Street, Toronto, Ontario
- Brucewood	20	48	flat	res.	sep.	closed conduits	P,R, RQ	2 yr	MacLaren Plansearch Inc. 33 Yonge Street, Toronto, Ontario MSE 1E7
Kingston - Calvin Park	38	29	-	res.	sep.	closed conduits	P,R, RQ	5 yr	Department of CivilEngineering, Queens's University, Kingston, Ontario K7L 3N6
London - Carling	18	66	flat	ind.	sep.	closed conduits	P,R, RQ	0,5 yr	Department of Geography University of West. Ontario, London,Ontario
Toronto - East York	155	49	flat	res.	comb.	closed conduits	P,R, RQ	2 yr	M.M. Dillon Ltd. 50 Holly Street, Toronto, Ontario M4P 2G5
Halifax - Fairfield	68	34	-	res., comm.	comb.	closed conduits	P,R	2 yr	Department of Civil Engineering, Tech. University of Nova Scotia, P.O. Box 1000, Halifax, N.S. B3J 2X4
Hamilton	55	41	flat	res.	comb.	closed conduits	P,R, RQ	2 yr	Gore & Storrie Ltd. 1670 Bayview Avenue, Toronto Ontario M4G 3C2
Ottawa - Kennedy-Burnett	45	32	-	res.	sep.	closed conduits	P,R, RQ	3 yr	Regional Municipality of Ottawa-Carleton 222 Queen Street, Ottawa, Ontario K1P 5Z3
Burlington - Malvern	23	30	flat	res.	sep.	closed conduits	P,R, RQ	5 yr	Hydraulics Division, National Water Research Institute, 867 Lakeshore Road, Burlington, Ontario L7R 4A6
CHINA, REP. OF									
Taipei	6700	70	flat	res., comm.	sep.	closed conduits, open channels, pres.pipe systems	-	10 y	The Chinese Institute of Civil and Hydraulic Engineering 4fl., No. 1, Section 2, Jen Ai Road, Taipei

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land- use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
CZECHOSLOVAKIA									
Bratislava - Strkovec	20,9	46,5	flat	res.	comb.	closed conduits	RQ	4 vr	Dpt. I. Šipkovský VUVH nábr.arm.gen.L. Svobodu 5, CS - 81249 Bratislava
- ul.Febr. vit'azstva	18,8	60	gently sloped	res.	comb.	closed conduits	RQ	8 yr	Ing. J. Pôbiš , CSC. VUVH nabr.arm.gen. L. Svobudu 5, CS - 81249 Bratislava
- collector B IV	474	22,5	flat up to steep	parks, res., comm.	comb.	closed conduits	P(3) RQ	2 yr	Ing. V. Petratur, CSc. Godrová 5, CS-81106 Bratislava
Praha - Karlovo nám	17,6	49,2	gently sloped	comm.	comb.	closed conduits	RQ	1 yr	Prof. P. Čizek, Dr.Sc. Jánskeho 365, Černošice CS - 25228 Praha
DENMARK									
Bispeengsbuen	0,036	100	flat	asphaltic concrete	surface runoff system	closed conduits	P,R	78 (2 mo)	Department of Environmental Engineering Technical University of Denmark Building 115, DK-2800 Lyngby
DTH ST 7802	0,068	0	flat	paving stones	surface runoff system	closed conduits	P,R	78-79 (8 mo)	- " -
Vestforbraending	0,016	0	flat	SF-stones	surface runoff system	closed conduits	P,R	80 (3 mo)	- " -
Nørrebro	1,46	78	flat	city	comb.	closed conduits	P,R	79-83 (48 mo)	- " -
Emdrup Banke	4,4	47	flat	urban (low houses)	comb.	closed conduits	P,R	79-83 (48 mo)	- " -
Cedervaenget	5,5	46	flat	urban (storey houses)	comb.	closed conduits	P,R RQ	80-83 (48 mo)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
DENMARK									
Vestre Paradisvej	18,0	20	flat	urban (low houses)	comb.	closed conduits	P,R RQ	80-83 (48 mo)	Dept. of Environmental Engineering Technical University of Denmark Building 115, DK-2800 Lyngby
Munkeris-Parken	6,5	40	flat	urban (low houses)	sep.	closed conduits	P,R RQ	80-83 (48 mo)	- " -
Vejdirektoratet-Bispeengsbuen	1,91	76	medium	traffic (urban)	sep.	closed conduits	P,R RQ	73-82 (10 yr)	- " -
Langedam-Birkerød	29	20	medium	urban (low houses)	comb.	closed conduits	P,R	77 - 83 (6 yr)	Department of Sanitary Engineering Danish Engineering Academy Building 372, DK-2800 Lyngby
Birkerød (A16)	19	23	flat	urban (low houses)	sep.	closed conduits	P,R RQ	79 - 80 (2 yr)	Birkerød Kommune Teknisk Forvaltning
Farum	12	29	flat	urban (low houses)	comb.	closed conduits	P,R RQ	79/80 (14 mo)	Farum Kommune Teknisk Forvaltning
Tårnby	244	25	flat	urban (low houses, storey houses)	comb.	closed conduits	P,R	75/76/77 (2 yr)	Dept. of Sanitary Engineering Danish Eng. Academy Building 372, DK-2800 Lyngby
Aalborg-Rihimäkivej	1,9	73	flat	indu-strial	comb.	closed conduits	P,R	78 - 79 (2 yr)	Department of Environmental Engineering University Center of Aalborg DK-9000 Aalborg

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land- use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
<u>FINLAND</u>									
Tampere-Hämeenpuisto	13,2	67	average sloped	city centre	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	National Board of Waters P.O. Box 250 SF-00101 Helsinki 10
-Nekala	14,2	40	gently sloped	ind.	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
Helsinki-Herttoniemi	14,7	35	gently sloped	traffic (motor- way)	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
Kajaani-Centre	18,5	64	gently sloped	city centre	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
Helsinki-Pakila	20,2	29	average sloped	res.	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
-Kontula	22,9	40	gently sloped	res.	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
Oulu-Kaukavainio	40,5	30	flat	res.	sep.	closed conduits	P, R PQ,RQ	77 - 79 (3 yr)	- " -
<u>FRANCE</u>									
Angers	225	47	0,8	res.	sep.	closed conduits	P(2),R(3)	76 - 77 (8 mo)	Ministry of Equipment S.T.U. 64, rue de la Fédération 75015 - Paris Ministry of Interior D.G.C.L. Technical Services 8-12, rue d'Aguesseau 75800 - Paris
Aix-Zup	25,6	78	2,9	res.	sep.	closed conduits	P(1),R(1) RQ(2)	80 - 82 (14 mo)	
-Nord	92	35	6,5	res. (low)	sep.	closed conduits	P(1),R(1) RQ(2)	80 - 82 (14 mo)	
Bordeaux	3490	25	0,31	res. to rural	sep.	closed c. and open channels	P(2),R(2) P(4),R(11)	76 - 77 (10 mo) 78 (4 mo)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
FRANCE									
Chambery	160	40	12	res. (dense)	comb.	closed conduits	P(2),R(3)	77 (3 mo)	Ministry of Equipment S.T.U. 63, rue de la Fédération 75015 Paris
Clermont-Ferrand	380	26	4,3	res.	comb.	closed conduits	P(2),R(2)	76 - 77 (7,5 mo)	Ministry of Interior D.G.C.L. Technical Services 8-12, rue d'Aguesseau 75800 Paris
Toulose-Colomiers	315	25	1,4	res. (low)	sep.	closed conduits	P(2),R(2)	76 - 77 (8,5 mo)	- " -
Lille-Villeneuve d'Asq	303	44	0,51	res. (low)	sep.	closed conduits	P(1),R(2)	76 - 77 (7,5 mo)	- " -
Montpellier	1586	21	1,6	res. to rural	sep.	closed c. and open channels	P, R	76 - 78 (10 mo)	- " -
Nancy-St. Anne	992	27	0,4	res. to rural	comb.	closed conduits	P(2),R(2)	76 - 77 (9,5 mo) 78	- " -
Maurepas	26,7	60	0,5	res.	sep.	closed conduits	P, R RQ	80 - 82 (15 mo)	- " -
Les Ulis	43,2	42	0,55	res.	sep.	closed conduits	P, R RQ	80 - 82 (15 mo)	- " -
Rouen	240	90	0,2	res. (low)	sep.	closed conduits	P, R	76 - 77 (5,8 mo)	- " -
Rungis	220	90	0,2	comm.	sep.	closed conduits	P, R	78 - 81 (3 yr)	- " -
Poissy	50,5	42	flat	res., ind.	comb.	closed conduits	P(1),R(1) RQ(1)	1 yr	Coyne et Bellier 5 rue d'Heliopolis 75017 Paris
Créteil	45,4	49	flat	res.	sep.	storm sewer	P(1),R(1) RQ(1)	1 yr	- " -
Velizy	53,3	54	flat	res., ind.	sep.	storm sewer	P(1),R(1) RQ(1)	1 yr	LROP 12 rue Teisserenc de Bort BP 108, 78190 Trappes

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
FRANCE									
Seine St. Denis County:									
-Rue Ste. Baudile	1145	30	flat	res. 91% undev. 9%	sep.	storm sewer	P(2),R(2) RQ(1)	1 yr	Arrondissement Operationnel N°3 Service departementale d'Assainissement 99 Av. Gen. de Gaulle 93 Rosny sous Bois
-Canal du Chesnay	560	19	flat	res. 68% undev.32%	sep.	storm sewer	P(2),R(1) RQ(1)	1 yr	
-Rue des Grammonts	144	25	flat	res.,	sep.	storm sewer	P(1),R(1) RQ(1)	1 yr	- " -
-Collecteur de la Malnoue	185	35	flat	res. 51%, ind. 26%, undev.23%	sep.	storm sewer	P(1),R(1) RQ(1)	1 yr	- " -
-Collecteur du Centre Urbain	226	24	flat	res. 54%, freew.7%, undev.39%	sep.	storm sewer	P(1),R(1) RQ(1)	1 yr	- " -
-Livry-Gargan	254	33	flat	res. 78%, park 10%, freew. 9%,	comb.	closed conduits	P(2),R(1) RQ(1)	1 yr	- " -
GERMANY, FED. REP. OF									
Aachen-Hüttenbach	28,6	50	steep	comm.	sep.	closed conduits	P, R PQ,RQ	73/74 (2 yr)	Institut für Siedlungs-wasserwirtschaft RWTH Aachen 5100 Aachen
-Muffet	8,6	49	strongly sloped	res.	sep.	closed conduits	P, R PQ,RQ	73/74 (2 yr)	- " -
-Tivoli	22,8	32	aver. sloped	res.	sep.	closed conduits	P, R PQ,RQ	73/74 (2 yr)	- " -
Augsburg-Bärenkeller	74	25	flat	res.	comb.	closed conduits	P, R PQ,RQ	74 (7 mo)	Tiefbauamt 8900 Augsburg

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
GERMANY, FED. REP. OF									
Autobahn - A 6	5,0	80	30	street	sep.	closed conduits	P, R PQ, RQ	78 (9 mo)	Institut für Siedlungswasserbau Universität Stuttgart Bandtäle 1, 7000 Stuttgart-Büsnau
Autobahn - A 81	1,3	100	40	street	sep.	closed conduits	P, R PQ, RQ	78 (8 mo)	- " -
Bremen-Ost	550	38	flat	res., ind., parks	comb.	closed conduits	P, R	78/79 (10 mo)	Amt für Stadtentwässerung und Stadtreinigung 2800 Bremen 1
Frankfurt-Emil-Klaar-Str.	72	80	flat	res., comm.	comb.	closed conduits	P, R PQ, RQ	79 - 81 (3 yr)	Stadt Frankfurt Stadtentwässerung 6000 Frankfurt/Main
-Sachsenhäuser Berg	46	35	strongly sloped	res.	comb.	closed conduits	P, R PQ, RQ	79 - 81 (3 yr)	- " -
-Westhausen	87	43	flat	res.	comb.	closed conduits	P, R PQ, RQ	79 - 81 (3 yr)	- " -
Hamm	800	40	flat	city centre	comb.	closed conduits	P, R PQ, RQ	80/81 (2 yr)	Lippeverband Kronprinzenstraße 24 4300 Essen
Hamburg			flat	city centre	comb.	closed conduits	P, R	73 - 76	Amt für Ingenieurwesen III Hauptabteilung Stadtentwässerung 2000 Hamburg 36
Hildesheim	81,8	29-93	strongly sloped	res.	sep.	closed conduits	P, R	76 - 79 (4 yr)	Institut für Wasserwirtschaft, Hydrologie und landwirtschaftl. Wasserbau Universität Hannover, 3000 Hannover 1
Kiel-Projensdorf	166	19	aver. sloped	res.	sep.	closed conduits	P, R	77/78 (1,5 yr)	- " -
-Rehsenerbach	26,5	1	steep	garden	sep.	closed conduits	P, R	77/78 (1,5 yr)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
GERMANY, FED. REP. OF									
Emscher-Lippe	4762	2-44	diff.	overall urban. area	diff.	closed conduits	P, R	contin.	Emschergenossenschaft/Lippeverband Kronprinzenstraße 24 4300 Essen
München-Harlaching	528	35	flat	res., comm.	comb.	closed conduits	P, R PQ,RQ	75 - 82 (6 yr)	Lehrstuhl für Wassergütewirtschaft Techn. Univ. München, Am Coulombwall, 8046 Garching
-Neuperlach	360	18	strongly sloped	res.	modif. comb.	closed conduits	P, R PQ,RQ	78 - 82 (4 yr)	- " -
-Ingolstädter-Straße	96	37	flat	mixed	comb.	closed conduits	P, R PQ,RQ	76/77 (10 mo)	Stadt München Abteilung Entwässerung 8000 München 2
-Josef-Wirth-Str.	34	16	flat	res.	mixed	closed conduits	P, R PQ,RQ	76/77 (10 mo)	- " -
Pullach (München)	23	36	flat	res.	sep.	closed conduits	P, R PQ,RQ	72 - 81 (7 yr)	Lehrstuhl für Wassergütewirtschaft Techn. Univ. München, Am Coulombwall, 8046 Garching
Stuttgart-Büsnau	32	38	steep	res.	comb.	closed conduits	P, R PQ,RQ	66 - 68 (2 yr)	Institut für Siedlungswasserbau der Universität Stuttgart Bandtäle 1, 7000 Stuttgart-Büsnau
Tübingen-Gniebel	35	50	strongly sloped	village	sep.	closed conduits	P, R	78/79 (1 yr)	Regierungspräsidium 7400 Tübingen
-Rübgarten	12	50	steep	village	sep.	closed conduits	P, R	78/79 (1 yr)	- " -
HUNGARY									
Heves	600	30	gently sloped	res., comm.	mixed	open and closed conduits	P,R	years	VITUKI Budapest Kvassay J. ut 1. H-1095 Budapest
Miskolc	75	27,6	strongly sloped	res., comm.	sep.	open and closed conduits	P,R	70 - 79 (9 yr)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
ITALY									
Milan	3100	67	flat	res., comm.	comb.	closed conduits	P(8), R(7)	5 yr	Istituto di Idraulica Politecnico di Milano Piazza Leonardo da Vinci, 32 I - 20133 Milano
Luzzi	*1,8*	*90*	*steep*	*res.*	*sep.*	*closed sewer*	P(1), R(1)	*years*	*Dipartimento Difesa del Suolo Università della Calabria I - 87040 Montalto Uffugo (Cosenza)*
JAPAN									
Shiga-prefecture - Hayama River basin	1610		flat	forest(48%), farm (21%), residents (13%), roads(1,7%), others(16%)	none, under constr.	open channels	P, R RQ	since 1982	Department of Environmental Engineering Osaka University
Hiroshima City - Yoshijima	215	47.0	flat	res., comm., ind.	comb. (pumped)	closed conduits (part. pumped)	P, R RQ	76 - 78	Hiroshima City
Kyoto City - Senbon	148	77.0	flat	ind., comm., res.	comb.	closed conduits	R, RQ P(1)	75 - 78 (36 mo)	Kyoto City
Chiba City - Northern Hill	93,2	26,2	gently sloped	res., comm.	sep.	open and closed conduits	P(5) R, RQ	71 - 72 (5 mo)	Chiba City
Toyohashi City - Haccho	68,4	73.0	flat	comm.	comb.	closed conduits	P(1) R, RQ	75 - 78 (28 mo)	Toyohashi City
Kitakyushu City - Ohji	57,6	26,0	aver. sloped	res.	comb.	closed conduits	P(1) R, RQ	75 - 77 (16 mo)	Kitakyushu City
Osaka City - Nakanoshima	45,5	86,0	flat	comm.	comb.	closed conduits	P(1) R, RQ	75 - 77 (26 mo)	Osaka City
Sapporo City - Mikaho	44,3	34,0	flat	res.	comb.	closed conduits	P(1) R, RQ	75 - 77 (2 yr)	Sapporo City
Nagoya City - Shirakawa	39,5	66,0	flat	comm.	comb.	closed conduits	P(1) R, RQ	75 - 77 (25 mo)	Nagoya City

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
JAPAN									
Kawasaki City - Kan-non	35,1	46,0	flat	res., comm.	comb.	closed conduits	P(1) R, RQ	75 - 77 (2 yr)	Kawasaki City
Kobe City - Kitasuma	26,8	45,6	aver. sloped	res.	sep.	open and closed conduits	P(1) R, RQ	76 - 79 (30 mo)	Kobe City
Yokohama City - Horinouchi	22,1	39,0	aver. sloped	res., comm.	comb.	closed conduits	P(1) R, RQ	75 - 78 (30 mo)	Yokohama City
Fukuoka City - Higashi-Nakusu	17,6	88,0	flat	comm.	comb.	closed conduits	P(1) R, RQ	75 - 78 (27 mo)	Fukuoka City
Kobe City - Hanakuma	17,2	91,4	gently sloped	res., comm., ind. incl.hosp. & schools	sep.	open conduits	P(1) R, RQ	73 - 75 (30 mo)	Kobe City
Kitakyushu City - northern coast/ central	14,9	42,3	strong-ly sloped	res., comm.	sep.	open conduits	P(4) R,RQ	80 - 81 (3 mo)	Kitakyushu City
Yamagata City - Midorimachi	13,7	53,3	aver. sloped	res., comm.	sep.	open conduits	P(3) R,RQ	78 - 79 (7 mo)	Yamagata City
Gifu City - Shimizu River	10,6	79,9	flat	comm.	sep.	open conduits	P(1) R,RQ	67 (2 mo)	Gifu City
MALI									
Bamako-Hydraulique	120	measure-ments in progress	gently sloped	tradit. housing, comm.	comb.	open channels	P(10),R(1)	78-80/82 (28 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadougou, Burkina Faso
-Bonbonniere	212	- " -	strongly sloped	comm., public build.	comb.	open channels	P(16),R(1)	78-80/82 (28 mo)	- " -
-Soudan Cinema	84	- " -	steep	comm., public build.	comb.	open channels	P(7),R(1)	78-80/82 (28 mo)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
MALI									
Bamako-Chemin de Fer	58	measurements in progress	steep	comm., public build.	comb.	open channels	P(5) R(1)	78-80/82 (28 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadougou, Burkina Faso
NETHERLANDS									
Amsterdam	8,9	49	flat	res.	sep.	closed conduits	P, R RQ	83	Stichting Onderzoek Reiniging von Afvalwater P.O. Box 414 NL - 2280 Ak Rijswijk
Bodegraven	45	49	flat	res.	comb.	closed conduits	P, R RQ	since 83 (5 yr)	- " -
Heerhugowaard	14,6	47	flat	res.	sep.	closed conduits	P, R RQ	83	- " -
Kerkrade	136	44	sloped	res.	comb.	closed conduits	P, R RQ	83	- " -
Lelystad-Pampus--Blokkerhoek	2	41	0	res.	sep.	closed conduits	P, R	68-86	Ijsselmeerpolders Development Authority Postbox 600 8200 AP Lelystad
-Norderwagenplein	0,7	99	0	comm. (parking lot)	sep.	closed conduits		69-86	- " -
-Bastion	4,5	66	0	res.	sep.	closed conduits	P, R PQ,RQ	82-85	- " -
Neede	200	30	2%	res.	comb.	closed conduits	P, R	72 - 77	Heidemij Adviesburo Lovinklaan 1 NL - Arnhem
Oesterhout	22,5	52	flat	res.	comb.	closed conduits	P, R RQ	since 83 (5 yr)	Stichting Onderzoek Reiniging van Afvalwater P.O. Box 414 NL - 2280 Ak Rijswijk
Loenen	81	20	gently sloped	res.	comb.	closed conduits	P, R RQ	since 82 (5 yr)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
NIGER									
Niamey-BV 1-PJ	71	measurements in progress	flat	public build.	comb.	open channels	P(9),R(1)	78 - 80 (18 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadoudou, Burkina Faso
-BV 2-Salaman	42	- " -	gently sloped	tradit. houses	comb.	open channels	P(5),R(1)	78 - 80 (18 mo)	- " -
-BV 3-Boukoky	73	- " -	flat	tradit. houses	comb.	open channels	P(10),R(1)	78 - 80 (18 mo)	- " -
POLAND									
Warszawa - Oszycka	298	45	strongly sloped	res., ind.	sep.	closed conduits	P(3),R(1) RQ	10 yr 2 yr	Instityt Ksztaltowania Srodowiska ul. L. Krzywickiego Nr. 9 PL - 02-078 Warszawa, P-34
- Mokotów	600	40	strongly sloped	res., comm.	comb.	closed conduits	P(4),R(1) RQ	3 yr indiv. events	- " -
PORTUGAL									
Lisboa-Alvalade	50,1	86	1,05 gently sloped	res., comm.	comb.	closed conduits	R(2)	since 1981 (6 yr)	Laboratorio Nacional de Engenharia Civil Sanitary Hydraulics Divisions CODEX Department of Hydraulics Av. do Brasil, 1799 Lisboa
Oporto-Antas	13,6	69	2,00 aver. sloped	res., comm.	sep.	closed conduits	P(1),R(1)	since 1984 (3 yr)	
SWEDEN									
Linköping 1	145	47	flat	res.	sep.	closed conduits	P,R	76 - 77 (12 m)	Chalmers University of Technology Department of Hydraulics S - 41296 Göteborg
Linköping 2	18,5	34	gently sloped	res.	sep.	closed conduits	P,R	76 - 77 (12 mo)	- " -
Linköping 3	3,5	57	flat	res.	sep.	closed conduits	P,R RQ	76 - 77 (12 mo)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
SWEDEN									
Göteborg - Floda	18,0	18	steep	res.	sep.	closed conduits	P,R RQ	1 yr	Chalmers University of Technology Department of Hydraulics S-41296 Göteborg
- Bergsjön	15,4	38	strongly sloped	res.	sep.	closed conduits	P,R RQ	5 yr	- " -
- Klostergarden	14,1	49	gently sloped	res., comm.	sep.	closed conduits	P,R clim. data	8 yr	- " -
Luleå-Porsön	13	30	aver. sloped	res.	sep.	closed conduits	P,R	indiv. events	Division of Water Resources Engineering University of Luleå S - 95187 Luleå
Lund - D1	440	23	gently sloped	res.	sep.	closed conduits	P,RQ R,PQ	17 mo	Department of Water Resources Engineering Lund Institute of Technology University of Lund
- D2	45	45	gently sloped	res.	sep.	closed conduits	P,RQ R, PQ	17 mo	Box 118 S - 22100 Lund
- D3	50	50	gently sloped	res.,	sep.	closed conduits	P,RQ R,PQ	17 mo	- " -
- D11	310	35	gently sloped	res., ind.	sep.	closed conduits	P,RQ R,PQ	17 mo	- " -
- D13	310	35	gently sloped	res.	sep.	closed conduits	P,RQ R,PQ	17 mo	- " -
- Total	1992	30	gently sloped	res., ind., comm.	sep.	closed conduits	P,RQ R,PQ, clim. data	17 mo	- " -
SWITZERLAND									
Zürich-Winterthur Highway	5,5	100	gently sloped	Highway	sep.	closed conduits	RQ,P R	1 yr	Federal Institute for Water Resources and Water Pollution Control (EAWAG) CH - 8600 Dübendorf
Zürich - Schwamendingen	10,1	40	flat	res.	sep.	closed conduits	R,PQ P	1 yr	- " -
- Friedacker	12,7	42	strongly sloped	res.	comb.	closed conduits	P,R	1 yr	- " -
Geneva - Seymaz	38,4	9	flat	res., comm., rural	sep., comb.	closed conduits	P(5),R(4)	78 - 83 (5 yr)	Institut Génie Rural En Bassenges CH - 1024 Ecublens-Lausanne

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
THAILAND									
Prathumthani-AIT Campus	4,5 acres	90	0,5 %	Campus	rain-water drainage	open channels	P	(9 mo)	Asian Institute of Technologies (Div. of Water Resources Eng.) P.O. Box 2754, Bangkok, Thailand
TOGO									
Lome-Cordon Littoral	11,9	measure-ments in progress	flat	urban.	comb.	open channels	P(6),R(1)	77 - 79 (21 mo)	Comité Interafricain d'Etudes Hydrauliques BP 369 Ouagadougou, Burkina Faso
-Plateau	11,1	- " -	flat	urban.	comb.	open channels	P(20),R(1)	77 - 79 (21 mo)	- " -
-Bassin 0	83	- " -	gently sloped	tradit. houses	comb.	open channels	P(20),R(1)	77 - 79 (21 mo)	- " -
UNITED KINGDOM									
Bracknell-Wildridings	11,67	45	3 %	res.	sep.	closed conduits	P(1)	74 - 81 (7,5 yr)	Hydraulics Research Station Howberry Park Wallingford, Oxon
Derby-St.-Marks Rd.	10,31	56	0,4 %	res.	sep.	closed conduits	P(1)	71 - 82 (11 yr)	- " -
Stevenage-Shephall	142,1	33	2 %	res.	sep.	closed conduits	P(2)	75 - 82 (7 yr)	- " -
Nottingham-Clifton Grove	10,6	42	4 %	res.	sep.	closed conduits	P(1)	80 (1 mo)	Dr. Pratt, Trent Polytechnic Burton St. Nottingham NG1 4BU

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
USA									
Anchorage/Alaska - Chester Cr./ Baxter Rd.	3.859	47	-	low dens. res.	sep.	-storm sewer -curb and gutter		2 yr	U.S. Geological Survey Hydrologic Records Sec. Water Resources Div., Box 25046 MS 415, Denver, Colorado 80225, USA
- Chester Cr./ 20th Ave.	1.036	66	-	res.	sep.	-storm sewer -curb and gutter		2 yr	
- Chester Cr./ 36th Ave.	259	85	-	comm.	sep.	-storm sewer -curb and gutter		2 yr	- " -
Fresno/Calif. - Comm.Urban Runoff Site	23.31	98.9	26	comm.	sep.	-storm sewer -curb and gutter		1 yr (5 mon)	- " -
- Multi-Dwell Res.Urb.Runoff Site	18,13	57	13	res.	sep.	-storm sewer -curb and gutter		1 yr (5 mon)	- " -
Lakewood/Co. - Villa Italia	51,8	91	-	comm.	sep.	-		1 yr (2 mon)	- " -
- North Ave.	33,67	54	-	res., comm.	sep.	-		1 yr (2 mon)	- " -
- North Ave.	28,49	59	-	res., comm.	sep.	-		4 mon	- " -
Northglenn/Co.	155,4	37	-	res.	sep.	-		1 yr (4 mon)	- " -
Denver/Co. - Cherry Knolls	23,051	64	-	res.	sep.	-		1 yr (1 mon)	- " -
- Asbury Park	51,8	31	-	comm.	sep.	-		1 yr (2 mon)	- " -
Fort Lauderdale/Fa.	528,36	97,9	-	comm.	sep.	storm sewer		1 yr (1 mon)	- " -

1	2	3	4	5	6	7	8	9	10
country, city name of catchment	area (ha)	ratio of imperv. (%)	slope	land-use	sewer concept	type of sewer system	phenomena monitored	study duration	responsible institution
USA									
Largo/ Fa. - Allen Creek	463,61	36	-	res., comm.	sep.	-		8 yr (8 mon)	U.S. Geological Survey Hydrologic Records Sec. Water Resources Div., Box 25046 MS 415 Denver, Colorado 80225, USA
Glen Ellyn/Ill. - Lake Ellyn	214,97	34	strongly sloped	comm., res.	sep.	-storm sewer -curb and gutter		1 yr (6 mon)	
Eden Prairie/ Mn.	33,67	11	flat	res.	sep.	-		11 mon	- " -
Golden Valley/Mn.	85,47	22	flat	res.	sep.	-		11 mon	- " -
Pittsford, N.Y. - Trib to Barge canal	67,34	43,8	-	res.	sep.	-storm sewer -curb and gutter		1 yr (2 mon)	- " -
Huntington, N.Y.	15,799	100	-	comm.	sep.	curb and gutter		3 yr (6 mon)	- " -
Upper Arlington, Oh. - Fishinger Rd.	116,55	60	flat	res.	sep.	-storm sewer -curb and gutter		1 yr (11 mon)	- " -
Salt Lake City, Ut. - Bells Canyon cond.	25,9	-	strongly sloped	res.	sep.	-storm sewer -curb and gutter		1 yr (7 mon)	- " -
- 21st south cond.	492,1	-	flat	res., comm., ind.	sep.	-		1 yr (11 mon)	- " -
Bellevue, Wa. - Lake Hills	38,591	36,1	steep	res.	sep.	-storm sewer -curb and gutter		2 yr (3 mon)	- " -
YUGOSLAVIA									
Miljakovac	25	44	steep	res.	sep.	storm sewer with gutters	P(2), R(2)	2 yr	Faculty of Civil Engineering Institute of Hydraulic Engineering P.O. Box 895, YU - 11000 Belgrade

Appendix B

RATIONAL METHOD CALCULATIONS FOR THE WATERSHED SHOWN IN FIGURE 8.1

For a 10-year return period, the local IDF curve is assumed to be described by equation

$$iM(t,\ 10) = 366\ t^{-0,44}$$

where iM(mm/h) is the rainfall intensity for duration t(min).

Case I - Pre-urban conditions

Using the data from Table 8.1, detailed calculations for Case I appear below.

(a) Area A1

(1) Time of concentration, at outlet 0', can be expressed as

$$tc_1 = to_1 + td_1$$

where to_1 is the overland flow travel time for area A1 and td_1 is the travel time in the drainage channel. Such times are calculated by dividing the travelled lengths by the corresponding mean flow velocities as shown below:

$$to_1 = 300/0,3 = 1000\ s$$

$$td_1 = 1200/1,2 = 1000\ s$$

$$tc_1 = 1000 + 1000 = 2000\ s = 33,3\ min$$

(2) Critical intensity for area A1 is calculated as

$$iM(tc_1,10) = 366 \cdot (33,3)^{-0,44} = 78,3\ mm/h$$

(3) Peak runoff Qp(T) at outlet 0' - in Equation 8.1, constant k equals $2,78 \times 10^{-3}$ if the intensity is given in mm/h, the area is in hectares, and the discharge in m^3/s. Assuming the correction factor F = 1, Equation 8.1 can be written as

$$Qp(T) = 2,78 \cdot 10^{-3} \cdot C \cdot iM(tc,T) \cdot A$$

and, for C = 0,3, the expression for Qp_1 then reads

$$Qp_1(10) = 2,78 \cdot 10^{-3} \cdot 0,3 \cdot 78,3 \cdot 40 = 2,61\ m^3/s$$

(b) Area A2

Calculations analogous to those for area A1 lead to the following results:

(1) Time of concentration tc_2 at outlet 0

$$tc_2 = (400/0,2) + (1600/0,8) = 4000 \text{ s} = 66,7 \text{ min}$$

(2) Critical intensity for area A_2

$$iM\ (tc_2, 10) = 366 \cdot (66,7)^{-0,44} = 57,6 \text{ mm/h}$$

(3) Peak runoff Qp_2 from A_2 only (C = 0,2)

$$Qp_2(10) = 2,78 \cdot 10^{-3} \cdot 0,2 \cdot 57,6 \cdot 60 = 1,92 \text{ m}^3/\text{s}$$

(c) Whole Watershed

(1) The total time of concentration tc at outlet 0 should be the greater of the two times, tc' and tc", calculated below:

$$tc' = tc_1 + (A_2 \text{ channel flow time})$$

and

$$tc'' = tc_2$$

After substitution, tc' = 2000 + 2000 = 4000 s and tc" = 4000 s. In this case, the two calculated times are equal. In other cases, tc' and tc" may differ and this may produce, in combination with various values of A and C, the so-called partial area effects which represent one of the deficiencies of the rational method (see Section 8.3.2).

(2) The critical intensity for the whole watershed is calculated as

$$iM(tc,10) = iM(tc',10) = iM(tc_2,10) = 57,6 \text{ mm/h}$$

(3) The mean runoff coefficient for the whole watershed is calculated from Equation 8.2 as

$$C = (C_1\ A_1 + C_2\ A_2) / (A_1 + A_2) = [(0,3 \cdot 40) + (0,2 \cdot 60)]/(60 + 40) = 0,24$$

(4) The total peak runoff, $Qp_t(10)$, at outlet 0 is then calculated as

$$Qp_t(10) = 2,78 \cdot 10^{-3} \cdot 0,24 \cdot 57,6 \cdot 100 = 3,8 \text{ m}^3/\text{s}$$

The reader will note that Qp_t is greater than both Qp_1 and Qp_2, but smaller than ($Qp_1 + Qp_2$).

Case II - Post-development Conditions

For post-development conditions, applied to A1, A2 or the whole watershed, the calculations are conducted in a similar manner as for Case I, using the data from Table 8.1. The results of all calculations are given in Table 8.2.

Appendix C

RATIONAL METHOD CALCULATIONS FOR THE SEWER SYSTEM SHOWN IN FIGURE 8.2.

I - Given Data

(a) Physiographic characteristics are summarized in Table C.1 below.

Table C.1 Subcatchment and sewer characteristics

Sub-Catchment	Area (ha)	Runoff coefficient	Drained by sewer	Sewer length (m)	Sewer slope So
A 1	1,5	0,4	L 12	100	0,01
A 2	0,7	0,3	L 23	100	0,005
			L 54	100	0,01
A 4	1,7	0,5	L 43	100	0,02
A 5	1,0	0,4	L 37	100	0,005
A 6	2,0	0,3	L 67	100	0,005
A 7	1,0	0,6	L 78	100	0,005

(b) Other Information

(1) Rainfall intensities are described by the following IDF curve for the return period of 5 years:

$$iM\ (tc,5)\ (mm/h) = 250\ t^{-0,5}\ (min)$$

(2) For time in minutes, areas in hectares, peak runoffs in litres per second, F = 1 and after substituting for iM, the rational method equation can be written as

$$Qp(1/s) = 695 \cdot C \cdot A \cdot tc^{-0,5}$$

(3) The Manning formula for circular pipes flowing full is given by Equation 6.17 in the form

$$Q = \frac{0,31}{n} D^{8/3} S_e^{1/2}$$

where Q is the discharge in m^3/s, n is the Manning roughness coefficient (see Table 6.1 in Chapter 6), D is the pipe diameter (m) and S_e is the slope of the energy grade line (assumed equal to the pipe invert slope, S_o). For concrete pipes in good condition, n was taken as n = 0,0125 and the above equation is rewritten as

$$Q = 24,9\ D^{8/3}\ S_o^{1/2}$$

II - Calculations

The results of all calculations are summarized in Table C.2. In such calculations, no corrections for partially-filled pipes were made and flow velocities were calculated for theoretical pipe diameters. In practical design, such diameters would be rounded off upward to the nearest available standard size. Furthermore, as commonly done in practice, a constant inlet time (t_o = 4 min) was assumed throughout the sewer system. Samples of calculations follow.

(a) Peak Runoff at a Subcatchment Outlet

Subcatchment A1 - the inlet time was assumed as 4 minutes (see Table C.2). The concentration time tc_{A1} can be expressed as

$$tc_{A1} = to_1 + td_1$$

In this case, to_1 = 4 and td_1 = 0 (because the sewer system starts at point 1) and, consequently, tc_{A1} = 4 min.

For subcatchment A1, C_{A1} = 0,4 and A = 1,5 ha (see Table C.2).

Table C.2 Results of design calculations of the sewer system shown in Figure 8.2

Design point	Total area upstream (ha)	Mean runoff coefficient	to (min)	td (min)	tc (min)	Peak runoff (l/s)	Sewer trunk	Flow velocity (m/s)	Pipe diameter (m)
1	1,5	0,4	4	-	4	208	L12	1,7	0,394
2	2,2	0,368	4	1	5	252	L23	1,4	0,483
5	1,0	0,4	4	-	4	139	L54	1,5	0,339
4	2,7	0,463	4	1	5	388	L43	2,6	0,438
3	4,9	0,42	4	2,2	6,2	574	L37	1,7	0,657
6	2,0	0,3	4	-	4	208	L67	1,5	0,449
7	7,9	0,413	4	3,2	7,2	845	L78	1,5	0,759

The peak runoff at point 1, according to the equation given earlier in Section I(2) is

$$Qp_1(A_1) = 695 \cdot 0{,}4 \cdot 1{,}5 \cdot (4)^{-0{,}5} = 208 \text{ l/s}$$

Similarly, the peak runoff at the outlet of subarea A_2 is given as

$$Qp_2(A_2) = 695 \cdot 0{,}3 \cdot 0{,}7 \cdot (4)^{-0{,}5} = 73 \text{ l/s}$$

for concentration time tc_{A2} = 4 min, and C_{A2} = 0,3 and A_2 = 0,7 ha.

(b) Sewer Design

The peak runoff inflow at the upstream end of the sewer is used to calculate the sewer diameter. For example, sewer L12, between points 1 and 2, is designed for peak runoff Qp1 (A1) from subcatchment A1.

The pipe diameter can be calculated from the equation given earlier in Section I(3) as

$$D = (Q/24{,}9\ S_o^{1/2})^{3/8}$$

After substituting for L12,

$$D(L12) = (0{,}208/(24{,}9 \cdot 0{,}01^{1/2}))^{3/8} = 0{,}394 \text{ (m)}$$

For a full-pipe flow, the flow velocity in L12 is calculated as

$$V(L12) = 4Q/\pi D^2 = 4 \cdot 0{,}208/(3{,}14 \cdot 0{,}394^2) = 1{,}7 \text{ m/s}$$

The travel time in L12 is calculated as

$$td\ (L12) = Length\ (L12)/V(L12) = 100/1,70 = 58,6\ s = 1\ min$$

(c) Calculations at a Sewer Junction

First, the longest time of concentration has to be determined for the given junction. For example, at point 2, there are two travel times:

$$tc_2 = to_2 = tc_{A2} = 4\ min$$

and

$$tc_1 = to_1 + td_{1,2} = 4 + 1 = 5\ min$$

Consequently, the greater of both times, tc_1 is considered in further calculations. As mentioned earlier, sewer L12 was assumed flowing full, which is not exactly true, because for times tc_1's greater than the critical time of concentration of 4 minutes the peak runoff will decrease, the pipe will be only partially filled and the flow velocity will differ from the design velocity. Taking these effects into consideration results in a tedious iterative procedure as mentioned in Section 8.3.2.

Checking for the longest time of concentration is done at each junction in the sewer system. For example, at point 7, there are six different travel times to that point from the upstream branches.

For point 2, the mean runoff coefficient is calculated as

$$C_{m2} = (C_1\ A_2 + C_2\ A_2)/(A_1 + A_2) = 0,368,$$

the contributing area is given as

$$A_{T2} = A_1 + A_2 = 1,5 + 0,7 = 2,2\ ha,$$

and the peak runoff is calculated as

$$Qp_2(A_{T2}) = 695 \cdot 0,368 \cdot 2,2 \cdot (5)^{-0,5} = 251\ l/s$$

It should be noted that Qp2(AT2) is greater than both Qp1(A1) and Qp2(A2), but smaller than their sum. Peak runoff Qp2(AT2) will be used to design the diameter of sewer L2,3.