13. Urban Water Systems

13 Urban Water Systems

'Today, a simple turn of the tap provides clean water – a precious resource. Engineering advances in managing this resource – with water treatment, supply, and distribution systems – changed life profoundly in the twentieth century, virtually eliminating waterborne diseases in developed nations, and providing clean and abundant water for communities, farms, and industries'.

So states the US National Academy of Engineering on its selection of water supply systems to be among the five greatest achievements of engineering in the twentieth century. As populations continually move to urban areas for improved opportunities and a higher standard of living, and as cities merge to form megacities, the design and management of water supply systems serving these urban areas becomes an increasingly important part of regional integrated water resources planning and management.

1. Introduction

Urban water infrastructure typically includes water collection and storage facilities at source sites, water transport via aqueducts (canals, tunnels and/or pipelines) from source sites to water treatment facilities; water treatment, storage and distribution systems; wastewater collection (sewage) systems and treatment; and urban drainage works. This is illustrated as a simple schematic in Figure 13.1.

Generic simulation models of components of urban water systems have been developed and are commonly applied to study specific component design and operation issues. Increasingly, optimization models are being used to estimate cost-effective designs and operating policies. Cost savings can be substantial, especially when applied to large complex urban systems (Dandy and Engelhardt, 2001; Savic and Walters, 1997).

Most urban water users require high-quality water, and natural surface and/or groundwater supplies, called raw water, often cannot meet the quality requirements of domestic and industrial users. In such situations, water treatment is required prior to its use. Once it is treated, urban water can then be stored and distributed within the urban area, usually through a network of storage tanks and pipes. Pipe flows in urban distribution systems should be under pressure to prevent contamination from groundwaters and to meet various user and fire-protection requirements.

After use, the 'wastewater' is collected in a network of sewers, or in some cases ditches, leading to a wastewater treatment plant or discharge site. In many urban areas the sewage system has a dual function. The sewers collect both wastewater from households and the runoff from streets and roofs during storm events. However, the transport capacity of the sewer network and the treatment facilities are limited. During intense rainfall, overflows from the sewage system discharge a mixture of surface runoff and wastewater to the surface waters. This has a negative impact on the water quality of urban surface waters.

Wastewater treatment plants remove some of the impurities in the wastewater before it is discharged into receiving water bodies or on land surfaces. Water bodies receiving effluents from point sources such as wastewater treatment plants may also receive runoff from the surrounding watershed area during storm events. The pollutants in both point and non-point discharges will

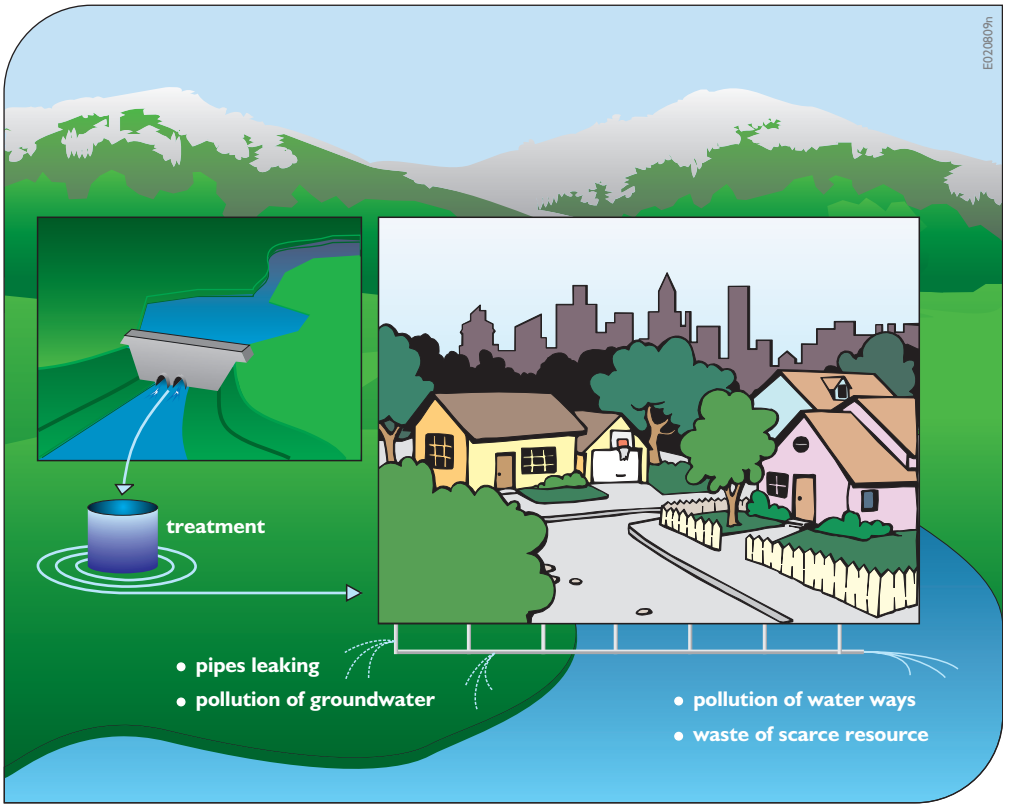

Figure 13.1. Schematic showing urban surface water source, water treatment prior to urban use, and some sources of non-point urban drainage and runoff and its impacts.

affect the quality of the water in those receiving water bodies. The fate and transport of these pollutants in these water bodies can be predicted by using water quality models similar to those discussed in Chapter 12.

This chapter briefly describes these urban water system components and reviews some of the general assumptions incorporated into optimization and simulation models used to plan urban water systems. The focus of urban water systems modelling is mainly on the prediction and management of quantity and quality of flows and pressure heads in water distribution networks, wastewater flows in gravity sewer networks, and on the design efficiencies of water and wastewater treatment plants. Other models can be used for the real-time operation of various components of urban systems.

2. Drinking Water

Drinking water issues include demand estimation, water treatment, and distribution.

2.1. Water Demand

A secure water supply is of vital importance for the health of the population and for the economy. Drinking water demand depends on:

- the number of inhabitants with access to drinking water
- meteorological and climatological conditions
- the price of drinking water
- the availability of drinking water
- an environmental policy that aims at moderate use of drinking water.

Table 13.1 shows an overview of the total annual water demand in various countries. The total water demand is sub-divided into domestic use and agricultural and industrial water use.

Drinking water demand for domestic use shows a daily and seasonal variation. There is no general formula for predicting drinking water demand. Drinking water suppliers tend to make predictions on the basis of their own experience and historical information about water demand in their region.

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country	demand m³/capita	domestic m³/capita	agriculture m³/capita	industrial m³/capita	year
Germany	490	67	2	389	1999
USA	1870	213	752	828	1990
Mexico	800	101	662	38	1999
Egypt	920	55	792	74	1993
Namibia	185	52	126	6	1990
China	439	22	338	78	1993
India	588	29	18	541	1990

source: OECD data compendium 2002

source: World Resources Institute

Table 13.1. Annual per-capita water demand in various countries in the world. Source: 1) OECD data compendium 2002 and 2) World Resources Institute.

2.2. Water Treatment

Before water is used for human consumption, its harmful impurities need to be removed. Communities that do not have adequate water treatment facilities, a common problem in developing regions, often have high incidences of disease and mortality due to drinking contaminated water. A range of syndromes, including acute dehydrating diarrhoea (cholera), prolonged febrile illness with abdominal symptoms (typhoid fever), acute bloody diarrhoea (dysentery) and chronic diarrhoea (Brainerd diarrhoea). Numerous health organizations point to the fact that contaminated water leads to over 3 billion episodes of diarrhoea and an estimated 2 million deaths, mostly among children, each year.

Contaminants in natural water supplies can also include microorganisms such as Cryptosporidium and Giardia lamblia as well as inorganic and organic cancer-causing chemicals (such as compounds containing arsenic, chromium, copper, lead and mercury) and radioactive material (such as radium and uranium). Herbicides and pesticides reduce the suitability of river water as a source of drinking water. Recently, traces of hormonal substances and medicines detected in river water are generating more and more concern.

To remove impurities and pathogens, a typical municipal water purification system involves a sequence of processes, from physical removal of impurities to chemical treatment. Physical and chemical removal processes include initial and final filtering, coagulation, flocculation, sedimentation and disinfection, as illustrated in the schematic of Figure 13.2.

As shown in Figure 13.2, one of the first steps in most water treatment plants involves passing raw water through coarse filters to remove sticks, leaves and other large solid objects. Sand and grit settle out of the water during this stage. Next a chemical such as alum is added to the raw water to facilitate coagulation. As the water is stirred, the alum causes the formation of sticky globs of small particles made up of bacteria, silt and other impurities. Once these globs of matter are formed, the water is routed to a series of settling tanks where the globs, or floc, sink to the bottom. This settling process is called flocculation.

After flocculation, the water is pumped slowly across another large settling basin. In this sedimentation or clarification process, much of the remaining floc and solid material accumulates at the bottom of the basin. The clarified water is then passed through layers of sand, coal and other granular material to remove microorganisms – including viruses, bacteria and protozoa such as Cryptosporidium – and any remaining floc and silt. This stage of purification mimics the natural filtration of water as it moves through the ground.

The filtered water is then treated with chemical disinfectants to kill any organisms that remain after the filtration process. An effective disinfectant is chlorine, but its use may cause potentially dangerous substances such as carcinogenic trihalomethanes.

Alternatives to chlorine include ozone oxidation (Figure 13.2). Unlike chlorine, ozone does not stay in the water after it leaves the treatment plant, so it offers

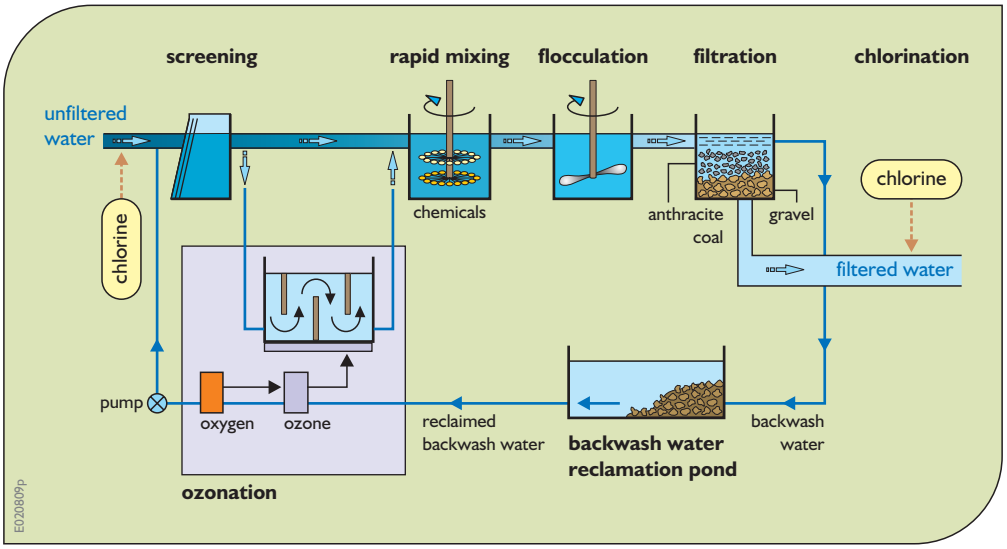

Figure 13.2. Typical processes in water treatment plants.

Figure 13.3. A 6-million gallon per day water treatment plant at San Luis Obispo, located about halfway between Los Angeles and San Francisco on the central coast of California.

no protection from bacteria that might be in the storage tanks and water pipes of the water distribution system. Water can also be treated with ultraviolet light to kill microorganisms, but this has the same limitation as oxidation: it is ineffective outside of the treatment plant.

Figure 13.3 is an aerial view of a water treatment plant serving a population of about 50,000.

Sometimes calcium carbonate is removed from drinking water in order to prevent it from accumulating in drinking water pipes and washing machines.

In arid coastal areas desalinated brackish or saline water is an important source of water for high-value uses. The cost of desalination is still high, but decreasing steadily. The two most common methods of desalination are distillation and reverse osmosis. Distillation requires more energy, while osmosis systems need frequent maintenance of the membranes.

2.3. Water Distribution

Water distribution systems include pumping stations, distribution storage and distribution piping. The hydraulic performance of each component depends upon the performance of the others. Of interest to designers are

both the flows and their pressures. Leakage of drinking water from the distribution system is a concern in many old drinking water systems.

The energy at any point within a network of pipes is often represented in three parts: the pressure head, p/γ, the elevation head, Z, and the velocity head, $V^2/2g$. (A more precise representation includes a kinetic energy correction factor, but that factor is small and can be ignored.) For open-channel flows, the elevation head is the distance from some datum to the top of the water surface. For pressure-pipe flow, the elevation head is the distance from some datum to the centre of the pipe. The parameter p is the pressure, for example Newtons per cubic metre (N/m^3), γ is the specific weight (N/m^2) of water, Z is the elevation above some base elevation (m), V is the velocity (m/s), and g is the gravitational acceleration (9.81 m/s^2).

Energy can be added to the system such as by a pump, or lost by, for example, friction. These changes in energy are referred to as head gains and losses. Balancing the energy across any two sites i and j in the system requires that the total heads, including any head gains H_G and losses H_L (m) are equal.

$$[p/\gamma + Z + V^2/2g + H_G]_{\text{site } i} = [p/\gamma + Z + V^2/2g + H_L]_{\text{site } j} \qquad (13.1)$$

The hydraulic grade is the sum of the pressure head and elevation head ($p/\gamma + Z$). For open-channel flow, the hydraulic grade is the water surface slope, since the pressure head at its surface is 0. For a pressure pipe, the hydraulic head is the height to which a water column would rise in a piezometer (a tube rising from the pipe). When plotted in profile along the length of the conveyance section, this is often referred to as the hydraulic grade line, or *HGL*. The hydraulic grade lines for open channels and pressure pipes are illustrated in Figures 13.4 and 13.5.

The energy grade is the sum of the hydraulic grade and the velocity head. This is the height to which a column of water would rise in a Pitot tube, but also accounts for fluid velocity. When plotted in profile, as in Figure 13.5, this is often referred to as the energy grade line, or *EGL*. At a lake or reservoir, where the velocity is essentially zero, the *EGL* is equal to the *HGL*.

Specific energy, E, is the sum of the depth of flow and the velocity head, $V^2/2g$. For open-channel flow, the depth of flow, y, is the elevation head minus the channel bottom elevation. For a given discharge, the specific energy is solely a function of channel depth. There may be more than one depth with the same specific energy. In one case the flow is subcritical (relatively higher depths, lower velocities) and in the other case the flow is critical (relatively lower depths and higher velocities). Whether or not the flow is above or below the critical depth (the depth that minimizes the specific energy) will depend in part on the channel slope.

Friction is the main cause of head loss. There are many equations that approximate friction loss associated with fluid flow through a given section of channel or pipe. These include Manning's or Strickler's equation, which is commonly used for open-channel flow, and Chezy's or Kutter's equation, Hazen–Williams equation, and

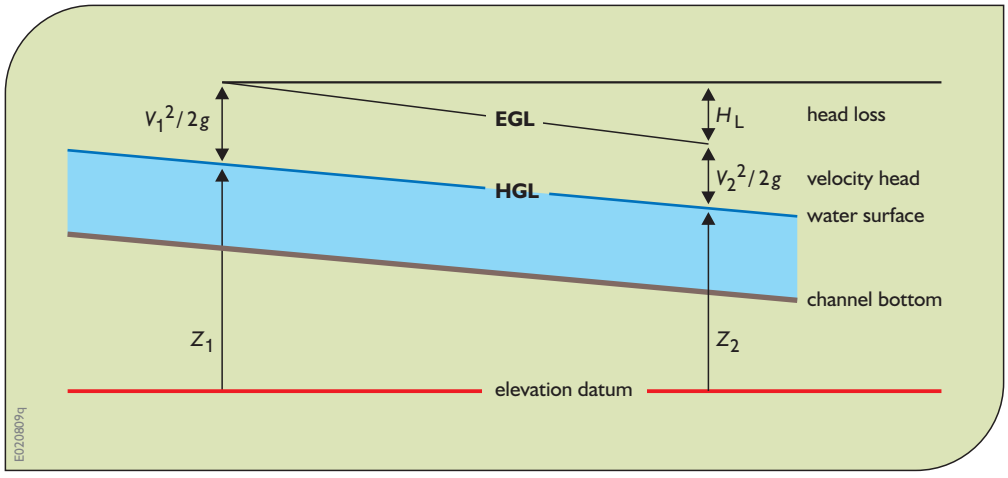

Figure 13.4. The energy components along an open channel.

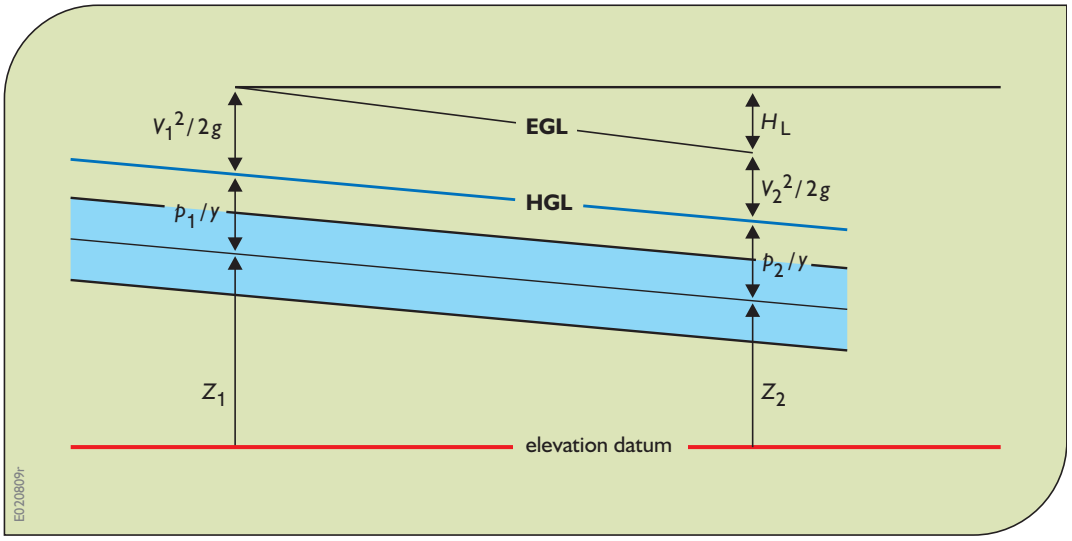

Figure 13.5. The energy components along a pressure pipe.

Darcy–Weisbach or Colebrook–White equations, which are used for pressure-pipe flow. They all define flow velocity, V (m/s), as an empirical function of a flow resistance factor, C, the hydraulic radius (cross-sectional area divided by wetted perimeter), R (m), and the friction or energy slope, $S = H_L$/Length.

$$V = kCR^xS^y \tag{13.2}$$

The terms k, x and y of Equation 13.2 are parameters. The roughness of the flow channel usually determines the flow resistance or roughness factor, C. The value of C may also be a function of the channel shape, depth and fluid velocity. Values of C for different types of pipes are listed in hydraulics texts or handbooks (e.g. Chin, 2000; Mays, 2000, 2005).

2.3.1. Open Channel Networks

For open-channel flow, Manning's or Strickler's equation is commonly used to predict the average velocity, V (m/s), and the flow, Q (m^3/s), associated with a given cross-sectional area, A (m^2). The velocity depends on the hydraulic radius R (m) and the slope S of the channel as well as a friction factor n.

$$V = (R^{2/3}S^{1/2})/n \tag{13.3}$$

$$Q = AV \tag{13.4}$$

The values of various friction factors n can be found in tables in hydraulics texts and handbooks.

The energy balance between two ends of a channel segment is defined in Equation 13.5. For open-channel flow the pressure heads are 0. Thus, for a channel containing water flowing from site i to site j:

$$[Z + V^2/2g]_{\text{site } i} = [Z + V^2/2g + H_L]_{\text{site } j} \tag{13.5}$$

The head loss H_L is assumed to be primarily due to friction.

The friction loss is computed on the basis of the average rate of friction loss along the segment and the length of the segment. This is the difference in the energy grade line elevations between sites i and j;

$$\begin{aligned} H_L &= (EGL_1 - EGL_2) \\ &= [Z + V^2/2g]_{\text{site } i} - [Z + V^2/2g]_{\text{site } j} \end{aligned} \tag{13.6}$$

The friction loss per unit distance along the channel is the average of the friction slopes at the two ends divided by the channel length. This defines the energy grade line, EGL.

2.3.2. Pressure Pipe Networks

The Hasen–Williams equation is commonly used to predict the flows or velocities in pressure pipes. Flows and velocities are again dependent on the slope, S, the hydraulic radius, R (m), (which equals half the pipe radius, r) and the cross-sectional area, A (m^2).

$$V = 0.849\ CR^{0.63}S^{0.54} \tag{13.7}$$

$$Q = AV = \pi r^2 V \tag{13.8}$$

The head loss along a length L (m) of pipe of diameter D (m) containing a flow of Q (m^3/s) is defined as

$$H_L = KQ^{1.85} \tag{13.9}$$

where K is the pipe coefficient defined by Equation 13.10.

$$K = [10.66\ L]/[C^{1.85}D^{4.87}] \tag{13.10}$$

Another pipe flow equation for head loss is the Darcy–Weisbach equation based on a friction factor f:

$$H_L = fLV^2/D\ 2g \tag{13.11}$$

The friction factor is dependent on the Reynolds number and the pipe roughness and diameter.

Given these equations, it is possible to compute the distribution of flows and heads throughout a network of open channels or pressure pipes. The two conditions are the continuity of flows at each node, and the continuity of head losses in loops for each time period t.

At each node i:

$$Storage_{it} + Q_{it}^{in} - Q_{it}^{out} = Storage_{i,t+1} \tag{13.12}$$

In each section between nodes i and j:

$$H_{Lit} = H_{Ljt} + H_{Lijt} \tag{13.13}$$

where the head loss between nodes i and j is H_{Lijt}.

To compute the flows and head losses at each node in Figure 13.6 requires two sets of equations, one for continuity of flows, and the other continuity of head losses. In this example, the direction of flow in two links, from A to C, and from B to C, are assumed unknown and hence each is represented by two non-negative flow variables.

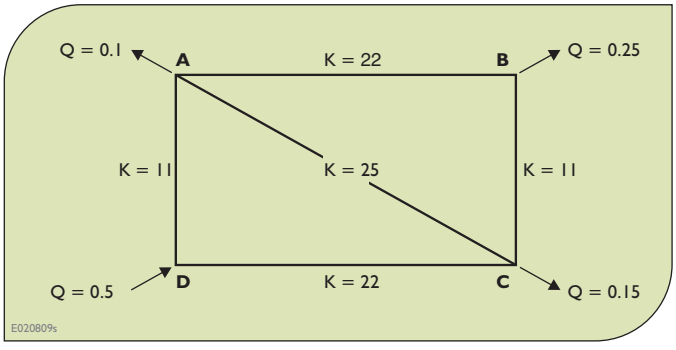

Figure 13.6. An example of a pipe network, showing the values of K for predicting head losses from Equation 13.10.

Let Q_{ij} be the flow from site i to site j and H_i be the head at site i. Continuity of flow in this network requires:

$$0.5 = Q_{DA} + Q_{DC} \tag{13.14}$$

$$0.1 = Q_{DA} - Q_{AC} + Q_{CA} - Q_{AB} \tag{13.15}$$

$$0.25 = Q_{AB} + Q_{CB} - Q_{BC} \tag{13.16}$$

$$0.15 = Q_{DC} + Q_{AC} - Q_{CA} + Q_{BC} - Q_{CB} \tag{13.17}$$

Continuity of heads at each node requires:

$$H_D = H_C + 22*(Q_{DC}^{1.85}) \tag{13.18}$$

$$H_D = H_A + 11*(Q_{DA}^{1.85}) \tag{13.19}$$

$$H_A = H_B + 22*(Q_{AB}^{1.85}) \tag{13.20}$$

$$H_C = H_A + 25*((Q_{CA} - Q_{AC})^{1.85}) \tag{13.21}$$

$$H_C = H_B + 11*((Q_{CB} - Q_{BC})^{1.85}) \tag{13.22}$$

Solving these Equations 13.14 to 13.22 simultaneously for the 5-flow and 4-head variables yields the flows Q_{ij} from nodes i to nodes j and heads H_i at nodes i listed in Table 13.2. Increasing H_D will increase the other heads accordingly.

The solution shown in Table 13.2 assumes no elevation heads, no storage capacity and no minor losses. Losses are usually expressed as a linear function of the velocity head, due to hydraulic structures (such as valves,

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Q_{DA}	=	0.29
Q_{DC}	=	0.21
Q_{AC}	=	0.07
Q_{CA}	=	0.00
Q_{AB}	=	0.12
Q_{CB}	=	0.13
Q_{BC}	=	0.00
H_A	=	0.43
H_B	=	0.00
H_C	=	0.26
H_D	=	1.52

Table 13.2. Flows and heads of the network shown in Figure 13.6.

restrictions or meters) at each node. This solution suggests that the pipe section between nodes *A* and *C* may not be economical, at least for these flow conditions. Other flow conditions may prove otherwise. But even if they do not, this pipe section increases the reliability of the system, and reliability is an important consideration in water supply distribution networks.

2.3.3. Water Quality

Many of the water quality models discussed in Chapter 12 can be used to predict water quality constituent concentrations in open channels and in pressure pipes. It is usually assumed that there is complete mixing, for example at junctions or in short segments of pipe. Reactions among constituents can occur as water travels through the system at predicted velocities. Water resident times (the ages of waters) in the various parts of the network are important variables for water quality prediction, as constituent decay, transformation and growth processes take place over time.

Computer models typically use numerical methods to find the hydraulic flow and head relationships as well as the resulting water quality concentrations. Most numerical models assume combinations of plug flow (advection) along pipe sections and complete mixing within segments of each pipe section at the end of each simulation time step. Some models also use Lagrangian approaches for tracking particles of constituents within a network. These methods are discussed in more detail in Chapter 12.

Computer programs (e.g. EPANET) exist that can perform simulations of the flows, heads and water quality behaviour within pressurized networks of pipes, pipe junctions, pumps, valves and storage tanks or reservoirs. These programs are designed to predict the movement and fate of water constituents within distribution systems. They can be used for many different kinds of application in distribution systems design, hydraulic model calibration, chlorine residual analysis and consumer exposure assessment. They can also be used to compare and evaluate the performance of alternative management strategies for improving water quality throughout a system. These can include:

- altering the sources within multiple source systems
- altering pumping and tank filling/emptying schedules
- use of satellite treatment, such as re-chlorination at storage tanks
- targeted pipe cleaning and replacement.

Computer models that simulate the hydraulic and water quality processes in water distribution networks must be run long enough for the system to reach equilibrium conditions, i.e. conditions not influenced by initial boundary assumptions. Equilibrium conditions within pipes are reached relatively quickly compared to those in storage tanks.

3. Wastewater

Wastewater issues include its production, its collection and its treatment prior to disposal.

3.1. Wastewater Production

Wastewater treatment plant influent is usually a mixture of wastewater from households and industries, urban runoff and infiltrating groundwater. The characterization of the influent, both in dry weather situations and during rainy weather, is of importance for the design and operation of the treatment facilities. In general, wastewater treatment plants can handle pure domestic wastewater better than diluted influent with low concentrations of pollutants. The discharge of urban runoff to the wastewater treatment plant dilutes the wastewater, thus affecting the treatment efficiency. The amount of infiltrating groundwater can also be significant in areas with old sewage systems.

3.2. Sewer Networks

Sewer flows and their pollutant concentrations vary throughout a typical day, a typical week, and over the seasons of a year. Flow conditions can range from free surface to surcharged flow, from steady to unsteady flow, and from uniform to gradually or rapidly varying non-uniform flow.

Urban drainage ditches normally have uniform cross sections along their lengths and uniform gradients. Because the dimensions of the cross sections are typically one or two orders of magnitude less than the lengths of the conduit, unsteady free-surface flow can be modelled using one-dimensional flow equations.

When modelling the hydraulics of flow it is important to distinguish between the speed of propagation of the

kinematic wave disturbance and the speed of the bulk of the water. In general the wave travels faster than the water particles. Thus if water is injected with a tracer, the tracer lags behind the wave. The speed of the wave disturbance depends on the depth, width and velocity of the flow.

Flood attenuation (or subsidence) is the decrease in the peak of the wave as it propagates downstream. Gravity tends to flatten, or spread out, the wave along the channel. The magnitude of the attenuation of a flood wave depends on the peak discharge, the curvature of the wave profile at the peak, and the width of flow. Flows can be distorted (changed in shape) by the particular channel characteristics.

Additional features of concern to hydraulic modellers are the entrance and exit losses to the conduit. Typically, at each end of the conduit is an access-hole. These are storage chambers that provide access to the conduits upstream and downstream. Access-holes induce some additional head loss.

Access-holes usually cause a major part of the head losses in sewage systems. An access-hole loss represents a combination of the expansion and contraction losses. For pressure flow, the head loss, H_L, due to contraction can be written as a function of the downstream velocity, V_D, and the upstream and downstream flow cross-sectional areas A_U and A_D:

$$H_L = K(V_D^2/2g)\,[1 - (A_D/A_U)]^2 \qquad (13.23)$$

The coefficient K varies between 0.5 for sudden contraction and about 0.1 for a well-designed gradual contraction.

An important parameter of a given open-channel conduit is its capacity: the flow that it can take without surcharging or flooding. Assuming normal depth flow where the hydraulic gradient is parallel to the bed of the conduit, each conduit has an upper limit to the flow that it can accept.

Pressurized flow is much more complex than free-surface flow. In marked contrast to the propagation speed of disturbances under free-surface flow conditions, the propagation of disturbances under pressurized flow in a 1 m circular conduit 100 m long can be less than a second. Some conduits can have the stable situation of free-surface flow upstream and pressurized flow downstream.

3.3. Wastewater Treatment

The wastewater generated by residences, businesses and industries in a community consists largely of water. It often contains less than 10% dissolved and suspended solid material. Its cloudiness is caused by suspended particles whose concentrations in untreated sewage range from 100 to 350 mg/l. One measure of the strength of the wastewater is its biochemical oxygen demand, or BOD_5. BOD_5 is the amount of dissolved oxygen aquatic microorganisms will require in five days as they metabolize the organic material in the wastewater. Untreated sewage typically has a BOD_5 concentration ranging from 100 mg/l to 300 mg/l.

Pathogens or disease-causing organisms are also present in sewage. Coliform bacteria are used as an indicator of disease-causing organisms. Sewage also contains nutrients (such as ammonia and phosphorus), minerals and metals. Ammonia can range from 12 to 50 mg/l and phosphorus can range from 6 to 20 mg/l in untreated sewage.

As illustrated in Figures 13.7 and 13.8, wastewater treatment is a multi-stage process. The goal is to reduce or remove organic matter, solids, nutrients, disease-causing organisms and other pollutants from wastewater before it is released into a body of water or on to the land, or is reused. The first stage of treatment is called preliminary treatment.

Preliminary treatment removes solid materials (sticks, rags, large particles, sand, gravel, toys, money, or anything people flush down toilets). Devices such as bar screens and grit chambers are used to filter the wastewater as it enters a treatment plant, and it then passes on to what is called primary treatment.

Clarifiers and septic tanks are generally used to provide primary treatment, which separates suspended solids and greases from wastewater. The wastewater is held in a tank for several hours, allowing the particles to settle to the bottom and the greases to float to the top. The solids that are drawn off the bottom and skimmed off the top receive further treatment as sludge. The clarified wastewater flows on to the next, secondary stage of wastewater treatment.

This secondary stage typically involves a biological treatment process designed to remove dissolved organic matter from wastewater. Sewage microorganisms cultivated

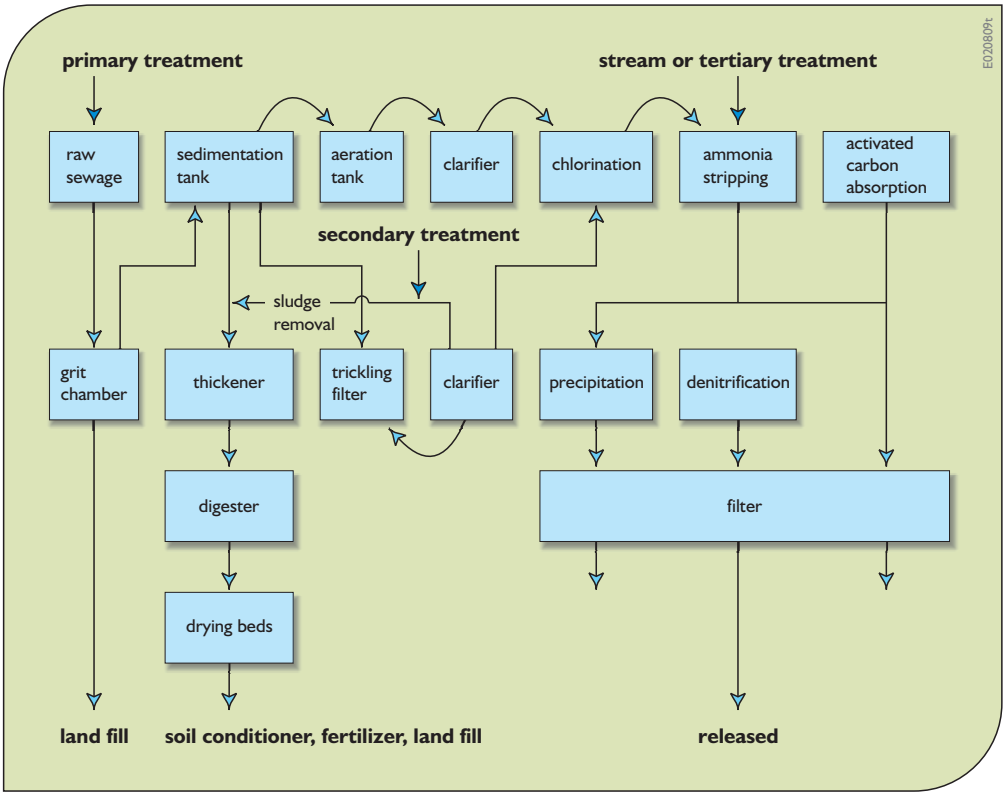

Figure 13.7. A typical wastewater treatment plant showing the sequence of processes for removing impurities.

Figure 13.8. Wastewater treatment plant in Soest, the Netherlands (Waterschap Vallei en Eem).

and added to the wastewater absorb organic matter from sewage as their food supply. Three approaches are commonly used to accomplish secondary treatment: fixed-film, suspended-film and lagoon systems.

Fixed-film systems grow microorganisms on substrates such as rocks, sand or plastic, over which the wastewater is poured. As organic matter and nutrients are absorbed from the wastewater, the film of microorganisms grows and thickens. Trickling filters, rotating biological contactors and sand filters are examples of fixed-film systems.

Suspended-film systems stir and suspend microorganisms in wastewater. As the microorganisms absorb organic matter and nutrients from the wastewater, they grow in size and number. After the microorganisms have been suspended in the wastewater for several hours, they are settled out as sludge. Some of the sludge is pumped back into the incoming wastewater to provide 'seed' microorganisms. The remainder is sent on to a sludge treatment process. Activated sludge, extended aeration, oxidation ditch and sequential batch reactor systems are all examples of suspended-film systems.

Lagoons, where used, are shallow basins that hold the wastewater for several months to allow for the natural degradation of sewage. These systems take advantage of natural aeration and microorganisms in the wastewater to renovate sewage.

Advanced treatment is necessary in some systems to remove nutrients from wastewater. Chemicals are sometimes added during the treatment process to help remove phosphorus or nitrogen. Some examples of nutrient removal systems are coagulant addition for phosphorus removal and air stripping for ammonia removal.

Final treatment focuses on removal of disease-causing organisms from wastewater. Treated wastewater can be disinfected by adding chlorine or by exposing it to sufficient ultraviolet light. High levels of chlorine may be harmful to aquatic life in receiving streams, so treatment systems often add a chlorine-neutralizing chemical to the treated wastewater before stream discharge.

Sludges are generated throughout the sewage treatment process. This sludge needs to be treated to reduce odours, remove some of the water and reduce volume, decompose some of the organic matter and kill disease-causing organisms. Following sludge treatment, liquid and cake sludges free of toxic compounds can be spread on fields, returning organic matter and nutrients to the soil.

Artificial wetlands and ponds are sometimes used for effluent polishing. In the wetlands the natural diurnal variation in the oxygen concentration is restored. Furthermore, artificial wetlands can reduce the nutrient content of the effluent by the uptake of nitrogen and phosphorus by algae or macrophytes. The organic matter may be harvested from the ponds and wetlands.

A typical model for the simulation of the treatment processes in wastewater treatment plants is the Activated Sludge Model (Gujer et al., 1999; Henze et al., 1999; Hvitved-Jacobsen et al., 1998). Activated sludge models predict the production of bacterial biomass and the subsequent conversion of organic matter and nutrients into sludge, CO_2 and N_2 gas.

4. Urban Drainage

Urban drainage involves:

- rainfall and surface runoff
- surface loading and washoff of pollutants
- stormwater sewer and pipe flow
- sediment transport
- structures and special flow characteristics
- separation of solids at structures
- outfalls.

These components or processes are briefly discussed in the following sub-sections.

4.1. Rainfall

Rainfall and the need to collect urban stormwater are the primary reasons for urban drainage systems. Storms are a major source of flow into the system. Even sanitary sewage systems that are claimed to be completely separate from storm drainage sewers are often influenced by rainfall through illicit connections or even infiltration.

Rainfall varies over time and space. These differences are normally small when considering short time periods and small distances, but they increase as time and distance increase. The ability to account for spatial differences in rainfall depends on the size of the catchment area and on the number of functioning rainfall recording points in the catchment. The use of radar permits more precision over space and time, rather as if more rain gauges were used and they were monitored more frequently. In practice, spatial effects are not measured at high resolution, and therefore events where significant spatial variations occur, such as summer thunderstorms, are usually not very accurately represented.

There are two categories of rainfall records: recorded (real) events and synthetic (not-real) events. Synthetic rainfall comes in two forms: as stochastically generated rainfall data and as design storms. The latter are derived from analyses of actual rainfall data and are used to augment or replace those historical (real) data.

Design events are a synthesized set of rainfall profiles that have been processed to produce storms with specific return periods; in other words, how often, on average, one can expect to observe rainfall events of that magnitude or greater. Design events are derived to reduce the number of runs needed to analyse system performance under design flow conditions.

4.1.1. Time Series Versus Design Storms

Professionals debate whether design rainfall is better represented by real rainfall or synthetic design events. The argument in favour of using synthetic storms is that they

are easy to use and require only a few events to assess the system performance. The argument in favour of a time series of real rainfall is that these data include a wider range of conditions, and therefore are likely to contain the conditions that are critical in each catchment.

The two methods are not contradictory. The use of real rainfall involves some synthesis in choosing which storms to use in a time series, and in adjusting them for use on a catchment other than the one where they were measured. Time series of rainfalls are generally used to look at aspects such as overflow spill frequencies and volumes. On the other hand, synthetic design storms can be generated for a wide range of conditions including the same conditions as represented by real rainfall. This is generally considered appropriate for looking at pipe network performance.

4.1.2. Spatial-Temporal Distributions

Rainfall varies in space as well as in time, and the two effects are related. Short storms typically come from small rain cells that have a short life, or that move rapidly over the catchment. As these cells are small (in the order of a kilometre in diameter), there is significant spatial variation in rainfall intensity. Longer storms tend to come from large rainfall cells associated with large weather systems. These have less spatial intensity variation.

Rainfall is generally measured at specific sites using rain gauges. The recorded rainfall amount and intensity will not be the same at each site. Thus, in order to use recorded rainfall data we need some way to account for this spatial and temporal variation. The average rainfall over the catchment in any period of time can be more or less than the measured values at one or more gauges. The runoff from a portion of a catchment exposed to a high-intensity rainfall will be more than the runoff from the same amount of rainfall spread evenly over the entire catchment.

4.1.3. Synthetic Rainfall

A convenient way of using rainfall data is to analyse long rainfall records to define the statistical characteristics of the rainfall, and then to use these statistics to produce synthetic rainstorms of various return periods and durations.

Three parameters are used to describe the statistics of rainfall depth.

- the rainfall intensity or depth of rain in a certain period
- the length of the period over which that intensity occurs
- the frequency with which it is likely to occur, or the probability of it occurring in any particular year.

In most of the work on urban drainage and river modelling, the risks of occurrence are expressed not by probabilities but by the inverse of probability, the return period. An event that has a probability of 0.2 of being equalled or exceeded each year has an expected return period of 1/0.2 or 5 years. An event having a probability of 0.5 of being equalled or exceeded has an expected return period of 1/0.5 = 2 years.

Rainfall data show an intensity–duration–frequency relationship. The intensity and duration are inversely related. As the rainfall duration increases, the intensity reduces. The frequency and intensity are inversely related, so that as the event becomes less frequent the intensity increases.

An important part of this duration–intensity relationship is the period of time over which the intensity is averaged. It is not necessarily the length of time for which it rained from start to finish. In fact any period of rainfall can be analysed for a large range of durations, and each duration could be assigned a different return period. The largest return period might be quoted as the 'return period of the storm', but it is only meaningful when quoted with its duration.

Intensity–duration–frequency relationships or depth–duration–frequency relationships, as shown in Figure 13.9, are derived by analysis of a long set of rainfall records. Intensity–duration–frequency data are commonly available all over the world and therefore it is important to be aware of how they are derived and ways they can be used for simulation modelling.

The depth of rainfall is the intensity times its duration integrated over the total storm duration.

4.1.4. Design Rainfall

Design rainfall events (hyetographs) for use in simulation models are derived from intensity–duration–frequency data.

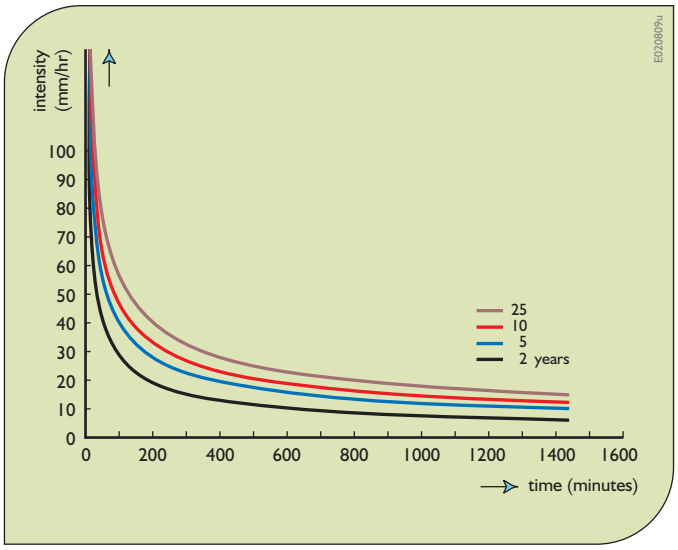

Figure 13.9. Rainfall intensity–duration–frequency (return period) curves.

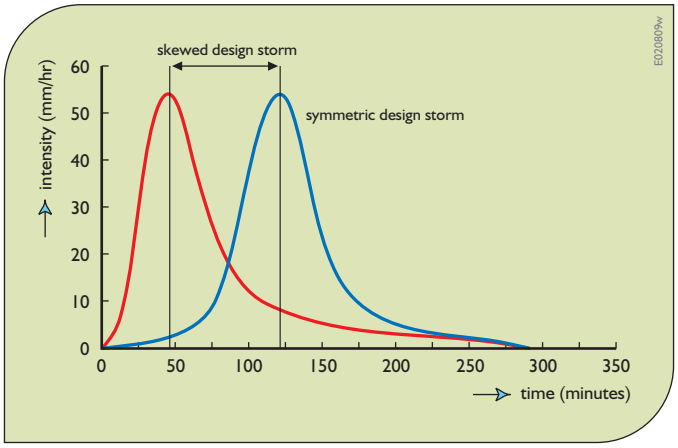

Figure 13.10. Storm peak skewness profiles.

The rainfall intensity during an event is not uniform in time, and its variation both in intensity and in the time when the peak intensity occurs during the storm can be characterized by the peakedness of the storm and the skew of the storm (Figure 13.10).

A design storm is a synthetic storm that has an appropriate peak intensity and storm profile.

4.2. Runoff

Runoff prediction is often the first step in obtaining stream and river flow data needed for various analyses. Often lack of detailed site-specific land cover, soil conditions and precipitation data as well as runoff processes limit the accuracy of runoff predictions.

4.2.1. Runoff Modelling

The runoff from rainfall involves a number of processes and events, as illustrated in Figure 13.11, and can be modelled using various methods. Most of these methods assume an initial loss, a continuing loss, and a remainder contributing to the system runoff.

Most models assume that the first part of a rainfall event goes to initial wetting of surfaces and filling of depression storage. The depth assumed to be lost is usually related to the surface type and condition. Rainwater can be intercepted by vegetation or can be trapped in depressions on the ground surface. It then either infiltrates into the ground and/or evaporates. Depression storage can occur on any surface, paved or otherwise.

Initial-loss depths are defined as the minimum quantity of rainfall causing overland runoff. The initial-loss depth of rainfall for catchment surfaces can be estimated as the intercept on the rainfall axis of plots of rainfall versus runoff (Figure 13.12). The runoff values shown in Figure 13.12 were obtained for various catchments in the United Kingdom (Price, 2002).

As rainfall increases, so does depression storage. The relationship between depression storage and surface slope S is assumed to be of the form aS^{-b}, where S is average slope of the sub-catchment and a and b are parameters between 0 and 1. The values of a and b depend in part on the surface type.

Evaporation, another source of initial loss, is generally considered to be relatively unimportant. For example, in the case of a heavy summer storm (25-mm rainfall depth) falling on hot asphalt (temperature say 60 °C falling to 20–30 °C as a result of sensible heat loss), a maximum evaporation loss of 1 mm is likely to occur.

Continuing losses are often separated into two parts: evapotranspiration and infiltration. These processes are usually assumed to continue throughout and beyond the storm event as long as water is available on the surface of the ground. Losses due to plant transpiration and general evaporation are not particularly an issue for single events, but can be during the inter-event periods where catchment drying takes place. This is applicable to models where time-series data are used and generated.

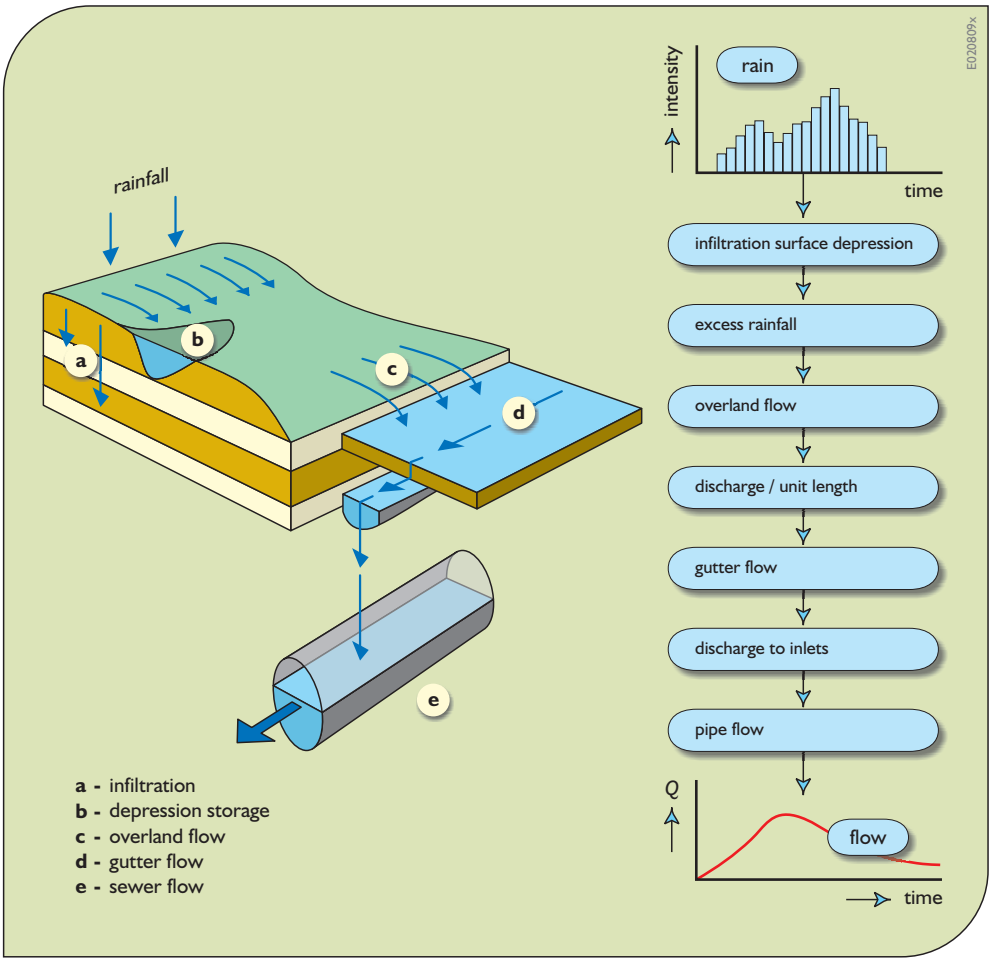

Figure 13.11. Schematic representation of urban rainfall–runoff processes.

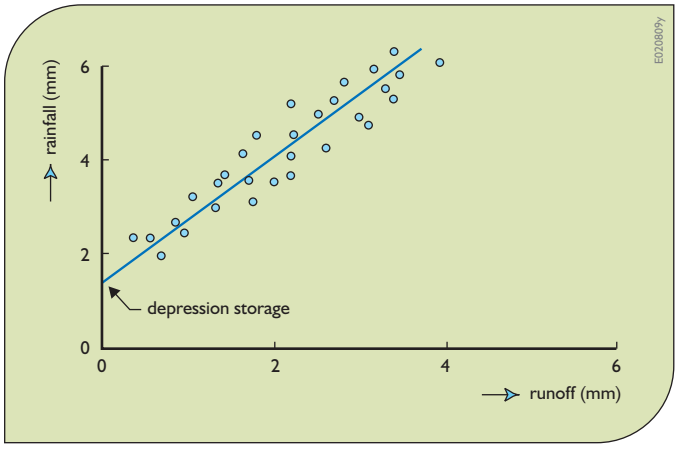

Figure 13.12. Estimation of depression storage based on data from catchments in the UK (Price, 2002).

Infiltration is usually assumed to account for the remaining rainfall that does not enter into the drainage system. The proportion of this loss can range from 100% for very permeable surfaces to 0% for completely impermeable surfaces.

Many models try to account for the wetting of the catchment and the increasing runoff that takes place as wetting increases. The effect of this is shown in Figure 13.13.

It is often impractical to take full account of the variability in urban topography and surface conditions. Fortunately for modellers, impervious (paved) surfaces are often dominant in an urban catchment, and the loss of rainfall prior to runoff is usually relatively small in periods with much rainfall.

Runoff-routing is the process of passing rainfall across the surface to enter the drainage network. This process results in attenuation and delay. These parameters are modelled by means of routing techniques that generally consider catchment area size, ground slope and rainfall intensity in determining the flow rate into the network.

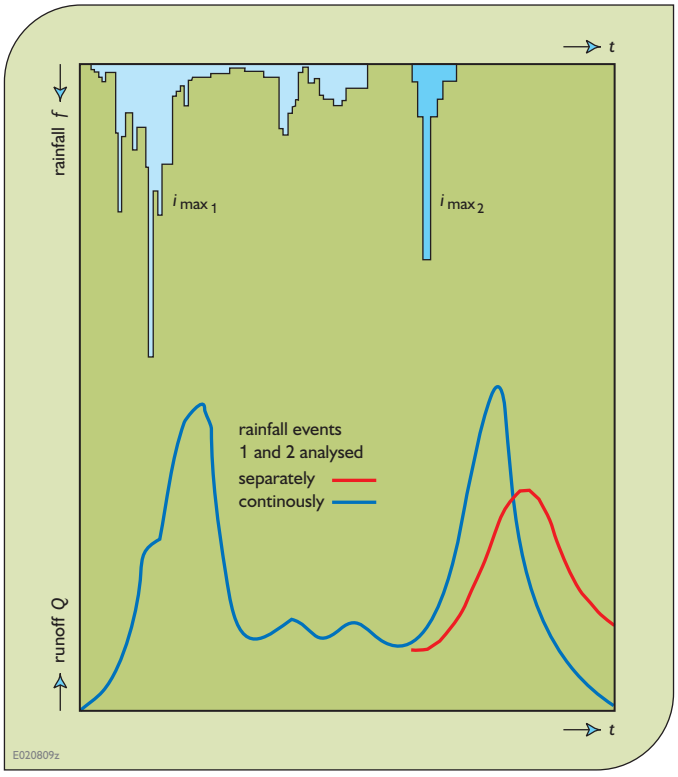

Figure 13.13. Effect of catchment wetness on runoff *Q* over time *t* (Price, 2002).

The topography and surface channels, and even upstream parts of the sewage system, are usually lumped together in this process and are not explicitly described in a model. The runoff-routing process is often linked to catchment surface type, and empirical calibration factors are used accordingly.

A number of models for rainfall–runoff and runoff-routing are available and are used in various parts of the world. Overland runoff on catchment surfaces can be represented by the kinematic wave equation. However, direct solution of this equation in combination with the continuity equation has not been a practical approach when applied to basins with a large number of contributing sub-catchments. Simpler reservoir-based models, which are less computationally and data demanding, represent the physical processes almost as accurately as the more complex physically based approaches (Price, 2002). In practice, models applied to catchments typically assume an average or combined behaviour of a number of overland flow surfaces, gutters and feeder pipes. Therefore, the parameters of a physically based approach (for example the roughness value) as applied would not relate directly to parameters representative of individual surfaces and structures.

Many overland flow-routing models are based on a linear reservoir-routing concept. A single reservoir model assumes the outflow, $Q(t)$ (m^3/s), at the catchment outlet is proportional to the volume of stormwater, $S(t)$ (m^3), present on the ground surface of that catchment, including the non-explicitly modelled network that contributes stormwater to that outlet point of the urban drainage system. To take into account the effects of depression storage and other initial losses, the first millimetre(s) of rainfall may not contribute to the runoff.

The basic equation for runoff $Q(t)$ (m^3/s) at time t is:

$$Q(t) = S(t)\,K \tag{13.24}$$

where K is a linear reservoir coefficient. This coefficient is sometimes a function of the catchment slope, area, length of longest sewer and rainfall intensity. For a two linear-reservoir model, two reservoirs are applied in series for each surface type with an equivalent storage – output relationship, as defined by Equation 13.24, for each reservoir.

The simplest models rely on fixed runoff coefficients K. They best apply to impervious areas where antecedent soil moisture conditions are not a factor.

Typical values for runoff coefficients are given in Table 13.3 (Price, 2002). Use of these coefficients should be supported either by field observations or by expert judgement.

4.2.2. The Horton Infiltration Model

The Horton model describes the increasing runoff from permeable surfaces while a rainfall event occurs by keeping track of decreasing infiltration as the soil moisture content increases. The runoff from paved surfaces is assumed to be constant while the runoff from permeable surfaces is a function of the conceptual wetting and infiltration processes.

On the basis of infiltrometer studies on small catchments, Horton defined the infiltration rate, f, either on pervious surfaces or on semi-pervious surfaces, as a function of time, t (hours), the initial infiltration rate, f_o (mm/hr), the minimum (limiting or critical) infiltration rate, f_c (mm/hr), and an infiltration rate constant, k (1/hr).

$$f = f_c + (f_o - f_c)e^{-kt} \tag{13.25}$$

Table 13.3. Typical values of the runoff fraction (coefficient *K*).

surface type	description	coefficient *K*
paved	high quality paved roads with gullies < 100m apart	1.00
paved	high quality paved roads with gullies > 100m apart	0.90
paved	medium quality paved roads	0.85
paved	poor quality paved roads	0.80
permeable	high to medium density housing	0.55 - 0.45
permeable	low density housing or industrial areas	0.35
permeable	open areas	0.00 - 0.25

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The minimum or limiting infiltration rate, f_c, is commonly set to the saturated groundwater hydraulic conductivity for the applicable soil type.

The integration of Equation 13.25 over time defines the cumulative infiltration $F(t)$:

$$F(t) = f_c t + (f_o - f_c)(1 - e^{-kt})/k \qquad (13.26)$$

The Horton equation variant as defined in Equation 13.26 represents the potential infiltration depth, F, as a function of time, t, assuming the rainfall rate is not limiting, i.e. it is higher than the potential infiltration rate. Expressed as a function of time, it is not suited for use in a continuous simulation model. The infiltration capacity should be reduced in proportion to the cumulative infiltration volume, F, rather than in proportion to time. To do this, Equation 13.26 may be solved iteratively to find the time it takes to cause ponding, t_p, as a function of F. That ponding time t_p is used in Equation 13.25 to establish the appropriate infiltration rate for the next time interval (Bedient and Huber, 1992). This procedure is used, for example, in the urban stormwater management model (SWMM) (Huber and Dickinson, 1988).

A flow chart of the calculations performed in a simulation program in which the rainfall can vary might be as shown in Figure 13.14.

Various values for Horton's infiltration model are available in the published literature. Values of f_o and f_c, as determined by infiltrometer studies, Table 13.4, are highly variable, even by an order of magnitude on seemingly similar soil types. Furthermore, the direct transfer of values as measured on rural catchments to urban catchments is not advised due to the compaction and vegetation differences associated with the latter surfaces.

4.2.3. The US Soil Conservation Method (SCS) Model

The SCS model (USDA, 1972) is widely used, especially in the United States, France, Germany, Australia and parts of Africa, for predicting runoff from rural catchments. It has also been used for the permeable component in a semi-urban environment. This runoff model allows for variation in runoff depending on catchment wetness. The model relies on what are called curve numbers, *CN*.

The basis of the method is the continuity equation. The total depth (mm) of rainfall, R, either evaporates or is otherwise lost, I_a, infiltrates and is retained in the soil, F, or runs off the land surface, Q. Thus,

$$R = I_a + F + Q \qquad (13.27)$$

The relationship between the depths (mm) of rainfall, R, runoff, Q, the actual retention, F, and the maximum potential retention storage, S (not including I_a), is assumed to be

$$F/S = Q/(R - I_a) \qquad (13.28)$$

when $R > I_a$. These equations combine to give the SCS model

$$Q = (R - I_a)^2/(R - I_a + S) \qquad (13.29)$$

This model can be modified for use in continuous simulation models.

Numerical representation of the derivative of Equation 13.29 can be written as Equation 13.30 for predicting

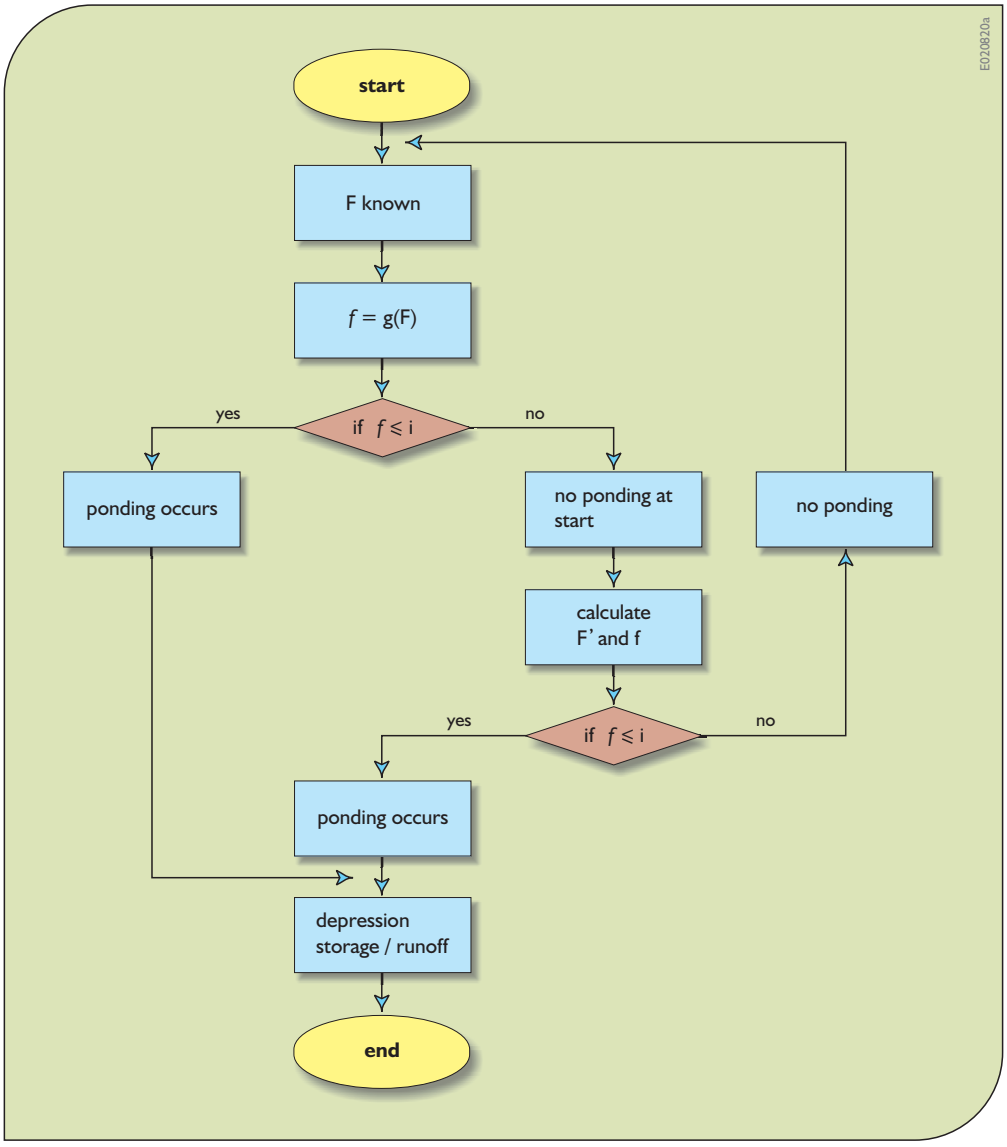

Figure 13.14. Flow chart of Horton model infiltration algorithm used in each time step of a simulation model.

SCS soil group	f_o (mm/hr)	f_c (mm/hr)	k (1 / hr)
A	250	25.4	2
B	200	12.7	2
C	125	6.3	2
D	76	2.5	2

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Table 13.4. Values for Horton's infiltration model for different soil groups as defined by the Soil Conservation Service. Note: see Table 13.6 for SCS soil groups.

the runoff, q (mm/Δt), over a time interval Δt given the rainfall r (mm/Δt), in that time interval.

$$q = r(R - I_a)(R - I_a + 2S)/(R - I_a + S)^2 \qquad (13.30)$$

This equation is used incrementally, enabling the rainfall and runoff coefficients, r and q, to change during the event.

The two parameters S and I_a are assumed to be linearly related by

$$I_a = kS \qquad (13.31)$$

where $0 < k < 0.2$.

The original SCS approach recommended $k = 0.2$. However other studies suggest k values between 0.05 and 0.1 may be more appropriate.

The storage variable, S, is itself related to an index known as the runoff curve number, CN, representing the combined influence of soil type, land management practices, vegetation cover, urban development and antecedent moisture conditions on hydrological response. CN values vary between 0 and 100, 0 representing no runoff and 100 representing 100% runoff.

The storage parameter S is related to the curve number CN by

$$S = (25400/CN) - 254 \qquad (13.32)$$

Curve number values depend on antecedent moisture condition classes (AMC) and hydrological soil group. The antecedent moisture conditions are divided into three classes, as defined in Table 13.5.

The four hydrological soil groups are defined in Table 13.6.

The CN value can either be defined globally for the catchment model or can be associated with specific surface types. CN values for different conditions are available from various sources. Table 13.7 lists some of those relevant to urban areas and antecedent moisture condition class AMC II.

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AMC	total 5-day antecedent rainfall (mm) season: dormant	growing
I	< 12.5	< 35.5
II	12.5 - 28.0	35.5 - 53.5
III	> 28.0	> 53.5

Table 13.5. Antecedent moisture condition classes (AMC) for determining curve numbers *CN*.

Figure 13.15 identifies the CN values for antecedent moisture condition classes (AMC) I and III based on class II values.

4.2.4. Other Rainfall–Runoff Models

The rainfall–runoff element of the popular SWMM model generally assumes 100% runoff from impermeable surfaces and uses the Horton or the Green–Ampt model

Table 13.6. SCS hydrological soil groups used in Tables 13.4 and 13.7.

soil type	definition
A	(low runoff potential) high infiltration rates even when thoroughly wetted. chiefly deep, well to excessively drained sands or gravels. high rate of water transmission.
B	moderate infiltration rates when thoroughly wetted. chiefly moderately deep to deep, moderately-well to well drained soils with moderately-fine to moderately-coarse textures. moderate rate of water transmission.
C	slow infiltration rates when thoroughly wetted. chiefly solids with layer that impedes downward movement of water, or soils with a moderately-fine to fine texture.sslow rate of water transmission.
D	(high runoff potential) very slow infiltration and transmission rates when thoroughly wetted. chiefly: • clay soils with a high swelling potential • soils with a permanent high water table • soils with a clay pan or clay layer at or near the surface • shallow soils over nearly impervious material

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land cover class	land cover / landuse/treatment	stormflow potential	hydrological soil group A	B	C	D
urban & sub-urban land use	open spaces, parks, cemeteries	75 % grass	39	61	74	80
	open spaces, parks, cemeteries	75 % grass	49	69	79	84
	commercial / business area	**% area impervious**	**A**	**B**	**C**	**D**
	industrial districts	85 %	89	92	94	95
	residential: lot size 500 m^2	72 %	81	88	91	93
	1000 m^2	65 %	77	85	90	92
	1350 m^2	38 %	61	75	83	87
	2000 m^2	30 %	57	72	81	86
	4000 m^2	25 %	54	70	80	85
	paved parking lots, roofs, etc.	20 %	51	68	78	84
	streets/roads: tarred, with storm sewers, curbs		98	98	98	98
	gravel		76	85	89	91
	dirt		72	82	87	89
	dirt-hard surface		74	84	90	92

Table 13.7. Initial *CN* values for AMC II with various urban land use, cover quality and hydrological soil groups (defined in Table 13.6).

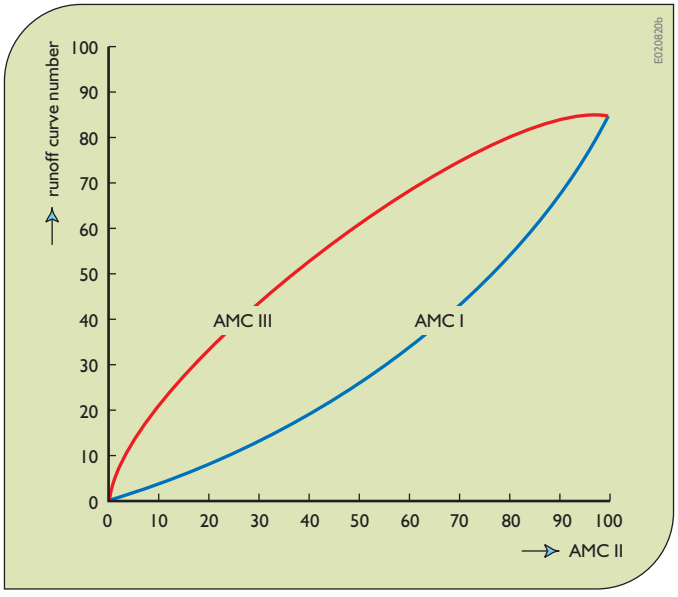

Figure 13.15. Runoff curve numbers for AMC classes I and III based on curve numbers for AMC II.

for permeable runoff. The Green–Ampt model is similar to the Horton model in that it has a conceptual infiltration rate that varies with time. It is therefore applicable to pervious or semi-pervious catchments (Huber and Dickinson, 1988; Roesner et.al., 1988).

4.3. Surface Pollutant Loading and Washoff

The modelling of surface pollutant loading and washoff into sewage systems is very imprecise. Pollutants that build up on the surface of an urban area originate from wind-blown dust, debris that is both natural and human-made, including vehicle emissions. When rainfall takes place, some of this material is washed into the stormwater sewers or gullies as dissolved pollutants and fine solids. During buildup time, many of the pollutants degrade.

Deposition of this material is not homogeneous but rather is a function of climate, geography, land use

and human activity. The mechanism of washoff is obviously a function of location, rainfall intensity, slope, flow rate, vehicle disturbance and so on. None of these factors is explicitly modelled in most washoff and sewer flow models.

Measurements made of pollutant accumulation and washoff have been the basis of empirical equations representing both loading and washoff processes. In practice the level of information available and the complexity of the processes being represented make the models of pollutant loading and washoff a tool whose outputs must be viewed for what they are – merely guesses.

4.3.1. Surface Loading

Pollutant loadings and accumulation on the surface of an urban catchment occur during dry periods between rainstorms.

A common hypothesis for pollutant accumulation during dry periods is that the mass loading rate, m_P (kg/ha/day), of pollutant P is constant. This assumed constant loading rate on the surface of the ground can vary over space and is related to the land use of that catchment. In reality, these loadings on the land surface will not be the same, either over space or over time. Hence, to be more statistically precise, a time series of loadings may be created from one or more probability distributions of observed loadings. (Just how this may be done is discussed in Chapter 7.) Different probability distributions may apply when, for example, weekend loadings differ from workday loadings. However, given all the other uncertain assumptions in any urban loading and washoff model, the effort may not be justified.

As masses of pollutants accumulate over a dry period they may degrade as well. The time rate of degradation of a pollutant P is commonly assumed to be proportional to its total accumulated mass M_P(kg/ha). Assuming a proportionality constant (decay rate constant) of k_P (1/day), the rate of change in the accumulated mass M_P over time t is

$$dM_P/dt = m_P - k_P M_P \qquad (13.33)$$

As the number of days during a dry period becomes very large, the limiting accumulation of a mass M_P of pollutant P is m_P/k_P. If there is no decay, then of course k_P is 0 and the limiting accumulation is infinite.

Integrating Equation 13.33 over the duration Δt (days) of a dry period yields the mass, $M_P(\Delta t)$ (kg/ha), of each pollutant available for washoff at the beginning of a rainstorm.

$$M_P(\Delta t) = M_P(0)e^{-kp\Delta t} + [m_P(1 - e^{-kp\Delta t})/k_P] \qquad (13.34)$$

where $M_P(0)$ is the initial mass of pollutant P on the catchment surface at the beginning of the dry period (that is, at the end of the previous rainstorm).

Sediments (which become suspended solids in the runoff) are among the pollutants that accumulate on the surface of urban catchments. They are important in themselves, but also because some of the other pollutants that accumulate become attached to them. Sediments are typically defined by their medium diameter size value (d_{50}). Normally a minimum of two sediment fractions are modelled: one coarse, high-density material (grit) and one fine (organics).

The sediments of each diameter size class are commonly assumed to have a fixed amount of pollutants attached to them. The fraction of each attached pollutant, sometimes referred to as the potency factor of the pollutant, is expressed as kg of pollutant per kg of sediment. Potency factors are one method for defining pollutant inputs into the system.

4.3.2. Surface Washoff

Pollutants in the washoff may be dissolved in the water, or be attached to the sediments. Many models of the transport of dissolved and particulate pollutants through a sewerage system assume each pollutant is conservative (that is, it does not degrade over time). For practical purposes this is a reasonable assumption when the time of flow in the sewers is relative short. Otherwise it may not be a good assumption, but at least it is a conservative one.

Pollutants can enter the sewage system from a number of sources. A major source is the washoff of pollutants from the catchment surface during a rainfall event. Their removal is caused by the impact of rain and by erosion from runoff flowing across the surface. Figure 13.16 shows schematically some sources of pollution in the washoff model.

The rate of pollutant washoff depends on an erosion coefficient, α_P, and the quantity, M_P, of available pollutant, P. As the storm event proceeds and pollutants

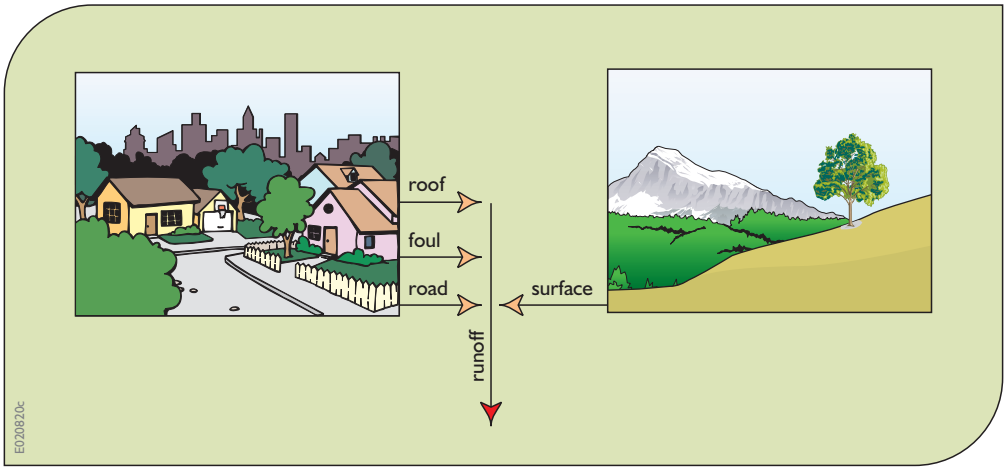

Figure 13.16. Some sources of pollutants in the washoff to sewage systems from land surfaces.

are removed from the catchment, the quantities of available pollutants decrease, hence the rate of pollutant washoff decreases even with the same runoff.

When runoff occurs, a fraction of the accumulated load may be contained in it. This fraction will depend on the extent of runoff. If a part of the surface loading of a pollutant is attached to sediments, its runoff will depend on the amount of sediment runoff, which in turn is dependent on the amount of surface water runoff.

The fraction of total surface loading mass contained in the runoff will depend on the runoff intensity. The following approximate relation may apply for the fraction, f_t^R, of pollutant P in the runoff R_t in period t:

$$f_t^P = \alpha_P R_t/(1 + \alpha_P R_t) \tag{13.35}$$

The greater the runoff R_t the greater will be the fraction f_t^P of the total remaining pollutant loading in that runoff. The values of the parameters α_P are indicators of the effectiveness of the runoff in picking up and transporting the particular pollutant mass. Their values are dependent on the type of pollutant P and on the land cover and topography of particular basin or drainage area. They can be determined on the basis of measured pollutant mass surface loadings and on the mass of pollutants contained in the rainfall and sediment runoff, preferably at the basin of interest. Since such data are difficult, or at least expensive, to obtain, they are usually based on experiments in laboratories.

A mass balance of pollutant loadings can define the total accumulated load, M_{Pt+1}, at the end of each simulation time period t or equivalently at the beginning of each time period $t+1$. Assuming a daily simulation time step,

$$M_{Pt+1} = (1 - f_t^P)\, M_{Pt}\, e^{-kP} + m_P \tag{13.36}$$

Of interest, of course, is the total pollutant mass in the runoff. For each pollutant type P in each period t, these will be $f_t^P(M_{Pt})$ for the dissolved part. The total mass of pollutant P in the runoff must also include those attached fractions (potency factors), if any, of each sediment size class being modelled as well.

As the sediments are routed through the system, those from different sources are mixed together. The concentrations of associated pollutants therefore change during the simulation as different proportions of sediment from different sources are mixed together. The results are given as concentrations of sediment, concentrations of dissolved pollutants and concentrations of pollutants associated with each sediment fraction.

4.3.3. Stormwater Sewer and Pipe Flow

Flows in pipes and sewers have been analysed extensively and their representation in models is generally accurately defined. The hydraulic characteristics of sewage are essentially the same as clean water. Time-dependent effects are, in part, a function of the change in storage in access-holes. Difficulties in obtaining convergence occur at pipes with steep to flat transitions, dry pipes and the like, and additional features and checks are therefore needed to achieve satisfactory model results.

4.3.4. Sediment Transport

Pollutant transport modelling of both sediment and dissolved fractions involves defining the processes of erosion and deposition, and advection and possibly dispersion. One-dimensional models by their very nature cannot predict the sediment gradient in the water column. In addition, the concept of the sewer being a bio-reactor is not included in most simulation models. Most models assume pollutants are conservative while in residence in the drainage system before being discharged into a water body. All the processes that take place in transit are generally either ignored or approximated using a range of assumptions.

4.3.5. Structures and Special Flow Characteristics

Access-holes, valves, pipes, pumping stations, overflow weirs and other elements that affect the flows and head losses in sewers can be explicitly included in deterministic simulation models. The impact of some of these structures can only be predicted using two- or three-dimensional models. However, the ever-increasing power of computers is making higher-dimensional fluid dynamic analyses increasingly available to practising engineers. The greatest limitation may be more related to data and calibration than to computer models and costs.

4.4. Water Quality Impacts

The quality of water in urban drainage and sewer systems can impact the flow conditions in those systems as well as the quality of the water into which these wastewater and drainage flows are discharged.

4.4.1. Slime

Slime can build up on the perimeters of sewers that contain domestic sewage. The buildup of slime may have a significant effect on roughness. In a combined system the effect will be less, as the maximum daily flow of domestic sewage will not usually be a significant part of pipe capacity.

The extent to which the roughness is increased by sliming depends on the relation between the sewage discharge and the pipe capacity. Sliming will occur over the whole of the perimeter below the water level that corresponds to the maximum daily flow. The slime growth will be heaviest in the region of the maximum water level. Over the lower part of the perimeter, the surface will still be slimed, but to a lesser extent than at the waterline. Above the maximum waterline the sewer surface will tend to be fairly free of slime.

4.4.2. Sediment

When sediment is present in the sewer, the roughness increases quite significantly. It is difficult to relate the roughness to the nature and time-history of the sediment deposits. Most stormwater sewers contain some sediment deposits, even if only temporarily. The only data available suggest that the increase in head loss can range from 30 to 300 mm, depending on the configuration of the deposit and on the flow conditions. The higher roughness value is more appropriate when the sewer is part full and when considerable energy is lost as a result of the generation of surface disturbances. In practice lower roughness values are used for flow states representing extreme events when sewers are operating in surcharge.

4.4.3. Pollution Impact on the Environment

The effects of combined sewer overflows (*CSO*s) or discharges are particularly difficult to quantify and regulate because of their intermittent and varied nature. Their immediate impact can only be measured during a spill event, and their chronic effects are often difficult to isolate from other pollution inputs. Yet they are among the major causes of poor river water quality. Standards and performance criteria specifically for intermittent discharges are therefore needed to reduce the pollutants in *CSO*s.

Drainage discharges that affect water quality can be divided into four groups:

- Those that contain oxygen-demanding substances. These can be either organic, such as fecal matter, or inorganic. (Heated discharges, such as cooling waters, reduce the saturated concentration of dissolved oxygen.)
- Those which contain substances that physically hinder re-oxygenation at the water surface, such as oils.
- Discharges containing toxic compounds, including ammonia, pesticides and some industrial effluents.
- Discharges that are high in suspended solids and thus inhibit biological activity by excluding light from the water or by blanketing the bed.

Problems arise when pollutant loads exceed the self-purification capacity of the receiving water, harming aquatic life and restricting the use of the water for consumption and for many industrial and recreational purposes. The assimilative capacity for many toxic substances is very low. Water polluted by drainage discharges can create nuisances such as unpleasant odours. It can also be a direct hazard to health, particularly in tropical regions where water-borne diseases such as cholera and typhoid are prevalent.

The aim of good drainage design, with respect to pollution, is to balance the effects of continuous and intermittent discharges against the assimilation capacity of the water, so as to achieve in a cost-effective way the desired quality of the receiving water.

Figure 13.17 shows the effect of a discharge that contains suspended solids and organic matter. The important indicators showing the effect of the discharge are the dissolved oxygen in diagram 'a' and the clean water fauna shown in diagram 'd'. The closeness with which the clean water fauna follow the dissolved oxygen reflects the reliance of a diverse fauna population on dissolved oxygen. These relationships are used by biologists to argue for greater emphasis on biological indicators of pollution, as these respond to intermittent discharges better than chemical tests which, if not continuous, may miss the pollution incident. There are a number of biological indexes in use in most countries in Europe.

In Figure 13.17 the *BOD* in diagram 'a' rises or is constant after release despite some of the pollutant being digested and depleting the dissolved oxygen. This is because there is a time lag of up to several days while the bacteria, which digest the pollutant, multiply. The suspended solids (*SS*) settle relatively quickly and they

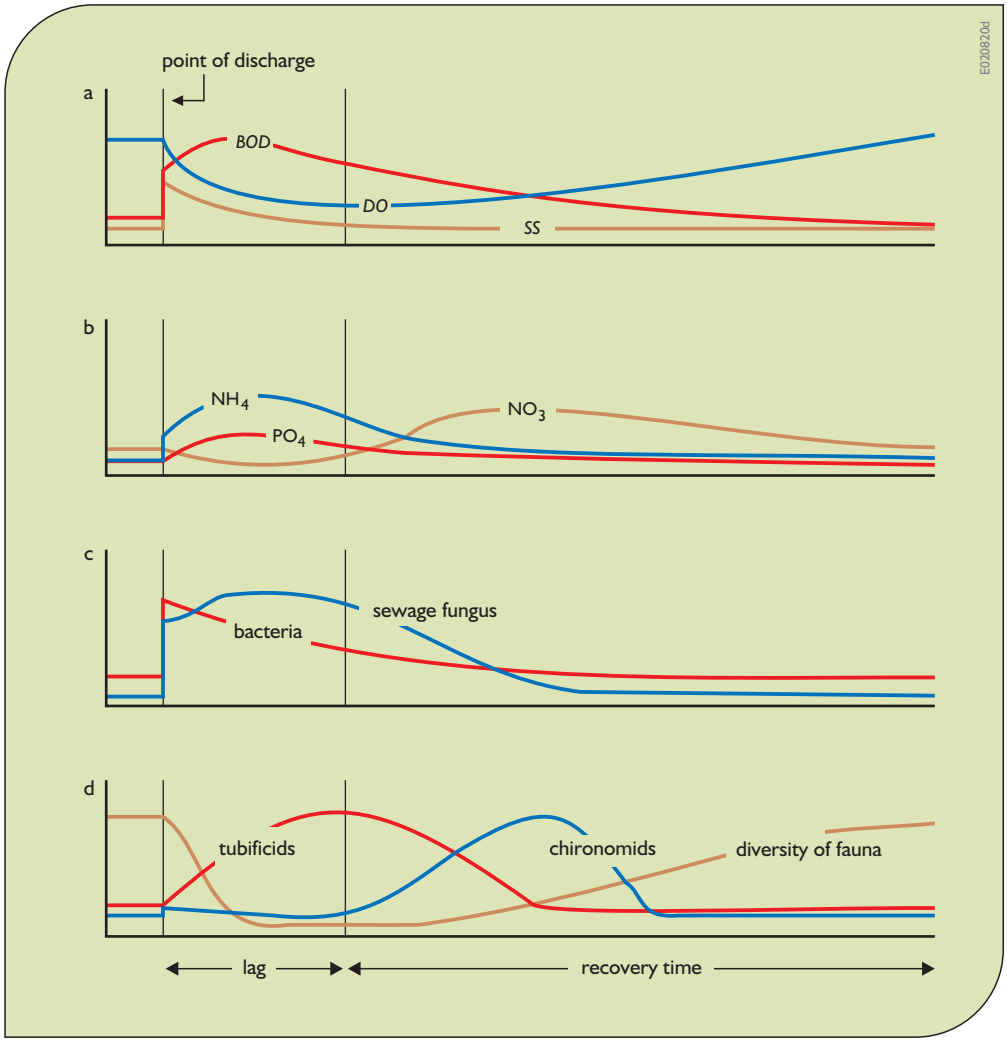

Figure 13.17. Pollution impact along a waterway downstream from its discharge.

can then be a source of pollutants if the bed is disturbed by high flows. This can create a subsequent pollution incident, especially if the suspended solids contain quantities of toxic heavy metals.

Diagram 'b' of Figure 13.17 shows ammonium ions (NH_4^+) that are discharged as part of the dissolved pollutants being oxidized to nitrates (NO_3^-). The rise in the ammonium concentration downstream of the discharge is relevant because of the very low tolerance many aquatic organisms, particularly fish, have to the chemical. The ammonium concentration rises if the conditions are anaerobic, and will decline once aerobic conditions return and the ammonium ions are oxidized to nitrates.

Diagrams 'c' and 'd' show the effect of combined sewer overflows on flora and fauna. The increased quantities of phosphate and nitrate nutrients that they consume can lead to eutrophication. The fauna show perhaps the clearest pattern of response. The predictability of this response has led to the development of the many biological indices of pollution. The rapid succession of organisms illustrates the pattern of dominance of only a few species in polluted conditions. For example, tubificid worms can exist in near-anaerobic conditions and, having few competitors, they can multiply prolifically. As the oxygen levels increase these organisms are succeeded by chironomids (midge larvae) and so on until in clean well-oxygenated water, there is a wide diversity of species all competing with each other.

In most circumstances the concentration of dissolved oxygen (*DO*) is the best indicator of the 'health' of a water source. A clean water source with little or no content of biodegradable pollutants will have an oxygen concentration of about 9–10 mg/l when in equilibrium with air in a temperate environment. This maximum saturation concentration is temperature dependent. The hotter the water, the lower the *DO* saturation concentration.

All higher forms of life in a river require oxygen. In the absence of toxic impurities there is a close correlation between *DO* and biodiversity. For example most game-fish die when the *DO* concentration falls below about 4 mg/l.

Perhaps of more pragmatic significance is the fact that oxygen is needed in the many natural treatment processes performed by microorganisms that live in natural water bodies. The quantity of oxygen required by these organisms to break down a given quantity of organic waste is, as previously discussed, the biochemical oxygen demand (*BOD*). It is expressed as mg of dissolved oxygen required by organisms to digest and stabilize the waste in a litre of water (mg/l). These organisms take time to fully digest and stabilize the waste. The rate at which they do so depends on the temperature of the water at the start, and the quantity of these organisms available in it.

Since the *BOD* test measures only the biodegradable material, it may not give an accurate assessment of the total quantity of oxidizable material in a sample in all circumstances, e.g. in the presence of substances toxic to the oxidizing bacteria. In addition, measuring the *BOD* of a sample of water typically takes a minimum of five days. The chemical oxygen demand (*COD*) test is a quicker method, and measures the total oxygen demand. It is a measure of the total amount of oxygen required to stabilize all the waste. While the value of *COD* is never less than the value of *BOD*, it is the faster reacting *BOD* that impacts water quality. And this is what most people care about. However, determining a relationship between *BOD* and *COD* at any site can provide guideline values for *BOD* based on *COD* values.

Tables 13.8 and 13.9 provide some general ranges of pollutant concentrations in *CSO*s from urban catchments.

Water quality models for urban drainage are similar to, or simpler versions of, water quality models for other water bodies (as discussed in Chapter 12). Often a simple mixing and dilution model will be sufficient to predict the concentration of pollutants at any point in a sewage system. For example, such models may be sufficient for some toxic substances that are not broken down. Once the flows in the *CSO* enter the receiving water body, models discussed in Chapter 12 can be used to estimate their fate as they travel with the water in the receiving water body.

A factor that makes predicting the impacts of overflow discharges particularly difficult is the non-continuous nature of the discharges and their pollutant concentrations. In the first sanitary or foul flush, the fine sediments deposited in the pipes during dry periods are swept up and washed out of the system. Most existing models, termed constant concentration models, do not account for this phenomenon. Since many of the most significant pollution events occur when the river has low flows, and hence low dilution factors, the quantity of spill in the first flush may be very important in the overall pollution impact.

constituent	raw	treated	overflow
suspended solids mg/l	300	5	200
BOD mg/l	367	10	300
COD mg/l	470	15	350
ammonia mg/l	39	7	30

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Table 13.8. Typical quality of domestic sewage.

4.4.4. Bacteriological and Pathogenic Factors

The modelling of pathogenic microorganisms is particularly difficult since there are a very large number of pathogenic organisms, each usually with a unique testing procedure, many of which are expensive. Also many pathogens may present a significant risk to human health in very small numbers. Incubation periods of over twenty-four hours are not uncommon and there are as yet no automatic real-time monitoring techniques in commercial use.

The detection of pathogens relies heavily on indicator organisms that are present in feces in far higher numbers than the pathogenic organisms. Escherichia Coliform (E. coli) bacteria is the most common fecal indicator, and is used throughout the world to test water samples for fecal contamination.

Because of the problems in measuring micro-biological parameters involved in *CSO*s, most sophisticated methods of determining the quality of sewer water restrict themselves to more easily measured determinants such as *BOD*, *COD*, suspended solids, ammonia, nitrates and similar constituents.

4.4.5. Oil and Toxic Contaminants

Oils are typically discharged into sewers by people and industries, or are picked up in the runoff from roads and road accidents. Since oil floats on water surfaces and disperses rapidly into a thin layer, a small quantity of oil discharged into a water body can prevent reoxygenation at the surface and thus suffocate the organisms living there. The dispersal rate changes with oil viscosity, and

constituent	highway runoff	residential area	commercial area	industrial area
suspended solids mg/l	28 - 1178	112 - 1104	230 - 1894	34 - 374
BOD mg/l	12 - 32	7 - 56	5 - 17	8 - 12
COD mg/l	128 - 171	37 - 120	74 - 160	40 - 70
ammonia mg/l	0.02 - 2.1	0.3 - 3.3	0.03 - 5.1	0.2 - 1.2
lead mg/l	0.15 - 2.9	0.09 - 0.44	0.1 - 0.4	0.6 - 1.2

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Table 13.9. Pollutant concentrations (mg/l) in urban runoff.

the length of time the oil is a problem will partly depend on the surface area of the receiving water as well.

There are three main sources of toxic contaminants that may be discharged from *CSO*s:

- *Industrial effluents*. These could be anything from heavy metals to herbicides.
- *Surface washoff contaminants*. These may be contaminants washed off the surface in heavy rainstorms, and in agricultural and suburban residential areas will probably include pesticides and herbicides. In many cases these contaminants make a larger contribution to the pollutant load than the domestic sanitary flow.
- *Substances produced naturally in the sewer*. Various poisonous gases are produced in sewers. From the point of view of water quality, ammonia is almost certainly the most important, though nitrogen sulphide can also be significant.

4.4.6. Suspended Solids

Discharges high in suspended solids pose a number of problems. They almost invariably exert an oxygen demand. If they remain in suspension they can prevent light from penetrating the water and thus inhibit photosynthesis. If deposited they become a reservoir of oxygen-demanding particles that can form an anaerobic layer on the bed, decreasing biodiversity. They can also degrade the bed for fish (such as salmon) spawning. If these suspended solids contain toxic substances, such as heavy metals, the problems can be more severe and complex.

5. Urban Water System Modelling

Optimization and simulation models are becoming increasingly available and are used to analyse a variety of design and operation problems involving urban water systems. Many are incorporated within graphic–user interfaces that facilitate the use of the models and the understanding and further analysis of their results.

5.1. Model Selection

A wide range of models is available for the simulation of hydrodynamics and water quality in urban systems. The selection of a particular model and the setup of a model schematization depends on the research question at hand, the behaviour of the system, the available time and budget, and future use of the model.

The research question and the behaviour of the water system determine the level of detail of the model schematization. The time scale of the dominating processes and the spatial distribution of the problem are key elements in the selection of a model, as is illustrated in Figure 13.18 and Table 13.10.

Figure 13.18 shows the time scales of the driving forces and their impact in urban water systems. It may be wise to consider the processes with largely different time scales separately, rather than joining them together in one model. For instance, the water quality of urban surface waters is affected by combined sewer overflows and by many diffusive sources of pollution. A combined sewer overflow lasts several hours and the impact of the discharge on the oxygen concentration in the surface water lasts for a couple of days. The accumulation of heavy metals and organic micro-pollutants in the sediment takes many years and the influx of the diffusive sources of pollution is more or less constant in time. The impact of combined sewer overflows on the oxygen concentration can be studied with a detailed, deterministic simulation model for the hydrodynamics and the water quality processes in the surface water system. A typical time step in such a model is minutes; a typical length segment is within the range from 10 to 100 metres. The accumulation of pollutants in the sediment can be modelled by means of a simple mass balance.

Another example is shown in Table 13.10. In this example the wastewater collection and treatment system in an urban area is modelled in three different ways. In the first approach, only the river is modelled. The discharge of effluent from a wastewater treatment plant is taken into account as a boundary condition. This is a useful approach for studying the impact of the discharge of effluent on water quality.

In the second approach, a detailed water and mass balance is made for an urban area. The main routes of water and pollution are considered. Generic measures, such as the disconnection of impervious areas from the sewage system, can be evaluated with this type of model schematization. In the third, most detailed, approach, a model schematization is made for the entire sewage system and, eventually, the wastewater treatment plant.

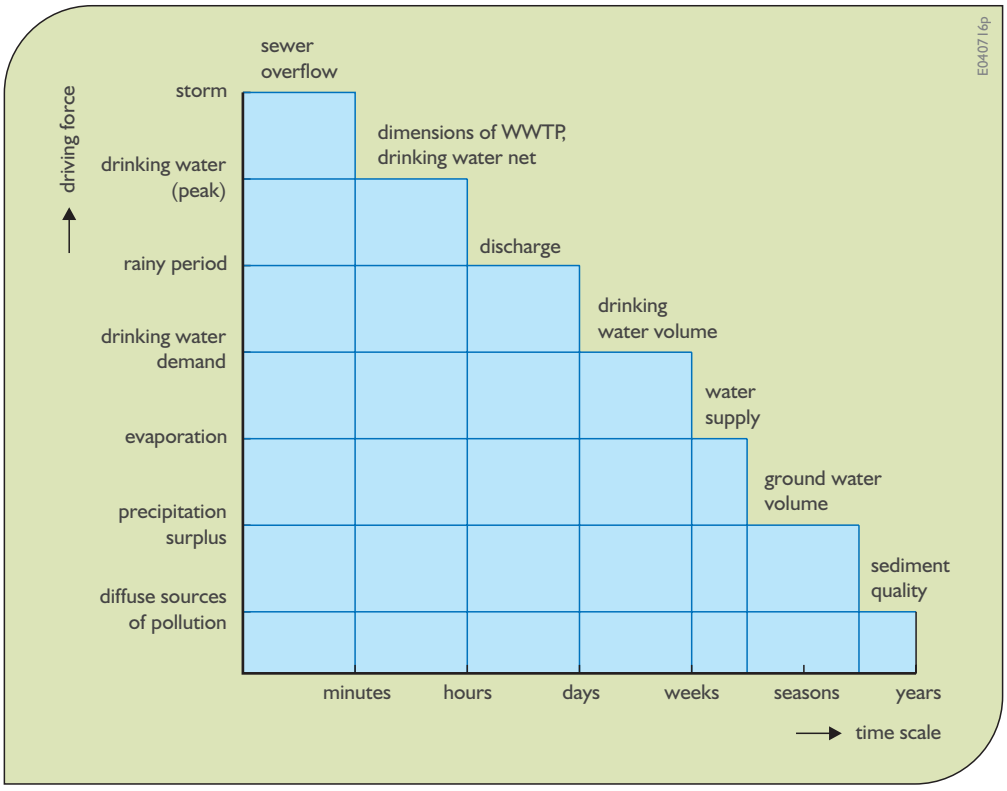

Figure 13.18. Time scales of driving forces and impacts in urban water systems.

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type of schematisation	implementation in code	examples
measured data of effluent quantity and quality	boundary conditions	end-of-pipe measures at the treatment plants
water balance and mass balance	rainfall runoff module, emissions module	analysis of main routes of rainfall discharge
detailed model schematisation	sewer flow module, water quality module	reduction of emissions via combined sewer overflows

Table 13.10. Three methods of making a model schematization for an urban water system.

This type of model is needed when defining specific measures concerning storage in the sewage system, enhanced treatment capacity at the wastewater treatment plant and the design of stormwater retention ponds.

5.2. Optimization

The use of the storage, transport and treatment capacity of existing urban infrastructure can be optimized in many cases. Optimization of urban water systems aims at finding the technical, environmental and financial best solution, considering and balancing measures in the sewage system, the wastewater treatment plant and the surface water system at the same time. For instance, the optimum use of the storage capacity in a sewage system by means of real-time control of the pumps may eliminate the need for a more expensive increase in treatment capacity at the wastewater treatment plant. Storage

Table 13.11. Storage, transport and treatment capacity of sewers, wastewater treatment plants and surface waters.

	sewer	WWTP	surface water
storage	moderate	none	limited - high
transport capacity	limited	limited	limited - high
treatment capacity	none	high	limited

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of runoff from streets and roofs in surface water may be better for river water quality than transporting the runoff to the wastewater treatment plant and subsequent discharging it as effluent. Table 13.11 shows a matrix with the key variables of storage, transport capacity and treatment capacity in the sewage system, the wastewater treatment plant and the surface water.

Methods for finding optimal solutions are becoming increasingly effective in the design and planning of urban infrastructure. Yet they are challenged by the complexity and non-linearity of water distribution networks, especially urban ones.

Numerous calibration procedures for water distribution system models have been developed since the 1970s. Trial and error approaches (Rahal et al., 1980; Walski, 1983) were replaced with explicit type models (Boulos and Wood, 1990; Ormsbee and Wood, 1986). More recently, calibration problems have been formulated and solved as optimization problems. Most of the approaches used so far are either local or global search methods. Local search gradient methods have been used by Shamir (1974), Lansey and Basnet (1991), Datta and Sridharan (1994), Reddy et al. (1996), Pudar and Liggett (1992), and Liggett and Chen (1994) to solve various steady-state and transient model calibration problems (Datta and Sridharan, 1994; Savic and Walters, 1995; Greco and Del Guidice, 1999; Vitkovsky et al., 2000).

Evolutionary search algorithms, discussed in Chapter 6, are now commonly used for the design and calibration of various highly non-linear hydraulic models of urban systems. They are particularly suited for search in large and complex decision spaces, e.g. in water treatment, storage and distribution networks. They do not need complex mathematical matrix inversion methods and they permit easy incorporation of additional calibration parameters and constraints into the optimization process (Savic and Walters, 1995; Vitkovsky and Simpson, 1997; Tucciarelli et al., 1999; Vitkovsky et al., 2000).

In addition to calibration, these evolutionary search methods have been used extensively to find least-cost designs of water distribution systems (Simpson et al., 1994; Dandy et al., 1996; Savic and Walters, 1997). Other applications include the development of optimal replacement strategies for water mains (Dandy and Engelhardt, 2001), finding the least expensive locations of water quality monitoring stations (Al-Zahrani and Moied, 2001), minimizing the cost of operating water distribution systems (Simpson et al., 1999), and identifying the least-cost development sequence of new water sources (Dandy and Connarty, 1995).

These search methods are also finding a role in developing master or capital improvement plans for water authorities (Murphy et al., 1996; Savic et al., 2000). In this role they have shown their ability to identify low-cost solutions for highly complex water distribution systems subject to a number of loading conditions and a large number of constraints. Constraints on the system include maximum and minimum pressures, maximum velocities in pipes, tank refill conditions and maximum and minimum tank levels.

As part of any planning process, water authorities need to schedule the capital improvements to their system over a specified planning period. These capital improvements could include water treatment plant upgrades or new water sources as well as new, duplicate or replacement pipes, tanks, pumps and valves. This scheduling process

requires estimates of how water demands are likely to grow over time in various parts of the system. The output of a scheduling exercise is a plan that identifies what facilities should be built, installed or replaced, to what capacity and when, over the planning horizon. This plan of how much to do and when to do it should be updated periodically long before the end of the planning horizon.

The application of optimization to master planning for complex urban water infrastructure presents a significant challenge. Using optimization methods to find the minimum-cost design of a system of several thousand pipes for a single demand at a single point in time is difficult enough on its own. The development of least-cost system designs over a number of time periods that experience multiple increasing demands can be much more challenging.

Consider, for example, developing a master plan for the next twenty years divided into four five-year construction periods. The obvious way to model this problem is to include the system design variables for each of the next four five-year periods, given the expected demands at those times. The objective function for this optimization model might be to minimize the present value of all construction, operation and maintenance costs. As mentioned previously, this is a very large problem that is probably unmanageable with the current state of technology for real water distribution systems.

Dandy et al. (2002) have developed and applied two alternative modelling approaches. One approach is to find the optimal solution for the system for only the final or 'target' year. The solution to this first optimization problem identifies those facilities that will need to be constructed sometime during the twenty-year planning period. A series of sub-problems are then optimized, one for each intermediate planning stage, to identify when each necessary facility should be built. For these sub-problems, the decisions are either to build or not to build to a predetermined capacity. If a component is to be built, its capacity has already been determined in the target year optimization.

For the second planning stage, all options selected in the first planning stage are locked in place and a choice is made from among the remaining options. Therefore, the search space is smaller for this case. A similar situation applies for the third planning stage.

An alternative approach is to solve the first optimization problem for just the first planning stage. All options and all sizes are available. The decisions chosen at this time are then fixed, and all options are considered in the next planning stage. These options include duplication of previously selected facilities. This pattern is repeated until the final 'target' year is reached.

Each method has its advantages and disadvantages. For the first, 'build-to-target' method, the optimum solution is found for the 'target year'. This is not necessarily the case for the 'build-up' method. On the other hand, the build-up method finds the optimal solution for the first planning stage, which the build-to-target method does not necessarily do. As the demands in the first planning stage are known more precisely than those for the 'target' year, this may be an advantage.

The build-up method allows small pipes to be placed at some locations in the first time planning stage, if warranted, and these can be duplicated at a later time; the build-to-target method does not. This allows greater flexibility, but may produce a solution that has a higher cost in present value terms.

The results obtained by these or any other optimization methods will depend on the assumed growth rate in demand, the durations of the planning intervals, the economic discount rate if present value of costs is being minimized, and the physical configuration of the system under consideration. Therefore, the use of both methods is recommended. Their outputs, together with engineering judgement, can be the basis for developing an adaptive master development plan. Remember, it is only the current construction period's solution that should be of interest. Prior to the end of that period, the planning exercise can be performed again with updated information to obtain a better estimate of what the next period's decisions should be.

5.3. Simulation

Dynamic simulation models are increasingly replacing steady-state models for analysing water quantity, pressure and water quality in distribution and collection networks. Dynamic models provide estimates of the time-variant behaviour of water flows and their contaminants in distribution networks, even arising from flow reversals. The use of long time-series analysis

provides a continuous representation of the variability of flow, pressure and quality variables throughout the system. It also facilitates the understanding of transient operational conditions that may influence, for example, the way contaminants are transported within the network. Dynamic simulation also lends itself well to statistical analyses of exposure. This methodology is practical for researchers and practitioners using readily available hardware and software (Harding and Walski, 2002).

Models used to simulate a sequence of time periods must be capable of simulating systems that operate under highly variable conditions. Urban water systems are driven by water use and rainfall, which by their natures are stochastic. Changes in water use, control of responses and dispatch of sources, and random storms over different parts of the catchment, all can affect flow quantities and direction, and thus the spatial distribution of contaminants. Because different water sources often have different quality, changing water sources can cause changes in the quality of water within the system.

The simulation of water quantities and qualities in urban catchments serves three general purposes:

- *Planning/Design*. These studies define system configurations, size or locate facilities, or define long-term operating policies. They adopt a long-term perspective but, under current practices, use short, hypothetical scenarios based on representative operating conditions. In principle, the statistical distribution of system conditions should be an important consideration, but in practice variability is considered only by analyses intended to represent worst-case conditions.
- *Operations*. These short-term studies analyse scenarios that are expected to occur in the immediate future so as to inform immediate operational decisions. These are based on current system conditions and expected operating conditions. These analyses are often driven by regulations.
- *Forensics*. These studies are used to link the presence of contaminants to the risk or actual occurrence of disease. Depending on whether the objective is cast in terms of acute or chronic exposures, such studies may adopt short or long-term perspectives. Because there are often dose–response relationships and issues of latency in the etiology of disease, explicit consideration of the spatial distribution, timing, frequency, duration and level of contamination is important to these studies (Rodenbeck and Maslia, 1998; Aral, et. al., 1996; Webler and Brown, 1993).

6. Conclusions

Urban water systems must include not only the reservoirs, groundwater wells and aqueducts that are the sources of water supplies needed to meet the varied demands in an urban area, but also the water treatment plants, the water distribution systems that transport that water, together with the pressures required, to where the demands are located. Once used, the now wastewater needs to be collected and transported to where it can be treated and discharged back into the environment. Underlying all of this hydraulic infrastructure and plumbing is the urban stormwater drainage system.

Well-designed and operated urban water systems are critically important for maintaining public health as well as for controlling the quality of the waters into which urban runoff is discharged. In most urban areas in developed regions, government regulations require designers and operators of urban water systems to meet three sets of standards. Pressures must be adequate for fire protection, water quality must be adequate to protect public health, and urban drainage of waste and stormwaters must meet effluent and receiving water body quality standards. This requires monitoring as well as the use of various models for detecting leaks and predicting the impacts of alternative urban water treatment and distribution, collection system designs and operating, maintenance and repair policies.

Modelling the water and wastewater flows, pressure heads and quality in urban water conveyance, treatment, distribution and collection systems is a challenging exercise, not only because of its hydraulic complexity, but also because of the stochastic inputs to and demands on the system. This chapter has attempted to provide an overview of some of the basic considerations used by modellers who develop computer-based optimization and simulation models for design and/or operation of parts of such systems. These same considerations should be in the minds of those who use such models as well. Much more detail can be found in many of the references listed below.

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