CHAPTER 5

CHAPTER 5

DESIGNING STORMWATER DRAINAGE SYSTEMS

5.1 INTRODUCTION

Drainage investigations involving both hydrologic and hydraulic studies feature prominently in many Civil Engineering projects ranging from simple culvert designs to intensive flood drainage studies and water quality studies.

As with any typical investigation involving streamflows or catchment runoff, the first task is the determination of the magnitudes of the flows being considered. This flow could be the design storm discharge of a catchment (in flood and drainage investigations) or a probabilistic low flow value (in water quality and pollutant dilution investigations). In all cases, some means of converting rainfall to runoff or determining dry-weather baseflow will be necessary. This form of flow determination is often referred to as the hydrologic investigation and involves investigating and simulating the processes of catchment (both urban and rural) hydrology. Typical outcomes of hydrologic investigations are flow magnitudes and flow durations for a range of probabilities of exceedence.

Once design flows are derived, the impacts of the flows on the physical water conveyance system need to be determined. This often involves computations based on the hydraulic characteristics of the system, e.g. flow level computations, hydraulic grade line analysis, hydrodynamic mixing, sediment and pollutant transport, sediment scouring and deposition etc. This form of analysis which simulates the hydraulic behaviour of open channel flow or closed conduits is referred to as the hydraulic investigation. Typical outcomes of hydraulic investigations are flood depth, flow velocities, flood extent, flow pattern etc.

5.2 A REVIEW OF FLOOD ESTIMATION PROCEDURES BY THE DRAINAGE AND IRRIGATION DEPARTMENT, MALAYSIA

The Drainage and Irrigation Department has over the years published a number of hydrological procedures for flood estimation in both rural and urban catchments in Peninsular Malaysia. The most recent procedures are as follows:

1. Rational Method- HP5 (DID, 1974)
2. Design Hydrograph Method- HP11 (DID, 1980)
3. Regional Flood Frequency Analysis- HP4 (DID, 1987)
4. Urban Rational Method- HP16 (DID, 1976)

The suitability and limitations of each method in terms of catchment size, types and basis of the method (whether rainfall or runoff based) are discussed by Quek (1993) and summarised as shown in Table 5.1. Note that all the above procedures are published in imperial units.

A brief summary of the basis of each of the flood estimation procedures is presented in Appendix 5.1.

TABLE 5.1
COMPARISON OF DID HYDROLOGICAL PROCEDURES HP5 (1974), HP11 (1980), HP4 (1987) AND HP16 (1976) (SOURCE: QUEK, 1993)

Methods	HP NO	Year Publised	Area km^2 (mi^2)	Catchment Type	Basis of method
Rational Method	5	1974	13-104 (5-40) *	Rural	Rainfall Based
Design Hydrograph Method	11	1980	< 518 (< 200)	Rural	Rainfall Based
Regional Flood Frequency Method	4	1987	> 20 (> 7.7)	Rural	Runoff Based
Urban Rational Method	16	1976	< 52 (< 20)	Urban	Rainfall Based

NB * May be applicable to area as small as 1.3 km^2 (0.5 mi^2).

All the DID procedures discussed above are rainfall-based with the exception of HP4 (1987) which is runoff-based. All the procedures are applicable to rural catchments only, except HP16 which is for urban catchments. Figure 5.1 shows the range of catchment areas applicable to each procedure. Note that the limits on catchment area for all the procedures are within the range expected for the Putrajaya site with a catchment area of about 60 km^2. The only exception is HP4 which is applicable only to catchment area in excess of 20 km^2.

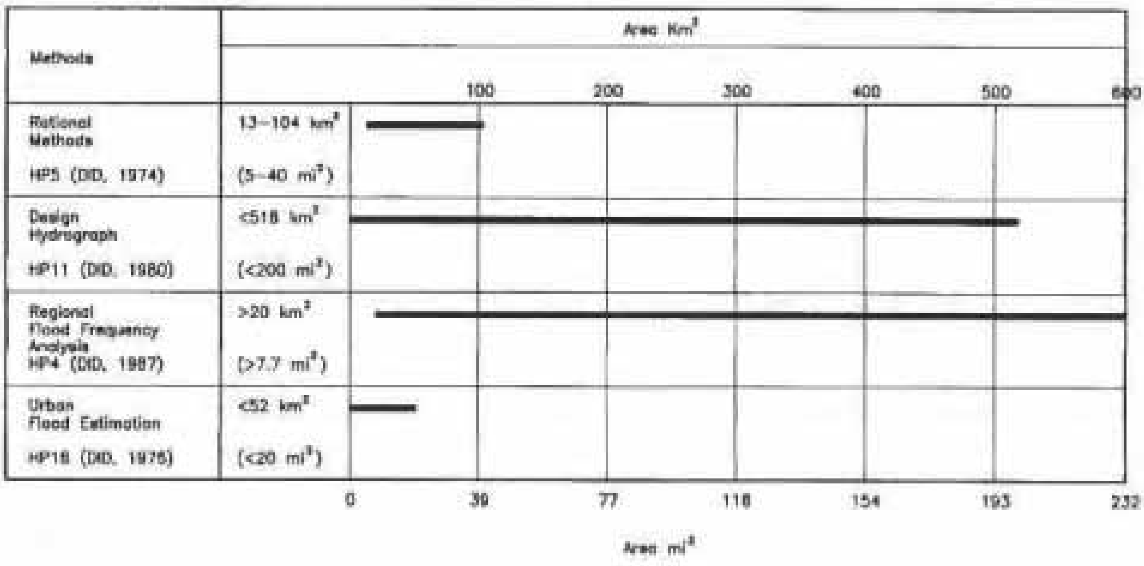


Figure 5.1 **Limits of catchment area for HP5, HP11 HP4 and HP16 (Source: Quek, 1993)**

5.2.1 Determination of the Rainfall Intensity-Frequency-Duration Curve

The above procedures which are rainfall-based i.e., HP 5, 11 and 16 require the estimation of the depth of design rainfall of various durations and recurrence intervals.

The Hydrological Procedure No. 1 (DID, 1982), provides a direct basis for estimating 2 and 20 year annual maximum rainfalls for durations between 0.5 and 72 hours, for any location in Peninsular Malaysia. Isohyetal maps are included in HP1 for durations of 0.5, 3, 24 and 72 hours, and for return periods of 2 and 20 years. The estimation of rainfall extremes at return periods other than those for which estimates are published is based on the assumption that the data fits to an EVI distribution, which can be defined from the published 2 and 20 year estimates. Linear interpolation of storm depth duration plots may be used to estimate rainfalls at intermediate durations for which data is not published. For the purpose of the present analysis, a rainfall depth-duration-frequency curve for the Putrajaya site has been prepared as shown in Figure 5.2.

5.2.2 Applicability of the Flood Estimation Procedures for Putrajaya

For rural catchment condition, the following Hydrological Procedures can be applied: HP5, HP11, and HP4. As discussed earlier, the only restriction is HP4 which is applicable only to area more than 20 km^2. As most of the Putrajaya site is undergoing rapid urbanisation, the only areas within the catchment boundary which may be classified as under rural condition comprised mainly of forest areas and areas outside the Putrajaya boundary which drain into Putrajaya.

The choice for which procedures to adopt will depend on a number of factors. For example, whether a peak discharge estimate alone is sufficient, or a hydrograph of flow versus time is required. In this respect, note that the HP5 and HP 4 only give estimate of the peak discharge, while HP11 gives estimate of both the peak discharge and the associated hydrogaph.

In situations where estimate of peak discharge is required, it is recommended to compute the peak discharges using all the three procedures, and an assessment be made as to which value to adopt. Quek (1993) carried out a comparison of the peak discharge estimates using all the procedures. It was found that for rural catchments below 200 ha, HP5 tends to overestimate the peak discharges by up to three times the HP11 method which is found to give more realistic estimate of peak discharges for small rural catchments.

Within Putrajaya boundary, the HP16 is recommended for use in estimating peak discharges associated with urban development. The procedure gives estimates of both the peak discharge and the associated hydrograph. The procedure is widely used for flood estimation in urban area in Peninsular Malaysia.

5.2.3 Urban Drainage Design Standards and Procedures for Peninsular Malaysia

Another document which is widely used currently in Malaysia for urban drainage design is the publication entitled *"Urban Drainage Design Standards and Procedures for Peninsular Malaysia"* (DID, 1975). The document contains a chapter on flood estimation which is largely based on the HP16 procedure using the Modified Rational Method. In addition, the document provides guidelines for the design of hydraulic structures such as open channels, culverts, bridges, energy dissipators and channel drops. It also provides guidelines on the design of erosion and sediment control structures. The document should be used in conjunction with the "Putrajaya Stormwater Management Design Guidelines."

It is unavoidable that certain subject matters are covered in both the above documents. Since the DID (1975) document was prepared more than 23 years ago, some of the design concepts and methods have changed. Furthermore, the document was prepared for use in the whole of Peninsular Malaysia, and may not be suitable for the Putrajaya site. As such, if a particular subject matter is covered in both documents, the present guidelines shall be adopted. If a subject matter is not covered in the present guidelines but is covered in the DID (1975) document, then the latter may be applied using one's professional judgment.

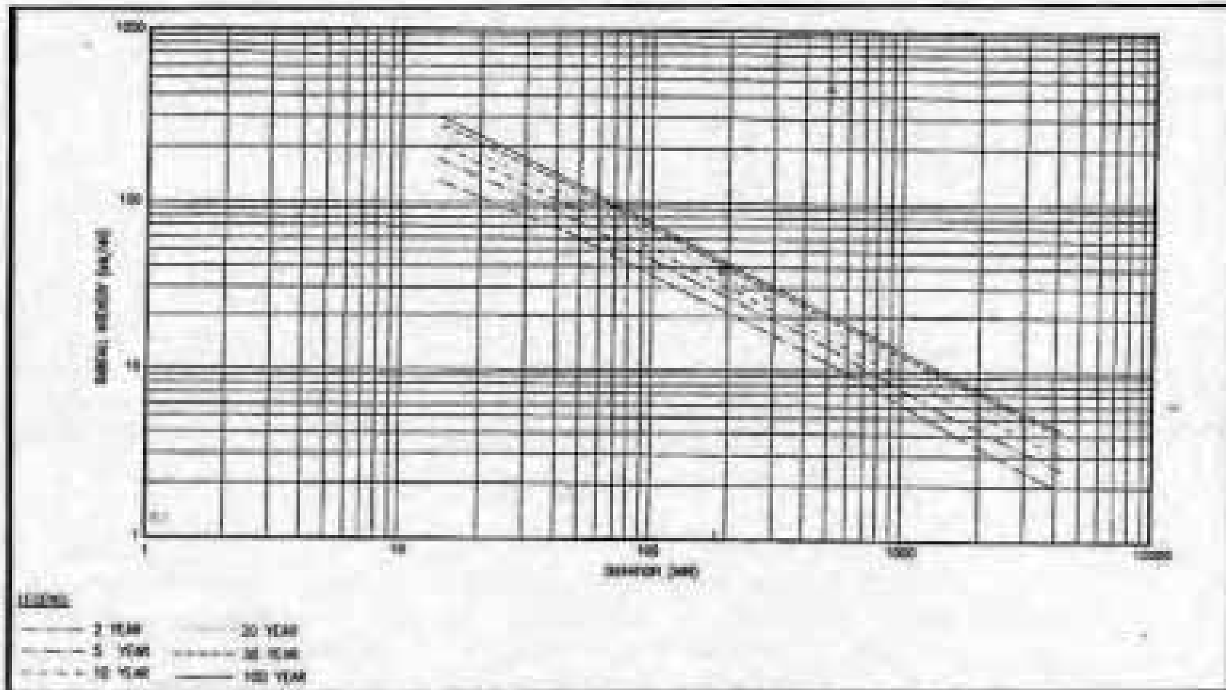


Figure 5.2 Rainfall Intensity Frequency Duration Curve for Putrajaya

5.3 HYDROLOGIC PROCEDURES

5.3.1 Streamflow-based Methods - Flood Frequency Analysis

Flood frequency analysis is a method of analysing the statistics of recorded flood data with the objective of defining the probability of distribution of the data for subsequent use in flood estimation. The most common application of flood frequency analysis is with observed flow data although analysis of flood depth can often be equally appropriate. The reader is referred to standard textbooks and publications by the Drainage and Irrigation Department of Malaysia for a complete description of flood frequency methods. These methods compute peak flow magnitudes of different average recurrence intervals and are only suitable for instantaneous flow and steady flow analysis. Flood frequency analysis is carried out using either the annual series or partial series approach. As a general rule, the annual series is used when estimating flows less frequent than the 10 year ARI while the partial series is used for estimating more frequent events.

A major advantage of streamflow-based methods is its ease of use. Implicit in using this approach, however, is the assumption that catchment characteristics remain unchanged throughout the period for which data was available.

5.3.1.1 Annual Flood Series

In the annual series approach, observed peak flows for each year (either calendar or water year) are fitted to a probability distribution. The use of the Log-Pearson Type III probability distribution is common practise. This probability distribution involves three parameters representing the statistical properties of the logarithmic values of the peak flows. These are the mean, standard deviation and the coefficient of skewness. When the coefficient of skewness is zero, the Log-Pearson Type III distribution becomes the two-parameter Log Normal distribution. Fitting and plotting the Log-Pearson Type III distribution to the logarithmic values of the observed peak flows involves deriving the three parameters for the peak flow data and using standard tables to provide appropriate probabilistic values corresponding to a number of plotting positions. Once fitted, the flood frequency curve can then be used to predict probabilistic flows of any Annual Exceedence Probability (AEP) or Average Recurrence Interval (ARI).

5.3.1.2 Partial Flood Series

The partial series approach adopts a "threshold discharge" value as the criterion in selecting flood events for statistical analysis compared to the calendar year or water year criterion of the annual series analysis. The use of a "threshold discharge" as the selection criterion is used in recognition of the fact that the annual flood series can be deficient in accounting for frequent flood events. It is important to ensure that all events selected are independent of each other.

Flood magnitudes for frequent events tend to be under-estimated by the annual flood series owing to the fact that the method requires only one event per year to be included in the statistical analysis regardless of (i) the magnitude of this event (could have been a drought year) and (ii) the number of other events of similar magnitude (as would occur in a wet year). In selecting all independent events with peak flows larger than a threshold value, a better estimation of the average recurrence interval of peak flows, especially for events of high frequency of occurrence, can be realised.

The partial series frequency curve allows peak discharges of average recurrence intervals of less than a year to be determined owing to the fact the number of flood events included in the analysis can exceed the number of years of record available. For any given site, the flow estimates for any given ARI by a partial series analysis will always be higher or equal to a corresponding estimate using an annual series analysis. A theoretical expression relating the ARIs of a given peak flow from a partial series analysis and an annual series analysis was derived by Langbein (1949) as follows:-

$$Y_A = \frac{e^{1/Y_P}}{e^{1/Y_P} - 1} \qquad - \qquad 5.1$$

where Y_A is the average recurrence interval derived from an annual series analysis.
Y_P is the average recurrence interval derived from a partial series analysis.

For AEP less than 10% (or average recurrence interval greater than 10 years) both approaches are expected to give similar design flows. Apart from requiring a higher degree of effort in carrying out a partial series analysis, this type of analysis also has the disadvantage of having no known probability distribution which can be fitted to the data. It is necessary to draw a line of best fit of the data plotted on a semi-log scale with the discharge in natural scale and the average recurrence interval plotted on logarithmic scale.

5.3.1.3 Plotting Flood Data

Plotting of the observed flow data is often helpful in understanding the statistical characteristics of the flow data. For annual series analysis, the logarithmic normal probability graph paper is often used while this together with a semi-logarithmic graph paper are used for partial series analysis.

To enable the observed flow data to be plotted, it is first necessary for their plotting positions to be evaluated. Their plotting positions may be viewed as estimates of their probability for exceedence and are evaluated by first ranking the flow data in descending order. The plotting position of each individual flow data is computed as follows:-

$$PP(m)=\frac{m-0.4}{N+0.2} \qquad - \qquad 5.2$$

If the plotting position is expressed in the form of an average recurrence interval (Y_P), Y_P is given as

$$Y_P=\frac{N+0.2}{m-0.4} \qquad - \qquad 5.3$$

The parameters m and N in equations 5.2 and 5.3 relate to the ranking of the individual flow and the number of years of available data respectively.

When applying the annual series, the highest rank value will equal the number of years of available data and the plotting position as computed in equation 5.2 will always be less than unity. With partial series, equation 5.3 is used to allow determination of average recurrence intervals of less than one year for flow data with rankings exceeding the number of years of available data.

It should be noted that the plotting positions derived from an annual series analysis and a partial series analysis have different interpretations. The average recurrence interval based on annual flood series data reflects the average interval between years in which a given discharge is exceeded, whether once or more than once while the ARI from a partial series analysis reflects the average interval between exceedence of the given discharge (Laurensen,1987).

5.3.1.4 Fitting the Log Pearson III distribution

In the annual flood series analysis, the Log-Pearson Type III probability distribution is often used to describe the flood frequency. The distribution can be fitted to the data by computation of the mean, standard deviation and coefficient of skewness of the logarithmic values of the flow data. These parameters, together with standard probability tables, can then be used to determine probabilistic flows for any AEPs.

The reader is referred to standard textbooks or publications by the Drainage and Irrigation Department of Malaysia for a detailed description of the procedure. The reader should also place particular attention to the procedure recommended in identifying and dealing with data uncertainties and in the computation of confidence limits in the derived flood frequency curve.

5.3.2 Rainfall-based Methods

Rainfall-based methods of flood estimation are methods which derive a flood of a selected probability of exceedence from a design rainfall event of the same probability. The application of these methods involved a two-stage approach, the first being the determination of the probabilistic rainfall event of the selected probability of exceedence and duration and the second being the conversion of the design rainfall to design flow. The main advantage in utilising probabilistic rainfall data in the derivation of design flows are:

- long periods of record;
- large number of rainfall stations from which regional analysis can be carried out; and
- rainfall data are largely independent of catchment conditions thus enabling a stronger basis for regional "pooling" of statistical information.

There are a number of rainfall-based methods of flood estimation which are widely used in Australian practice. They differ fundamentally in the approach adopted in converting the design rainfall to flows. Analysis of rainfall data from daily and pluviograph rainfall station throughout Malaysia have been carried out by the Drainage and Irrigation Department of Malaysia. The Drainage and Irrigation Department has published a document entitled Hydrological Procedure No 1- Estimations of the Design Rainstorm in Peninsular Malaysia (DID, 1982).

5.3.2.1 The Rational Method

Theoretical Considerations
The Rational formula expresses a relationship between rainfall intensity and catchment area as independent variables and the peak flood discharge resulting from the rainfall as the dependent variable. It has been used for over 150 years, and known as the Rational Formula for nearly 100 years. It is widely used in the design of stormwater drainage systems, farm dam spillways, and small culverts in road and railway embankments.

The Rational Formula for estimation of the peak discharge is:-

$$Q = 0.278\ CIA \qquad - \qquad 5.4$$

where
Q = peak discharge (m^3/s);
C = a dimensionless runoff coefficient;
I = mean rainfall intensity (mm/h) of a storm of the design average recurrence interval (ARI) and duration equal to the time of concentration tc;
A = catchment area (km^2)

The coefficient 0.278 converts km^2 and mm/hr to m^3/s.

To estimate a flood using the Rational Formula, the following factors must be determined:-

- A - catchment Area
- C - runoff coefficient
- I - mean rainfall intensity

To determine I, the following factors must be determined:-

- t_c - time of concentration
- ARI - average recurrence interval of the storm

The Rational Formula should strictly be used for peak flow estimation only although many have assumed a simple triangular hydrograph for design. This is only the case when the storm duration selected equates to the time of concentration of the catchment. In the event that the storm duration is different from the time of concentration of the catchment, the shape of the assumed hydrograph is not triangular as shown in Figure 5.3.

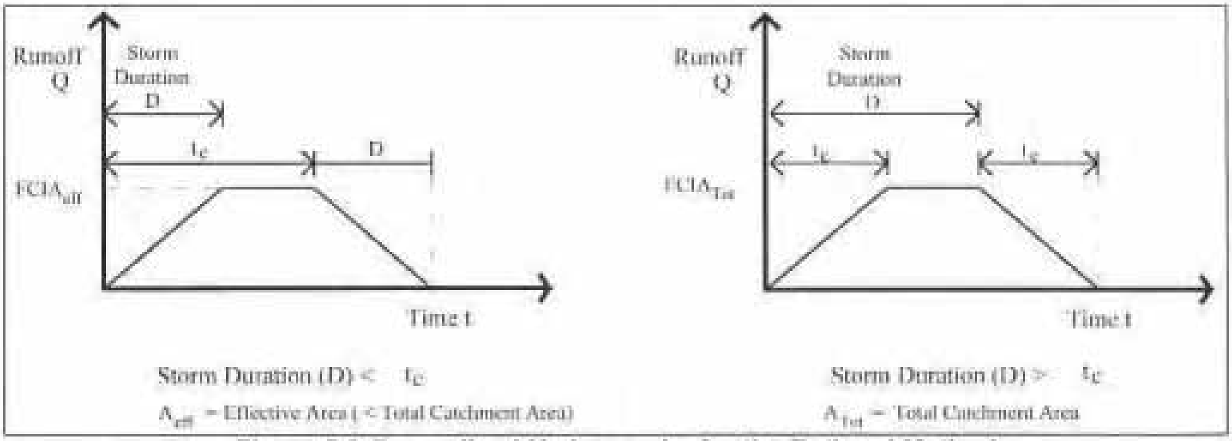


Figure 5.3 Generalised Hydrographs for the Rational Method

With the Rational Method, the assumption of uniform, steady rainfall tends towards underestimation of the peak discharge. On the other hand, neglecting of storage effects on the catchment tends towards overestimation of the peak discharge. It follows that the catchments most conducive to valid results from the rational formula are those with small channel storage and short design storms (ie. short time of concentration), since short storms tend to be more uniform in time than long ones. These constraints limit reliable applications of the rational method to urban drainage systems and small rural catchments.

Design IFD curves always show decreasing design intensities for increasing storm duration for a given rainfall average recurrence interval. In designing stormwater systems, selection of storm durations longer than the time of concentration of the catchment will always lead to lower peak discharges than storms of duration equal to t_c. Analysis of design storms longer than the time of concentration of the catchment is therefore not necessary unless the stormwater drainage systems involve stormwater detention basin(s). With stormwater detention basin(s) design, the combination of both runoff volume and instantaneous discharges influence the selection of storage volume and outlet discharge capacity.

Storms shorter than t_c have a higher intensity but the maximum area simultaneously contributing runoff to the peak discharge at the outlet is less than the catchment area A. It is usually assumed that the reduction in contributing area outweighs the increase in rainfall intensity relative to that of a storm of duration equal to t_c and the peak discharge from the shorter storm is, therefore, the lesser. It follows that the time of concentration is normally the ***critical storm duration***, i.e. the duration producing the highest discharge for a given average recurrence interval and runoff coefficient.

In the situations where the increase in rainfall intensity, for a storm duration shorter than t_c, outweighs the reduction in contributing area, the peak discharge resulting from the shorter duration storm would be higher than that generated from the storm of t_c duration. This is often referred to as the ***Partial Area Effect***.

Application of Rational Method

To estimate a flood using the Rational Formula, the following factors must be determined:-

- A - catchment Area
- C - runoff coefficient
- I - mean rainfall intensity

To determine I, the following factors must be determined:-

- t_c - time of concentration
- ARI - average recurrence interval of the storm.

5.3.2.2 Runoff Routing Methods

General

Runoff routing methods are becoming widely adopted as an industry-standard for hydrologic investigation for flood estimation. These methods involve determining the rainfall-excess and routing it through a model of the catchment storage by flood routing procedure. Computer models are invariably used due to the level of detail necessary in modelling the distributed nature of the catchment storage. Catchment storage include the temporary storage of runoff in the stream channels and floodplains and on ground surface as well as temporary storage in reservoir and retarding basins. Most runoff routing models involves the modelling of the two dominant hydrologic processes of catchment losses (to determine the rainfall excess hyetograph) and runoff routing.

Runoff routing models allow the user to sub-divide the catchment into a number of sub-catchments to model the runoff generation and flow routing in individual sub-catchments. This allows for the accounting of areal distribution of rainfall, landuse, catchment and stream characteristics as well as future modifications to these parameters. Runoff routing models were developed to overcome some of the inherent problems associated with the Unit-hydrograph method such as areal lumping of catchment and rainfall characteristics as well as the utilisation of system linear theory. Research studies have found catchment flood response to be non-linear and runoff routing methods allow (to some degree) for the modelling the nonlinearity in catchment response. Studies have shown that there may be a need for further improvements in current runoff routing models to account for non-linearity in rivers with extensive floodplains.

Runoff routing models currently available to the industry for urban and rural flood estimation include ILSAX, RORB, RAFTS and WBNM.

Basis of Runoff Routing

The establishment of a runoff routing model of a catchment commence with the sub-division of the catchment into sub-areas according to the stream network, catchment topography and landuse, rainfall variability and location of interest. A node would be assigned at the centroid of each sub-catchment and at the confluence of stream. Figure 5.4 illustrates a typical catchment sub-division for a runoff routing model.

A design or historic rainfall pattern would be assigned to each sub-catchment node and the resulting rainfall excess determined. In a design situation, often one common design storm pattern would be assigned to sub-catchments while in the case of reproducing a historical storm event, the number of different storm patterns applied to the catchment would be dependent on the data available.

While having the same storm pattern, most runoff routing models also allow different total storm rainfall to be assigned to each sub-catchment. It is quite common even in design, for sub-catchments to be assigned different storm depth while having the same storm temporal pattern to represent topographic effect on areal distribution of rainfall.

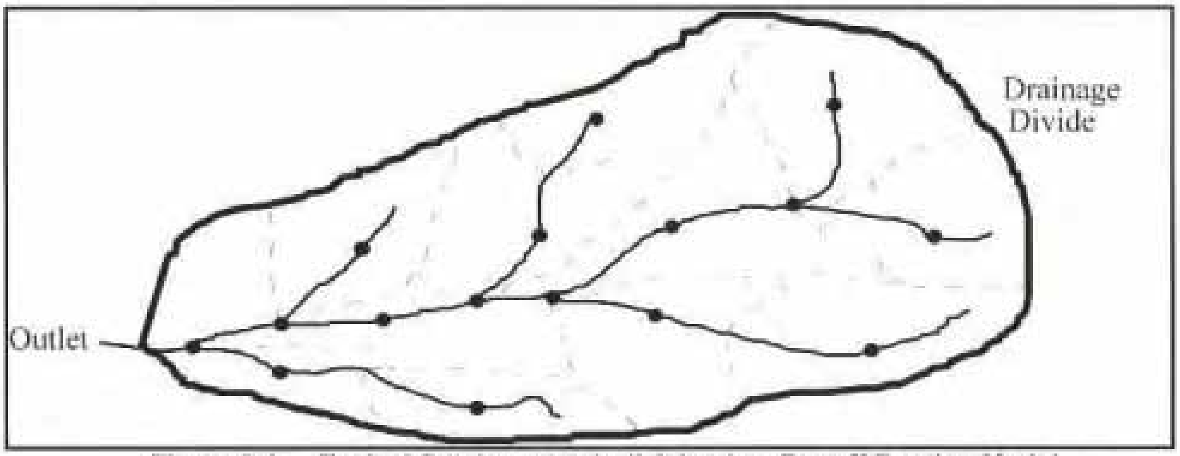


Figure 5.4 Typical Catchment sub-division in a Runoff Routing Model

Loss Modelling

Rainfall excess hyetograph from each sub-catchment is calculated by applying the catchment loss model to each sub-catchment. The loss models in all runoff routing models consist of a combination of pervious area loss modelling and impervious area loss modelling. Common type of loss modelling for pervious area include the initial loss/continuing loss model, the initial loss/runoff coefficient model, the Green-Ampt loss model, the Horton Infiltration Loss model. The most common loss model for impervious is simply a runoff coefficient model (usually a runoff coefficient of 0.9). The user defines the landuse in each sub-catchment by adjusting the proportion of pervious and impervious area in the sub-catchment.

The rainfall excess from each sub-catchment would be routed from the centroid of that sub-catchment, along the main stream, to the next downstream node where the runoff hydrograph is combined with (i) runoff hydrographs from other tributaries and/or (ii) rainfall excess hyetograph from the sub-catchment of the downstream node reach. The combined runoff hydrograph is then routed downstream to the next node and so on.

Flow Routing

Catchment storage has the effect of modifying the runoff hydrograph such that the peak flow is attenuated and translated in time as illustrated in Figure 5.5. This effect can be simulated by flood routing technique in which the input (i.e. rainfall excess hyetograph or the runoff hydrograph from an upstream node) routed through a series of catchment storages.

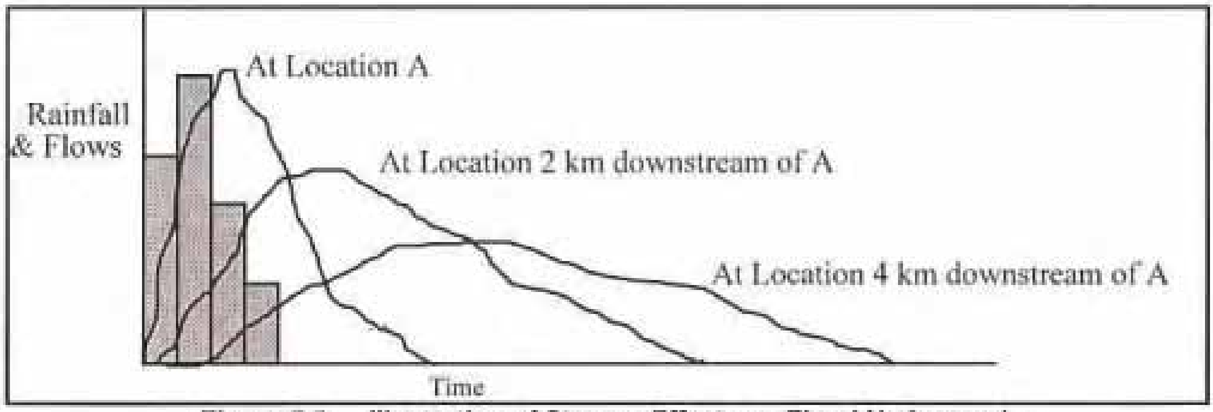


Figure 5.5 Illustration of Storage Effects on Flood Hydrographs

Most runoff routing model utilises a simple storage routing algorithm based on the continuity equation and a storage equation of the general form $S = kQ^m$. The storage equation is non-linear and this form of the storage equation is justifiable by both theoretical and empirical means. Theoretical derivation using a simple flow formula such as the Manning's Equation has shown that the parameter k represents the effect of such river characteristics as the hydraulic roughness, bed slope, length and cross section geometry. The value of the exponent m, which affects the degree of non-linearity of the catchment flood response, has been shown to be related to the shape of the river channel with typical values of m being 0.6 for wide rectangular channel (i.e. under fully developed floodplain flow conditions perhaps) and 0.75 for triangular or trapezoidal sections. Like any models (no matter how simplified), the parameters of the model needs to be calibrated. The parameters k and m are calibrated from observed flood events and thus the reliability of the method reduces when the availability and the quality of calibration data are limited.

Runoff Routing Parameters for Ungauged Catchment

For ungauged catchment, the use of regional data is often adopted and the results are usually expected to have a higher uncertainty. The recognition that catchment flood response is non-linear in nature in runoff routing has nevertheless improved the reliability of extreme flood prediction, especially when recorded events for model calibration have high probabilities of exceedence. The traditional approach of the Unit-hydrograph method has been shown to have the tendency to under-estimate extreme floods in such circumstance. As mentioned earlier, the degree of non-linearity and the appropriate form of the storage equation (other than the power function commonly assumed in the industry) is still a subject of intense research.

5.4 HYDRAULIC PROCEDURES

Hydraulic design of waterways and associated hydraulic structures are necessary to safely and efficiently convey stormwater runoff, to facilitate design of crossings of roads over creeks, floodways and rivers. The hydrologic procedures outlined in the previous section provide the basis from which design flows for these hydraulic structures can be estimated.

Hydraulic procedures are required to compute flow characteristics in closed conduits and open channels during the passage of a runoff hydrograph. In all cases, hydraulic calculations are aimed at computing energy losses in the flow conveyance system such that the hydraulic grade line (HGL) of the system can be established. The HGL is often used to determine the location of the hydrostatic head along the water conveyance system. In the case of open channel flow, the HGL generally coincides with the water surface. In closed conduit flow conveyance systems, flow conditions can be pressurised and water rises to the HGL level at junction pits. The design process is commonly a series of iterations involving modifying the various hydraulic structures such that the resulting HGL complies with the design standards specified. These design standards can include:

- Frequency of surcharge of underground drainage systems;
- Flood levels and flow velocities along the river and floodplain;
- Flood attenuation caused by hydrograph characteristics (a function of the hydrograph shape);
- Flood attenuation caused by channel and floodplain storage (a function of the river and floodplain geometrical properties and hydraulic roughness);
- Flow directions, flood breakouts and returns, off-channel storage, etc.

In closed conduit flow conditions, calculation of energy losses within the flow conveyance system is most commonly based on applying energy loss factors to the velocity head ($v^2/2g$). The energy loss attributed to pipe friction is best calculated by the Darcy-Weisbach Equation. Localised energy losses due to junction and inlet pits are computed in a similar manner using standard coefficients derived from extensive laboratory tests of these structures.

The simplest model available for computing flood levels and flow velocities is the use of empirical formulae such as the Chezy Equation and the Manning's Equation. These equations are referred to as slope-area methods on the basis that they utilises simple relationships between the discharge in a river to the energy gradient, flow cross-sectional area and hydraulic roughness. These formulae are suited for computing water levels only at a single location and for a single discharge value (i.e. steady flow assumptions apply) under uniform flow conditions.

These methods at best would address the second of the above five design considerations related to the hydraulics behaviour of the flow conveyance system. Often the use of slope-area methods are inappropriate due to the fact that natural river flows are not uniform.

Open channel flow conditions can often be categorised as either rapidly varying or gradually varying. Rapidly varied flow occurs whenever there is a abrupt change in the geometry of the channel or in the flow regime of the flow. In regions of rapidly varied flow, the water surface profile changes rapidly. Examples of rapidly varying flow include flow over weirs and through regions of rapid changes in bed elevation or channel width (i.e. abrupt change in geometry) and hydraulic jumps (i.e. change in flow regime). Simulations of rapidly varied flow conditions require the solution to the equations of conservation of mass and conservation of momentum in fluid flow (i.e. the Saint Venant Equations).

The flow condition generally occurring in natural river and floodplain systems can be categorised as that of gradually varied flow; that is, conditions in which the flow characteristics are non-uniform but vary gradually with distance along the channel due to gradual variation of bed slope, channel geometry and hydraulic roughness.

5.4.1 Darcy-Weisbach Equation

For closed conduit operating under full flow conditions (either pressurised or non-pressurised), the energy loss associated with the pipe friction is best computed by the Darcy-Weisbach Equation, expressed as follows:-

$$h_L = \frac{fL}{D} \times \frac{v^2}{2g} \qquad - \qquad 5.5$$

where

h_L	is the energy loss along the length of the pipe (m)
f	is the friction factor which is dependent on the type of pipe, its diameter and the flow Reynolds Number. The appropriate value of f can be read from the standard "Moody Diagram"
L	is the length of the pipe (m)
D	is the diameter (or equivalent diameter) of the pipe
v	is the flow velocity within the pipe (m/s)
g	is the gravitational acceleration (m/s^2)

For localised energy losses associated with junction and inlet pits, the term fL/D in the above equation is replaced by a head loss coefficient. Values of the energy loss coefficients may be obtained from standard charts produced from various laboratory studies, particular the "Missouri Charts" which were based on extensive laboratory studies by Sangster et al. (1958) of the University of Missouri.

5.4.2 Slope-Area Methods

Slope-area methods are useful in providing a quick means of determining flows. They are particularly useful when estimating historical flows from flood marks. Equations commonly used include the Manning's Equation and the Chezy Equation as expressed in Equations (5.6) and (5.7) respectively.

$$Q = \frac{A \times R^{2/3} \times S_f^{1/2}}{n} \qquad \text{(Manning's Equation)} \qquad - \qquad 5.6$$

and

$$Q = C \times A \times R \times S_f^{1/2} \qquad \text{(Chezy Equation)} \qquad - \qquad 5.7$$

where

Q	is the discharge (m^3/s)
A	is the area of the flow cross section (m^2)
R	is the hydraulic radius of the flow section and is the ratio of the flow area to the wetted perimeter (m)
S_f	is the friction slope, often approximated by the bed slope of the channel
n	is the Manning's roughness coefficient
C	is the Chezy roughness coefficient

It is evident that the two equations are similar and both equations assume uniform one-dimensional flow conditions.

5.4.3 Designing Closed Conduit Systems

Hydraulic design of underground stormwater drainage systems involves the selection of appropriate pipe sizes to meet the design objectives for the system in terms of frequency of stormwater surcharge from the system. This requires the drainage network to be first established and catchment runoff rates estimated at key locations using methods outlined in Section 2. The estimation of probabilistic flows at these locations should be carried out for a range of storm durations and the maximum discharges at each locations selected for sizing the inlet structures and underground pipes.

The Hydraulic Grade Line (HGL) defines the hydrostatic pressure levels throughout the drainage system. This, together with the Total Energy Line (TEL), enables the design engineer to ascertain the performance of the drainage system under design conditions. The TEL represents the total energy available to flow and consists of the velocity head, pressure head and potential head. The TEL is located a distance equal to the velocity head ($v^2/2g$) above the HGL. In assessing the performance of a pipe drainage system, the HGL determines the water elevation at pits and thus the degree of surcharge and overflows.

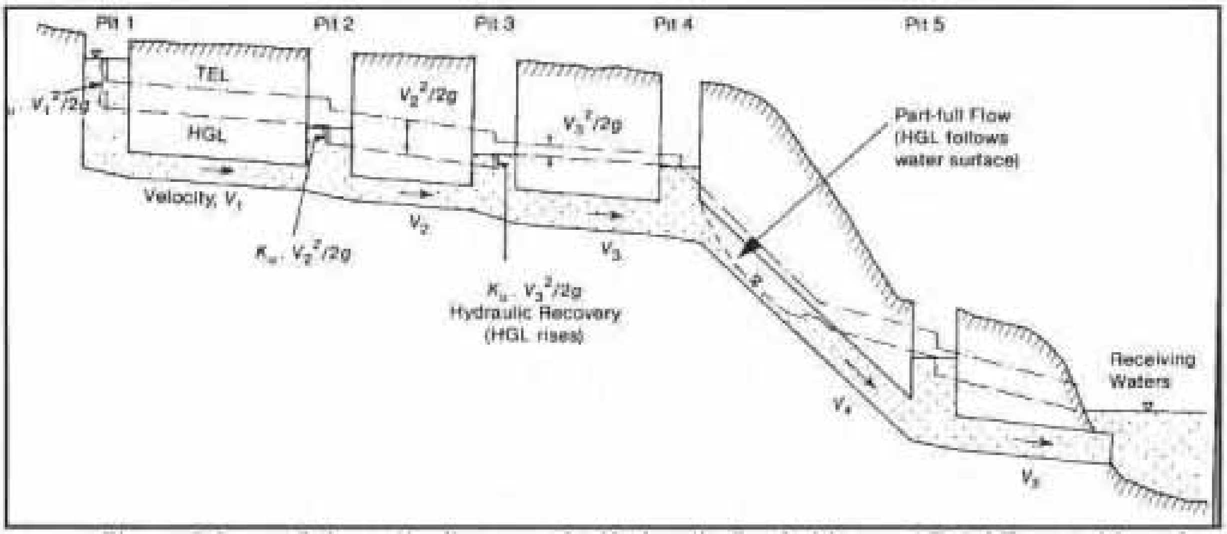


Figure 5.6 Schematic diagram of a Hydraulic Grade Line and Total Energy Line of a Stormwater Pipe System. source: Inst. Engrs., Aust., 1987

Figure 5.6 illustrates typical TEL and HGL characteristics in a surcharged pipe system. It is often convenient to compute the HGL and TEL of the drainage network by assuming steady flow conditions in each of the individual pipelines with the steady flow being their respectively peak discharges corresponding to the selected ARI. The appropriate pipe sizes in each of these lines are determined by progressively adjusting them until the computed HGL levels at all locations are below the corresponding ground elevations. Any locations at which the HGL levels are above the corresponding ground elevations indicate the likelihood of stormwater overflow unless these locations are appropriately sealed.

Following determination of the appropriate design flows at each of the key locations within the drainage system, the design process of the drainage system will require hydraulic calculations to commence from the most downstream location of the drainage network, with calculations progressively working in the upstream direction. There are however procedures adopted in practice which combines the hydrologic and hydraulic calculations and estimates the design flows at each location followed by a preliminary sizing of the downstream section of the underground pipe before proceeding to the estimation of local inflows at the next downstream location. In such cases, an assumption that the pipe will be flowing full but not under pressure is often adopted and calculations commence from the most upstream location of the drainage network, working in the downstream direction. A second, backwards pass will then become necessary to compute the HGL for the selected pipe sizes to confirm their suitability.

5.4.3.1 Gully pits

Common Stormwater Inlet Structures
Gully pits efficiently convey storm flows from the surface to the underground pipe system. When selecting and locating gully inlets, consideration should be given to hydraulic efficiency, public safety, debris collection potential and consequential impact on the hydraulic capacity, and maintenance problems.

There are two types of inlets structures in common use which may be installed singly or in multiple arrangements. These types are:-

- the grated pits which have high flow capacity owing to the availability of all three sides (other than the kerb side) for inflow. At cul-de-sac there are situations where all four sides of the grated pit can be utilised for stormwater inflow;
- side entry pits which rely on the ability of the opening under the backstone or lintel to capture flow. Side entry pits are usually depressed at the invert of the channel to improve their capture capacity. There are circumstances where the side entry pit is combined with a grated inlet.

The discharge at a pit opening is calculated using the local area contributing to it, and the corresponding time of concentration.

Design Standards for Gutterflow Spread
The gutterflow is defined as the storm water runoff flowing down the gutter, along the face of the kerb. This runoff water accumulates from the road, the verge and other off road catchments.

The gutterflow spread is defined as the width of the water flowing in front of the kerb. The spread can be minimised by catching the runoff water via gullies or kerb openings. This spread needs to be limited to provide a safe driving environment and minimise inconvenience suffered by the public.

Issues of safety and public convenience which need to be considered in determining the spread of gutter flow include the following:-

- vehicles inadvertently drive into the gutterflow causing possible aquaplaning and loss of control;
- vehicles endeavour to drive around the water flowing in the gutter, pushing them into the adjacent traffic lane;
- an inconvenience to pedestrians near the road if they are sprayed when vehicles drive within the gutter flows, or to pedestrians alighting from buses on to the road.

Acceptable spread widths for high traffic volume roads are usually specified for a 2 year ARI storm. This allows for higher intensity storms with durations less than the minimum that can be calculated using Australian Rainfall and Runoff.

This standard is normally lower than that used for the pipe system design. This is because ponding in excess of the specified spread limit is acceptable for storms larger than the 2 year ARI event. In these cases, vehicles tend to significantly lower their speed.

For highways without shoulder, the maximum, spread widths that can be specified for a 2 year ARI storm are:

- Up to a 1/2 a lane on the outside (left lane) of carriageway; and
- Up to a 1/3 of a lane on the median side of carriageway

Spread width limits less than these maximum values should be specified for roads with large traffic volumes.

On high speed roads with shoulders (freeways and major highways) the gutterflow spread should kept within road shoulders for the 2 year ARI storm event.

On most urban highways, more gullies will be necessary than those required to keep the spread to an acceptable level. Additional gullies will need to be provided to minimise flow across side streets, superelevation transitions and eliminate nuisance to pedestrians in the vicinity of the highway (especially at bus stops and high volume pedestrian crossings).

At low points, the designer must locate a secondary gully approximately 50 mm higher than the low point as a contingency if the low point gully becomes blocked. Furthermore, it is necessary to ensure that sufficient gullies are placed to accept the runoff from the design storm considered in the pipe system design (normally ARI greater than that for gullies).

5.4.3.2 Inlet Capacities

(i) Grated Pits (GP)
Grated pits are generally located clear of carriageways, such as in medians, table drains, or catch drains. In general, grated inlets should not be used in a sag of a roadway owing to the potential for debris to significantly reduce the inlet capacity. The capacity of the inlet depends on the depth of ponding.

For a depth less than 120 mm, inflows conditions is similar to that over a weir with crest length equal to the effective outside perimeter of the inlet (neglect kerb side). The inlet capacity can be computed as follows:-

$$Q_i = 1.66 \,.\, P \,.\, d^{1.5} \qquad - \qquad 5.8$$

where
- Q_i is the rate of flow into grate (m^3/s)
- P is the perimeter of grate opening (m), (disregarding bars but subtracting the side against the kerb.)
- d is the depth of water over the grate (mm).

Where the depth exceeds 400 mm, the grate operates as an orifice and the formula below applies:-

$$Q_i = 0.67 \,.\, A \,.\, \sqrt{2gd} \qquad - \qquad 5.9$$

where A is the area of clear opening of grate (m^2).

For a depth between 120 mm and 400 mm, the operation of the grate is indeterminate and both formulae should be applied and the lower discharge value adopted.

Transverse steel bars must be used where pits are likely to be traversed by bicycle traffic. However, transverse bars reduce hydraulic efficiency. As grates are prone to blockage by debris, their use in trapped low points on carriageways is not advisable. Side entry pits are more suitable in this case. Table 5.2 lists suggested factors to be applied to the computed inlet capacity to account for possible blockage due to litter and other flow debris.

Table 5.2
Inlet Capacity Reduction Factors

Condition	Inlet Type	Inlet Capacity Reduction Factor
Sag	Kerb opening	0.80
Sag	Grated	0.50
Continuous Grade	Kerb opening	0.80
Continuous Grade	Longitudinal bar grate	0.60
Continuous Grade	Transverse bar grate or longitudinal bar grate incorporating transverse bars	0.50

(ii) Side Entry Pit (SEP)

Side entry pits can be provided with good entry conditions with little chance of blocking and maximisation of entry efficiency by depressing the throat opening below the line of the channel invert, provision of deflectors (ribs or groves which direct water into the inlet), depressed section in the front of and adjacent to the pit.

The capacities of side entry pits with no inlet depression beside kerb and channel or median barrier are calculated in a similar fashion with Equation (5.10) with weir flow conditions for ponding of stormwater up to 1.4 times the height of the inlet (h), ie.

$$Q_i = 1.66 \,.\, L \,.\, d^{1.5} \qquad - \qquad 5.10$$

where L is the inlet width (m)

For ponding levels higher than 1.4 times the height of the inlet, the orifice flow equation for the inlet capacity is as follows:-

$$Q_i = 0.67 \,.\, A \,.\, \sqrt{2g(d - h/2)} \qquad - \qquad 5.11$$

where h is the height of the kerb inlet (m)

While it is acknowledged that the inlet capacity can be improved significantly by a depressed kerb inlet, there is insufficient data to quantify this. The capture efficiency of the side entry pits is influenced by a number of factors including the slope of the road, the roughness of the road, cross slope etc. In general terms, the capture efficiency decreases with increasing approach flow velocity in the gutter. The quantity of water by-passing the side entry inlet increases as the percentage capture decreases and these flows need to be added to the design flow for the next downstream pit.

To design spacing for 100% capture is generally not necessary. The lower range of percentage captures can be used to advantage on slopes above approximately 5 %. Where 95% capture is used, it is not usually necessary to consider bypass flow in the calculations. When evaluating alternative pit spacings, the aim is to achieve economical design by minimising the total number of pits in the system. Suggested reduction factors for side entry inlets are listed in Table 5.1

5.4.3.3 Inlet and Junction Pits

Change in Horizontal Alignment

Junction pits are required at changes in both horizontal and vertical alignment of a pipeline and at confluences of two or more pipelines where there is no need to admit surface water to the pipe system. Changè of direction and lateral inflow contribute significantly to the change in pressure head due to flow dispersion. Proper direction of flow by means of deflectors are often worthwhile improvements to the hydraulic flow conditions. Shaping of the floor of the pit should be to provide a vertical deflection of flow as shown in Figure 5.7. Figure 5.8 illustrates incorrect installation of flow deflectors which would result in higher energy loss in the pit.

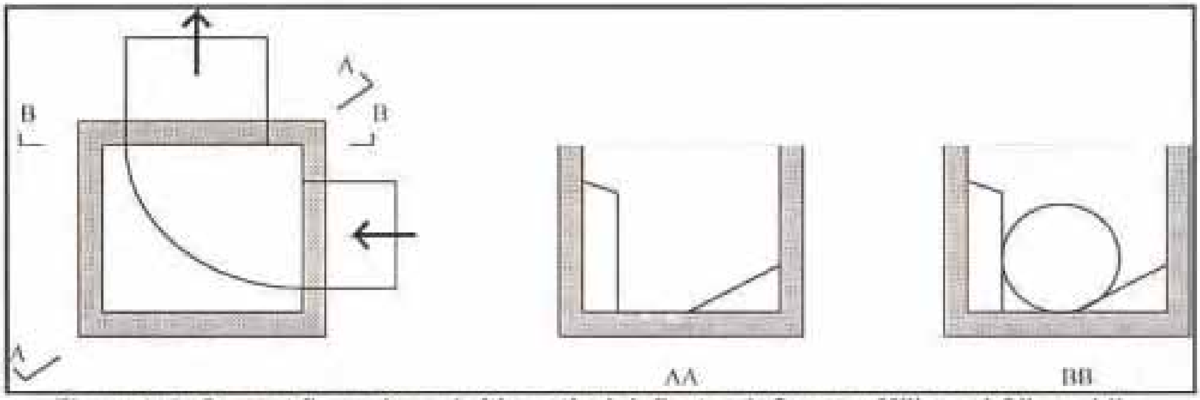


Figure 5.7 Correct floor shape (with vertical deflectors) Source: Mills and O'Loughlin (1986)

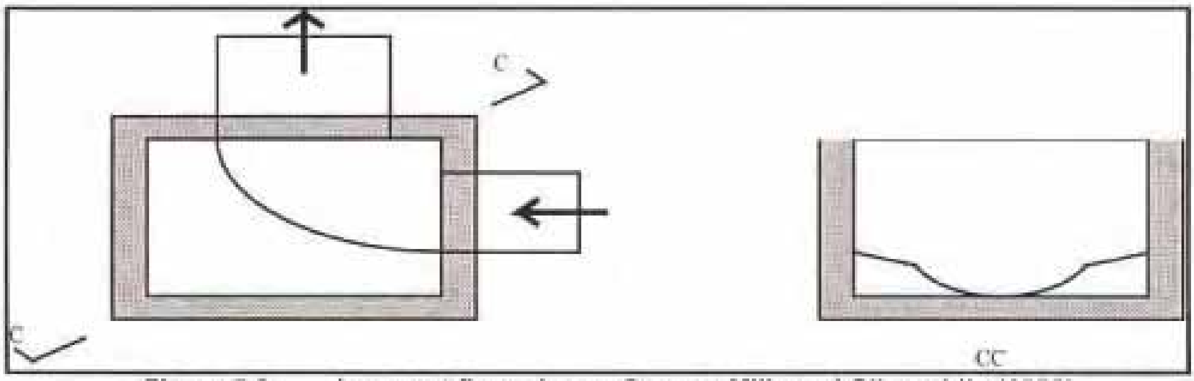
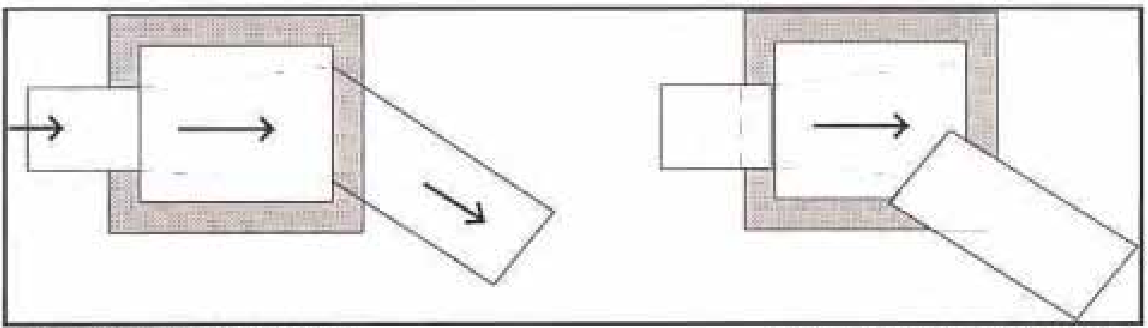


Figure 5.8 Incorrect floor shape. Source: Mills and O'Loughlin (1986)

Other good practices in minimising the energy loss at junctions include the proper alignment of the outlet pit, as shown in the Figure 5.9. For example, at bends, the proper location of the outflow pipe is with its face at the intersection of the pipe centre line.



Correct Alignment of Outlet Pipe
(Mills and O'Loughlin, 1986)

Incorrect Alignment of Outlet Pipe
(Mills and O'Loughlin, 1986)

Figure 5.9 Alignment Of Pipes At Bends

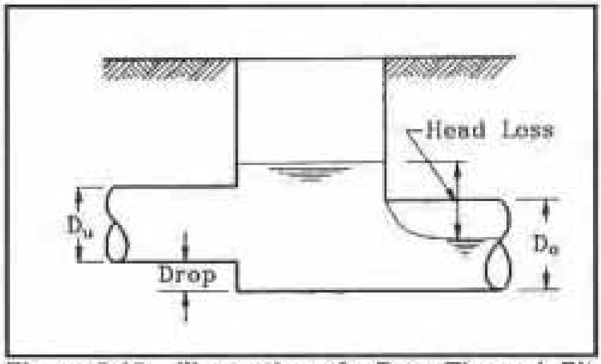


Figure 5.10 Illustration of a Drop Through Pit

Change in Vertical Alignment

It is desirable to provide drop through pits in situations of steep terrain or when a significant change in the vertical alignment of the pipeline is required. Drop through pits are desirable to avoid the need for excessive energy dissipation at the end of the pipe. The drop measured between inlet and outlet pipe inverts is only equal to the head loss if the pipes are flowing full and are of equal diameter. A nominal drop of 0.1 metres is usually sufficient, except for pits

Where major flows join, or flow direction changes in which cases the head loss should be calculated. In flat country, where the fall may not be sufficient to provide the nominal drop, the pipe inverts may lie on a continuous gradeline, provided that the bottom of the pit is shaped to match the lower half of the pipe, and the hydraulic gradeline is checked to detect any adverse effects.

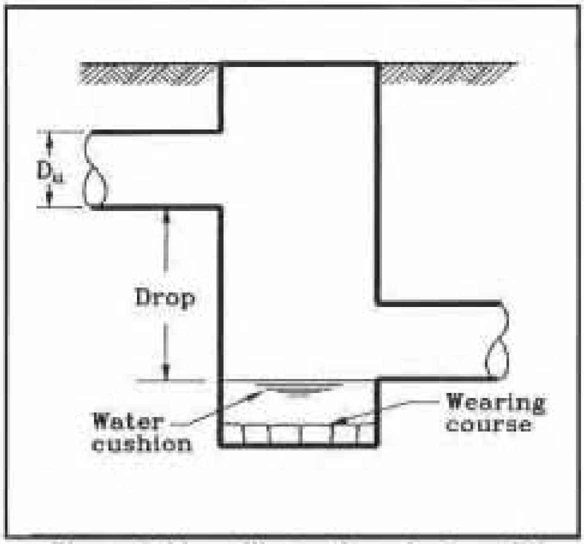


Figure 5.11 Illustration of a Drop Pit

Significant changes in level through pits may be necessary in order to avoid existing public utility services, or to convey water down a batter, or as a deliberate means of reducing the energy of flow. This type of pit is called a drop pit as shown in Figure 5.11. Where the drop exceeds 2 metres the pit floor should be protected by a wearing course of concrete or rock, and the outlet should be placed about 0.3m above the floor to leave a permanent water cushion.

5.4.3.4 Computing the Hydraulic Grade Line

The hydraulic grade line of the drainage system is determined by applying the equation of conservation of energy. The energy loss between within a pipeline is made up of energy loss due to pipe friction and those due to localised "shock" losses.

Energy losses due to friction along the pipe is reflected in the gradual reduction in the TEL as one moves in the downstream direction. The gradual change in the TEL is often interrupted by significant energy loss at inlet and junction pits. Energy losses are expressed as a function of the velocity in the outlet or downstream pipe, ie.

$$h_L = k\frac{v_o^2}{2g} \qquad - \qquad 5.12$$

where
- h_L is the energy loss (m)
- k is the energy loss coefficient (dimensionless)
- v_o is the flow velocity immediately downstream of the point of interest (m/s)

Pipe Friction

For pipe friction, the value of the energy loss coefficient is often defined as follows:

$$k = \frac{fL}{D} \qquad - \qquad 5.13$$

where
- L is the length of the pipe (m)
- D is the diameter or equivalent diameter of the pipe (m)
- f is the friction factor which can be obtained from standard graphs such as the "Moody Diagram" or approximated by the equation

$$f = \frac{1.325}{\left[\ln\left(\frac{e}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right]^2} \qquad - \qquad 5.14$$

where e is the pipe wall roughness (m)
Re is the Reynolds Number

Table 5.3 lists typical values of the pipe wall roughness (e).

Table 5.3
Typical Pipe Roughness (e) in mm (ref. ARR)

Pipe Material	Pipe Conditions		
	Good	Normal	Poor
Concrete			
Precast with, "O" Ring Joints	0.06	0.15	0.6
Spun precast, "O" Ring Joints	0.06	0.15	0.3
Monolithic construction against steel forms	0.3	0.6	1.5
Monolithic construction against rough form	0.6	1.5	
Asbestos Cement	0.015	0.03	
UPVC			
with chemically cemented joints		0.03	
with spigot and socket joints		0.06	

Pit Losses

The pressure head change at pits are computed in the same way as energy losses, with the change expressed as a function of the outflow velocity head, ie.

$$\frac{\Delta P}{\gamma} = k_u \frac{V_o^2}{2g} \qquad - \qquad 5.15$$

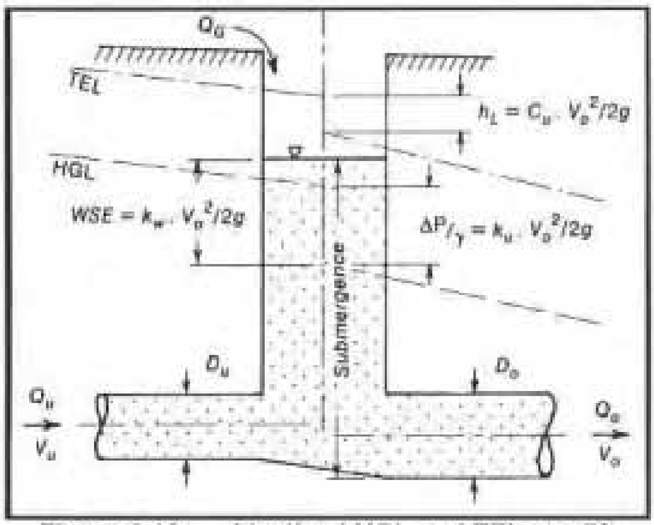


Figure 5.12 Idealised HGL and TEL at a Pit
source: Inst. Engrs., Aust., 1987

The water level in the pit is slightly higher than the HGL as illustrated in Figure 5.12 owing to the reduction in the velocity head from that at the inlet pipe (converted to pressure head) as the water enters the pit. However, for most situations, the water level in a pit is assumed to coincide with the HGL level. Computation of pressure changes at pits thus centred around the determination of the appropriate pressure change coefficient. Probably the most significant data available for the determination of the pressure change coefficients are the "Missouri Charts" published by Sangster et al. (1958). The charts are voluminous and cover a number of pit configurations.

The Missouri Charts

The Missouri Charts dealt with pipes flowing full for pit configurations involving grate flow, lateral pipe inflow and through flow. Thus, in applying the charts, the computed velocity head at the outlet pipe needs to be based on pipe full flow conditions even if it is known that the pipe flows part full. The values of the pressure change coefficient for the different inflow configurations depend on such parameters as:

- the submergence ratio (depth of water/diameter of outflow pipe, d/D_O)
- the ratio of upstream and downstream pipe diameters (D_U/D_O)
- flow ratios (eg. lateral inflow to outflow Q_L/Q_O, grate flow to outflow (Q_G/Q_O), etc.)
- pit dimension ratios (eg. width to outlet diameter, B/D_O)

Grate inflows cause the highest change in pressure head and the submergence ratio d/D_o has a strong influence on the pressure change coefficient in situations involving grate inflows. In the case of grate flows only, the pressure change coefficients varied from 6.0 for a submergence ratio of 1.6 to a k_u value of 1.8 for submergence ratio exceeding 7.0. In these situations, an iterative computation procedure needs to be adopted where an estimate of the pit water level is first made to enable the submergence ratio to be computed. This would then allow the value of k_u to be read from the chart which can then be used to compute the pressure change from the HGL at the outflow pipe and thus the water level in the pit. This water level is then compared with the initially estimated water level and if found to be significantly different, the computed water level is then used to re-evaluate the submergence ratio and hence the value of k_u. This process continues until convergence is achieved.

The ratio of upstream and downstream pipe diameters, D_u/D_o , has a significantly influence on the pressure change coefficient for flow through pipe configurations. K_u values can range from -3.5 to 1.1 for diameter ratio (D_u/D_o) of 0.6 to 1.4 respectively. Negative values of k_u suggest a pressure recovery as a result of a lower downstream velocity head due to a larger pipe diameter. K_u values increases with the introduction of grate flow, with the increase being a function of the proportion of the total outflow attributable to the grate inflow.

Pressure change coefficients for lateral inflows in rectangular and square pits are treated separately. For the case of lateral inflow in addition to a flow through situation in a rectangular pit, the pressure change coefficient is dependent on the proportion of the total outflow attributable to the lateral inflow and the ratio of the respective pipe diameters to the outflow pipe diameter. In the case of lateral inflows (from both sides of the pit), pressure change coefficients due to each individual lateral inflow are computed. The corresponding water levels are computed from the two pressure change coefficient and the lower of the two water levels is adopted. In all cases, the inclusion of grate flow would further increase the pressure change coefficient.

For square pits, an additional parameter affects the pressure change coefficient. The parameter is the ratio of the pit width to the outflow pipe diameter B/D_o. The pressure change coefficient increases with increasing B/D_o ratios but converges towards a narrow band of values for when the ratio of the lateral pipe diameter to the outflow pipe diameter is high.

Adjustments for deflectors are also included in the Missouri Charts. Deflectors essentially eliminates the influence of the shape of the pit.

Simplified Method for Calculating Pit Losses

Argue (1986) presented some simplifying tables from which estimates of the pressure change coefficients can be made. These data are presented in Tables 5.4 and 5.5, the first indicating no grate flow and the other indicating the inclusion of grate flow.

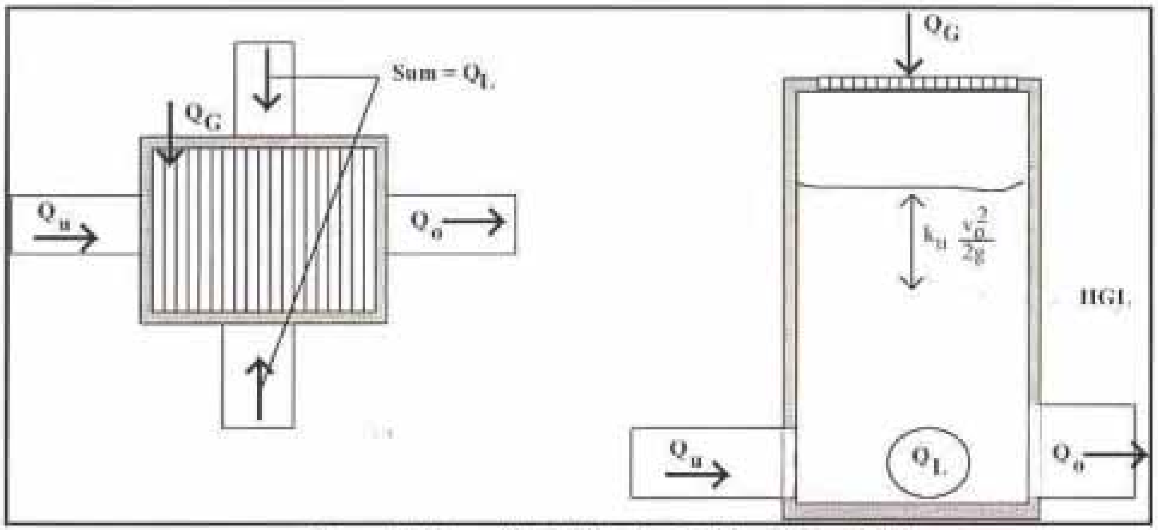


Figure 5.13 Illustration for Tables 5.3 and 5.4

Table 5.4
Junction Pits Without Grate Flow (Argue, 1986)

Description	Q_u~	Q_L~	Q_G~	k_u
Junction pit on through pipeline, ie. $Q_u = Q_o$	Q_o	-	-	0.2
Junction on through pipe with lateral inflows				
1. $Q_u >> Q_L$	Q_o	some		0.5
2. $Q_u \sim Q_L$	$Q_o/2$	$Q_o/2$		1.0
3. $Q_u << Q_L$	some	Q_o		2.0
Junction piton "L" pipe junction, ie. Q_u=0	-	Q_o	-	2.0
Junction pit on "T" pipe junction, ie. Q_u=0				
1. Opposed Lateral	-	Q_o	-	2.5
2. Offset Lateral	-	Q_o	-	2.0

Table 5.5
Inlet/Junction Pits With Grate Flow (Argue, 1986)

Description	Q_u~	Q_L~	Q_G~	k_u
Inlet pit with outflow pipe only	-	-	Q_o	5.0
Inlet on through pipeline				
1. $Q_u \sim Q_G$	$Q_o/2$	-	$Q_o/2$	2.0
2. $Q_u = Q_o$	Q_o	-	some	0.5
Inlet on through pipe with lateral inflows				
1. $Q_u >> Q_L$	Q_o	some	some	0.5
2. $Q_u >> Q_L$	$Q_o/2$	some	$Q_o/2$	1.5
3. $Q_u \sim Q_L$	$Q_o/2$	$Q_o/2$	some	1.5
4. $Q_u << Q_L$	some	Q_o	some	2.0
5. $Q_u << Q_L$	some	$Q_o/2$	$Q_o/2$	2.5
Inlet on "L" pipe junction, ie. Q_u=0	-	Q_o	some	2.5
Junction pit on "T" pipe junction, ie. Q_u=0				
1. Opposed Lateral	-	Q_o	some	3.0
2. Offset Lateral	-	Q_o	some	2.5

Adjustments to the above values of k_u for any change in grade were given as follows:-

- change in grade from flat to steep in the direction of flow : add 0.5
- change in grade from steep to flat in the direction of flow: subtract 0.5

5.4.4 Designing Open Channel Systems

Open channels are commonly used to convey stormwater through both the urban and rural landscape. Proper design of such open channels should give consideration to a number of aspects including the following :

Optimum channel route

- land use considerations;
- economic considerations;
- site topography constraints.

Longitudinal invert slope

- leave as naturally existing;
- flatten, to reduce velocities;
- by using drop; structures or meanders;
- steepen, to increase velocities by using a more direct route.

Cross-sectional shape

- available easement/reserve width limitations;
- flow velocity/depth considerations;
- overall planning considerations.

Channel lining material

- leave as naturally existing;
- hard or soft facing;
- aesthetics and recreational considerations;
- maintenance and safety considerations.

Flow characteristics

- subcritical flow conditions;
- supercritical flow conditions.

In most situations, open channels will operate under subcritical flow conditions and most of the procedures and the recommendations presented are relevant to the design of typical open channels operating under subcritical flow conditions. Open channels operating under or approaching supercritical conditions should normally be avoided owing to significantly high flow velocity and the likelihood of occurrence of hydraulic jumps causing erosion problems. Where such situations cannot be avoided it should be noted that specialist design knowledge is required.

5.4.4.1 Open Drain Types

In the rural environment, the main function of open drains are to:

- Intercept, collect and direct runoff which is draining naturally towards the roadway from adjacent catchment areas;
- collect runoff from the road formation and adjacent cutting and embankment slopes;
- Provide the means for containing and draining these waters to points of disposal.

The types of drains commonly utilised in drainage of main roads and highways are summarised in Table 5.6. A typical longitudinal drain is shown in Figure 5.14.

Table 5.6
Types and Functions of Drains

Types of Drain	Function	Design Considerations
Table Drains	collects and convey the surface runoff generated from the roadway and adjoining area may also be used to intercept subsoil seepage	typical cross section of table drains is triangular with a side slope of 1 in 4 or flatter the minimum depth from the edge of the shoulder of the road to the invert is 0.3 m if higher discharge capacity is required, a trapezoidal shaped section may be used table drains should be free of long term water pondage and the slope of the drain should be selected to ensure that water is readily conveyed to the drainage outlet
Diversion Drain	in rural roads, diversion drains are used to convey water collected at the table drain to the designated drainage outlet	typical cross section of table drains is triangular with a side slope of 1 in 4 or flatter stormwater conveyed by the diversion drain is often discharged into side drains, creeks or allowed to spread freely over the countryside in order that stormwater collected by the table drain are efficiently conveyed away from the road diversion drains are often necessary at between 150 m to 300 m intervals along the road for undulating terrain, reducing to 100 m on steeper topography it is necessary to block the table drain immediately downstream of its confluence with the diversion drain to ensure effective diversion of stormwater away from the table drain.
Catch Drain	catch drains are used to intercept and convey stormwater water runoff along the top of a cutting to designated discharge points so as to prevent runoff from flowing down the unprotected cut batter	typical cross section of table drains is triangular with a side slope of 1 in 4 or flatter catch drains are generally steep channels to effectively and economically convey runoff to designated discharge points and thus often experience high flow velocities consideration need to be given to channel lining and energy dissipation at the designated discharge points
Side Drain	side drains are used to isolate the road from runoff generated from adjoining areas	typical cross section of table drains is triangular with a side slope of 1 in 4 or flatter side drains can often be constructed by the use of a small levee when it is undesirable to excavate a drain

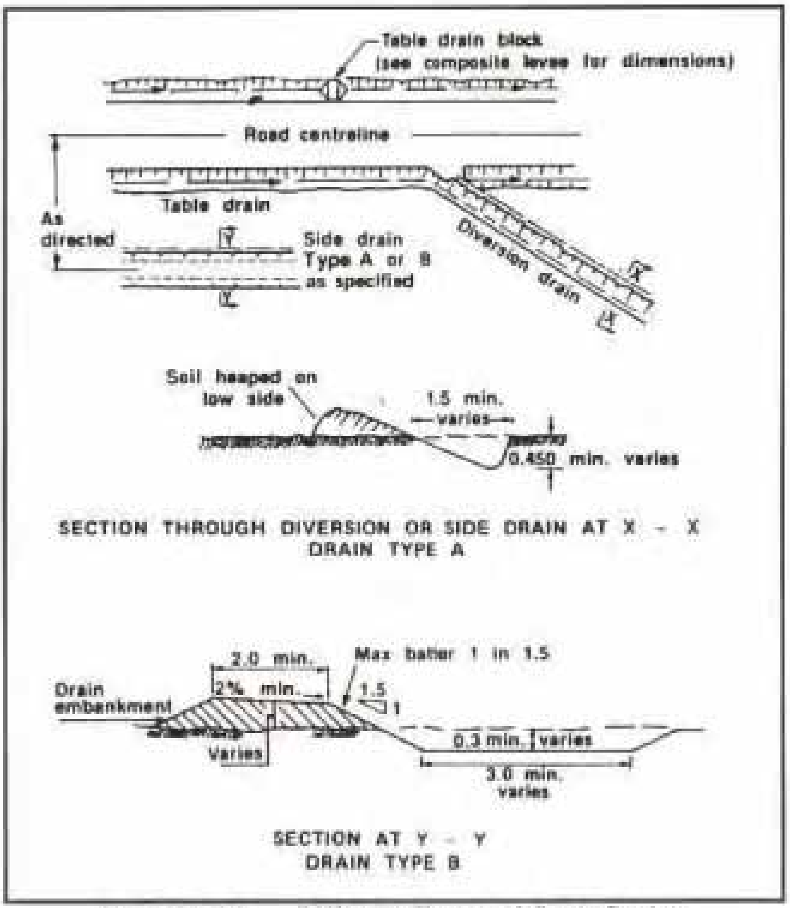


Figure 5.14 Different Types of Open Drains

In selecting the appropriate gradients of open channels, consideration should be given to the soil type in order to avoid erosion and scour of the channel. Earthen channels with grades steeper than 4% will almost invariably be subjected to erosion and scour problems. Consideration should be given to erosion mitigation measures with the use of drop structures to control the slope of the channel being preferred. If catchment conditions are not suited to the use of drop structures, erosion mitigation measures will include channel lining varying from grassing to bituminous surfacing and stone pitching. Channel side slopes should be designed and constructed with due consideration of public safety and ease of maintenance with equipment normally available in the area.

5.4.4.2 Hydraulic Design Procedure

Manning's equation and other empirical slope-area methods are most commonly used throughout Australia for the purpose of computing flow depths and velocities in open channel. These equations are based on the assumptions of flow in a uniform prismatic channel and the flow depth computed is referred to as the normal depth. The Manning's equation is perhaps the most commonly used and design data for various reach types are readily available from standard hydraulic text books (as outlined later in this Chapter).

Manning's equation may be expressed in terms of flow velocity or discharge. The uniform flow velocity is expressed as:

$$V = \frac{1}{n} R^{\frac{2}{3}} \sqrt{S} \qquad - \qquad 5.16$$

and Equation 5.16 becomes the same as Equation 5.6 when the discharge is expressed as

$$Q = A \times V \qquad - \qquad 5.17$$

where V = velocity, (m/s)

n	= Manning's roughness coefficient
A	= cross-sectional area of flow, (m^2)
R	= hydraulic radius; R = A/P
P	= wetted perimeter (m)
S	= slope of channel bed, (m/m)
Q	= flow, (m^3/s)

The design of open channels using the slope-area method involves the selection of the longitudinal invert slope, overall channel size and cross-section shape such that the resulting normal flow depths and flow velocities satisfy the design objectives. The use of slope area methods is strictly applicable for long uniformly shaped channels with no special structures on the watercourse, typical of table drains and side drains.

For natural channels, slope-area methods such as Manning's equation can only be used for preliminary estimation of flow depths and velocities for open channel design. This is because normal depth flow conditions occur only rarely in practice in natural watercourses, the general conditions being that of gradually varied flow with water levels (and consequently flow velocities) being influenced by irregular channel shapes, changing bed slope, variation in hydraulic roughness, backwater effects and water level controls at critical sections along the watercourse. Water surface profiles in natural watercourses could include possible fluctuations between subcritical and supercritical flow along the length of the channel. In such circumstances, the design engineer must identify any variations in channel flow conditions and to determine the extent of their influence. Water surface profile calculations involving iterative finite difference calculations towards solving the energy equation will need to be carried out at a series of discrete cross-sections along the channel.

It must be emphasised that all of the procedures and computer programs associated with the design of open channel systems require the design engineer to be fully familiar with the relevant hydraulic processes, and in the case of computer programs, their computational procedures and limitations.

5.4.4.3 Subcritical and Supercritical Flow

The flow in open channels may be classified according to the level of energy contained in the flow itself as represented by the Froude number. The subcritical range has Froude numbers less than 1 and is characterised by low velocities and high depths found typically on hydraulically mild slopes. Supercritical flow has a Froude number greater than 1 and is characterised by high velocities and shallow depths developed in a hydraulically steep slope channel. Critical flow conditions occur when the Froude number equals 1 and the normal depth is equal to the critical depth for the flow element.

The Froude number is defined as:

$$F = \frac{V}{(gd_m)^{1/2}} \qquad - \qquad 5.18$$

where V is the average velocity in the cross-section
g is the acceleration due to gravity, and
d_m is the hydraulic mean depth, expressed as the ratio of the cross sectional area (A) and the top water width (T); A/T:

The classification of the flow regime according to subcritical, critical, or supercritical conditions is important for a number of reasons, most important of which is its used in defining the general shape of the water surface profile and to enable the Design Engineer to appreciate how changes in localised flow conditions would influence the water surface profiles upstream and downstream of the section in question. Figure 5.15 shows the six standard water surface profiles for mild slope and step slope conditions.

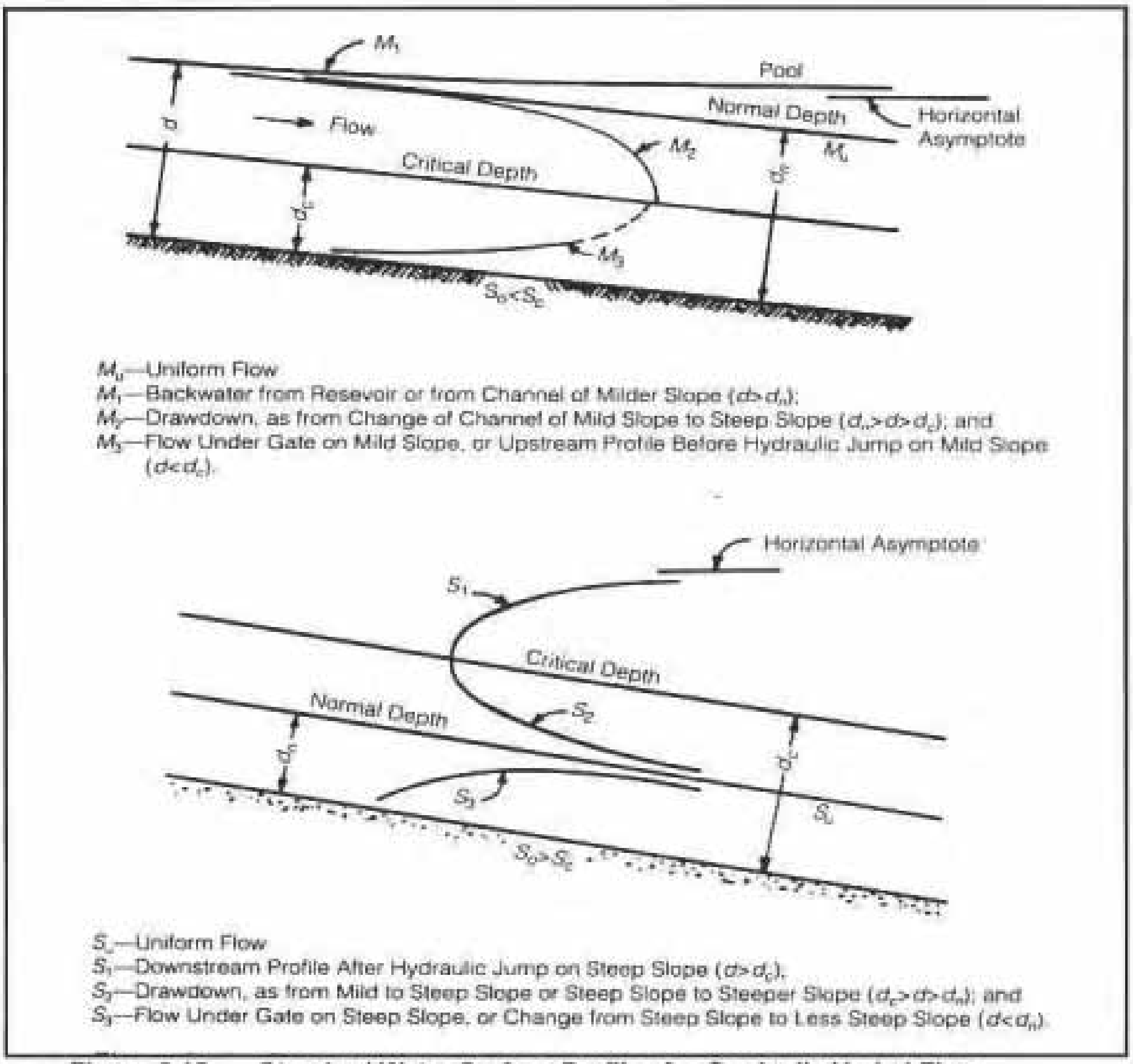


Figure 5.15 Standard Water Surface Profiles for Gradually Varied Flow

For mild slope and subcritical flow conditions, the water surface profiles depart from the normal depth levels due to downstream influences. This is shown by the M_1 and M_2 profiles. The M_1 profile demonstrates the situation where the water surface profile is influence by high tailwater level where the water surface elevations along the open channel increase from that associated with the normal depth towards the elevation determined by downstream conditions associated with a permanent water level, a constriction in the channel or a flattening of the channel slope. Similarly, the M_2 profile reflects the situation where flow depths along the channel depart from the normal depth and the water surface profile is being drawn down towards a water level controlled by downstream conditions associated with a weir or change (increase) in channel slope. In these two cases (ie. M_1 and M_2 profiles) the water surface profiles are influenced by downstream factors and the length of water elevation transition from normal depth to the downstream control depth is dependent on the degree in which the downstream depth differ from the normal depth. Consequently, water surface profile computations always begin at the downstream control section and proceed upstream.

For steep slope and supercritical flow conditions, the water surface profiles approaches the normal depths and the shape of the water surface profiles is defined by water elevations fixed by upstream influences. In the case of the S_2 profile, upstream influences have led to a water level at the upstream end of the profile which is higher than the elevation corresponding to the normal depth, typically of flow profiles immediately downstream of a weir or other critical depth control sections or when flow is discharged into a steeper channel. The water surface profile is downstream of this control section is drawn down towards the normal depth water profile. Similarly, the S_3 profile demonstrates the condition where water elevations approach the normal depth elevation from a lower starting level, typical of profiles under sluice gates or from a channel of steeper slope to one with a less steep slope. In these two cases (ie. M_2 and M_3 profiles) the water surface profiles are influenced by upstream factors and the length of water elevation transition from the upstream water elevation to the normal depth elevation is dependent on the degree in which the upstream depth differ from the normal depth. Consequently, water surface profile computations always begin at the upstream control section and proceed downstream.

Water surface profiles M_3 and S_1 represent conditions preceding and following a hydraulic jump respectively. In these situations, the water surface profiles are generally of less relevance to design and the design focus would generally be directed at structures to initiate the hydraulic jump and to contain the hydraulic jump region. The design engineer is referred to design guides published by the US Bureau of Reclamation (1973) when designing these structures.

5.4.4.4 Use of Manning's Equation

In applying the Manning's Equation, particularly to natural channels, the greatest difficulty lies in the determination of Manning's roughness coefficient "*n*". The choice of an appropriate value for the roughness coefficient of an open channel is often critical in the overall design procedure and requires a considerable degree of judgement.

The Manning's roughness coefficient is dependent on a number of variables including surface roughness, vegetation, channel irregularity, the presence of obstructions, channel alignment and the likelihood of sedimentation or scour. It is important to note also that it is dependent on the relative depth of flow at a cross-section. Generally as flow depths increase, the relative effect of bed roughness decreases, while the effect of bank roughness may increase.

The above variables can be considered as individual components of an overall effective Manning's roughness coefficient for the particular channel. Various references and procedures are available to assist the designer to estimate the applicable channel roughness coefficients:

Tables relating the channel type and its associated lining, to recommended roughness coefficients, e.g. Argue (1986) Table 6.1, ARR-1987 Tables 5.1 and 14.17, Henderson (1966) Table 5.2, Chow (1959) Table 5.6;

Photographs and descriptions of channels with known roughness coefficients, e.g. Chow (1959), Barnes (1967) and French (1985);

Equations to derive estimates of channel roughness and which incorporate modifying factors representing the individual components of the effective Manning's roughness coefficient, e.g. ARR-1987 Section 5.8, Chow (1959) Table 5.5 and French (1985);

Determination of Manning's roughness coefficients along a gauged naturally existing channel reach, by calibrating calculated water surface profiles to observed flood heights.

Designers are referred to Section 5.8 of ARR-1987 where each of the above aspects are discussed in considerable detail.

5.4.4.5 Manning's Roughness Coefficient of Composite Channels

Estimation of an equivalent or composite Manning's roughness value in a channel of varying roughness is often required where there is a marked variation in boundary roughness across an individual cross section. In the case of an open channel designed within a typical urban environment, examples of this situation could include a grassed channel containing a lined low flow drain within the channel invert, or a channel containing a low level access berm along one side of the channel. The procedures recommended below involve the determination of wetted perimeter, hydraulic radius and Manning's roughness coefficient for each segment representing the varying zones of roughness across the channel section.

Equations for determining Manning's roughness coefficients in channels of composite roughness include:

$$n = (\sum[P_i.n^{3/2}] / P)^{2/3}$$

$$\Rightarrow P.R^{5/3} / \sum[P_i.R_i^{5/3} / n_i)]$$

$$\Rightarrow (A^{5/3}.P^{2/3}) / \sum(A_i^{5/3} / n_i.P_i^{2/3}) \qquad 5.19$$

where
- n = equivalent composite Manning's roughness coefficient for entire cross-section
- P = wetted perimeter of whole cross-section (m)
- R = hydraulic radius of whole cross-section = A/P
- n_i = Manning's roughness coefficient for segment i
- P_i = wetted perimeter of segment I (m)
- R_i = hydraulic radius of segment I (m)

It is recommended that use of Equation 5.19 be restricted to simple cross sections with no extensive overbank flow areas.

The design engineer should note that both of the procedures presented above have limitations which are largely dependent on the cross-sectional shape of the channel, and on the configuration of the respective segmented zones of varying roughness. Use of these equations may, in certain circumstances, generate distorted composite Manning's roughness coefficients which would lead to inaccurate predictions of channel flow conditions. In the situation where a channel consists of a relatively deep and narrow central flow area, with relatively shallow and wide overbank flow areas, use of Equation 5.19 may generate a composite roughness coefficient which is excessively weighted by the large wetted perimeter of the overbank flow areas, even though these areas may carry only a small proportion of the total flow.

The reverse situation would lead to a composite roughness coefficient which is lower than any of the individual roughness coefficients of the contributing channel segments.

5.4.6 Design Velocities for Grassed and Earth Channels

Flow velocities in channels lined with soft facings should be limited to avoid scour. There are various means of restricting flow velocities, including the reduction of channel invert slopes by the installation of channel drop structures, the increase in flow resistance through the selection of appropriate channel cross sections with relatively large wetted perimeters, the reduction of channel longitudinal slopes by increasing the length of flow path, and the increase in channel roughness with vegetation and rocks.

Design of drop structures should ensure that the associated high turbulence zone is contained within the structure itself. This may involve the provision of baffle blocks downstream of the drop face and the provision of flexible rock mattress protection at the upstream face of the drop structure to cater for increased velocities associated with flow draw down effects. The Design engineer is referred to the procedure of Peterka (1984) when designing drop structures in open channels.

Recommended design flow velocities for consolidated bare earth and vegetated channels are summarised in Table 5.6. It is recommended that the average flow velocity under major storm design conditions not exceed the indicated permissible velocity in Table 5.7. Care should also be taken to ensure that localised flow velocities are not excessive (e.g. at the edge of low flow channels) under major and lesser storm conditions. Where channels of composite shape or surface cover are involved the flow should be sectorised and the average flow velocity in each sector compared with the permissible values in Table 5.7.

Channel Side Slopes

The maximum channel side slopes for grass lined sections should preferably be 1 in 6 (1V on 6H), with an absolute maximum of 1 in 4 (1V : 4H). If grass lined channels are designed with side slopes steeper than 1 on 4 (1V : 4H), regular maintenance may become impractical and the channel may eventually become overgrown with vegetation which would consequently reduce the flow capacity of the channel.

Channels lined with ground covers may be permitted to have maximum side slopes of 1 on 2 (1V : 2H), however design of such channels must include considerations of potential future increases in channel roughness, and considerations of channel side slope stability.

Table 5.7
Permissible Velocities for Consolidated Bare Earth Channels andVegetated Channels

Channel Gradient (%)	**Permissible velocities (m/s) when percentage of stable vegetation Gradient cover (¹) is:**			
	0 % (2)	**50 %**	**70 %**	**100 %**
Erosion resistant soils				
1	0.7	1.6	2.1	2.8
2	0.6	1.4	1.8	2.5
3	0.	1.3	1.7	2.4
4		1.3	1.6	2.3
5		1.2	1.6	2.2
6			1.5	2.1
8			1.5	2.0
10			1.4	1.9
15			1.3	1.8
20			1.3	1.7
	Easily eroded soils			
1		0.5 1.2	1.5	2.1
2		0.5 1.1	1.4	1.9
3		0.4 1.0	1.3	1.8
4		1.0	1.2	1.7
		0.9	1.2	1.6
6			1.1	1.6
8			1.1	1.5
10			1.1	1.5
15			1.0	1.4
20			0.9	1.3

Notes:

(1) Designers should assess the percentage of stable vegetal cover likely to persist under design flow conditions. However it should be assumed that under average conditions the following species are not likely to provide more than the percentage of stable vegetal cover indicated:

- Kikuyu, Pangola and well maintained Couch species - 100%
- Rhodes Grass, poorly maintained Couch species - 70%
- Native species, tussock grasses - *50%*

(2) Applies to surface consolidated, but not cultivated.

This appendix presents a brief summary of the following Hydrological Procedures published by DID as discussed in the paper by Quek (1993):

1. Rational Method- HP5 (DID, 1974)
2. Design Hydrograph Method- HP11 (DID, 1980)
3. Regional Flood Frequency Analysis- HP4 (DID, 1987)
4. Urban Rational Method- HP16 (DID, 1976)

The readers are advised to refer to the references for a full description of the procedures. The material presented herein is aimed at giving the readers a broad overview of the theory and basis in the derivation of each method. As all the procedures discussed herein are published in imperial units, the following is a conversion table for SI and imperial units:

1 mm= 0.03937 in	1 in= 25.40 mm
1 m= 3.281 ft	1 ft= 0.3048 m
1 km= 0.6214 mi	1 mi= 1.609 km
1 m^3/s= 35.31 ft^3/s	1 ft^3/s= 0.02832 m^3/s
1 km^2= 100 ha= 0.3861 mi^2= 247.1 ac	1 mi^2= 640 ac= 2.59 km^2

1 The Rational Method, HP5 (DID, 1974)

The Rational Method in HP 5 (DID, 1974) relates the peak discharge statistically to the rainfall via the formula:

$$Q_T = C\, i_T\, A \qquad (1)$$

where

Q_T is the peak discharge of the design flood in ft^3/s with return period T years

i_T is the average intensity of the design rainstorm of duration equal to the time of concentration T_c and of return period T year in in/hr

A is the catchment area in acres

C is the coefficient of runoff which is dimensionless.

The time of concentration T_c in hours is given by the relation

$$T_c = \frac{0.434\, A^{0.111}\, L}{S^{0.467}} \qquad (2)$$

where

L is the length of the main stream in miles

A is the catchment area in square miles

S is the slope from the main stream-catchment boundary intersection to the design point, measured along the main stream in per cent.

The basic steps involved in the application of the procedure are as follows:

(a) Estimate the critical duration of the design storm T_c using Equation 2;

(b) Compute the mean intensity i_T for duration T_c and return period T using HP1 (DID, 1982);

(c) Estimate C from storm and catchment characteristics;

(d) Compute Q_T for various return period T using Equation 1.

In HP5 (DID, 1974), the selection of the runoff coefficient C (Figure 3 of the Procedure) is related to the intensity of the one in ten year design storm and catchment location in Peninsular Malaysia, which is divided into Region A for East Coast and B for West Coast for the determination of the runoff coefficient.

A total of 16 gauged small rural catchments situated throughout the Peninsular, with areas ranging from 13 to 104 km^2 (5 to 40 mi^2), were used in the development of the procedure.

2 Design Hydrograph Method- HP11 (DID, 1980)

The development of HP11 (DID, 1980), which gives the complete hydrograph as well as the peak discharge of a catchment, consists of three basic components:

(a) a design storm
(b) a rainfall-runoff relationship which converts the design storm into volume of runoff, and
(c) a dimensionless triangular hydrograph which gives the time distribution of the volume of runoff.

The design storm rainfall, P, is calculated for a range of durations using HP1 (DID, 1982).

The rainfall-runoff relationship was developed based on data from 97 storms from 19 catchments. The depth of the direct runoff Q in inches is given by Equation 3.1 of HP11 (DID, 1980) for storm rainfall less than three inches:

$$Q = 0.33\, P \qquad 3$$

or from Equation 3.2 of HP11 (DID, 1980) for storm rainfall greater than three inches:

$$Q = \frac{P^2}{(P+6)} \qquad 4$$

The peak discharge of the triangular hydrograph, q_p, in ft^3/s, is given by Equation 3.5 of HP11 (DID, 1980):

$$q_p = \frac{D_p \cdot A \cdot 640 \cdot Q}{(L_g + D/2)} \qquad 5$$

where

D_p is the peak ordinate of the dimensionless hydrograph that is characteristic of the catchment, whose value is given in Table 2 of HP11 (DID, 1980) for three hydrological groups
L_g is the catchment lag in hours
Q is the direct runoff in inches
D is the duration of storm rainfall in hours
A is the catchment area in mi^2.

The catchment lag is related to the physical characteristics that determine the storage behaviour of the catchment, the values of which were derived from analysis of data from 38 catchments in Peninsular Malaysia. Equation 3.12 of HP11 (DID, 1980) gives the lag time in hours for the catchment via:

$$L_g = C_t\ (L \cdot L_c / \sqrt{S})^n \qquad \textbf{6}$$

where

L	is the main stream length from the outlet to the catchment boundary in miles
L_c	is the main stream length from the outlet to the catchment centroid in miles
S	is the weighted mean stream slope in ft/mi
C_t, n	are constants whose values are given in Table 1 of HP11 (DID, 1980) for the three different hydrological groups.

The product LL_c is a measure of the size and shape of the catchment, and S is a measure of catchment topography.

A design baseflow of 5 ft^3/s per square mile is added to the computed peak discharge to obtain the total design flood peak.

3 Regional Flood Frequency Analysis- HP4 (DID, 1987)

In regional flood frequency analysis, individual frequency curves from gauged sites are averaged to form a regional curve which is postulated to apply to all catchments in the same region. Basically, the method involves the development of two components:

(a) A set of regional regression equations relating the mean annual flood to the catchment characteristics of catchment area and mean annual catchment rainfall; and
(b) A set of dimensionless regional frequency curves relating Q_T/MAF to the return period T where Q_T is the peak discharge of T year return period and MAF is the mean annual flood.

A total of 61 stations with lengths of records vary from 8 to 36 years, with areas varying from 20 to 19000 km^2, in Peninsular Malaysia were selected for the analysis.

The mean annual catchment rainfall for the region is estimated from the Mean Annual Rainfall Isohyetal Map for Peninsular Malaysia for 1950 to 1985 as given in Appendix C of the Water Resources Publication No. 19 (DID, 1988). Map 2 of HP4 (DID, 1987) divides the whole of Peninsular Malaysia into six MAF regions.

The catchment characteristics are assumed to be related to the MAF via the regression equation:

$$MAF = c\ A^a\ R^b \qquad \textbf{7}$$

where

MAF	is the mean annual flood in m^3/s;
A	is the catchment area in km^2;
R	is the mean annual catchment rainfall in m; and
c,a,b	are the catchment characteristics constants as given in Table 2 of the procedure.

The flood frequency (FF) region of the catchment is determined from Map 1 of HP4. The value of Q_T/MAF is determined from the flood frequency curve in Figure 1 of the procedure. The value of Q_T is determined as the product of Q_T/MAF and MAF from Equation 7:

$$Q_T = \frac{Q_T}{MAF} \cdot MAF \quad (8)$$

4 Urban Rational Method- HP16 (DID, 1976)

In HP16 (DID, 1976), the peak discharge is related to the rainfall intensity and catchment area via the Modified Form of the Rational Method which has an additional coefficient C_s to take into account the effect of channel storage:

$$Q_T = C_S \; C \; i_T \; A \quad (9)$$

where

Q_T is the peak discharge of the design flood in ft^3/s with return period T years

i_T is the average intensity of the design rainstorm of duration equal to the time of concentration T_c and of return period T year in in/hr

A is the drainage area in acres

C is the coefficient of runoff depending on the characteristics of the drainage area, and

C_s is the storage coefficient.

The time of concentration, T_c, in hours is the sum of the overland flow time, T_o, and the time of flow in the drain, T_d, as follows:

$$T_c = T_o + T_d \quad (10)$$

The storage coefficient is given by:

$$C_s = \frac{2\,T_c}{2\,T_c + T_d} \quad (11)$$

The overland flow time is determined from the Rantz chart as given in Figure 3.2 of HP16 given the overland flow distance, slope and runoff coefficient. HP16 recommends that an overland flow time of 10 to 15 minutes be assumed for areas where these parameters cannot be determined such as an area yet to be mapped out.

The time of flow in the drain can be determined from normal hydraulic formula, given the channel cross section, length, roughness and slope. In areas yet to be mapped out, T_d can be determined by dividing the estimated drain length by 10 ft/s for proposed lined drains or by the average velocities as given in Table 3.3 of the procedure.

The rainfall intensity is based on HP1 (1982) for the critical duration i.e., the time of concentration T_c.

The runoff coefficient is estimated from Table 3.1 of HP16 which expresses the value of C as a function of the land use. The procedure is applicable to urban catchment area of up to 52 km^2 (20 mi^2).