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16.1 INTRODUCTION

The objective of the Chapter is to show how the designer performs hydrologic system design by combining the various stormwater quantity and quality components, which are described in later Parts, into an integrated stormwater system.

Design is essentially a creative process. The engineer carrying out a stormwater system design needs to create, visualise and define his or her ideas and then confirm them by *analysis*. If analysis shows that the design does not fulfil its objectives, then it is changed and the process repeated until a satisfactory design is obtained within the applicable constraints.

This Chapter follows the above process. Although it is not possible to give formal procedures for the 'creative' process, it does give recommended practice based on experience. Then follows a section on the stormwater system design procedure.

16.2 DRAINAGE SYSTEM

When urban development takes place in a natural catchment, the ordered system of natural drainage lines is replaced by a system of roof and property drains, inlets, swales, pipes and channels. Although outwardly different from the natural catchment drainage network, the formal paths of the developed landscape should display a structure which is similar to that of the natural system they have replaced (Argue, 1986).

Planning and design of an integrated urban stormwater management/drainage network for a new development requires a database on the following:

(a) catchment natural drainage direction,

(b) runoff outfall point,

(c) catchment boundary,

(d) internal node points: locations of interest where flows, flood levels and possibly water quality need to be assessed. These may be, for example, at major road crossings or landscape features,

(e) drainage network: the layout of the surface channels which convey runoff originating in the catchment, and

(f) catchment sub-areas.

An example of a natural catchment drainage system, which illustrates items (a) to (f), is shown in Figure 16.1.

16.2.1 General Structure of Urban Catchments

A structure similar to that described above for rural catchments can generally be observed in urban catchments. Differences arise primarily where man-made components of the built environment interrupt the movement of runoff. A typical example is where roadways cross natural drainage paths.

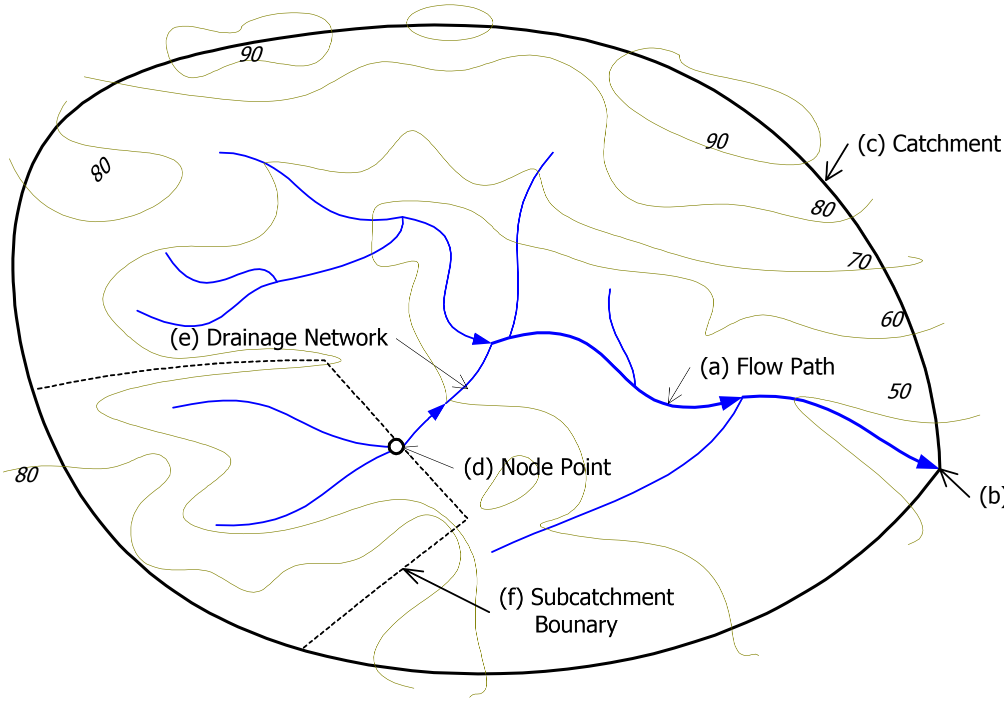


Figure 16.1 Runoff Travel Path and Features of a Natural Catchment

At a large scale, such as over a city or town area, the interference of man has virtually no effect on the natural drainage directions and only a marginal effect, in a few cases, on catchment boundaries. Urbanisation does have a pronounced effect on the runoff response of the catchment, and on water quality. Structures such as bridges and culverts have an effect on flood levels, while the presence of ponds, dams and the like has an effect on storage characteristics.

Stormwater strategy planning studies (see Chapter 8) of large urban areas therefore focus primarily on catchment quantity and quality response, catchment storage characteristics and on the effects of major structures within the overall (natural) catchment boundary. At this scale, it is the usual practice to represent the small community-level drainage systems by single sub-areas with suitable sub-catchment characteristics.

16.2.2 Structure of Small Urban Catchments

When the detailed drainage planning or design of each segment of an urban landscape is undertaken, it is found that man-made structures have a great influence on the behaviour of stormwater runoff. It is not unusual for surface flows and channelled or underground flows to travel to different points.

Node points in urban catchments are usually assigned to the principal road intersections where cross flow cannot be allowed, to sag points (low points), and to points where the direction of flow changes significantly. They should also include locations where flow will enter from major sub-divisions or from commercial/ industrial estates. Each node has its own sub-catchment area contributing flow.

Drainage network nodes are linked by flow paths using existing or planned conduits (pipes or open channels), down-slope roadways, swales, and overland flow paths. All of these flow paths are collectively termed 'links'. Thus the network structure consists of *nodes* and *links*.

16.3 DESIGN PROCESS

The design process involves a combination of hydrology, hydraulics and water quality. Hydrologic considerations determine the flows at each point in the drainage system. Runoff flows will collect pollutant loads. The resulting hydrographs and pollutographs are then routed downstream according to hydraulic principles. Water quality can be improved by treatment devices, which in turn affect the runoff and hydraulics (by, for example, introducing additional head losses). Because of the interaction between hydrology, hydraulics, and water quality all aspects need to be considered together.

The designer should treat the design effort as an iterative process involving the developer, regulators, planners and landscape architects, the general project civil engineer, etc. A design that is optimal from a stormwater point of view may not be achieved. However, designers should develop skills that enable them to exploit the potential of given urban layouts and the terrain to achieve their objectives.

16.3.1 Major and Minor System Design

Chapter 11 discussed the major/minor drainage system design concept and criteria. The *major system* is designed for a major event, typically of 100 year ARI. The *minor system* is designed for a minor event, which may range between 2 year and 20 year ARI. The major/minor concept may be described as a 'system within a system' for it comprises two distinct but conjunctive drainage networks.

The major and minor systems are closely inter-related, and their design needs to be done in tandem. In most cases it will be necessary to change the preliminary concepts in order to meet design criteria. A flowchart of the design process is given in Figure 16.4.

Most stormwater quality management facilities are sized for optimum performance in the *"water quality design storm"*. As discussed in Chapter 4, this is typically the 1 in 3 month ARI event. The flowchart in Figure 16.4 also includes consideration of this event. The facilities may be used in combination, as a 'treatment train'. Generally each component is intended to treat different pollutants. However, it is also necessary to consider the combined removal efficiency of the combination of measures and this is discussed in Section 16.6.

16.3.2 Preliminaries

This Section assumes that project goals and objectives and the applicable regulations, have already been defined and that basic data has been collected at the Master Planning stage (see Chapter 9). These steps are vital pre-requisites.

The importance of thorough data collection cannot be over-emphasised. The designer should be aware of the location of all major services and underground installations and arrange the network layout to avoid these.

At the preliminary design stage it is not practicable to accurately assess all hydraulic losses. For preliminary design purposes, a Manning's "n" value, or other roughness coefficient, about 25% above that contemplated for final design should be used in calculation to allow for the effect of minor losses. Flow estimates can then be based on these sizes and assumed grades.

Design runoff rates may need to be adjusted to reflect detention requirements, in accordance with the procedures described in Chapter 14. Pollutant load estimation, transport and retention follows the principles set out in Chapter 15.

16.3.3 Computer Methods

Computer methods can be used for all of the procedures described in this Chapter. Indeed, for all but the simplest systems the required calculations are too complex to carry out manually. The designer still needs to have a clear understanding of the steps involved. Computer methods are discussed in Chapter 17.

16.4 SYSTEM NUMBERING

A systematic method for numbering stormwater system nodes and links will assist in the design process. The numbers serve as identifiers in either hand or computer calculations.

Local authorities may choose to adopt a standardised numbering system for their area, for asset management and other purposes. The numbering system should be able to accommodate future system extensions.

Two different numbering systems are presented in this Section. The first is mainly applicable to studies carried out at a Strategic Planning level, while the second is suitable for the Master Planning and Detailed Design level.

Some computer models have restrictions on the format and/or number of digits in the numbering system. This Manual does not attempt to discuss these various restrictions. There is no one 'ideal' system; different systems may be found to be more suitable in different areas. Therefore only guidelines are given.

16.4.1 Hydrologic Model Numbering

Hydrologic calculations generally start at the furthest upstream sub-catchment, and then proceed down the drainage line on which the first sub-catchment is located. When a branch line is encountered, calculations re-start at the furthest upstream catchment on the branch line, and proceed downstream to the branch pilot. Once the flows of the two upstream links are known, calculations can proceed downstream of the branch point. Most numbering systems for hydrologic models follow this principle.

For a new analysis, or for an hydrologic model of a catchment at a Strategic Planning level, the numbering system should be based on the links in the existing natural drainage system. The following procedure is suggested:

Link Numbering System

A base plan of the existing natural drainage system should first be prepared (Figure 16.2). The catchment is discretised into sub-catchments following guidelines given in previous sections.

The decimal classification of the drain lengths between junctions shown in the figure is recommended. Each link is given a number in the form U.V where U, the "integral part" of the number, denotes the branch to which the length belongs and V, the "fractional part" of the number, denotes the position of the length within the branch. The integral part of the number is called the branch number.

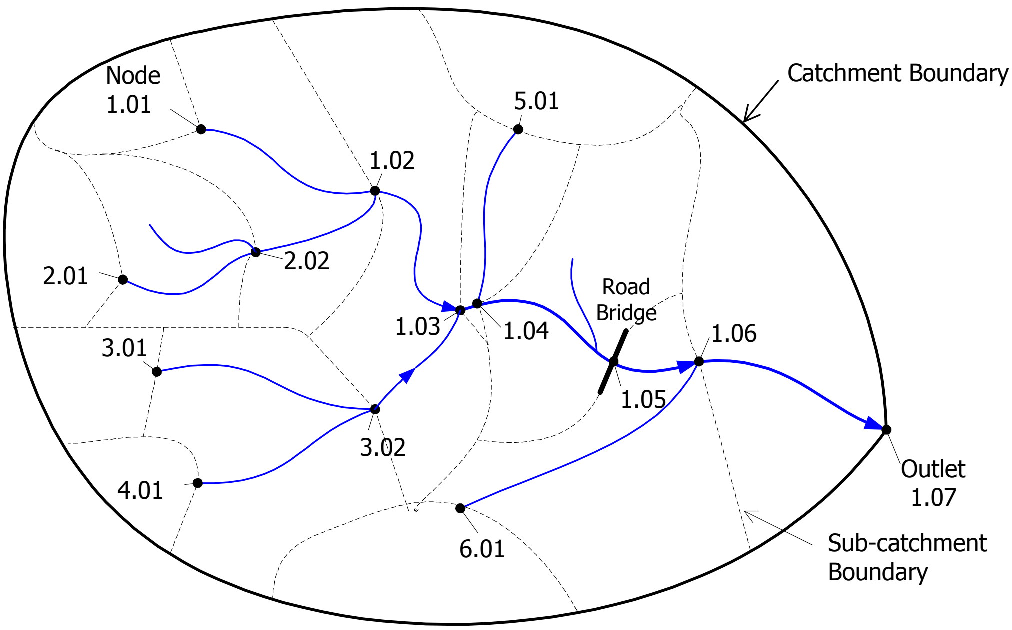


Figure 16.2 An Example of Link and Sub-Area (Node) Numbering System for Hydrologic Model

The suggested procedure for numbering links is as follows:

- the longest channel upstream from the catchment outlet is numbered 1;
- individual links on channel 1 are numbered 1.01; 1.02…. 1.09; 1.10; 1.11; 1.12; and so on starting from the most upstream link.

Node Numbering System

Nodes can be numbered according to the number of the *downstream* link. Note that in a normal dendritic drainage system there can only be one link or channel flowing out of a node (See Figure 16.2).

An alternative convention, which may be used, is to name nodes for place names, locations or nearby features such as town, bridge or river confluence. This may be found to be more convenient for ease of reference in strategic studies of large river catchments.

16.4.2 Detailed Numbering System

In existing areas the minor drainage system layout is already well defined by existing structures. If there is no numbering system already, one should be established for asset management purposes as well as for drainage studies. A supplementary numbering system is required to enable components other than nodes to be identified.

The numbering system for new development would then follow on from the existing system. For example, if a drain is extended upstream the numbers continue on from the last number in the existing system.

No universal guidelines can be given as different systems are better suited to different manual or computer calculation techniques. An example is shown in Figure 16.3.

Node Numbering System

A base plan of the existing or proposed drain system should first be prepared. Each node or structure is given a number in the form U.V where U, the "integral part" of the number, denotes the branch to which the length belongs and V, the "fractional part" of the number, denotes the position of the structure within the branch. The integral part of the number is called the branch number.

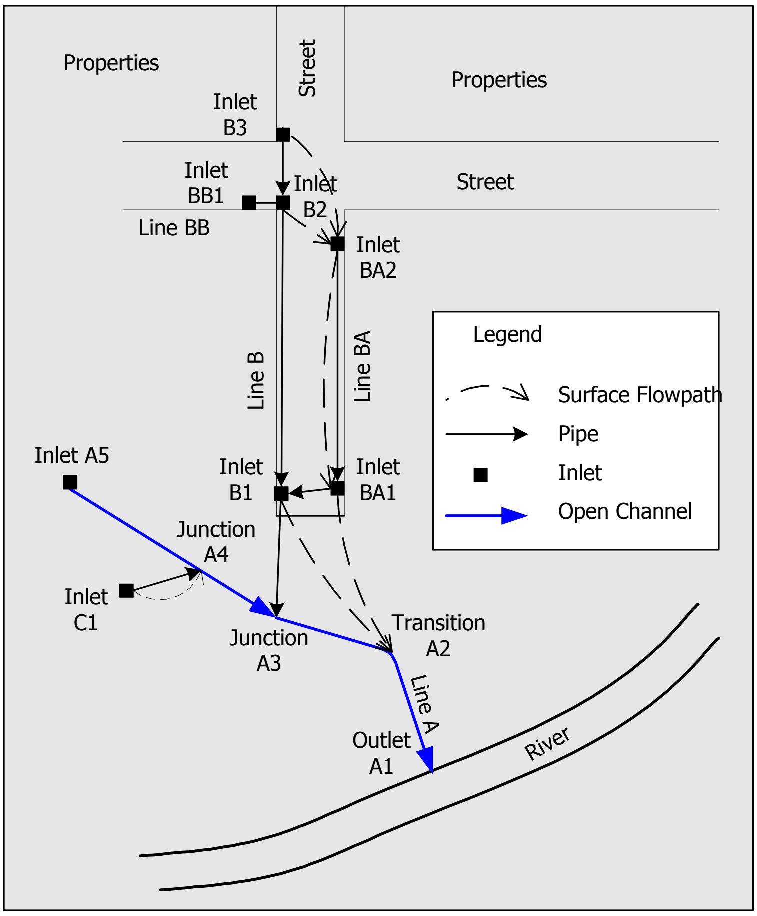


Figure 16.3 An Example of Detailed Numbering System for Conveyance Facilities

The main line is identified alphabetically, for example as line A and the structure furthest *downstream* is assigned structure A1. It is useful for ease of reference if the designation of the main line has a physical significance, such as the abbreviation of a suburb, administrative area, watercourse name, or road name. Thus the main channel in administrative area XY could be numbered XY-A, and so on.

Working upstream, number the remaining structures on the main line A2, A3, A4, etc. Branch lines are identified as B, C etc. and their nodes are numbered as line B (B1, B2, B3, . . .), line C (C1, C2, C3, . ..) etc. This system allows for future upstream extensions or additions. If new structures are inserted, for example between nodes B.2 and B.3, they are numbered B.2A, B.2B etc. (see Figure 16.3). Note that this system is different to that in Figure 16.2, but as they are used for different purposes there should be no confusion.

Link Numbering

A separate link numbering system is necessary in complex systems where it is necessary to distinguish between, say, channel and overland flow links. In simple hand calculations each link, such as a channel or pipe section, can be referred to by the designations of the upstream and downstream nodes.

Some computer models use automatic (default) link numbers for the links between nodes. The user does not need to use these numbers unless he or she so wishes.

If a link numbering system is to be adopted it should follow that of the nodes but not so closely as to cause confusion. For example, the channel lengths on Branch BA could be numbered as BA-P1, BA-P2 and so on, starting from the most downstream length. The corresponding road gutter sections can be numbered BA-G1, BA-G2.

16.5 GENERAL PLANNING AND DESIGN PROCEDURE

There are ten steps that must be undertaken to plan or design a drainage/stormwater management system for a typical new development. They are:

STEP 1: CATCHMENT DEFINITION AND DISCRETISATION

STEP 2: FLOW ESTIMATION AND CHECK

STEP 3: QUANTITY AND QUALITY CONTROL STRATEGY

STEP 4: MINOR SYSTEM INITIAL ASSESSMENT

STEP 5: MAJOR SYSTEM CHECK

STEP 6: PRELIMINARY SIZING

STEP 7: WATER QUALITY SYSTEM DESIGN

STEP 8: NETWORK REVIEW

STEP 9: EVALUATION

STEP 10: FINAL DESIGN DETAILING

A Flowchart of the typical procedure is given in Figure 16.4. This procedure is necessarily generalised; it will need to be adjusted to suit each individual problem.

Reviews of existing drainage systems should be carried out using the same general sequence. Step 1 will, of course, take into account the existing constructed drainage system. It is often found that older existing systems are lacking in consideration of overland surface flows (the 'major system' flows).

16.5.1 Catchment Definition and Discretisation

(a) Obtain Catchment Plan and Define Flow Paths

Identify the catchment area boundary (watershed) from maps or base plans. Classify probable future development within the catchment in accordance with its effects on hydrology, hydraulics and stormwater quality. Off-site areas that drain onto the site, not just the site itself, must be included.

Identify location of discharge points (outfalls), along with their capacity and downstream constraints. Identify natural drainage paths through the site.

Catchment discretisation is generally based on drainage patterns, surface slopes and landuse patterns. It is usually desirable for sub-catchments to be chosen so that they have homogeneous physical characteristics.

(b) Obtain Site Development or Master Plan and Formulate Conceptual Alternatives

The stormwater drainage system should be developed in conjunction with overall development plans and Master Plans. Master Planning processes are discussed in Chapter 9.

(c) Select Design ARI Criteria

Recommended initial values of design acceptance criteria for different types of development are provided in Chapter 4. Those suggestions should cover the majority of situations encountered in drainage practice.

Different design acceptance criteria are recommended for different types of development. This is necessary in order to ensure that the surface flows remain within acceptable limits. In practice, the design process may lead to modifications to these values. The eventual objective is for *"the total system to be designed to convey peak flows during major and minor storm events . . whilst adhering to public safety and convenience criteria separately applicable under (this range of) conditions"* (QUDM, 1992).

16.5.2 Flow Estimation and Check

Establish a hydrologic model of the catchment, for existing conditions. This requires the use of design rainfall data, and the estimation of hydrologic parameters such as percentage of impervious area or runoff coefficient.

Use the model to estimate design flows for the required design storm or storms. The details of this procedure will depend on the modelling approach chosen, and are described in Chapter 14.

It is desirable to check the discharge estimate using another independent method. The ideal check would be against flow gauging data; or against recorded flood depths in the drain concerned. If this is not possible, the check could be carried out using another hydrologic method provided that it is valid for the catchment situation. For example, the Rational Method should not be used as a check if the catchment area is greater than 80 hectares. In urban areas there are only a limited number of applicable methods. One suitable method is to use a different computer model to that used in the design, in a simple, lumped-catchment mode. Computer hydrologic models are discussed in Chapter 17.

Experienced designers of urban drainage systems may choose to omit the check procedure. In that case the accuracy of the estimate will depend on the designer's ability to estimate appropriate values for the catchment parameters.

A worked example with calculation steps for the system design procedure at a community level is given in Appendix 16.A.2. This encompasses runoff quantity, quality and conveyance facilities for Sg. Rokam in Ipoh. System design can be undertaken at various other levels. The smallest level is at on-site, such as the roof and property drainage.

16.5.3 Determine Quantity and Quality Control Strategy

Identify what combination of on-site detention/retention and/or community detention/retention, will be used in the system. This information will come from the Stormwater Strategic Plan if one is available. Locate and estimate size of proposed regional/ community facilities.

Decisions taken in this step will affect the subsequent system design procedure:

- if on-site detention (OSD) is used, the catchment characteristics should be modified accordingly to reproduce the OSD behaviour.
- if community/regional ponds or detention storages are used, the pond characteristics will affect the flow hydrograph in the downstream drainage system.
- if retention facilities are proposed, they will need to be explicitly represented in the drainage system design. This includes representation of diversions (if off-line), "loss" of flow due to infiltration, storage effects, and the behaviour of the basin during storms exceeding the design flow.

In most cases it will be necessary to study a range of design storms of different ARI to ensure that all performance requirements are met.

It can be seen from the above list that the calculations required are complex and repetitive. Current practice is to use computer models to carry out these calculations in all but the simplest and smallest of catchments. System design computer models (see Chapter 17) are set up to efficiently carry out the large number of detailed computations required. When using these models as tools, the designer should still follow the same steps as set out below and in the flowchart, Figure 16.4.

16.5.4 Minor System Initial Assessment

(a) Conceptual System Layout

Design begins with system layout - approximately defining the minor and major flow routes, and broad water quality control strategy. System layout includes the selection of an outfall, defining drainage area boundaries, and identifying the locations of trunk and main drains, and water quality control structures. Initial layouts can usually be done from topographic maps.

Existing drainage alignments, whether clearly defined channels or more subtle swales, should normally be set aside as major drainageways. Rolling, hilly terrain usually has natural drainage patterns that cannot be significantly modified. If existing major drainageways are to be blocked by land development, alternative drainage capacity must be provided.

The safety, convenience and cost effectiveness of preserving natural drainage to serve as “outfalls” for stormwater drain discharges are well recognised, and the designer is encouraged to identify and utilise the natural system to its fullest advantage. In doing so it is necessary to ensure that the flow rates and loadings are altered as little as possible so that the natural channel remains stable.

(b) Initial Assessment

Starting from the top of the catchment, an initial assessment calculation is carried out for the minor drainage system. This involves the following steps:

- select trial inlet locations. For piped drainage, select inlet capacity.
- calculate the flow in the minor system design storm
- add flows bypassed from upstream inlets, if any
- check that any bypassed flows are able to be safely handled by the major system. Where the major system involves flow along roads, the surface flow must be limited so that it meets the flooding and safety criteria listed in Chapter 4. If the surface flow is not acceptable, change the inlet locations.
- check whether it is feasible to provide the required inlet capacity. Inlet capacity is discussed in Chapter 24. More inlets may need to be added.
- the required capacity is then the sum of the upstream drain flow (except for the most upstream drain) and the inlet flow.
- repeat the above steps for the rest of the minor system.

16.5.5 Major System Check

After the initial minor system is developed, the next step is to carry out a check of flows in the major design storm to ensure that they are also within acceptable criteria.

The methods for determining and routing design runoff in the major storm are discussed in Chapter 14. Catchment parameters may need to be adjusted to reflect on-site detention or retention, in accordance with the procedures described in Section 16.5.3. Community/ regional detention and retention BMPs should be directly included in the calculations. This will almost certainly require the use of computer models.

One design aim is to limit the 'gap' flow, defined as follows:

$$Q_{gap} = Q_D - (Q_m \times F_B) \tag{16.1}$$

where,

Q_{gap} = gap flow

Q_D = flow in major design storm

Q_m = minor system design flow

F_B = blockage factor, see Section 24.4.

The major system check involves the following steps (see the Flowchart in Figure 16.4):

- starting from the top of the catchment, calculate flows Q_D in the major design storm at points of interest such as road crossings or flow junctions
- determine the minor system flow at the upstream end of the section under consideration Q_m
- calculate the net surface flow $Q_S = Q_D - Q_m$
- if the major storm causes flooding of local roads, check whether Q_S is within acceptable limits for the road cross-section. If not, adjust the minor system design to increase *both* inlet capacity and drain capacity to reduce the surface flow.
- check that the required drain size or pipe capacity at this location is not less than that upstream. Generally, a reduction of drain width or pipe diameter from one section to the next downstream section is not permitted.
- for pipe drain inlets, calculate the required inlet capacity Q_G (see Chapter 24)
- repeat for the next section downstream.

The larger of the two pipe size/ inlet combinations from the minor storm or major storm assessments, is then adopted for the remainder of the design.

If it is impractical to accommodate the major system design flows using the combination of the street and stormwater drain carrying capacity, the engineer has these options: (1) change the major system design basis (which may require an important policy decision), (2) increase the minor system capacity, (3) elevate or otherwise protect buildings, (4) begin a formal major drainageway, or (5) provide upstream detention to decrease downstream discharge. Each of these choices will have economic implications.

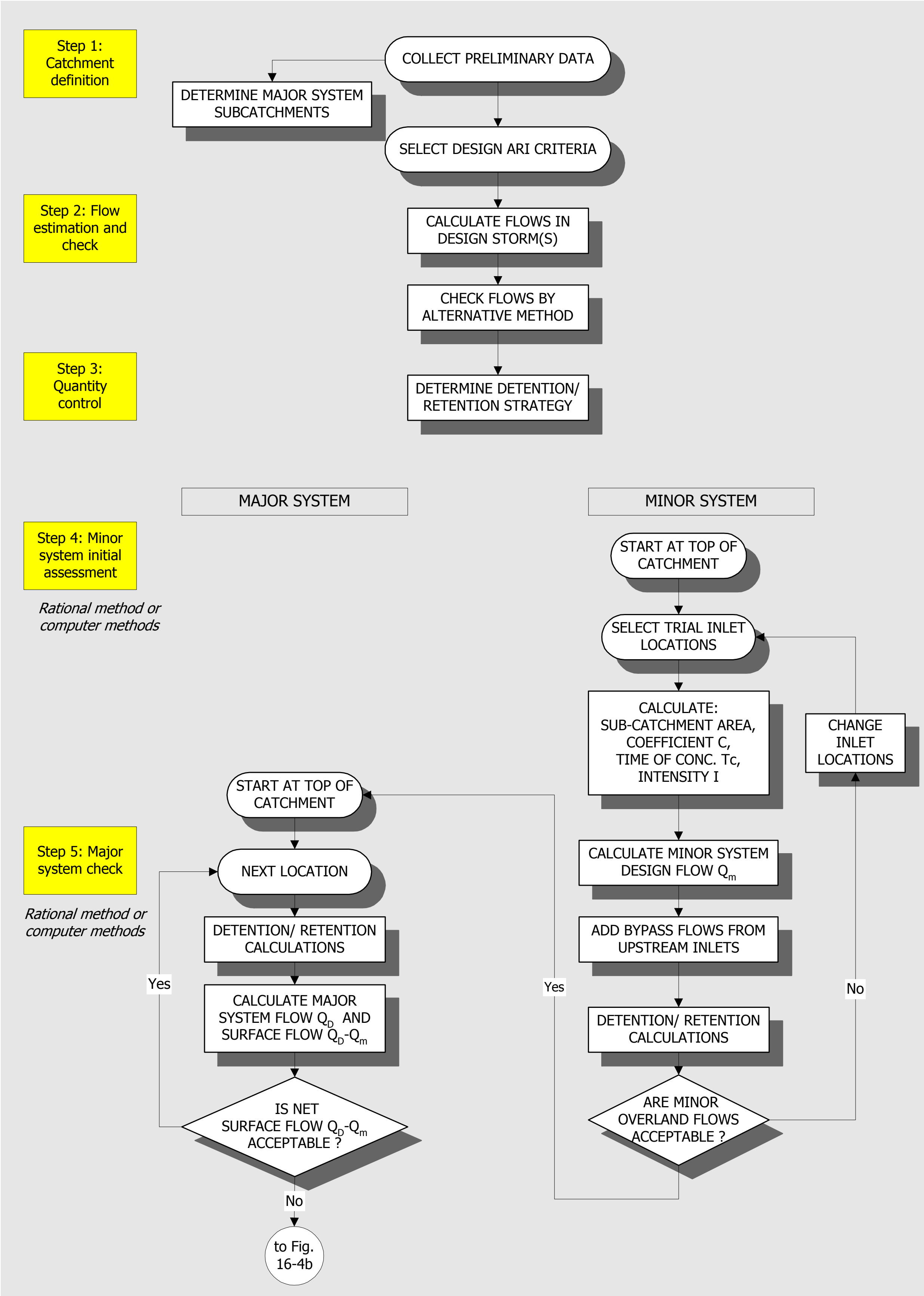
16.5.6 Preliminary Design

Using the above initial analyses, proceed with preliminary design of the major/minor systems and stormwater management systems. Consideration should also be given to alternatives, which may result in a more economic design.

In the investigation of an individual scheme, the full range of design alternatives should be considered to determine the "most cost-effective" alternative. Benefits and costs in both the short and long terms should be considered although least capital cost is commonly the method used to choose the "most economic" alternative.

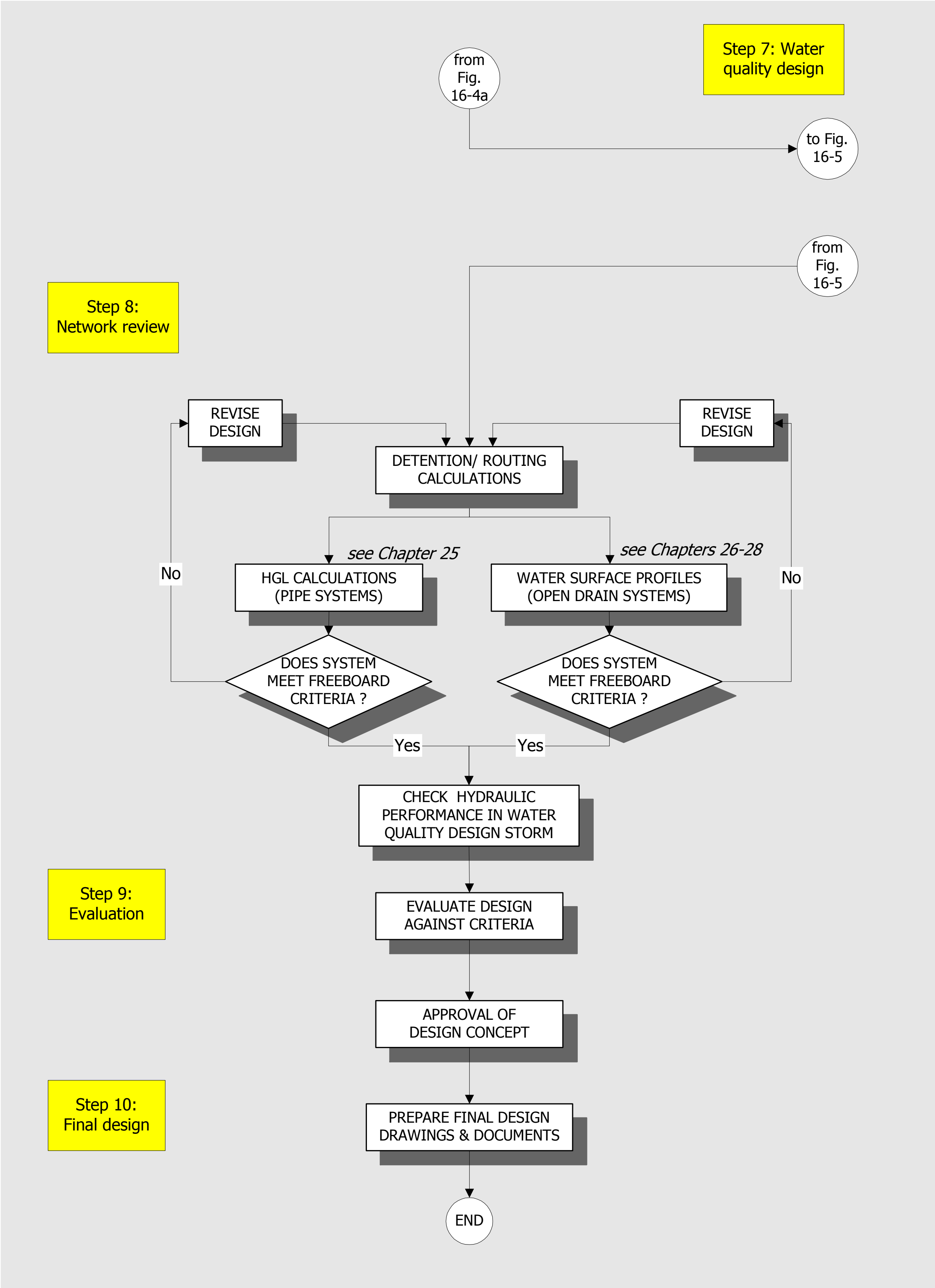
Design alternatives may include:

- doing nothing;
- reduce design criteria;
- provide above and below ground detention storage in parks, road reserves and private properties;
- partial augmentation;



(a) Flow Chart - 1

Figure 16.4 System Design Procedure



(b) Flow Chart - 2

Figure 16.4 System Design Procedure

- major augmentation;
- purchase and removal of affected buildings

Preliminary design involves the following activities. These tasks should be conducted for each of the alternatives. The degree of detail and the number of alternatives to be investigated will vary for different types of development.

(a) Define alignments and grades for stormwater drainage channels. Factors that will influence alignment and grade, include sewer lines and other utilities *(crucial)*, railways, tree preservation, embankments, buildings, etc. Other factors that influence channel grade include existing slope (of natural channels), erodibility, and available right of way and channel lining. Several preliminary layouts need to may be considered.

Open drains are acceptable unless the Local Authority determines otherwise. However if pipe drains are used, open drains or engineered waterways will be required when pipe sizes become so large as to be uneconomical. Guidelines on selection of stormwater conveyance systems are given in Chapter 10. In existing built-up urban areas there may be insufficient land for an open drain. In that case, larger pipe diameters and/or multiple pipes or box culverts will be required. Piped drains are normally located as described in Chapter 25. On occasion, it may be necessary to locate them in right-of-way easements, but locating them on public property such as roads is preferred.

Crossings with other underground utilities should be avoided whenever possible, but, if necessary, should be at an angle greater than 45 degrees. Utility crossings are a major design factor (and cost) when retrofitting in urban areas, and considerable effort is required to comprehensively define and locate utilities in the field.

Pipe drains can continue under roads provided that there is sufficient vertical clearance. Open drains will need to be culverted. In either case, road crossings should be made at an angle greater than 45°.

Identify land requirements for the drainage system. The major system should use open space reserves to convey surface flows. This land will need to be set aside as a perpetual Reserve.

In the uppermost area of a drainage catchment, major drainageways may not consist of readily distinguishable channels: they may be home-owners' yards, swales or streets or whatever "low ground" exists and for the entire study area, the designer should determine the path that the major system design runoff event will take. Preliminary design-level sizing for the surface drainage paths can normally be done via hand calculations using normal depth techniques and assumptions regarding location of hydraulic "controls".

(b) Locate and size inlets, remembering that open drains may have pipe inlets. Refer to Chapter 24 for details.

(c) Given the stormwater drainage system alignment, grades and inflows, the water surface profile and energy grade line for the stormwater drain should be computed. Adjust alignment and grade as required to comply with freeboard criteria.

(d) Given designated major drainageways (with right-of-way set aside to accommodate the design runoff) and a functioning minor system, evaluate where the flood waters, in excess of those being conveyed in the stormwater drain, will flow. Also evaluate how site topography must be adjusted to assure that these flows will be conveyed safely down gradient without threatening lives, safety, or property.

(e) Evaluate behaviour of detention facilities during the major system design flow condition. Assure that detention dams, which could pose a threat to human safety or property if they fail, are designed to handle extreme flows.

(f) Special evaluation of culverts and bridges, in accordance with the procedures described in Chapter 27, will be required. Assure that applicable criteria are not violated. Adjust the size and characteristics of the conveyances until the amount of backwater during the major system design runoff event meets target levels. Define the area affected by backwater flooding during the major event and indicate that this land should remain undeveloped for perpetuity.

16.5.7 Water Quality System Design

After the design has been completed to this point, the next step is to review and refine the preliminary sizing of water quality control structures. These facilities are designed to be effective in the "water quality design storm". Refer to Section 16.6 for details of the water quality design procedure.

16.5.8 System Design Review

Prepare preliminary-design level capital and operation/ maintenance costs for the alternatives. If feasible and appropriate, use life cycle cost theory for economic evaluation. The design cost objective should be to minimise the total annual costs of the drainage facilities and flood-related damage.

Evaluate the alternatives with respect to important qualitative criteria such as preservation of open space, enhanced wildlife habitat, impact on wetlands and water quality benefits (if no formal attempt has been made to contrast the pollutant removal capabilities of various alternatives).

Prepare a preliminary design report that contrasts the alternatives quantitatively and qualitatively (in a form suitable for submission to the client and regulators).

Meet with client, project planner, general project civil engineer and the Local Authority to review preliminary design concepts and to select a preferred alternative. Unbiased evaluation of the good and bad points of each alternative is necessary if the most desirable system is to be identified. Quantitative and qualitative factors should be assessed. If any potential problems exist, such as effects on downstream landowners, conflicts with utilities, or difficulty in acquiring easements, they should be thoroughly reviewed and resolved.

Adjust the preferred alternatives (still at the preliminary-design level) to accommodate the requests of the local Authority and/or the Client and his advisors.

16.5.9 Re evaluation

The preliminary drainage network developed in previous sections, and adjusted as a result of review, should now be re-evaluated. This involves repeating the preliminary calculations for the 'adopted' design to verify that it will meet the design targets.

The re-evaluation should be performed at a similar or greater degree of detail to the preliminary design. It may involve either hand or computer methods, depending on the size and complexity of the system.

Hydrologic assumptions, catchment boundaries, sub-catchment delineation, street classifications, pollutant load estimates, assumed or calculated pollutant removal efficiencies, and any other preliminary design values that will be used subsequently in final design should be reviewed for accuracy and applicability to final design. All unresolved questions must be answered at this time.

16.5.10 Final Design Detailing

The preceding steps constitute the preliminary design effort. Up to this point, the engineer has been compiling the information necessary to make an informed decision on which system to use for final design. The following steps will complete the process.

(a) Obtain Final Street Grades, Geometry, Elevations, etc.

Often it will be necessary to revise street construction details to facilitate drainage. This may include adjusting cross fall on streets, raising required ground elevation at buildings adjacent to streets to accommodate major drainage, or increasing street gradient to achieve sufficient capacity within the street. It is important to assure that floor elevations of buildings are well above street crown elevations to prevent repeated flooding.

(b) Hydraulically Design the Open Channel and Pipe Drainage Systems

The final hydraulic design of a system should be on the basis of procedures set out in Chapters 24 and 25 for pipe drains, and Chapter 26 for open drains. A realistic Manning "n" value for final design should be determined and applied, treating the conduits as either open channels or pipes flowing full, as appropriate. For open channel flow, the energy grade line should be used.

For piped stormwater drains, the design engineer must review the hydraulic grade line for various runoff conditions (initial design runoff and others that the larger) to ensure that the hydraulic grade line is consistent with desired system performance.

The effects of any water quality treatment structures must be allowed for in this analysis. Trash racks, oil separators and GPTs will produce a head loss and the design should allow for blockage, with an alternative flow path in case of complete blockage. Detention or retention storages will have an influence by reducing downstream flows.

(c) Design of Water Quality Controls

Complete the final design of the water quality control measures. Design requirements applicable to each type of measure are discussed in Section 16.6 and Part G of this Manual.

(d) Complete all Other Aspects of the Design Effort

The final design should address all other factors, including structural and geotechnical design, land requirements, approvals, construction documents, co-ordination with other aspects of the project, etc.

Final design detailing involves the following steps for either hand or computer methods:

- fixing pit floor levels,
- checking clearance of utility services, and
- adjustments if necessary

Clashes with utility services may necessitate substantial adjustments to the design. Drop inlets may have to be included or other changes made, forcing recalculation of the parts of the network. It is rare for the preliminary design to be directly adopted without some changes. Much of the need for change can be avoided by thorough data collection in the early stages of the project.

The final design for open drains will provide the following information:

- gutter and sag inlets
  - locations, types, sizes
- combined inlet/junction pits
  - locations, types, sizes, pipe entries and exits
- property ties and other flow entry points
  - locations, sizes
- channels
  - locations, types, cross-section, lengths, sizes

For piped systems, detailed design of the stormwater drainage inlets is also necessary as described in Chapter 24. The manual HGL procedure shall be carried out for pipes as described in Chapter 25. Alternatively, computer models can be used to carry out the detailed design. These models may be developments of those used in the preliminary design. The final design for piped systems should provide the following additional information:

- inlets, junction inlets and manholes
  - locations, types, sizes, pipe entries and exits
- property ties and other flow entry points
  - locations, sizes
- pipes
  - locations, types, strength, grades, lengths, diameters

16.6 DESIGN OF STORMWATER QUALITY CONTROL SYSTEMS

16.6.1 System Design Procedure

The recommended system design procedure for water quality control measures is shown as a flowchart in Figure 16.5.

The procedure uses average annual pollutant loadings as a basis for design, and as a measure of BMP effectiveness. Assuming that the pollutants are conservative, a simple mass balance calculation is used. Calculations are performed separately for each pollutant. These loadings and removals can be directly related to the water quality Design Acceptance Criteria given in Chapter 4. The main steps are as follows:

- Calculate average annual runoff volumes using the Volumetric Rational Method described in Chapter 15. Estimation of the weighted average runoff coefficient is based on sub-catchment areas and percentage imperviousness reflecting landuse.
- Calculate pollutant loads from runoff volume using the pollutant EMC.
- Add runoff volumes and loads to the upstream volume and load, to get total volume and loads at this point.

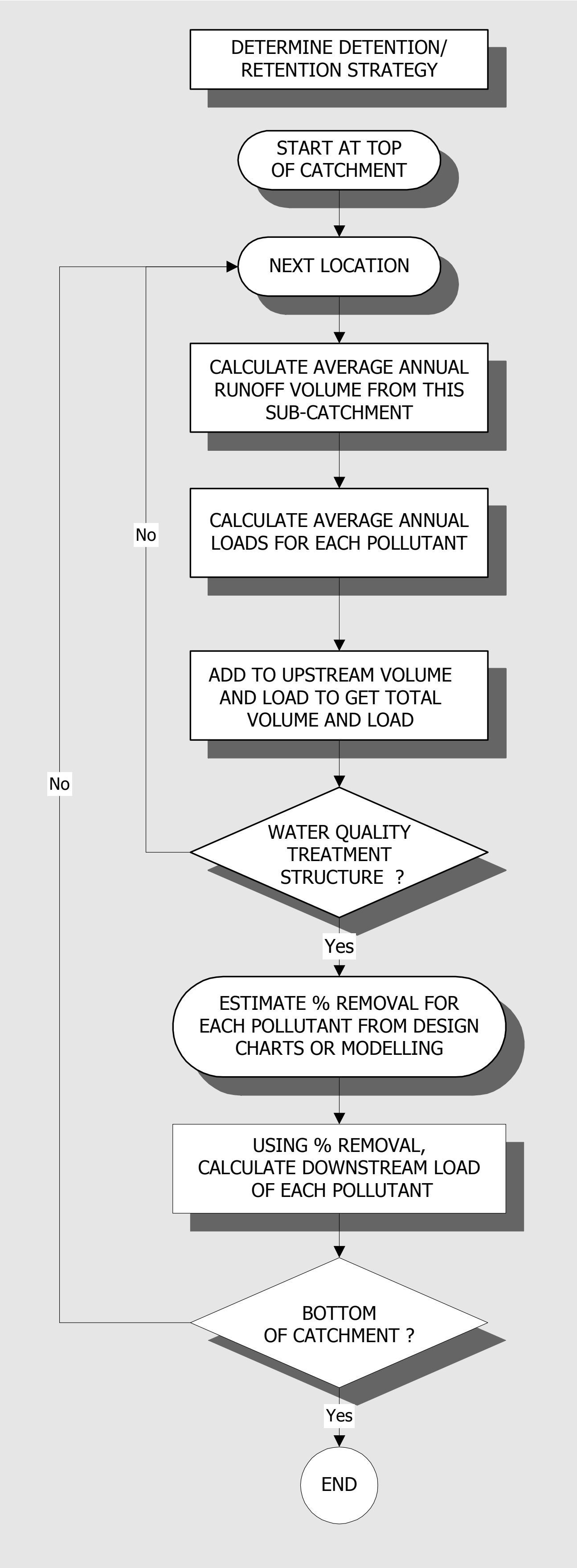


Figure 16.5 Flowchart for Water Quality System Design

- At each water quality control structure, estimate the percentage removal for each pollutant from design curves given in Part G of the Manual.
- Using this % removal, calculate the amount of load, which continues downstream. Note that these values must be calculated separately for each pollutant.
- Calculate the overall pollutant removal efficiency by comparing the final load at the bottom of the catchment, with the total catchment pollutant load (Refer to Section 16.6.2). The efficiency will in general be different for different pollutants.

Usually this procedure would be performed separately for pre-development and post-development conditions. The pre-development and post-development loads can then be compared to give an indication of the likely impact of the development, and of its water quality controls.

Note that this method *cannot* be used for non-conservative pollutants, such as bacteria.

16.6.2 Estimating the Efficiency of Treatment Trains

A preliminary estimate of the pollutant removal efficiency of combinations of treatment devices, including treatment trains, can be calculated using a mass loading approach on an annual basis. The data required is the system layout, the annual loads generated in each subcatchment, and the pollutant removal efficiencies of each device.

For measures in *series*, a simple approximation to the pollutant removal efficiency of a treatment train is that it is equal to the product of the removal efficiencies for the individual components. An example of this simple calculation is given in Figure 16.6. In the Figure, if the GPT removes 70% of sediment of a certain size and the removal efficiency of a downstream pond for that same size of sediment is 40%, the combined removal efficiency of this treatment train is approximately 80 %. A worked example showing the application of this procedure is presented in Appendix 16.A.2 for the same catchment of Sg. Rokam in Ipoh (see Appendix 16.A.1).

There are objections to this simplified approach on the grounds that pollutant removal efficiency depends on the inflow concentration (Roesner, *Personal communication*). Therefore the combined efficiency is likely to be less than the product of the individual components. However there is insufficient data to allow this effect to be quantified.

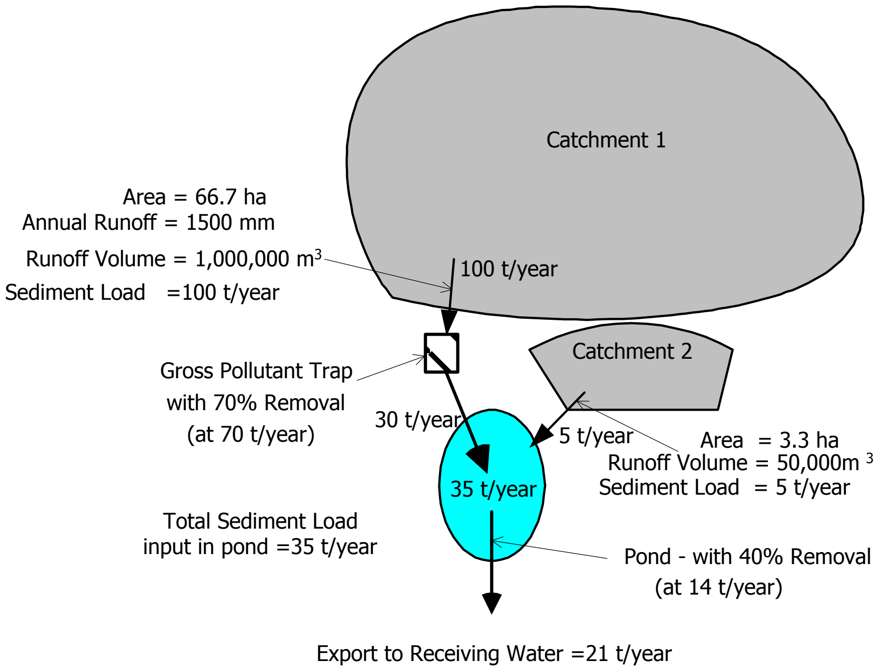


Figure 16.6 Example of Approximate Pollutant Removal Calculation for a Treatment Train in series

APPENDIX 16.A WORKED EXAMPLE

Given a small urban catchment (1.2 km^2) at Sg. Rokam, Rapat Setia in Ipoh. Most of the catchment is partly developed with residential facilities. The catchment is moderately flat with rural, residential and commercial landuses. Schematic layout of the existing main drainage system with landuse distribution is shown in Figure 16.A1. Components of the proposed community level stormwater network is shown in Figure 16.A2. No calibration data are available for stormwater quality and quantity. Worked examples are provided for the following:

1. Develop and examine a stormwater system network for quantity and compare the analysis for pre- and post-development conditions.

2. Calculate the average annual load and overall removal efficiency of the same proposed stormwater system for sediment and Total Nitrogen (TN). Find out the reduction in annual load compared with pre-development conditions.

3. Analyse part of the network system for stormwater quantity conveyance using Rational Method

16.A.1 Network Design for Stormwater Quantity Facilities Using Computer Models

(a) Summary of Typical Analysis Procedure

The procedure shown here is for XP-SWMM version 6.1 software. Similar general procedures would be followed for other computer modelling software.

1) Create a New file, load in a GIS background, draw the network and name or number the nodes.
2) Determine the design ARI standard(s) to be used. Define a series of design storms using the Temporal Patterns set up in Step 1 and design IDF values from Chapter 13.
3) Define the losses or infiltration procedure.
4) Enter the post-development catchment details anduse the Time -Area procedure for generating the hydrograph.
5) Each of the layers has Job Control parameters that are used to control the simulation.
6) To transfer the flows from one layer to the next, an interface file is set up. This enables the creation of hydrographs for various storms and then selectively routing these through the drainage network.
7) The drainage system details need to be entered in the hydraulics layer (EXTRAN) so that the model can route the hydrographs generated in the hydrology layer (RUNOFF).
8) Mode Properties are set to control whether to run the hydrology and hydraulics at the same time or to run one layer at a time.
9) Run the model to generate the results.
10) Once the analysis has been performed then the results may be displayed on screen, or printed out.
11) Check the discharge estimate against alternative method (*optional*).
12) Alter the catchment parameters to represent post-development conditions, and re-run the model for the minor system design storm.
13) Adjust the minor system design if necessary to achieve satisfactory performance in the minor system design storm.
14) Re-run the model for the major storm to check the design, and adjust if necessary.

Figure 16.A1 Landuse Pattern and Existing Drain of Sg. Rokam Catchment

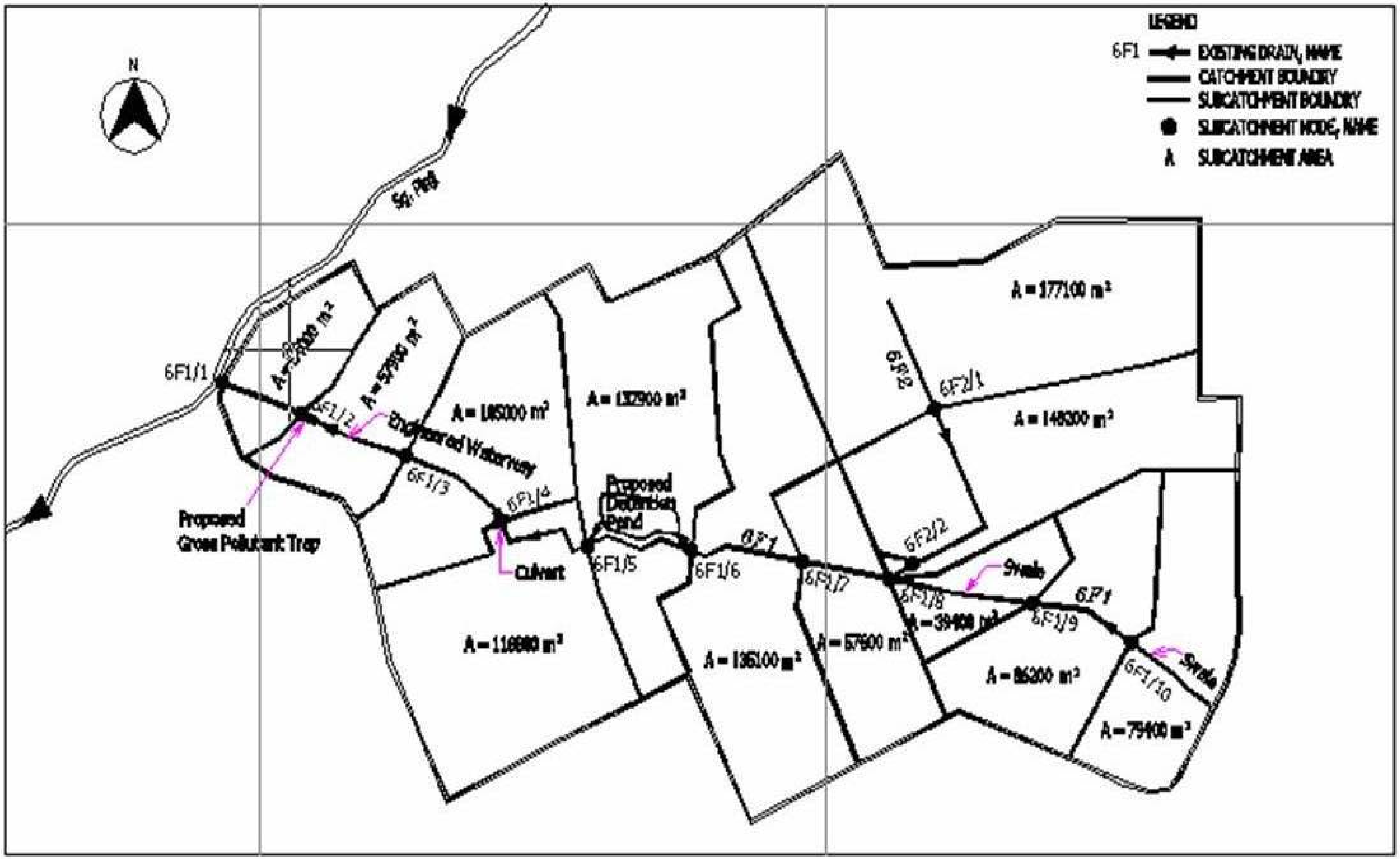


Figure 16.A2 Components of the Stormwater System Network Considered for the Sg. Rokam Catchment Analysis

(b) Solution

The calculations were performed using XP-SWMM version 6.1 software. Summary input and output tables are provided. Similar general procedures would be followed for other computer modelling software.

1) Create a new file, load in a GIS background, draw the network and name or number the nodes.

2) Determine the design ARI standard(s) to be used and define a series of design storms.

   For this example, the minor system design storm is 5 year ARI and the major, 100 year ARI.

3) Define the losses or infiltration procedure. This example uses a Horton infiltration curve in the Global Database, which can be used in the various catchments. The following values are used:

   Impervious area: depression storage = 1 mm, n = 0.014

   Pervious area: depression storage = 5 mm, n = 0.030

   Horton infiltration (only applies to pervious area): F_o = 75.0 mm/hr, F_c = 3.8 mm/hr, α = 0.0011 sec^{-1}

4) Enter the catchment details and use the Time-Area procedure for generating the hydrograph, as shown in Table 16.A1 and Table 16.A2.

Table 16.A1 Catchment Details, Existing System

Node Name	Routing Method	Infiltration Method	Contributing Area (ha)	Impervious (%)	Time of Concentration (min)	Slope (%)
6F2/1	Time-Area.	Normal	17.71	5	25	1.0
6F1/10	Time-Area.	Normal	7.94	20	14	2.0
6F1/9	Time-Area.	Normal	8.62	30	13	1.3
6F1/8	Time-Area.	Normal	14.82	5	28	1.0
	Time-Area.	Normal	3.94	30	14	1.0
6F1/7	Time-Area.	Normal	5.76	40	13	1.0
6F1/6	Time-Area.	Normal	13.51	40	14	1.0
6F1/5us	Time-Area.	Normal	13.29	30	17	0.5
6F1/4us	Time-Area.	Normal	11.88	30	15	0.5
6F1/3	Time-Area.	Normal	10.5	20	16	0.5
6F1/2	Time-Area.	Normal	5.79	20	13	0.5
6F1/1	Time-Area.	Normal	5.4	20	13	0.5
Outlet	-	-	-	-	-	-

5) Each of the layers has Job Control parameters that are used to control the simulation.

6) To transfer the flows from one layer to the next, an interface file is set up. This enables the creation of hydrographs for various storms and then selectively routing these through the drainage network.

7) The drainage system details need to be entered in the hydraulics layer (EXTRAN) so that the model can route the hydrographs generated in the hydrology layer (RUNOFF).

Table 16.A2 Drainage System Details, Existing System

Link Name	U/S Node	D/S Node	No.	Type	Width (m)	Depth (m)	Sideslope (Z)	Length (m)	IL u/s (m)	IL d/s (m)	Manning n
25	6F2/1	6F1/8	-	Trapezoidal	2.0	1.5	0	400	46.40	41.00	0.017
26	6F1/10	6F1/9	-	Trapezoidal	1.0	1.5	4	200	47.00	42.50	0.040
27	6F1/9	6F1/8	-	Trapezoidal	1.0	1.5	4	250	42.50	41.00	0.040
28	6F1/8	6F1/7	-	Trapezoidal	2.5	2.5	0	150	41.00	40.70	0.017
29	6F1/7	6F1/6	-	Trapezoidal	2.5	2.5	0	200	40.70	39.00	0.017
30	6F1/6	6F1/5us	-	Trapezoidal	2.5	2.5	0	200	39.00	38.00	0.017
O/flow1	6F1/6	6F1/4ds	-	Trapezoidal	10.0	1.2	2	370	39.00	37.10	0.040
41	6F1/5us	6F1/5ds	-	Trapezoidal	2.5	3.0	0	20	38.00	37.80	0.017
38	6F1/5ds	6F1/4us	-	Trapezoidal	3.0	3.0	0	180	37.80	37.20	0.017
39	6F1/4us	6F1/4ds	-	-	-	-	-	-	37.20	37.10	-
-	-	-	2	Rectangle	1.5	1.5	-	10	-	-	0.014
-	-	-	-	Trapezoidal	20.0	1.2	5	10	-	-	0.025
32	6F1/4ds	6F1/3	-	Trapezoidal	10.0	3.0	4	200	37.10	36.60	0.025
33	6F1/3	6F1/2	-	Trapezoidal	10.0	3.0	4	200	36.60	36.00	0.025
34	6F1/2	6F1/1	-	Trapezoidal	10.0	3.0	4	200	36.00	35.30	0.025
36	6F1/1	Outlet	-	Trapezoidal	10.0	3.0	4	100	35.30	34.80	0.025

8) Mode Properties are set to control whether to run the hydrology and hydraulics at the same time or to run one layer at a time.

9) Run the model to generate the results.

10) Once the analysis has been performed then the results may be displayed on screen, or printed out. The Spatial Report produces a graphical summary, and the 'REPORT' function is used to generate summary tabular outputs. These are shown in Table 16.A3 and Table 16.A4 for two cases, the 100 year ARI and 5 year ARI 60 minute duration storms.

Table 16.A3 100 year ARI 60 minute Storm, Existing System

Link Name	U/S node	D/S node	IL u/s (m)	IL d/s (m)	Manning n	Level u/s (m)	Surcharge time (min)	Q_{max} (m^3/s)	Velocity (m/s)	V x D (m^2/s)
25	6F2/1	6F1/8	46.4	41.0	0.017	47.46	0.0	8.00	3.54	4.07
26	6F1/10	6F1/9	47.0	42.5	0.040	47.66	0.0	4.62	1.48	1.12
27	6F1/9	6F1/8	42.5	41.0	0.040	44.08	0.0	7.66	0.98	1.52
28	6F1/8	6F1/7	41.0	40.7	0.017	43.89	0.0	21.40	3.51	9.26
29	6F1/7	6F1/6	40.7	39.0	0.017	43.25	6.6	23.16	3.88	10.73
30	6F1/6	6F1/5us	39.0	38.0	0.017	42.24	0.0	27.79	4.45	13.32
O/flow1	6F1/6	6F1/4ds	39.0	37.1	0.040	42.24	0.0	1.06	0.57	0.10
41	6F1/5us	6F1/5ds	38.0	37.8	0.017	40.81	0.0	33.59	4.74	13.45
38	6F1/5ds	6F1/4us	37.8	37.2	0.017	40.67	0.0	33.62	4.28	11.09
39	6F1/4us	6F1/4ds	37.2	37.1	-	39.53	0.0	-	-	-
-	-	-	-	-	0.014	-	-	15.29	3.63	6.54
-	-	-	-	-	0.025	-	-	24.51	4.73	0.00
32	6F1/4ds	6F1/3	37.1	36.6	0.025	38.44	0.0	40.18	1.95	2.62
33	6F1/3	6F1/2	36.6	36.0	0.025	37.96	0.0	45.51	2.17	2.95
34	6F1/2	6F1/1	36.0	35.3	0.025	37.37	0.0	48.67	2.41	3.17
36	6F1/1	Outlet	35.3	34.8	0.025	36.57	0.0	51.44	2.78	3.43

Table 16.A4 5 year ARI 60 minute Storm, Existing System

Link Name	U/S Node	D/S Node	IL u/s (m)	IL d/s (m)	Manning n	Level u/s (m)	Surcharge Time (min)	Q_{max} (m^3/s)	Velocity (m/s)	V x D (m^2/s)
25	6F2/1	6F1/8	46.4	41.0	0.017	47.17	0.0	5.22	3.38	2.61
26	6F1/10	6F1/9	47.0	42.5	0.040	47.56	0.0	3.07	1.41	0.88
27	6F1/9	6F1/8	42.5	41.0	0.040	43.48	0.0	6.37	0.86	1.31
28	6F1/8	6F1/7	41.0	40.7	0.017	43.14	0.0	15.73	3.32	6.20
29	6F1/7	6F1/6	40.7	39.0	0.017	42.32	0.0	17.35	3.59	7.08
30	6F1/6	6F1/5us	39.0	38.0	0.017	41.40	0.0	21.91	3.65	8.76
O/flow1	6F1/6	6F1/4ds	39.0	37.1	0.040	41.40	0.0	0.00	0.00	0.00
41	6F1/5us	6F1/5ds	38.0	37.8	0.017	40.40	0.0	26.12	4.28	10.47
38	6F1/5ds	6F1/4us	37.8	37.2	0.017	40.29	0.0	26.12	3.68	8.66
39	6F1/4us	6F1/4ds	37.2	37.1	-	39.42	0.0	-	-	-
-	-	-	-	-	0.014	-	-	14.51	3.62	6.07
-	-	-	-	-	0.025	-	-	15.53	4.10	0.00
32	6F1/4ds	6F1/3	37.1	36.6	0.025	38.24	0.0	30.06	1.80	2.06
33	6F1/3	6F1/2	36.6	36.0	0.025	37.75	0.0	33.37	1.98	2.28
34	6F1/2	6F1/1	36.0	35.3	0.025	37.16	0.0	35.17	2.19	2.42
36	6F1/1	Outlet	35.3	34.8	0.025	36.36	0.0	36.72	2.26	2.55

11) Check the discharge estimate against another method *(optional)*. No checking was done for this example.

12) Alter the catchment parameters for post-development conditions, as shown in the Table 16.A5.

Table 16.A5 Catchment Details, Post-development

Node Name	Routing Method	Infiltration Method	Contributing Area (ha)	Impervious % (%)	Time of Concentration (min)	Slope % (%)
6F2/1	Time-Area.	Normal	17.71	50	20	1.0
6F1/10	Time-Area.	Normal	7.94	24	14	2.0
6F1/9	Time-Area.	Normal	8.62	36	13	1.3
6F1/8	Time-Area.	Normal	14.82	50	20	1.0
-	-	-	3.94	36	-	1.0
-	Time-Area.	Normal	-	-	14	-
6F1/7	Time-Area.	Normal	5.76	50	13	1.0
6F1/6	Time-Area.	Normal	13.51	50	14	1.0
6F1/5ds	-	-	-	-	-	-
Basin-us	-	-	-	-	-	-
Basin	Time-Area.	Normal	13.29	36	17	0.5
6F1/4A	-	-	-	-	-	-
6F1/4us	Time-Area.	Normal	11.88	36	15	0.5
6F1/4ds	-	-	-	-	-	-
6F1/3	Time-Area.	Normal	10.50	24	16	0.5
6F1/2	Time-Area.	Normal	5.79	24	13	0.5
6F1/1	Time-Area.	Normal	5.40	24	13	0.5
Outlet	-	-	-	-	-	-

13) Re-run the model for the minor storm, in this case 5 year ARI. If post-development flow exceeds that for existing conditions, detention measures will be required. In this case an off-line detention storage was added.

14) Adjust the minor system design if necessary to achieve satisfactory performance in the minor system design storm. In this case, the design was considered to be acceptable.

15) Re-run the model for the major storm to check the design including overland flow paths, and adjust if necessary. The resulting design is shown in the Table 16.A6.

Table 16.A6 Drainage System Details, Existing System

Link Name	U/S Node	D/S Node	No.	Type	Width (m)	Depth (m)	Sideslope (Z)	Length (m)	IL u/s (m)	IL d/s (m)	Manning n
25	6F2/1	6F1/8	-	Trapezoidal	2.0	1.5	0	400	46.4	41.0	0.017
26	6F1/10	6F1/9	-	Trapezoidal	1.0	1.5	4	200	47.0	42.5	0.040
27	6F1/9	6F1/8	-	Trapezoidal	1.0	1.5	4	250	42.5	41.0	0.040
28	6F1/8	6F1/7	-	Trapezoidal	2.5	2.5	0	150	41.0	40.7	0.017
29	6F1/7	6F1/6	-	Trapezoidal	2.5	2.5	0	200	40.7	39.0	0.017
30	6F1/6	6F1/5ds	-	Trapezoidal	2.5	2.5	0	200	39.0	37.8	0.017
46	6F1/6	Basin-us	-	-	-	-	-	-	39.0	39.5	-
45	6F1/5ds	6F1/4A	-	Trapezoidal	3.0	3.0	0	60	37.8	37.6	0.017
47	Basin-us	Basin	-	Trapezoidal	1.0	1.0	4	200	39.5	39.5	0.025
41	Basin	6F1/4A	-	-	-	-	-	-	39.5	37.6	-
-	-	-	-	Circular	-	1.4	-	30	-	-	0.014
38	6F1/4A	6F1/4us	-	Trapezoidal	3.0	3.0	0	140	37.6	37.2	0.017
39	6F1/4us	6F1/4ds	-	-	-	-	-	-	37.2	37.1	-
-	-	-	2	Rectangle	1.5	1.5	-	10	-	-	0.014
-	-	-	-	Trapezoidal	10.0	0.2	5	5	-	-	0.025
32	6F1/4ds	6F1/3	-	Trapezoidal	10.0	3.0	4	200	37.1	36.6	0.025
33	6F1/3	6F1/2	1	Trapezoidal	10.0	3.0	4	200	36.6	36.0	0.025
34	6F1/2	6F1/1	-	Trapezoidal	10.0	3.0	4	200	36.0	35.3	0.025
36	6F1/1	Outlet	-	Trapezoidal	10.0	3.0	4	100	35.3	34.8	0.025

16) Table 16.A7 and 16.A8 show the results for the designed system for 100 year ARI and 5 year ARI 60 minute duration events. Note that the design should also be checked for other durations, because the addition of a detention storage may make the critical duration longer. That step was omitted in this Example. Also run the model for the 3 month ARI storm to calculate the design flow for the GPT, which will be designed in Chapter 34.

Table 16.A7 100 year ARI 60 minute Storm, Post development

Link Name	U/S Node	D/S Node	IL u/s (m)	IL d/s (m)	Manning n	Level u/s (m)	Surcharge Time (min)	Q_{max} (m^3/s)	Velocity (m/s)	V x D (m^2/s)
25	6F2/1	6F1/8	46.4	41.0	0.017	47.61	0.0	9.54	3.81	4.87
26	6F1/10	6F1/9	47.0	42.5	0.040	47.67	0.0	4.64	1.50	1.06
27	6F1/9	6F1/8	42.5	41.0	0.040	44.24	0.0	7.63	0.84	1.69
28	6F1/8	6F1/7	41.0	40.7	0.017	43.96	7.0	24.38	4.06	10.63
29	6F1/7	6F1/6	40.7	39.0	0.017	43.06	0.0	27.19	4.49	11.16
30	6F1/6	6F1/5ds	39.0	37.8	0.017	41.62	11.1	24.72	4.01	10.12
46	6F1/6	Basin-us	39.0	39.5	-	41.62	11.1	-	-	-
45	6F1/5ds	6F1/4A	37.8	37.6	0.017	40.43	0.0	24.68	3.13	8.23
47	Basin-us	Basin	39.5	39.5	0.025	41.57	38.0	9.46	1.89	2.89
41	Basin	6F1/4A	39.5	37.6	-	41.03	0.0	-	-	-
-	-	-	-	-	0.014	-	-	6.56	5.30	7.45
38	6F1/4A	6F1/4us	37.6	37.2	0.017	40.25	0.0	29.00	3.92	9.60
39	6F1/4us	6F1/4ds	37.2	37.1	-	39.47	0.0	-	-	-
-	-	-	-	-	0.014	-	-	14.89	3.62	6.29
-	-	-	-	-	0.025	-	-	19.57	4.35	0.00
32	6F1/4ds	6F1/3	37.1	36.6	0.025	38.38	0.0	34.39	1.61	2.23
33	6F1/3	6F1/2	36.6	36.0	0.025	38.10	0.0	38.98	1.80	2.48
34	6F1/2	6F1/1	36.0	35.3	0.025	37.26	0.0	41.46	2.30	2.78
36	6F1/1	Outlet	35.3	34.8	0.025	36.46	0.0	43.83	2.53	2.98

Table 16.A8 5 year ARI 60 minute Storm, Post-development

Link Name	U/S Node	D/S Node	IL u/s (m)	IL d/s (m)	Manning n	Level u/s (m)	Surcharge Time (min)	Q_{max} (m^3/s)	Velocity (m/s)	V x D (m^2/s)
25	6F2/1	6F1/8	46.4	41.0	0.017	47.31	0.0	6.50	3.55	3.26
26	6F1/10	6F1/9	47.0	42.5	0.040	47.56	0.0	3.09	1.41	0.88
27	6F1/9	6F1/8	42.5	41.0	0.040	43.61	0.0	5.83	0.80	1.22
28	6F1/8	6F1/7	41.0	40.7	0.017	43.35	0.0	17.98	3.43	7.10
29	6F1/7	6F1/6	40.7	39.0	0.017	42.50	0.0	19.98	4.15	8.05
30	6F1/6	6F1/5ds	39.0	37.8	0.017	41.08	0.0	18.35	3.58	7.33
46	6F1/6	Basin-us	39.0	39.5	-	41.08	0.0	-	-	-
45	6F1/5ds	6F1/4A	37.8	37.6	0.017	40.03	0.0	18.32	2.73	6.11
47	Basin-us	Basin	39.5	39.5	0.025	41.03	24.8	6.51	1.54	1.76
41	Basin	6F1/4A	39.5	37.6	-	40.56	0.0	-	-	-
-	-	-	-	-	0.014	-	-	5.08	4.91	5.47
38	6F1/4A	6F1/4us	37.6	37.2	0.017	39.89	0.0	21.65	3.25	7.20
39	6F1/4us	6F1/4ds	37.2	37.1	-	39.37	0.0	-	-	-
-	-	-	-	-	0.014	-	-	14.07	3.60	5.81
-	-	-	-	-	0.025	-	-	11.46	3.68	0.00
32	6F1/4ds	6F1/3	37.1	36.6	0.025	38.17	0.0	25.48	1.52	1.76
33	6F1/3	6F1/2	36.6	36.0	0.025	37.85	0.0	28.53	1.67	1.93
34	6F1/2	6F1/1	36.0	35.3	0.025	37.06	0.0	29.92	2.08	2.11
36	6F1/1	Outlet	35.3	34.8	0.025	36.27	0.0	31.40	2.03	2.21

Figure 16.A3 Network Analysis Results for Stormwater Quantity of 5 year ARI for Existing Condition

Figure 16.A4 Network Analysis Results for Stormwater Quantity of 100 year ARI for Existing Condition

Figure 16.A5 Network Analysis Results for Stormwater Quantity of 5 year ARI for Post-development Condition

Figure 16.A6 Network Analysis Results for Stormwater Quantity of 100 year ARI for Post-development Condition

Figure 16.A7 Network Analysis Results for Stormwater Quantity of 3 month ARI for Post-development Condition

16.A.2 Network Design of Treatment Train for Stormwater Quality Control

(a) Solution

The calculations are performed on an average annual basis, using a spreadsheet and EMCs.

1) The calculation of weighted average volumetric runoff coefficient C_v for pre and post-development cases is shown in Table 16.A9. Note that the C_v values are changed to represent new development.

Table 16.A9 Weighted Average Volumetric Runoff Coefficient

Node	Catchment Area (ha)	Total Catchment Area (ha)	Pre-development Landuse	Pre-development Curve on Chart*	Pre-development Coefficient Cv	Post-development Landuse	Post-development Curve on Chart*	Coefficient Cv
6F2/1	17.71		Rural, sandy, open crop	D	0.50	urban residential	4	0.59
6F1/10	7.94		urban residential	4	0.59	urban residential	4	0.59
6F1/9	8.62		urban residential	4	0.59	urban residential	4	0.59
6F1/8	14.82		Rural, sandy, open crop	D	0.50	urban residential	4	0.59
-	3.94		urban residential	4	0.59	urban residential	4	0.59
6F1/7	5.76		urban residential	4	0.59	urban residential	4	0.59
6F1/6	13.51		urban residential	4	0.59	urban residential	4	0.59
Total		72.30			0.55			0.59
6F1/5us	13.29		urban residential	4	0.59	urban residential	4	0.59
6F1/5ds	-		-			-		
6F1/4us	11.88		urban residential	4	0.59	urban residential	4	0.59
6F1/4ds	-		-			-		
6F1/3	10.50		urban residential	4	0.59	urban residential	4	0.59
6F1/2	5.79		urban residential	4	0.59	urban residential	4	0.59
Total		41.46			0.59			0.59
6F1/1	5.40		urban residential	4	0.59	urban residential	4	0.59
Total		5.40			0.59			0.59
Outlet	-	-	-			-		
TOTAL	119.16	119.16						

NOTE: * Curves 1-8 give runoff coefficients for urban catchments, see Design Chart 14.3
* Curves A-F give runoff coefficients for rural catchments, see Design Chart 14.4
To estimate C_v, use the rainfall intensity from a 60 minute duration, 1 year ARI storm
${}^{1}I_{1}$ = 52.8 mm/hr

2) Using these Cv values, hydrologic data from Appendix 16.A.3 and information from other sections of this Manual, the calculation of the average annual pre-development load is shown in Table 16.A10.

   The calculation is done on a mass balance basis starting from the top of the catchment. Separate columns must be used for each pollutant. Assume that the only pollutant sources are from catchment runoff.

3) Similarly, the calculation of the average annual post-development load is shown in Table 16.A11. The table also shows the calculation of pollutant removal in the water quality control devices (off-line basin, GPT and pond). The hydraulic effects of diversions have been estimated for the purposes of this example. In practice, design calculations should involve computer simulation to estimate the annual average volume of flow diverted and infiltrated.

Table 16.A10 Average Annual Load Calculation, Pre Development

No.		Catchment Parameters			Sediment	TN	TOTAL LOAD (kg) Sediment	TOTAL LOAD (kg) TN
1.		~~Rural Catchment to 6F1/6~~						
		Catchment area	32.53	ha	(nodes 6F2/1, 6F1/8)			
		Annual rainfall depth	2200	mm				
		Volumetric Runoff Coefficient C_v	0.50					
		Annual runoff depth	1100	mm				
		Annual runoff volume	357,830	m^3				
		EMC of pollutant (Table 15.2)		mg/L	85	0.2		
		Pollutant load from rural areas		kg	30,416	72		
2.		~~Urban Catchment to 6F1/6~~						
		Catchment area	39.77	ha				
		Annual rainfall depth	2200	mm				
	#	Volumetric Runoff Coefficient C_v	0.59					
		Annual runoff depth	1298	mm				
		Annual runoff volume	516,215	m^3				
	*	EMC of pollutant (Table 15.2)		mg/L	100	1.2		
		Pollutant load from urban areas		kg	51,621	619	82,037	691
3.		~~Catchment from 6F1/6 to 6F1/2~~						
		Catchment area	41.46	ha				
		Annual rainfall depth	2200	mm				
	#	Volumetric Runoff Coefficient C_v	0.59					
		Annual runoff depth	1298	mm				
		Annual runoff volume	538,151	m^3				
	*	EMC of pollutant (Table 15.2)		mg/L	100	1.2		
		Pollutant load from this catchment		kg	53,815	646	53,815	646
		~~Total catchment to 6F1/2~~						
		Total catchment area	81.2	ha				
		Total runoff volume	1,054,365	m^3				
		Total sediment load at 6F1/2	105,437	tonnes			135,852	1,337
4.		~~Catchment from 6F1/2 to 6F1/1~~						
		Catchment area	5.40	ha				
		Annual rainfall depth	2200	mm				
	#	Volumetric Runoff Coefficient C_v	0.59					
		Annual runoff depth	1298	mm				
		Annual runoff volume	70,092	m^3				
	*	EMC of pollutant (Table 15.2)		mg/L	100	1.2		
		Pollutant load from this catchment		kg	7,009	84	7,009	84
		~~Total catchment to Pond~~						
		Total catchment area	86.63	ha				
		Total runoff volume	1,124,457	m^3				
		Total load at 6F1/1		kg			142,861	1,421

~~*NOTE:*~~

\# for calculation of weighted average volumetric runoff coefficient, see Table 16.A9

* indicates that values are derived from the referenced Chapter, or estimated

Table 16.A11 Average Annual Load Calculation, Post Development

No.	Catchment Parameters			Sediment	TN	TOTAL LOAD (kg) Sediment	TOTAL LOAD (kg) TN
1.	~~Catchment to 6F1/6~~						
	Catchment area	72.30	ha				
	Annual rainfall depth	2200	mm				
	# Volumetric Runoff Coefficient C_v	0.59					
	Annual runoff depth	1298	mm				
	Annual runoff volume	938,454	m^3				
	* EMC of pollutant (Table 15.2)		mg/L	100	1.2		
	Pollutant load at 6F1/6		kg	93,845	1,126	93,845	1,126
	~~Off-line basin~~						
	* Flow diverted to detention (estimate)	2%	of annual total volume				
	@ (Assuming that flows above 3 month ARI are diverted by the side weir)						
	Flow diverted to detention pond	18,769	m^3				
	Average concentration of pollutant in flow		mg/L	100	1.2		
	Pollutant diverted to detention pond		kg	1,877	23	-1,877	-23
	@ Flow and pollutant removal by infiltration (estimate):			50%			
	Pollutant load removed		kg	938	11		
	Pollutant returned to main stream		kg	938	11	938	11
	Total load downstream of detention pond					92,907	1,115
2.	~~Catchment from 6F1/6 to 6F1/2~~						
	Catchment area	41.46	ha				
	Annual rainfall depth	2200	mm				
	# Volumetric Runoff Coefficient C_v	0.59					
	Annual runoff depth	1298	mm				
	Annual runoff volume	538,151	m^3				
	* EMC of pollutant (Table 15.2)		mg/L	100	1.2		
	Load from this catchment		kg	53,815	646	53,815	646
	~~Total catchment to GPT at 6F1/2~~						
	Total catchment area	113.8	ha				
	Total runoff volume	1,476,605	m^3				
	Total load at 6F1/2	92,907	kg			146,722	1,761
	GPT area:	140	m^2				
	At/Ac	1.2E-04					
	U	80%					
	Soil type	average soil					
	* Sediment retention of reference soil	(Design Chart 34-1)		33%	10%		
	* Volume correction factor F_v	(Design Chart 34-2)		1.00			
	Pollutant removed		kg	30,659		-30,659	-176
	Pollutant returned to main stream		kg	62,248		116,063	1,585
3.	~~Catchment from 6F1/2 to 6F1/1~~						
	Catchment area	5.40	ha				
	Annual rainfall depth	2200	mm				
	# Volumetric Runoff Coefficient C_v	0.68					
	Annual runoff depth	1496	mm				
	Annual runoff volume	80,784	m^3				
	* EMC of pollutant (Table 15.2)		mg/L	100	1.2		
	Load from this catchment		kg	8,078	97	8,078	97
	~~Total catchment to Pond~~						
	Total catchment area	119.16	ha				
	Total runoff volume	1,557,389	m^3				
	Total pollutant load at 6F1/1		kg			124,141	1,682
	Pond area:	1.19	ha				
	Ap/Ac	1.0%					
	DCIA	50%					
	* Pollutant removal in pond	(from Design Chart 35.A1)		65%	40%		
	Pollutant removed		kg	80,692	673	-80,692	-673
	Pollutant remaining	(output)	kg	43,449	1,009	43,449	1,009

~~NOTE:~~

for calculation of weighted average volumetric runoff coefficient, see Table 16.A9

* indicates that values are derived from the referenced Chapter, or estimated

@ this estimate should be confirmed by a long-term simulation *(b) Summary of Results*

Once the annual load calculations have been performed, it is possible to compare the pre-development and post-development cases by assessing the overall efficiency and pollutant load reduction (post-development) of the water quality control measures. The results are shown in Table 16.A12.

Table 16.A12 Average Annual Load and Removal Efficiency Calculation, Post Development

Calculation for		Unit	Sediment	TN
Load Reduction:				
Annual load pre-development	=	kg	142,861	1,421
Annual load post-development	=	kg	43,449	1,009
Pollutant load reduction	=		(142,861-43,449)/142,861	(1,421-1,009)/1,421
Reduction percentage	=	%	69.6	29.0
Removal Efficicency:				
Total sediment load generated from the catchment (input)	=		93,845+53,815+8,078	1,126+646+97
	=	kg	155,739	1,869
Overall removal efficiency			(Input - output)/Input	
of treatment train	=		(155,739-43,449)/155,739	(1,869-1,009)/1,869
Efficiency in percent	=	%	72.1	46.0

Compare these results with the target system Design Acceptance Criteria in Chapter 4. In this example, the proposed treatment system is predicted to achieve almost 70% reduction in sediment and 29% reduction in TN load compared with existing conditions. These figures exceed the minimum target criteria for drainage system upgrading in Table 4.5.

16.A.3 Network Design for Stormwater Conveyance Facilities Using Rational Method

(a) Calculation Procedure

The Rational Method has probably been the most popular method for designing storm systems. It has been applied all over the world and many refinements of the method have been produced.

For small catchment areas, it continues to be a reasonable method, provided that it is used properly and that results and design concepts are assessed for reasonableness. For a detailed discussion on the application/limitations of the method, see Chapter 14.

This section outlines a simple, Rational Method procedure for system design calculations for catchment areas less than 80 hectares that *DO NOT* contain detention or retention storages. This procedure is suitable for small systems where the establishment of a computer model is not warranted.

The steps in the Rational Method calculation procedure are summarised below:

- The drainage area is first subdivided into sub-areas with homogeneous landuse according to the existing or planned development.
- For each sub-area, estimate the runoff coefficient *C* and the corresponding area *A*.
- The layout of the drainage system is then drawn according to the topography, the existing or planned streets and roads and local design practices.

- Inlet points are then defined according to the detail of design considerations. For main drains, for example, the outlets of the earlier mentioned homogeneous sub-areas should serve as the inlet nodes. On the other hand in very detailed calculations, all the inlet points should be defined according to local design practices.
- After the inlet points have been chosen, the designer must specify the drainage sub-area for each inlet point A and the corresponding mean runoff coefficient *C*. If the sub-area for a given inlet has non-homogeneous landuse, a weighted coefficient may be estimated.
- The runoff calculations are then done by means of the general Rational Method equations for each inlet point, proceeding from the upper parts of the watershed to the final outlet. The peak runoff, which is calculated at each point, is then used to determine the size of the downstream trunk drain using a hydraulic formula for open channel/pipes flowing full.

After the preliminary minor system is designed and checked for its interaction with the major system, reviews are made of alternatives, hydrological assumptions are verified, new computations are made, and final data obtained on street grades and elevations. The engineer then should proceed with final hydraulic design of the system.

(b) Calculation Worksheet

A worksheet for the hand calculation procedure is given in Table 16.A13. This worksheet can be used for open drains or, with minor alterations, for piped systems. The procedure can also be set up on a computer spreadsheet. In this example the network is analysed for the upper catchment only (until Node 6F1/6 of Figure 16.A2) to make sure that the total catchment area is less than 80 ha.

The procedure is for the average situation; variations may be necessary to fit actual field conditions.

1) Column 1 – Determine design point location and list. This design point should correspond to the sub-catchment illustrated on the preliminary layout map. List sub-catchments contributing runoff to this point that have not previously been analysed. The sub-catchment at downstream point will be only noted at drainage junction point. The sub-catchment nodes are numbered as discussed in Section 16.4.
2) Column 2 – The drainage system will be numbered as discussed in Section 16.4.
3) Column 3 – Enter length of flow path between previous design point and design point under consideration.
4) Column 4 – The area, *A* in hectare of the sub-catchments listed Column 1 is tabulated here. Subtract ponding areas, which do not contribute to direct runoff.
5) Column 5 - Runoff coefficient, *C*, for post development conditions for the sub-catchments listed Column 1, should be determined and listed. The *C* value should be weighted if the subcatchments contain areas with different *C* values.
6) Column 6 – The equivalent area, *C* x *A* of each sub-catchment.
7) Column 7 - Determine the inlet time for the particular design point. For the first design point of a system, the inlet time will be equal to the time of concentration (t_c). Remember that t_c is the wave travel time, and includes both overland flow time and travel time in a discrete channel. For subsequent design points, inlet time should also be tabulated to determine if it may be of greater magnitude than the accumulated time of concentration from upstream sub-catchments. If the inlet time exceeds the time of concentration from upstream catchment, and the area tributary to the inlet is of sufficient magnitude, the inlet time should be substituted for time of concentration and used for this and subsequent sub-catchments.

   In other words, at each design point in the system, the engineer should ascertain whether the total drainage area with a composite t_c or the given individual upstream catchment (with a different t_c) produces the higher discharge.
8) Column 8 – Enter the appropriate flow time (wave travel time) between the previous design point and the design point under consideration.
9) Column 9 – The sub-area time of concentration, t_c is found by adding columns 7 and 8.
10) Column 10 – The appropriate design return period for the drain, depend on the type of landuse within the sub-catchment.
11) Column 11 – The total equivalent area is the summation of *C* x *A* of the previous design point.
12) Column 12 – The critical time of concentration, t_c is the longer t_c summation of the previous design point time of concentration and the intervening flow time.

13) Column 13 – The total time in drain is found by adding the previous time in drain to the time in drain in current section.
14) Column 14 – The intensity to be applied to the sub-catchments under consideration is obtained from the intensity-duration-frequency curve developed for the specific area under consideration based upon depth-duration-frequency information. The intensity is determined from the time of concentration and the return frequency for this particular design point. A hypothetical IDF relationship was used in Table 16.A13.
15) Column 15 – Direct runoff from the tributary sub-catchments listed in Column 1 is calculated and tabulated here by multiplying columns 11 and 14 together.
16) Column 16 – List the proposed channel/pipe gradient.
17) Column 17 – List the required channel/pipe size to convey the quantity of flow. For standard open drains or pipes, round up to the next commercially available size.
18) Column 18 – List the capacity of the channel/pipe flowing full (with the slope expressed in Column 16).
19) Column 19 – Tabulate the actual velocity of flow in the proposed channel/pipe for the design Section.
20) Column 20 – By dividing the length of the channel/pipe by its velocity, the time of flow in the drain can be determined.
21) Column 21 – Proposed drain invert level.
22) Column 22 – Calculate the discharge for the 100 year ARI to check the capacity of the drainage system for major storms.
23) Additional Column – This column is optional to include any remarks or comments that may affect or explain the design. The allowable quantity of carry-over across street intersections, if any, should often be listed for the minor design storm. When routing the major storm through the system, required elevations for adjacent construction can often be listed in this column.